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FINITE-ELEMENT ASSISTED DESIGN OF EAVES JOINT OF COLD-FORMED STEEL PORTAL FRAMES HAVING SINGLE CHANNEL-SECTIONS

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Abstract

A finite element model is described for the eaves joint of a cold-formed steel portal frame that comprises a single channel-section and bracket connected through both screws and bolts. Such a joint detail is commonly used in practice in New Zealand and Australia; as the bolt-holes are detailed for nominal clearance the intended purpose of the screws is to prevent slip of the joint during frame erection. The results of the finite element model are compared against experimental test results. In the experimental tests, the screws failed in shear followed by twisting of the joint bracket. It is shown that at first screw failure, the bolts had not engaged i.e. they still had not fully slipped. It is also shown that the gradient of load-displacement curve can be accurately predicted from the equations of screw stiffness of Zaharia and Dubina (2000). The finite element model is used to determine upper and lower bounds to the experimental failure load, and thus potentially can be used to assist designers when designing and detailing such eaves joints in practice.

Keywords

Cold-formed steel; Portal frames; Eaves joint; Channel-section; Finite element analysis

Nomenclature

B	Channel section flange width;
D	Measured screw or bolt diameter;
D_l	Channel section lip width;
DES	Design;
e	Lever arm of the eaves joint;
E_s	Steel modulus of elasticity;
EXP	Experiment;
F	Applied load (see Fig.6);
$F_{DES,AISI}$	Maximum design load which can be resisted by the joint assuming failure of the channel section to AISI S100 (2016);
FEA	Finite element analysis;
F_{EXP}	Load recorded experimentally;
$F_{EXP,S}$	Load recorded at first screw failure;
$F_{EXP,U}$	Peak experimental load;
F_{FEA}	Applied load from FEA;
$F_{FEA,S}$	Applied load in FEA when screw shear reaches 8.5 kN;
$F_{FEA,U}$	Peak load predicted by FEA;
f_i	Calculated shear force in screws or bolts;
G	Gradient of load-displacement curve;
G_{EXP}	Gradient of experiment load-displacement curve;
G_{FEA}	Gradient of FEA load-displacement curve;
H	Channel section height;
K_θ	Rotational stiffness of the bolt and screw group;
$K_{\theta,C}$	K_θ for column;

$K_{\theta,R}$	K_{θ} for rafter;
K_j	Rotational stiffness of eaves joint;
$K_{j,EXP}$	Maximum design load which can be resisted by the joint assuming failure of the channel section to AISI S100 (2016);
$K_{j,FEA}$	Rotational stiffness predicted by FEA;
$K_{j,FEA,r}$	FEA- predicted rotational stiffness assuming out of plane restraint of the eaves joint;
$K_{j,DES}$	Rotational stiffness using hand calculation;
$K_{j,DES,L}$	Calculated rotational stiffness without contribution of the bolts;
$K_{j,DES,U}$	Calculated rotational stiffness including contribution of screws and bolts;
k_{ZD}	In-plane hole elongation stiffness of screw or bolt from Zaharia and Dubina (2000);
M	Applied bending moment;
M_a	Available flexural strength of a lipped channel section about centroidal axis, determined in accordance with AISI S100 (2016);
M_{DES}	Designed bending moment capacity of a lipped channel section;
$\bar{M}$	Required flexural strengths due to loads;
P_a	Available axial strength of a lipped channel section in compression determined in accordance with AISI S100 (2016);
$\bar{P}$	Required compressive axial strength due to loads;
r_i	Screw or bolt distance from centre of rotation;
t	Thickness of plate;
V_a	Available shear strength of a lipped channel section determined in accordance with AISI S100 (2016);
x_{cr}	x-coordinate of centre of rotation;

x_i	x-coordinate of screw or bolt in the joint;
y_{cr}	y-coordinate of centre of rotation;
y_i	y-coordinate of screw or bolt in the joint.

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1 Introduction

In countries such as New Zealand and Australia, with no or low snow loading, cold-formed steel (CFS) portal frames are a feasible and cost-effective alternative to hot-rolled portal frames and can be used for frame spans of up to 25 m. Advantages of CFS portal frames include being lightweight and easy to transport, as well as not requiring skilled workers for on-site assembly (Lim, Wrzesien et al., 2016). It is well-known that the behaviour of the eaves and apex joints are critical, as they need to transfer bending moment (as well as axial and shear force) from the columns to the rafters. The typical shape of the bending moment diagram of a portal frame under gravity load is shown in Fig.1.

