Application of component method for bolted cold-formed steel joints

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ABSTRACT:

The paper summarises the results of an experimental program carried out in order to evaluate performance of ridge (apex) and eaves (knee) joints of pitched roof cold-formed steel portal frames under monotonic and cyclic loading. 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. Application of the component method for determination of characteristics of connections in cold-formed members with both flange and web bolts was studied. Fair agreement between analytical and experimental results was obtained.

1 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 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.

2 SUMMARY OF TESTING PROGRAM

2.1 *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: self weight 0.35 kN/m^2 (with a partial safety factor of γ_{ULS}=1.1 for the ultimate limit state), technological load 0.15 kN/m^2 (γ_{ULS}=1.1) and snow load 0.72 kN/m^2 (γ_{ULS}=2.0). These loads were 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. The size of knee and ridge specimens and testing setup were chosen to obtain in the connected members a distribution of bending moment similar to the one observed in the designed structure.

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).

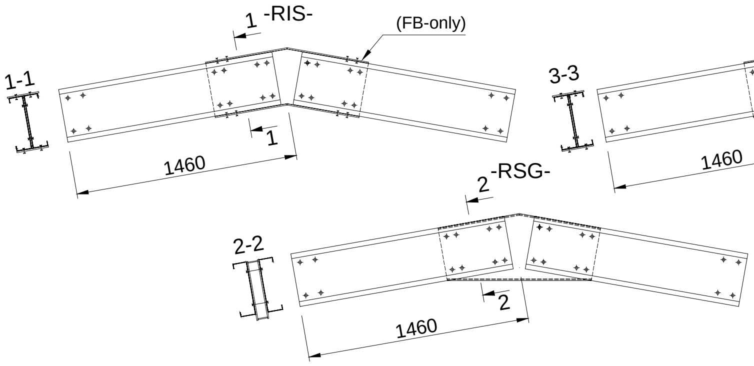


Figure 1. Configurations of ridge joints.

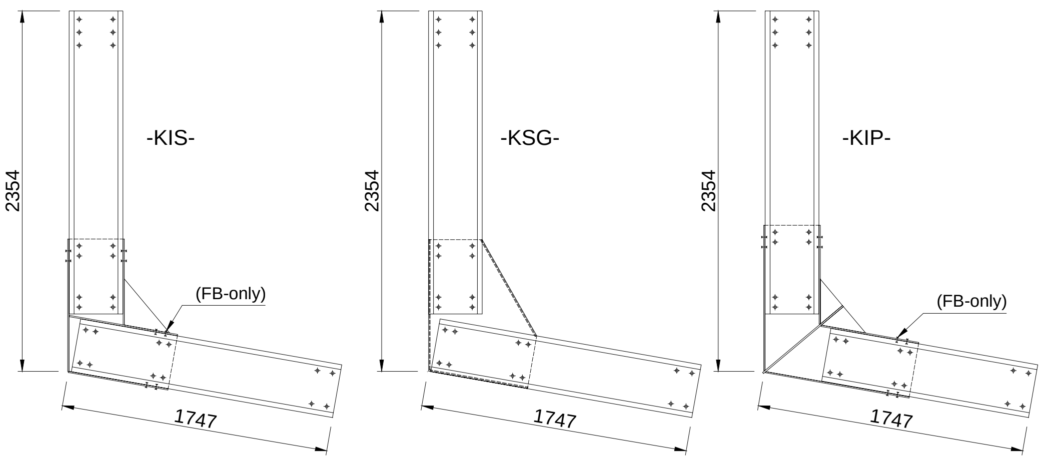


Figure 2. configurations of knee joints.

Table 1. Tested specimens.

