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Cold-formed Steel Portal Frame Joints a Review

A. M. Wrzesein

J. B. P. Lim

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Cold-formed steel portal frame joints: a review

A.M. Wrzesien[1], J.B.P. Lim[2]

Abstract

This paper reviews research published on cold-formed steel portal joints, beginning with the laboratory tests of Baignent and Hancock (1982) and ending with those of Rhodes and Burns (2006). The moment-capacity of the cold-formed steel channel-sections being connected in the portal framing systems ranges from 3.6 kNm to 128.5 kNm, with each type of framing system employing a different joint detail. While in accordance with the Eurocode 3 joint classification system, the joints arrangements reported would be classified as semi-rigid, for the purpose of design the majority of the joints would be sufficiently rigid for the frames to be designed safely to the ultimate limit state using a rigid-joint assumption, with the joints capable of sustaining almost the full-moment capacity of the cold-formed steel channel-sections being connected. However, in order for the assumption of rigid joints to be valid, the number of bolts or specialist components required may, in some countries, result in the joints being uneconomical to fabricate. It is seen that of all the joints reviewed, the joint arrangement tested by Rhodes and Burns is distinctive as rigid-joints are formed inexpensively through the use of knee braces. This, however, is at the expense of losing clear height to the eaves. Using UK design practice, a parametric study of sixteen frames, having spans ranging from 8 m to 14 m, is described that compares the economy of rigid-jointed frames against that of knee-braced frames. It is shown that use of a knee-braced frame results in a 10% increase in load carrying capacity, and a 36% reduction in horizontal deflections.

[1] PhD Student, Department of Civil Engineering, University of Strathclyde, Glasgow, UK

[2] Lecturer, Department of Civil Engineering, University of Strathclyde, Glasgow, UK

Introduction

In the UK, for portal frames having spans of up to 14 m (or more), the use of cold-formed steel sections for the column and rafter members can be a viable alternative to conventional hot-rolled steel sections. Some of the advantages of using cold-formed steel include a higher strength-to-weight ratio, reduced erection costs, and reduced acquisition and transportation costs (since both the primary members as well as the secondary members can be purchased from the same supplier).

However, in order for a valid comparison to be made between both types of framing system, the cost of fabrication of the joints must be taken into account. In the case of a typical hot-rolled steel portal frame, Tomà (1993) estimated that as much as 40% of the total frame cost is due to the fabrication of the joints. While it can be expected that this percentage will be lower for a typical cold-formed steel portal frame, it cannot be expected to be significantly lower.

Furthermore, with conventional hot-rolled steel portal frame joints, which are designed plastically, one of the key requirements is that the joints are designed to function as rigid. On the other hand, with cold-formed steel portal frames, which are designed elastically, the requirement of rigid joints that are expensive to fabricate may not be as important.

In this paper, research published in the literature on cold-formed steel portal joints is reviewed, beginning with the laboratory tests of Baignent and Hancock (1982) and ending with those of Rhodes and Burns (2006). The moment capacity of the joints in the review ranges from 3.6 kNm to 128.54 kNm.

The majority of the joints described attempt to form a rigid joint through the use of haunch brackets and bolts. Rhodes and Burns (2006), however, describe a haunch connection formed through knee brace member. The effect of having a knee brace is investigated further by the authors.

A parametric study is undertaken, comprising sixteen frames having spans ranging from 8 m to 14 m, comparing the economy of rigid-jointed frames to that of knee-braced frames, taking into account both ultimate and serviceability limit state design.

Literature review

Over the past thirty years, different researchers have undertake tests on different arrangements for the eaves and apex joints of cold-formed steel portal framing systems. Table 1 summarises the joints reported in the literature by each researcher, including the moment-capacity of the cold-formed steel sections being connected, and the number of components and fasteners required to form the joint.

The earliest tests reported in the literature on cold-formed steel portal frame joints are those by Baigent and Hancock (1982). Details of this joint are given in Fig. 1. As can be seen, the joints were formed through the web of the channel-sections used for column and rafter members. The moment-capacity of the channel-sections being connected was 9.19 kNm. The thickness of the channel-sections was 1.86 mm, while the thickness of the plate used to connect the joints was 12 mm. Due to high-tensile grip bolts, the joints could be considered as being rigid.