Experimental and numerical studies relating to the design of CFS portal frames and their joints under static loading have previously been reported in the literature by Baigent et al. (1982), Kirk (1986), Bryan (1993), Lim and Nethercot (2002 and 2004), Mills and LaBoube (2004), Dubina et al. (2009), Kwon et al. (2008), Wrzesien et al. (2012) Wrzesien et al. (2015), Öztürk, F. and S. Pul (2015), Zhang et al. (2015), Zhang et al. (2016), Blum and Rasmussen (2018), and Wrzesien et al. (2018); Fig.2 shows some of these joint arrangements. The research on the structural behaviour of CFS portal frames under a seismic loading was reported by Dubina et al. (2004), Sato et al. (2009), Sabbagh et al. (2013), Mojtabaei et al. (2018) and McCrum et al. (2019).

Fig.3 shows the joint arrangement considered in this paper, which has not previously been considered in the literature. As can be seen, it comprises a single channel-section and bracket, connected through bolts and screws. Such a joint arrangement is commonly used in practice in New Zealand and Australia. For the joint arrangement described in this paper, the bolts are M16 and the diameter of the bolt-holes are 18 mm as they are detailed for nominal clearance. As the bolts are free to

move in clearance, the intended purpose of the screws is to prevent slip of the joint during frame erection. Fig.4 defines the parameters used to describe the channel-section.

2 Experimental investigation

Two eaves joints, of the arrangement shown in Fig.3, designated as Test Specimens A and B, were tested under a combination of closing moment, axial, and shear load. The channel-section and bracket were fabricated from high strength AS1397 G450 steel. The tests were conducted in the University of Auckland Structural Test Hall.

A photograph of the laboratory test arrangement is shown in Fig.5. As can be seen, the eaves joint was tested vertically. The test set-up was similar to that used by Mills and LaBoube (2004). However, unlike the test set-up of Mills and LaBoube (2004), the test arrangement adopted includes an eaves purlin to provide lateral restraint, as would be the case in practice; the length of this eaves purlin used was 1.95 m. The dimensions of the test set-up are shown in Fig.6 for Test Specimen A. (The dimensions are also identical for Test Specimen B.) As can be seen from Fig.6, the lever arm of the eaves joint is 0.705 m, as measured from the pin to the centreline of the joint. As can be seen, the channel-sections are under combined actions; Fig.7 shows the bending, axial and shear forces under a unit jacking force.

From Fig.3, C15024 single channel-sections were used for the column and rafter members, and C10015 single channel-sections for the eaves purlin. Table 1 shows the average measured centreline dimensions. Table 1 also shows the flexural strength, M_{DES} , and axial strength of the channel section, P_{DES} calculated in accordance with the direct strength method (DSM) in AISI S100 (2016), assuming a yield and ultimate strength of steel of 450 MPa. Considering only the strength of the channel-sections, the

maximum expected jacking force, $F_{DES,AISI}$ is 18.77 kN calculated in accordance with AISI S100 (2016) and based on Eq. (1):

$$\frac{\bar{P}}{P_a} + \frac{\bar{M}}{M_a} \leq 1.0 \quad (1)$$

where $\bar{P}$ and $\bar{M}$ are the required compressive axial and flexural strengths due to loads, and P_a and M_a are available compressive axial flexural strengths.

Fig.8 shows the eaves bracket size and bolt-group and screw position for Test Specimens A and B. The average measured thickness of the brackets was 2.48 mm. As can be seen, the dimensions of the bracket sizes, and therefore eaves joint arrangement, differed for Test Specimens A and B. Apart from the sizes of the brackets, the test arrangement was identical for Test Specimens A and B.

The channel-section was connected to the eaves bracket using No. 12-14x20 Tek screws and M16 Grade 4.8 bolts. The bolts were finger-tightened. As mentioned previously, the bolt-holes are detailed for nominal clearance and so were clearance holes with 18 mm diameter. According to the screw supplier catalogue (Buildex, 2018), the screws have a single shear capacity of 8.5 kN.

The eaves purlin was connected to the eaves joint via a 3 mm thick angle bracket screwed to the bracket with eight screws (see Fig.3 and 4). As can be seen from Fig.5, the other end of the eaves purlin was fixed to a rigid support.