Element type	Code	Loading type
RIS (Ridge connection with I Simple profile)	RIS-FB-M	Monotonic
	*RIS-FB-C1**	Cyclic: modified ECCS
	*RIS-FB-C2**	Cyclic: low cycle fatigue
RSG (Ridge connection with Spaced Gusset)	RSG-M	Monotonic
	RSG-C1	Cyclic: ECCS procedure
	RSG-C2	Cyclic: Modified ECCS
RIP (Ridge connection with I profile and end Plate)	RIP-M	Monotonic
	RIP-M	Monotonic
	RIP-C1	Cyclic - ECCS proc.
KSG (Knee connection with Spaced Gusset)	KSG-M	Monotonic
	KSG-C1	Cyclic - Modified ECCS
	KSG-C2	Cyclic - Low cycle fatigue
KIS (Knee connection with I Simple profile)	KIS-M	Monotonic
	*KIS-FB-M**	Monotonic
	*KIS-FB-C**	Cyclic - Modified ECCS
KIP (Knee connection with I profile and end Plate)	KIP-M	Monotonic
	*KIP-FB-M**	Monotonic
	*KIP-FB-C**	Cyclic - Modified ECCS

*FB Specimens (RIS, RIP, KIS, KIP) with supplementary bolts on the flange

The connecting bolts are subjected to shear and their design was carried out assuming the rotation of the joint around the centroid of the bolt group and a linear distribution of forces in each bolt, proportional to their distance from the centre of rotation. The bending moment reduced in the centre of rotation of the joint was considered for design of joints, not the theoretical one at the corner of the frame.

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 (see Table 1).

2.2 *Test setup*

Monotonic and cyclic experiments were performed for each specimen typology, all specimens being tested statically. Figure 3 shows the test setup and specimen instrumentation. For monotonically loaded specimens the loading velocity was approximately 3.33 mm/min, and 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.

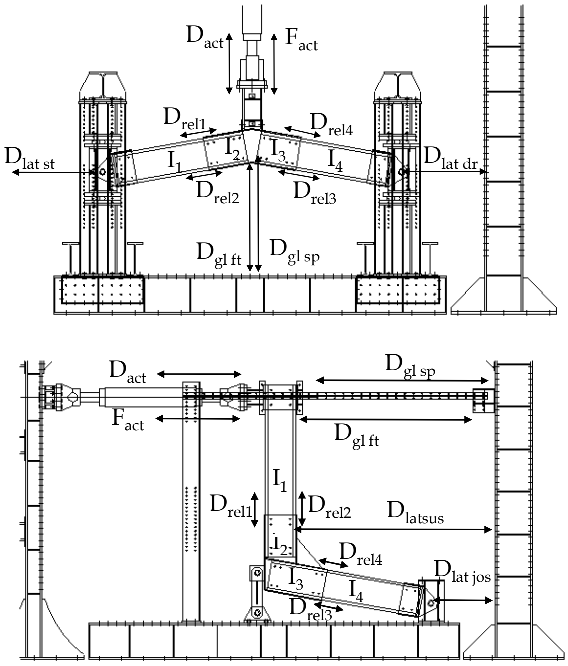


Figure 3. Loading scheme and instrumentation.

2.3 *Monotonic tests*

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 (Figure 4; Figure 5).