The next set of tests reported were those by Kirk (1986) on the Swagebeam portal framing system. These tests were undertaken by Professor Bryan at Salford University. Figure 2 shows details of the joints. As can be seen, back-to-back channel sections were used for the column and rafter members. The joints were formed through back-to-back brackets bolted between the webs of the channel-sections. The moment-capacity of the back-to-back channel-sections was 32 kNm; the thickness of the channel-sections was 2.4 mm and the thickness of each bracket was 3.0 mm. The primary innovation was that the joints could formed through the swages rolled in the brackets which connected with matching swages in the webs of the channel-sections.

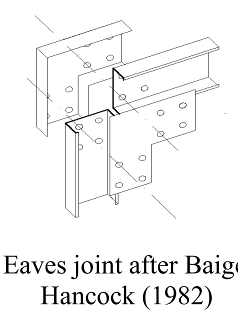


Fig.1 Eaves joint after Baigent and Hancock (1982)

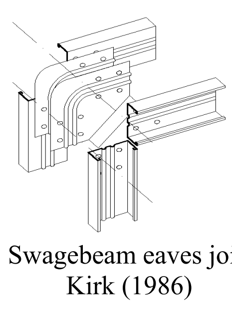


Fig.2 Swagebeam eaves joint after Kirk (1986)

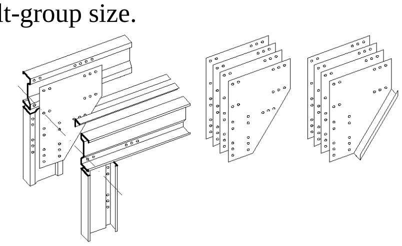
Table 1. Joints reported in the literature

Author	Sections				Bracket				Fasteners	
		d x b x t (mm)	σ_y (N/mm^2)	M_c (kNm)	l_e (mm)	t_b (mm)	σ_{yb} (N/mm^2)	No. of brackets	Description	No. of fixings/ joint
Baigent (1982)		153x79x1.86	325.8	9.2	260	12	-	1	M19mm H.T. bolts	8
Kirk (1986)		220x65x2.4	280*	32.0	620	3	280	2	M16 G.8.8	8
Mäkeläinen (1996)		300x75**x3.0	350*	77.0	-	12	355	1	M16 G.8.8	20
Chung (1998)		150x64x1.6	450*	17.9	460^{h} 460^{hs}	6 2.5	343 475	1 2	M16 G.8.8	8
Lim (2002)		340x90x3	280*	82.8	746^{hs}	3	209	2	M16 G.8.8	16
Mills (2004)		200x76x1.5	450*	10.8	End plate joint Mitred joint Sections screwed back-to-back				Bolts Bolts Screws	2 2 12
Dubina (2004)		350x100x3	452	117.8	940	10	235	1	M20 G.6.6	32
Dundu (2005)		300x75x3	468.9	51.6	Sections bolted back-to-back				M20 G.8.8 M20 G.8.8	4 8
Kwon (2006)		PRY 150x40x0.8	570*	3.6	261.6	2.3	240	1	Screws ø 4.8mm	16
Rhodes (2006)		342x97x2.5 rafter 402x97x3.2 column	343 352	76.7 128.5	Connection angles C 202x69x2 knee brace		275* 350*	2 2	M16 G.8.8	32

* design yield strength, ** dimension not reported but assumed following standard size, h haunch, hs haunch with stiffener

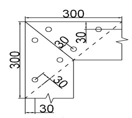
Mäkeläinen (1996) described tests on a portal framing system constructed from back-to-back sigma sections connected though the web via brackets. To provide additional stiffness to the frame, a tie bar (double angle 50 x 50 x 2.5 mm) was bolted to both eaves brackets (Fig. 3a). The depth of the sections used for the tests were 250 mm, 300 mm, and 400 mm; thicknesses of 2.5 mm and 3.0 mm were considered. Figure 3 shows details of the joint brackets. These included a single plate of thicknesses of 8 mm, 10 mm and 12 mm (see Fig. 3a), four cold-formed plates thickness of 2.5mm each (see Fig. 3b), and four cold-formed plates with two outer plates outwardly lipped (see Fig. 3c). Although the moment capacities of the sections were not provided, similar compound member made from back-to-back standard sigma section 300 mm deep, 75 mm wide, and 3.0 mm were calculated to have a moment capacity of 77 kNm.