As shown in Fig.6, two 6 mm stiffening plates were bolted to the webs of the channel-section to prevent local failure at the pin locations. At each loading point, a 30 mm diameter pin was inserted through holes drilled in the stiffening plates and the webs of the column and rafter. A displacement-controlled load via a 500 kN capacity MTS machine was applied to the specimens. The cross-head of the testing machine was set to move at a constant rate of 1 mm/min.

Fig.9 shows the variation of the load against displacement for Test Specimens A and B. Load cycles to remove any slack or friction were not conducted, as any such load cycles would result in irreversible deformation of the screw and bolt holes. The horizontal line, $F_{DES,AISI}$, is also shown in Fig. 9 For reference, $F_{DES,AISI}$ is also shown. As can be expected, $F_{DES,AISI}$ is much higher than the experimental failure load, since $F_{DES,AISI}$ is a check only for the channel-sections.

Fig.10 shows photographs taken after ultimate stage of loading i.e. 50 mm for Test Specimen A. The eaves joint failed in the following sequence:

- (a) shear failure of screw or screws
- (b) twisting of the channel-sections
- (c) formation of a yield line in the bracket

Table 2 (a) summaries the load at which first screw failure occurred, $F_{EXP,S}$, the peak load, $F_{EXP,U}$, and the gradient of the load-displacement, G_{EXP} , for Test Specimens A and B.

Based on the gradient of the load-displacement curves (see Table 2(b)), G_{EXP} , the rotational stiffness of the joints, $K_{j,EXP}$, that fits the model shown in Fig.11 can be determined. It should be noted that the simple model shown in Fig.11 assumes only one rotational spring for the joint. The results of the rotational stiffness that fits this model are shown in Table 2 (c); as can be seen, the rotational stiffness of both joints are similar.

It should be noted that no bedding down load cycles were conducted and also that there is a slightly higher initial stiffness up to approximately 2 kN. This increased initial stiffness can be attributed to friction either in the test setup or between the steel plates; this higher initial stiffness will be ignored in this paper. Beyond this slightly higher initial stiffness, the gradient of the load-displacement curve remained approximately constant until the first screw failed in shear after the load reached 8.04 kN. For the case

of Test Specimen A, a second screw failed almost immediately after the first screw failure. Following the sudden loss of stiffness, the load continued to increase again until a peak load, $F_{EXP,U}$, was reached of 8.67 kN, failure occurring due to yielding of the bracket.

For the case of Test Specimen B, the first and second screws failed in shear after the load reached 8.53 kN and 9.25 kN, respectively. Following failure of the second screw, the load increased gradually until the bracket failed due to yielding at 8.07 kN.

3 Finite element investigation

Fig.12 shows the finite element mesh used for analysis of Test Specimens. The finite element program ABAQUS (ABAQUS Manual, 2017) was used for the analyses. The channel-sections and bracket were modelled with 4-node doubly curved S4R shell elements with linear interpolation, reduced integration and hourglass control. As mentioned in the ABAQUS Manual (2017), the S4R element is suitable for complex buckling behaviour. S4R has six degrees of freedom per node and provides accurate solutions to most applications. A minimum mesh size of 5 mm x 5 mm was used for the channel-sections and bracket. Contact interaction with normal hard behaviour and frictionless tangential behaviour were modelled between all surfaces.

Instead of modelling the screws and bolt-holes physically, the screws and bolts in the joints were modelled using point-based fasteners, which act as multi-point constraints. The axial stiffness of the bolts was taken as $E_s A_b / L_b$ (where E_s is elastic modulus of elasticity, taken as 200 GPa, A_b is cross sectional area of the bolts, L_b is length of the bolt which is taken to be $t_B + t_C$ where t_B is thickness of the bracket and t_C is thickness of the channel-section). The axial stiffness of the screws were assumed to be 1 kN/mm, as per experimental test results on screw pull-out behaviour.

In addition to axial stiffness, Table 3 summarises the values of the screw-hole / bolt-hole stiffness used in the FEA model, attributed to tilting and bearing of the screw-hole/bolt-hole against the screw/bolt. This value is calculated from the semi-empirical equation proposed by Zaharia and Dubina (2000):

$$k_{zd} = 6.8 \frac{\sqrt{D}}{\left(\frac{5}{t_B} + \frac{5}{t_c} - 1\right)} \quad (\text{kN / mm}) \quad (2)$$

where D is the measured screw or bolt diameter.