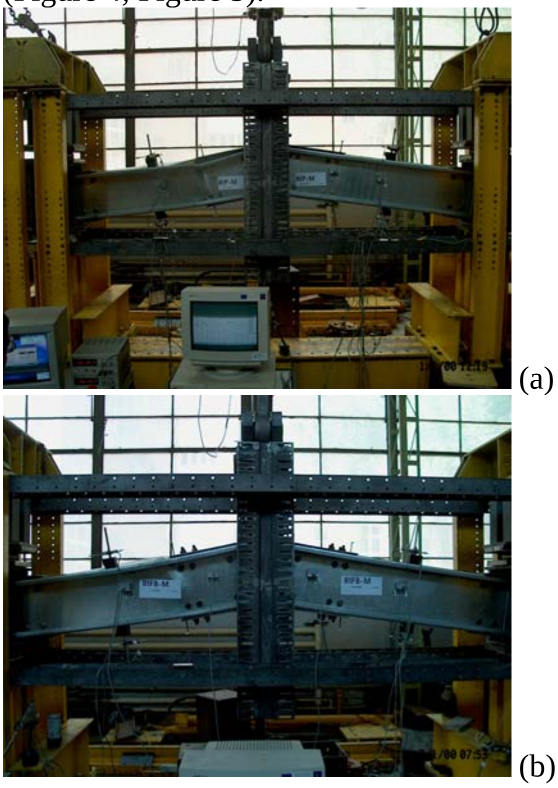


Figure 4. Failure of ridge specimens RIP-M (a) and RIS-FB-M (b).

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.

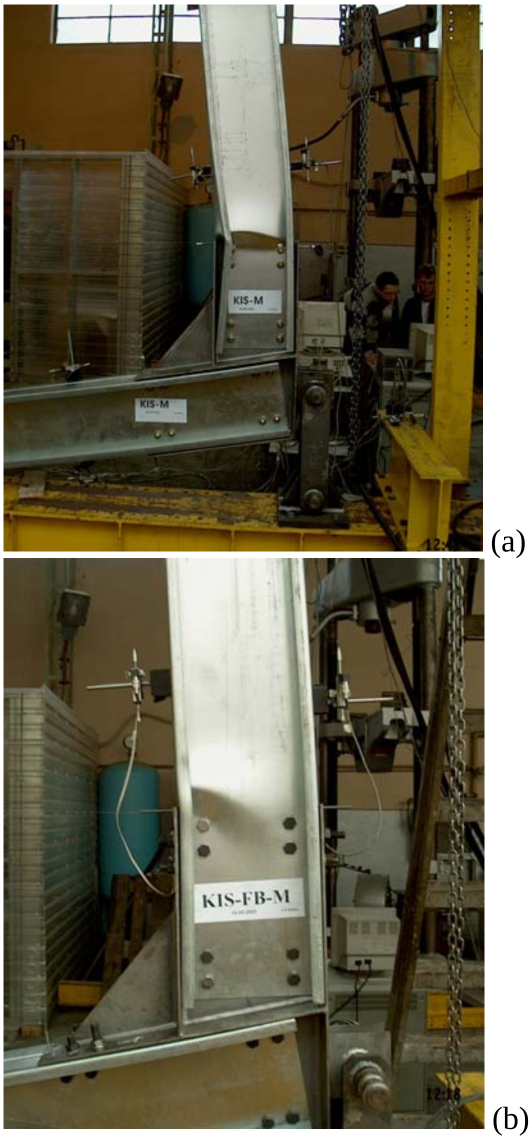
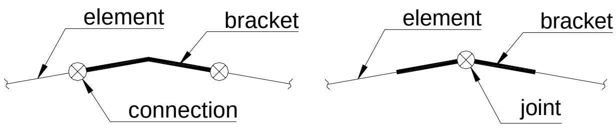


Figure 5. Failure of knee specimens KIS-M (a) and KIS-FB-M (b).



(a) (b)

Figure 6. Two possible models for ridge joints: detailed (a) and simplified (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 6a), and a simplified one, which considers the characteristics of the connection concentrated in one joint only (see Figure 6b). 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). For the simplified joint representation (as in Figure 6b), both moment (M_j) and rotations were considered at the intersection of the element centrelines.

Comparative experimental curves for ridge and knee connections are presented in Figure 7. There are no significant differences among the specimens without flange bolts (RSG-M, RIP-M, and KSG-M, KIS-M). This could be explained by the higher stiffness and capacity of the connecting bolts compared to the other components of the joint. On the other hand, there is an important gain in load bearing capacity when bolts are installed also on the flanges, although this joint type is more difficult to fabricate (RIS-FB-M and KIS-FB-M).