While the semi-empirical equation was developed for bolted connections in clearance holes, it was demonstrated by Wrzesien (2016) that it can also be used for estimating the screw-hole stiffness. It should be noted that Wrzesien (2016) conducted a series of tests on single shear screwed joints and compared the experimental shear stiffness against calculated values.

Fig.13 shows the idealisation of the point-based fasteners used in this paper. As can be seen, the idealisation of the point-based fasteners does not take into account failure of the screws (and also of the bolts). As can also be seen, the screw-hole stiffness has been modelled without any slip. The point-based fasteners, however, can allow slip to be modelled if required. In this Section, an arbitrary slip of 1 mm slip is considered for the bolts.

In the out-of-plane direction, for the case of the screws and bolts, it was assumed that the bolts could freely rotate axially; the other two rotational degrees of freedom were considered to be rigid.

Static general analysis with geometric and material nonlinear properties was used. Young's modulus and Poisson's ratio were taken to be 200 GPa and 0.3, respectively. An elastic-perfectly plastic stress-strain curve was used, with a yield stress of 450 MPa.

The finite element results are also shown in Fig.9. The failure mode predicted by the finite element model are also shown in Fig.10. It can be seen that the FEA predicted the mode of failure similar to those of the experimental tests.

As can be seen from Table 3, using the values for screw-hole / bolt-hole stiffness from Eq. 2 (see Table 3), G_{FEA} is similar to G_{EXP} with less than 3% difference. Table 2 summaries the values of both G_{EXP} and G_{FEA} .

As mentioned previously, the finite element method does not take into account failure of the screws (and also of the bolts). The failure load of the eaves joint predicted by the FEA can, therefore, be considered to be an upper bound to the failure load of the joint i.e. it assumes that there is no failure of the screws (and also of the bolts).

It is useful also to identify the point on the FEA load-displacement curve that the screws reach the shear failure load of 8.5 kN (as given by Buildex, 2018). This point is also shown in Fig.9. As can be seen, for both Test Specimens A and B, it occurs below that of the experimental failure load, and so can be considered to be a lower bound to the failure load of the joint (see Table 2). For comparison, the point on the load-displacement curve that the screws reach a shear load of 11 kN is also shown. However, it can be expected statistically that the shear resistance of a single screw would be higher than the 8.5 kN recommended for design.

4 Effect of bolt slip

In the previous Section, a bolt slip of 1 mm was assumed. In this Section, two finite element idealisations are considered for the bolt:

- bolt-slip of 5 mm. Such a bolt-slip would be reasonable if the bolt-holes are slotted.
- no bolt-slip. Having no-bolt slip implies no clearance and that diameter of the bolt and bolt-holes are the same.

Fig.14 shows the effect of the bolt-slip of 5 mm. It can be seen that increasing the bolt-slip from 1 mm to 5 mm has had no effect on the gradient of the load-displacement curve. Fig.14 also shows the point where the shear in the screw reaches 8.5 kN. If this is assumed to represent a lower bound to the failure load, it can be seen that this point lies above the experimental failure load when bolts engage and do not slip.

Fig.15 shows the FEA-predicted stiffness relative to the values assumed from Zaharia and Dubina (2000). As can be seen, Fig.15 demonstrates the contribution of the screw-hole / bolt-hole stiffness to the overall stiffness of the eaves joint. As can be seen, the gradient of the load-displacement curve, G_{FEA} , increases by about 40%, when bolt-slip is ignored. The upper bound stiffness is also shown, obtained from a finite element model in which point-based fasteners, for both the screws and bolts, are idealised as rigid. For such a rigid assumption, the difference in gradient is 2.94 and 2.74 times the gradient of the Test Specimens A and B, respectively.

Furthermore, it can be seen from Fig.16 that this rigid assumption results in a stiffness of 2.06 and 1.82 times the gradient of the Test Specimens A and B, respectively. It is thus concluded that it is the screws that control the stiffness of the joints.

5 Effect of screw-hole stiffness

In the previous section it was demonstrated that it is the screw-hole stiffness that controls the gradient of the load load-displacement curve i.e. the bolts have not slipped sufficiently for them to engage. In this Section, a sensitivity study is conducted on the screw-hole stiffness.

Fig.16 shows the effect of the screw-hole stiffness. The parameter α is a factor describing the linear increase of screw-hole stiffness relative k_{zd} . As can be seen from Fig.16, the gradients are sensitive to the value of k_{zd} (i.e. it is not on a plateau). Also

shown in Fig.16 is the theoretical upper bound to the G_{FEA} , predicted from a finite element model in which the screw-hole stiffness was very large. It can be seen that the upper bound cannot be obtained even if the screw-hole stiffness was as high as 5 times that of k_{zd} .

6 Rotational stiffness

In Section 3, the rotational stiffness was previously determined and tabulated in Table 2(c), assuming one rotational spring for the joint. However, based on Lim and Nethercot (2002), Wrzesien et al. (2015) and Lim et al. (2016), it well-known that there are two centre of rotations, one in the column and one in the rafter (see Fig.17). Using the finite element model, the rotational stiffness of each can be determined. These are shown in Fig.11. It should be noted that this ignores any bracket deformation.

Having validated the finite element model against the experimental test results, the rotational stiffness of the joints, K_j , of Test Specimens A and B can now be determined from the finite element model. Table 4 summarises the rotational stiffness calculated.

Crawford and Kulak (1971) proposed a hand-calculation method for calculating the rotational stiffness of loaded fastener groups, that can be used for design purposes. Previously, using the results of full-scale frame tests, Lim and Nethercot (2004) demonstrated that this method can be applied to cold-formed steel portal frames having bolt moment-connections. By the method of Crawford and Kulak (1971), upper and lower bounds to the rotational stiffness can be determined:

- $K_{j,DES,U}$. This value is calculated assuming no bolt-slip and represents a design upper bound to the joint stiffness

- $K_{j,DES}$. This value is calculated assuming that the bolts do not engage and represents a design lower bound to the joint stiffness, assuming that the bolts slip.

Table 4(b) shows these upper and lower bounds. (The calculation method is shown in Appendix 1.) As can be seen, the rotational stiffness from finite element results lies between these two values. It should be noted that these calculations ignore the axial contribution for the flange bolts is ignored using these calculations. It is interesting to note that an average of the upper and lower bounds of rotational stiffness is close to the rotational stiffness predicated using the finite element results.

Effect of eaves purlin on rotational stiffness

In order to investigate the effect of the eaves purlin connection on the rotational stiffness, the finite element models were also re-run without the lateral restraint provided by the purlin; instead, lateral restraint was provided only at the position of the bolts and screws in the web. The value of $K_{j,FEA}$ increased only by 17% and 20% for Test Specimens A and B, respectively.

Effect of pure bending moment

The finite element model was used to investigate the effect of a constant bending moment. The rotational stiffness is shown in Table 4(c). It was found that the rotational stiffness of both Joints A and B increased by only around 5%, compared with the combined actions case.

7 Design using combined actions

Fig.18 shows the experimental and finite element results plotted as the variation of moment against displacement. Two horizontal lines are shown:

- M_a is the nominal moment capacity of the channel section used as shown in Table 1.

- $M_{DES,AISI}$ is the designed bending moment capacity of the lipped channel section used in the experiments and modelling based on AISI S100 (2016) considering combined axial compression and bending which is 12.85kN.m (= $F_{DES,AISI} \cdot e$ where e is the lever arm of the eaves joint which is 0.705 m, shown in Fig.6).

It can thus be seen that the axial and shear forces of the members only have a small effect on the failure load of the joint. As can be seen, the bending moment capacity with combined actions is relatively close to the pure bending capacity with about 12% difference. Also, model with pure bending moment showed a relatively similar capacity to the model with combined action with about 6% difference.

8 Conclusions

The eaves joint of a cold-formed steel portal frame, comprising a single channel-section and bracket, connected through both screws and bolts has been investigated using the finite element method. The following general conclusions can be drawn:

1. The screws are nominally provided in practice to prevent slip of the joints during frame erection. However, it has been shown it is the screws that control the stiffness of the eaves joint as the bolts have still not engaged. For clearance bolt-holes, bolt-slip needs to be modelled, otherwise the rotational stiffness of the joints will be overestimated.
2. The stiffness of the joints, attributable screw-hole stiffness, can be predicted from the semi-empirical equation by Zaharia and Dubina (2000).
3. The finite element model can be used to determine both upper and lower-bounds to the strength, by considering screw failure, and so can be used for the safe design of eaves joints. The upper-bound solution assumes no failure of the screws.