In Table 2 the yield and ultimate rotation ($\theta_{C,y}{}^*$; $\theta_{C,u}{}^*$), the initial stiffness (K_{iniC}), and the maximum bending moment ($M_{C,max}$) are presented and compared for all monotonically tested specimens, for the failed connection. The initial stiffness was determined by a linear fit of moment-rotation values data between 0.25 and 0.9 of the maximum moment. Yield rotation was determined as the point on the initial stiffness line corresponding to maximum moment. Ultimate rotation was defined as the one corresponding to a 10% drop of moment capacity relative to the maximum moment.

Obviously, the specimens with unbolted flanges that failed prematurely by web buckling due to stress concentration around the outer bolt rows, would be the weakest part of portal frames. Consequently, this joint typology is not recommended to be used in practice.

Table 2. Monotonic results: parameters of connection moment-rotation curves.

Specimen	K_{iniC} kNm/rad	$\theta_{C,y}{}^*$ rad	$\theta_{C,u}{}^*$ rad	μ	M_{Cmax} kNm
RSG-M	4891.3	0.021	0.034	1.6	77.1
RIS-FB-M	6011.1	0.017	0.025	1.4	108.0
RIP-M	5806.8	0.018	0.028	1.6	74.3
RIP-M2	6541.2	0.012	0.013	1.1	72.9
KSG-M	6031.6	0.009	0.023	2.5	53.3
KIS-M	4115.0	0.020	0.033	1.6	78.4
KIS-FB-M	6432.3	0.016	0.029	1.8	102.9
KIP-M	7863.9	0.010	0.019	2.0	90.0
KIP-FB-M	6956.5	0.015	0.025	1.6	116.7

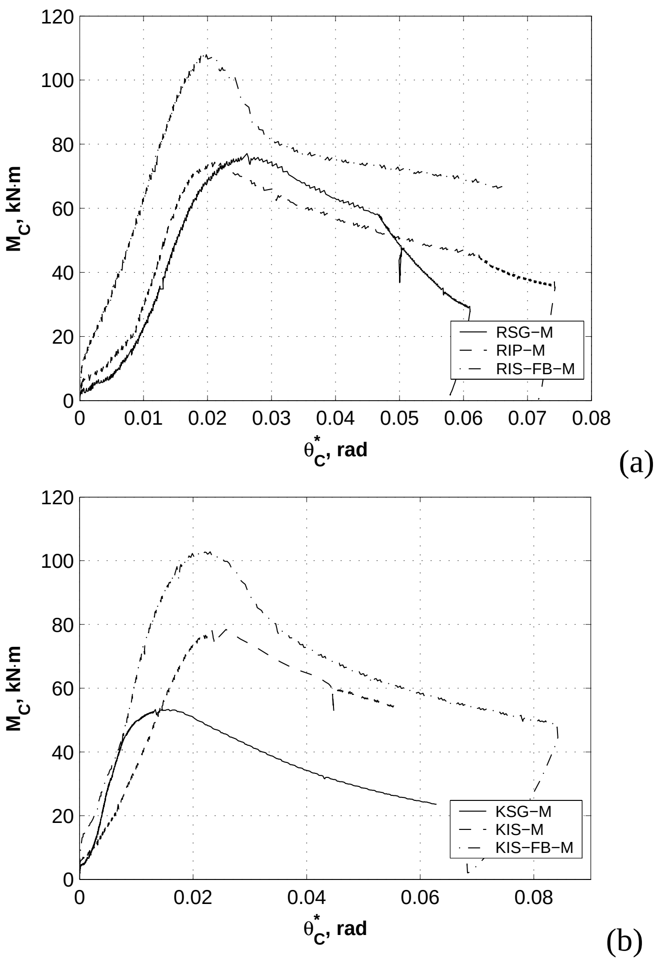


Figure 7. Comparative results from monotonic tests for ridge (a) and knee (b) joints.

The ductility $\mu = \theta_u / \theta_y$ is 1.8 for knee joint KIP-FB-M, and 1.5 for apex joint – RIS-FB-M. The reduction of the maximum moment ($M_{C,max}$) and of ultimate rotation ($\theta_{C,u}{}^*$) in the case of RIS-FB-M specimen is due to the effect of axial compression, which is significant in this case.