4. The results of the rotational stiffness are compared against a hand calculation method proposed by Crawford and Kulak (1971). It is interesting to note that an average of an upper and lower bounds is close to the rotational stiffness of the experimental test results.
5. Using the test-setup presented in this paper the rotational stiffness as well as flexural capacity of the eaves joint can be predicted. Axial and shear effects are shown to only affect the rotational stiffness and the flexural strength by about 13% and 5%, respectively.

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Appendix 1 Hand calculation method for rotational stiffness

Fig.17 shows a screw and bolt group of an eaves joint used for Test Specimen A. When this group resists an applied moment M , the screw and bolt group rotates the bracket about a point known as the centre of rotation. The induced force f_i is proportional to its distance r_i from the centre of rotation and acts in a direction perpendicular to the line drawn from the bolt to the centre of rotation. It is assumed that the columns and rafter members are rigid.

The centre of rotation can be calculated based on Eq. (3):

$$x_{CR} = \frac{\sum_{i=1}^n k_{zd,i} x_i}{\sum_{i=1}^n k_{zd,i}}, y_{CR} = \frac{\sum_{i=1}^n k_{zd,i} y_i}{\sum_{i=1}^n k_{zd,i}} \quad (3)$$

where n is the number of bolts and screws in the screw and bolt group. $K_{zd,i}$ is the stiffness of the bolt or screw based on Eq. (2) (see Table 3). x_i and y_i are the coordinates of the bolts and screws. From this, the induced force at the bolt or screw will be proportional to its distance from the centre of the bolts and screws:

$$f_i = c_i r_i \quad (4)$$

where c_i is a constant.

From this:

$$\sum_{i=1}^n f_i r_i = M \Rightarrow M = \sum_{i=1}^n c_i r_i r_i \quad (5)$$

On the other hand, the hole elongation δ_i based on Hooke's law would be:

$$\delta_i = \frac{f_i}{k_{zd,i}} = \frac{c_i r_i}{k_{zd,i}} \quad (6)$$

The rotation of the screw and bolt group, θ , can then be calculated by dividing the hole elongation of each screw and bolt by its distance from the centre of rotation:

$$\theta = \frac{\delta_i}{d_i} = \frac{c_i}{k_{zd,i}} \quad (7)$$

Therefore, the rotational stiffness of the bolt-group K_θ would be:

$$K_\theta = \frac{M}{\theta} = \frac{\sum_{i=1}^n m_i}{\theta} = \frac{\sum_{i=1}^n f_i r_i}{\theta} = \frac{\sum_{i=1}^n (c_i r_i) r_i}{\theta} = \frac{\sum_{i=1}^n k_{zd,i} \theta r_i^2}{\theta} = \frac{\theta \sum_{i=1}^n k_{zd,i} r_i^2}{\theta} = \sum_{i=1}^n k_{zd,i} r_i^2 \quad (8)$$

Having the rotational stiffness of the column, $K_{\theta,C}$, and rafter, $K_{\theta,R}$, bolt groups, the rotational stiffness of the eaves joint, $K_{j,DES}$, can be calculated as Eq. (8).

$$\frac{1}{K_{j,DES}} = \frac{1}{K_{\theta,C}} + \frac{1}{K_{\theta,R}} \quad (9)$$

References

- AISI S100. North American Specification for the design of cold-formed steel structural members. AISI S100-16, North American Cold-formed Steel Specification. American Iron and Steel Institute, Washington, D.C.; 2016.
- Bagheri Sabbagh, A., Petkovski, M., Pilakoutas, K., Mirghaderi, R. (2013). Cyclic behaviour of bolted cold-formed steel moment connections: FE modelling including slip. *Journal of Constructional Steel Research* 80: 100-108.
- Baigent, A.H., Hancock, G.J. (1982). Structural analysis of assemblages of thin-walled members. *Engineering Structures* 4: 207-216.
- Blum H.B., Rasmussen K.J.R. (2018). Elastic buckling of columns with a discrete elastic torsional restraint. *Thin-Walled Structures* 129:502-11.
- Bryan, E. R. (1993). The design of bolted joints in cold-formed steel sections. *Thin-walled Structures* 16: 239-262.
- Crawford S.F., Kulak G.L. (1971). Eccentrically loaded bolted connections. *Journal of the Structural Division* 97:765-83.
- Dubina, D., A. Stratan, Nagy, Z. (2009). Full-scale tests on cold-formed steel pitched-roof portal frames with bolted joints. *Advanced Steel Construction* 5 (2): 175-194.
- Dubina, D., Stratan, A., Ciutina, A., Fulop, L., Zsolt, N. (2004). Monotonic and cyclic performance of joints of cold formed steel portal frames. 4th International Conference on Thin-walled structures. Loughborough, UK: 381-388.
- Kirk, P. (1986). Design of a cold formed section portal frame building system. 8th International Specialty Conference on Cold-Formed Steel Structures. St. Louis, Missouri, USA: 295-310.