2.4 *Cyclic tests*

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 crack gradually opened in the flange and web, causing an important decrease of the load bearing capacity in each consecutive cycle.

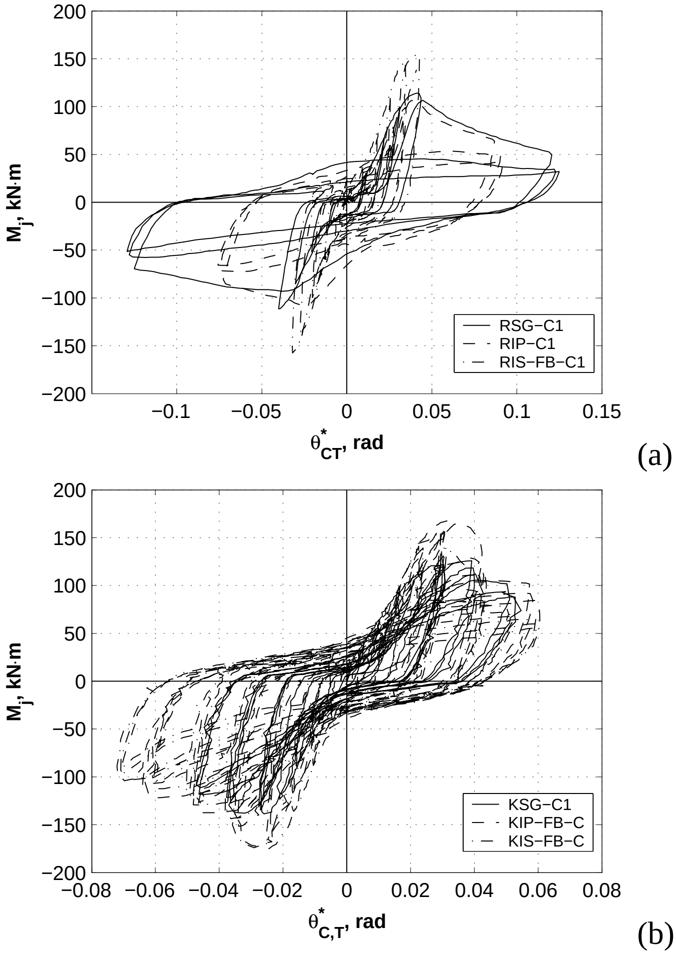


Figure 8. Comparative results from cyclic tests.

Table 3. Cyclic results: parameters of connection moment-rotation curves.

Specimen	K_{iniC} kNm/rad	$\theta_{C,y}^{*}$ rad	$\theta_{C,u}^{*}$ rad	μ	M_{Cmax} kNm
RSG-C1	5060.0	0.017	0.028	1.7	78.8
	-5400.3	-0.019	*	*	-76.8
RSG-C2	4502.9	0.018	0.028	1.6	76.0
	-2792.5	-0.029	-0.036	1.2	-78.1
RIS-FB-C1	*	*	*	*	106.9
	*	*	*	*	-108.6
RIS-FB-C2	*	*	*	*	100.2
	*	*	*	*	-111.5
RIP-C1	6642.1	0.014	*	*	73.8
	-6585.1	-0.013	*	*	-74.7
KSG-C1	5395.5	0.013	0.022	1.7	82.5
	-6672.4	-0.015	-0.028	1.9	-90.9
KSG-C2	5067.6	0.014	0.022	1.5	84.5
	-4684.1	-0.014	-0.017	1.2	-76.8
KIS-FB-C	6914.7	0.014	0.021	1.5	102.3
	-9201.5	-0.012	-0.023	2.0	-114.4
KIP-FB-C	10051.8	0.012	0.026	2.1	102.2
	-8193.5	-0.011	-0.021	1.9	-105.1

* results not available

The hysteretic M-θ curves show a stable behaviour up to the yield limit ($\theta_{C,y}^{*}$) with a sudden decrease of the load bearing capacity afterwards (Figure 8). Therefore the low ductility of the specimens must be underlined again. Further, the cycles show the effect of slippage in the joint (i.e. pinching) and strength degradation in repeated cycles. The strength degradation is stronger in the first repetition, while in the consequent cycles the behaviour is more stable. Based on the unstabilised envelope of the cyclic curves the strength, capacity and ductility characteristics of the connections have been determined, and are reported in Table 3. Again, joints without flange bolts were weaker.

3 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 is investigated in the present paper.

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, present study investigates only joints with both web and flange bolts (RIS-FB-M, KIS-FB-M, and KIP-FB-M). Qualitative FEM simulation (see Figure 9) 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 10), 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. This assumption significantly differ in comparison with the behaviour models considered in the papers of the list of reference, which, all, consider the centroid of the bolt group as rotation centre.

Centre of compression of the connection was considered at the exterior flange of the cold-formed member (see Figure 10). 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 (similarly with Lim and Nethercot 2004)

– 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.

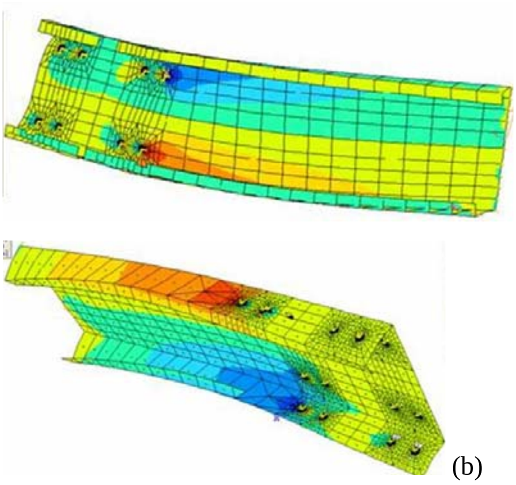

(a)
(b)

Figure 9. Stress concentration in the case of specimens with web bolts only (a), and both web and flange bolts (b).

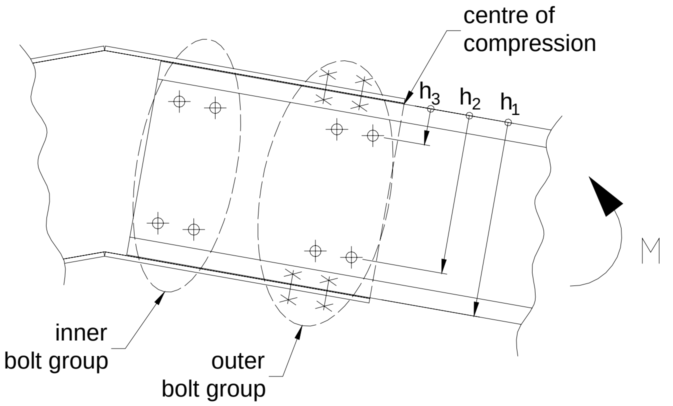


Figure 10. Bolt groups considered in analysis.

Only three components were considered to contribute to stiffness of the connection: bolts in shear (denoted $k_{v,f}$ for flange bolts and $k_{v,w}$ for web bolts), bolts in bearing on cold-formed member (denoted $k_{b,cff}$ for flange bolts and $k_{b,cfw}$ for web bolts), and bolts in bearing on the bracket (denoted $k_{b,bf}$ for flange bolts and $k_{b,bw}$ for web bolts), see Figure 11a. Formulas for determination of stiffness coefficients are available in EN1993-1-8 (2003). For each of the bolt rows r, an effective stiffness coefficient $k_{eff,r}$ is determined, by combining the individual stiffness coefficients using the following relationship (EN1993-1-8, 2003, see Figure 11b):

$$k_{eff,r} = \frac{1}{\sum_i \frac{1}{k_{i,r}}} \tag{1}$$

The effective stiffness coefficients of the bolt rows in "tension" zone are replaced by an equivalent spring k_{eq} (EN1993-1-8, 2003, see Figure 11c):

$$k_{eq} = \frac{\sum_r k_{eff,r} h_r}{z_{eq}} \tag{2}$$

where h_r is the distance between bolt row r and the centre of compression; z_{eq} is determined using equation (3).