- Kwon, Y.B., H.S. Chung, H.S., Kim, G.D. (2008). Development of cold-formed steel portal frames with PRY sections. *Advances in Structural Engineering* 11(6): 633-649.
- Lim, J.B.P., Nethercot D.A. (2002). Design and development of a general cold-formed steel portal framing system. *The Structural Engineer* 80: 31-40.
- Lim, J.B.P., Nethercot D.A. (2004). Finite Element Idealization of a Cold-Formed Steel Portal Frame. *Journal of Structural Engineering* 130: 78-94.
- Lim, J.B.P., Hancock, G.J., Clifton, G.C., Pham, C.H., Das R. (2016). DSM for ultimate strength of bolted moment-connections between cold-formed steel channel members. *Journal of Constructional Steel Research* 117:196-203.
- Lim, J.B.P., Wrzesien, A.M., Nethercot, D.A. (2016). Sustainable applications of cold-formed steel structures: Portal frames. *Recent Trends in Cold-Formed Steel Construction*. C. Yu. Sawston, UK, Woodhead Publishing: 265-303.
- McCrum, D.P., Simon, J., Grimes, M., Broderick, B.M., Lim, J.B.P., Wrzesien, A.M. (2019). Experimental cyclic performance of cold-formed steel bolted moment resisting frames. *Engineering Structures* 181: 1-14.
- Mechanical properties. Buildex Web site. <https://www.buildex.com.au/technical-advice/application-specification-guides/mechanical-properties/>. Updated 2018.
- Mills, J., LaBoube, R., (2004). Self-drilling screw joints for cold-formed channel portal frames. *Journal of Structural Engineering* 130: 1799-1806.
- Mojtabaei, S.M., Kabir, M.Z., Hajirasouliha, I., Kargar, M. (2018). Analytical and experimental study on the seismic performance of cold-formed steel frames. *Journal of Constructional Steel Research* 143: 18-31.
- Öztürk, F., Pul, S. (2015). Experimental and numerical study on a full scale apex connection of cold-formed steel portal frames. *Thin-Walled Structures* 94: 79-88.

Sato, A., Uang, C.M. (2009). Seismic design procedure development for cold-formed steel–special bolted moment frames. *Journal of Constructional Steel Research* 65(4): 860-868.

Smith, M. (2017). *ABAQUS/Standard User's Manual, Version 2017*. Providence, RI: Simulia.

Wrzesien, A., Pouladi, P., Lim, J.B.P., Grimes, M., Broderick, B., McCrum, D. Effect of bolt holes clearance on the sway deflections of cold-formed steel portal frames. *International Conference on Engineering Research and Practice for Steel Construction 2018 (ICSC2018)*. Hong Kong, China.

Wrzesien, A.M. (2016). Effect of stressed skin action on the behaviour of cold-formed steel portal frames with non-linear flexible joints and top-hat purlins. Department of Civil and Environmental Engineering. Glasgow, The University of Strathclyde. Ph.D. Thesis.

Wrzesien, A.M., Lim, J.B.P., Nethercot, D.A. (2012). Optimum joint detail for a general cold-formed steel portal frame. *Advances in Structural Engineering*. 15(9): 1623-1639.

Wrzesien, A.M., Lim, J.B.P., Xu, Y., MacLeod, I.A., Lawson, R.M. (2015). Effect of stressed skin action on the behaviour of cold-formed steel portal frames. *Engineering Structures* 105: 123-136.

Zaharia, R., Dubina, D. (2000). Behaviour of cold-formed steel truss bolted joints. *The IV'th international workshop on connections in steel structures*.

Zhang, X., Rasmussen, K.J.R., Zhang, H. (2015). Structural modeling of cold-formed steel portal frames. *Structures* 4:58-68.

Zhang, X., Rasmussen, K.J.R., Zhang, H. (2016). Experimental investigation of locally and distortionally buckled portal frames. *Journal of Constructional Steel Research* 2016;122:571-83.