$$z_{eq} = \frac{\sum_r k_{eff,r} h_r^{\,2}}{\sum_r k_{eff,r} h_r} \tag{3}$$

Finally, the initial connection stiffness is determined as (see Figure 11d):

$$S_{j,ini} = \frac{E z_{eq}^2}{\sum_i 1/k_i} \tag{4}$$

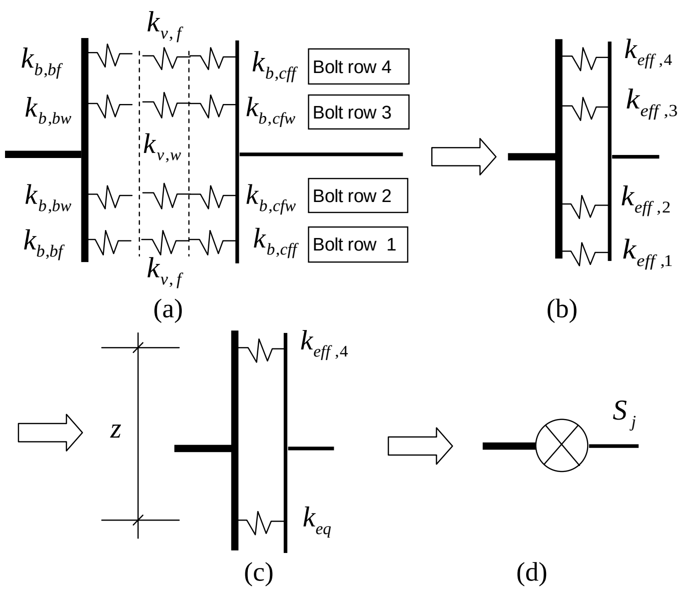


Figure 11. Main steps in the assembly of components for determination of connection stiffness.

Moment resistance of the bolted connection was determined using a two-step procedure. In the first step, only components related to bolt resistance were included in order to determine the moment resistance of the bolted connection $M^b_{C,Rd}$. In a second step, connection moment resistance was obtained as the minimum of moment resistance of the bolted connection $M^b_{C,Rd}$ and one of the connected cold-formed member $M_{beam,Rd}$:

$$M_{C,Rd} = \min\left(M^{b}_{C,Rd}, M_{beam,Rd}\right) \tag{5}$$

Moment resistance of the bolted connection was determined as (EN1993-1-8, 2003):

$$M^{b}_{C,Rd} = \sum_{r} F_{tr,Rd}\, h_r \tag{6}$$

where $F_{tr,Rd}$ is the effective tension resistance of bolt row r (minimum value of components related to bolt row r); h_r is the distance between bolt row r and the centre of compression.

Moment resistance of the cold-formed member $M_{beam,Rd}$ was determined using measured geometrical and mechanical characteristics, using effective cross-section modulus.

It was considered appropriate to use a linear distribution of forces on bolts in the case of a connection to light-gauge members. Therefore, the effective tension resistance of bolt rows was limited according to the following relationship:

$$F_{tr,Rd} \le F_{t1,Rd} \frac{h_r}{h_1} \tag{7}$$

where $F_{t1,Rd}$ is the effective tension resistance of bolt row *1* (farthest from the centre of compression); h_1 is the distance between bolt row *1* and the centre of compression.

Though the resistance of the cold-formed member is taken into account in the final moment resistance of the connection, the approach adopted for determination of connection moment resistance allows to easily determine if the connection is full-strength or partial strength.