Zhang, X., Rasmussen, K.J.R., Zhang, H. (2016). Second-order effects in locally and/or distortionally buckled frames and design based on beam element analysis. *Journal of Constructional Steel Research* 122:57-69.

Table 1 Measured average centerline dimensions of channel-sections

Section	H (mm)	B (mm)	D_l (mm)	t (mm)	M_a (kNm)	P_a (kN)	V_a (kN)
C15024 Column/rafter	149.24	65.64	16.05	2.48	14.55	179.58	32.63
C10015 Eaves purlin	100.19	51.06	12.17	1.56	-	-	-

Table 2 Experimental test results

(a) Failure Loads

Test Specimen	$F_{EXP,S}$ (kN)	$F_{EXP,U}$ (kN)	$F_{FEA,S}$ (kN)	$F_{FEA,U}$ (kN)	$F_{DES,AISI}$ (kN)
A	8.04	8.67	5.35	11.20	18.77
B	8.50	9.25	5.84	10.85	18.77

(b) Gradient of load-displacement curves

Test Specimen	G_{EXP} (kN/mm)	G_{FEA} (kN/mm)
A	0.34	0.33
B	0.31	0.31

(c) Rotational stiffness of load-displacement curves

Test Specimen	$K_{j,EXP}$ (kN.m/rad)	$K_{j,FEA}$ (kN.m/rad)
A	192	203
B	173	187

Table 3 Values of k_{ZD} of the screws and bolts as per Zaharia and Dubina (2000)

	d (mm)	t_B (mm)	t_C (mm)	k_{ZD} (kN/mm)
Screw	5.36	2.47	2.48	5.17
Bolt	15.80	2.47	2.48	8.87

Table 4 Rotational stiffness of joints (kNm/rad)

(a) FEA of experimental test (combined actions)

Test Specimens	$K_{\theta,R,FEA}$	$K_{\theta,C,FEA}$	$K_{j,FEA}$	$K_{j,FEA,r}$
A	317	566	203	244
B	278	569	187	220

(b) Hand calculation after Crawford and Kulak (1971)

Test Specimens	Lower bound			Upper bound		
	$K_{\theta,R}$	$K_{\theta,C}$	$K_{J,DES,L}$	$K_{\theta,R}$	$K_{\theta,C}$	$K_{j,DES,U}$
A	238	649	174	498	1296	360
B	186	526	137	381	1016	277

(c) FEA (pure bending)

Test Specimens	$K_{\theta,R,FEA}$	$K_{\theta,C,FEA}$	$K_{j,FEA}$
A	329	614	214
B	316	525	197

Fig.1 Typical bending moment diagram of portal frame under gravity load

(a)

(b)

(c)

Fig.2 Eaves joints tested by (a) Kirk (1986), (b) Mills and LaBoube (2004), (c) Blum and Rasmussen (2018)

Fig.3 Eaves joint arrangement with single channel-section and bracket, connected through screws and bolts: (a) general assembly; (b) eaves bracket

Fig.4 Lipped channel section parameters [based on centerline](#)

Fig.5 Photograph of laboratory test set-up of eaves joint with purlin

Fig.6 Schematic drawing of eaves joint Test Specimen A

Fig.7 Internal action diagrams of Test Specimens

All dimensions are in

(a) Test Specimen A

(b) Test Specimen B

Fig.8 Details of bolt groups and screw positions and corresponding sizes of eaves brackets

(a) Test Specimen A

(b) Test Specimen B

Fig.9 Variation of load against displacement

(a) Front View

(b) Back View

(c) Front View

(d) Back view

Fig.10 Mode of failures for Test Specimen A

Fig.11 Frame model for the eaves joint of a portal frame based on current test setup

Fig.12 Isometric view of the finite element mesh of Test Specimen A

Fig.13 Force-displacement relationship of spring element used to idealise in-plane behaviour of screws and bolts

(a) Test Specimen A

(b) Test Specimen B

Fig.14 Effect of bolt-slip

(a) Test Specimen A

(b) Test Specimen B

Fig.15 Effect of bolt-slip on gradient of the load-displacement curves

(a) Test Specimen A

(b) Test Specimen B

Fig.16 Effect of screw stiffness (bolts assumed not to engage)

Fig.17 Model based on Crawford and Kulak (1971) to calculate rotational stiffness

(a) Test Specimen A

(b) Test Specimen B

Fig.18 Variation of moment against displacement