Table 4 and Table 5 present resistance and stiffness of bolt rows. The weakest component of flange bolts is bearing on cold formed member, while in the case of web bolts it is bearing on bracket (see Table 4). The difference is due to the fact that bolts are in simple shear on flanges and in double shear on web, as well as due to different number of bolts on flanges (4 bolts per row) and web (2 bolts per row). The main contribution to the flexibility of the connection is bearing on the cold-formed member, as well as bearing on bracket in the case of web bolts (see Table 5).

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 6 and Figure 12. 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 4. Resistance of connection components.

Bolt row	Component Bolts in shear, kN	Bolts in bearing on the cold-formed member, kN	Bolts in bearing on the bracket, kN	Bolt-row resistance $F_{tr,Rd}$, kN
1	361.4	290.6	527.0	290.6
2	361.4	290.6	288.0	288.0
3	361.4	290.6	288.0	288.0
4	361.4	290.6	527.0	290.6

Table 5. Stiffness of connection components.

Bolt row	Component Bolts in shear, mm	Bolts in bearing on the cold-formed member, mm	Bolts in bearing on the bracket, mm	Bolt-row effective stiffness $k_{eff,r}$, mm
1	2.286	0.7785	1.3886	0.4095
2	2.286	0.7785	0.7714	0.3313
3	2.286	0.7785	0.7714	0.3313
4	2.286	0.7785	1.3886	0.4095

Table 6. 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

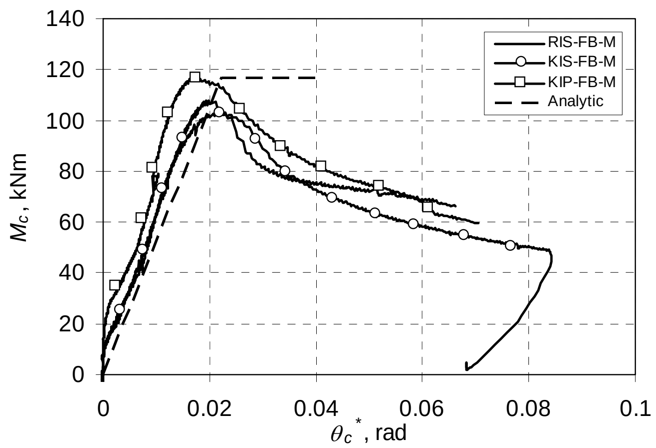


Figure 12. Experimental vs. analytical moment-rotation curves.

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.

4 CONCLUSIONS

The classical calculation model for the connection, assuming its centre of rotation to be located in the centroid of the bolt group and a linear distribution of the forces on each bolt is not correct. The force distribution is unequal due to the flexibility of the connected member. In fact, the force is an order of magnitude bigger in the outer bolt rows compared to most inner one. A connection with bolts only on the web causes concentrated forces in the web of the connected member and leads to premature web buckling, reducing the joint moment capacity. These types of joints are always partial strength. If the load bearing capacity of the connected beam is to be matched by the connection strength, bolts on the flanges become necessary.

The ductility of the connection is limited both under monotonic and cyclic loads and the design, including the design for earthquake loads, should take into account only the conventional elastic capacity. Because there is no significant post-elastic strength, there are no significant differences in ductility and capacity of cyclically tested specimens compared with the monotonic ones. However, if the joints are loaded under the limit of their maximum capacity, even cyclically, their strength is not too much affected.

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), connection 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. Since the cyclic loading does not cause too much degradation in strength and stiffness of the connection, compared with monotonic loading, the results obtained by component method, in this particular case, can be safely used for seismic design of these joints, provided a γ_{M2}=1.25 partial safety factor is considered. However, the results and conclusions of this paper have to be limited to the joint typologies and range of section dimensions used in the experimental program. Slender sections and more compact bolt-groups can lead to different results. Consequently, further studies are necessary to generalise these conclusions.

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