Chung (1998) and Lim and Nethercot (2002) independently reported tests on an arrangement where the joint was formed through back-to-back brackets bolted between the webs of the channel-sections being connected. In the tests described by Chung, the moment-capacity of the sections was 17.88 kNm, while that of Lim and Nethercot was 82.8 kNm. Figures 4a to 4d shows the different shape of the brackets studied by Chung. In the case of the joint details shown in Fig. 4c and 4d, the joints ware tested twice. In the first stage, the joints were formed through a hot-rolled steel single gusset plates of thickness 6mm. In the second phase, the joints were formed through two back-to-back cold-formed steel brackets, each 2.5mm thick and with lip stiffeners along the catheti and hypotenuse of the bracket respectively (Fig. 4c and 4d). Unlike Chung, the joints tested by Lim and Nethercot isolated failure of the brackets from that of the channel-sections. Having ensured that the brackets themselves would not fail, research was focused on the strength and stiffness of the channel-sections, as influenced by the bolt-group size.

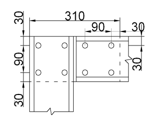


a) single layer b) four layers c) four layers with lip stiffeners

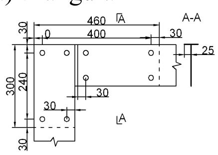
Fig.3 Eaves joint having different brackets configuration after Mäkeläinen and Kankaanpää (1996)



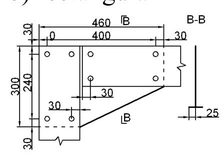
a) triangular



b) rectangular



c) L-shape with stiffener



d) haunched with stiffener

Fig.4 Eaves joint brackets after Chung and Lau (1998)

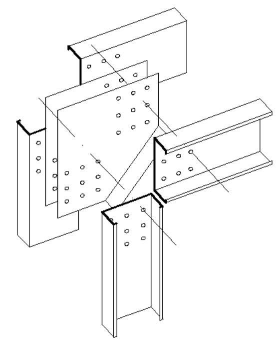
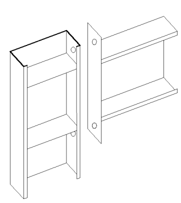
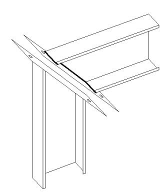


Fig.5 Eaves joint after Lim and Nethercot (2002)

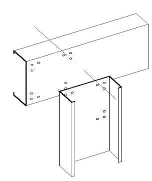
Mills and LaBoube (2004) conducted experimental studies on joints currently used in Australia for cold-formed steel portal frame sheds. The joints were constructed from single channel-sections with a moment capacity of 10.84 kNm. Popular joints included an end plate connection bolted to the column and welded to the rafter (Fig. 6a), and a mitred joint (Fig. 6b). Self-drilling screws were used as an alternative to conventional bolting. A similar arrangement for the apex joint was also studied, in which double lipped channel-sections were used as the gusset plate and screwed back-to-back to the rafters.



a) bolted end plate joint



b) mitred joint

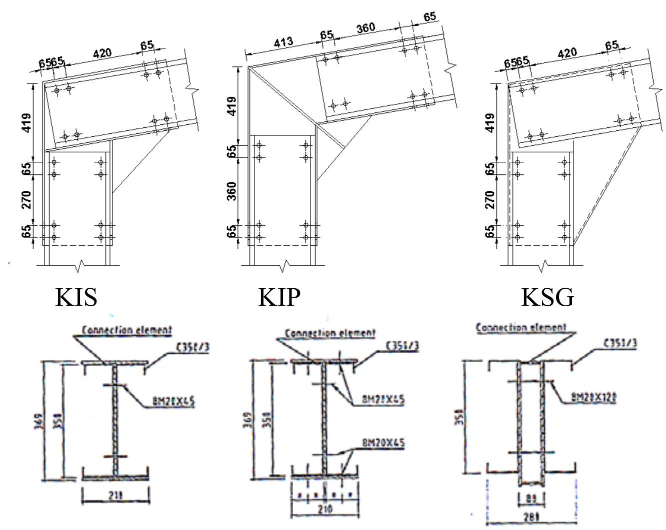


c) self-drilling screw joint

Fig.6 Eaves joints after Mills and LaBoube (2004)

Dubina *et al* (2004) described the three different type of joints. As can be seen in Figure 7a and 7c, the channel-sections were bolted only through the web of welded I-section brackets (KIS, KIP) and spaced gussets bracket (KSG). In second variant, bolts were located both on the web and on the flange (Fig. 7b) of I-section bracket (KIS) and I-section bracket with plate bisector (KIP). The moment-capacity of the channel-sections being connected was 117.8 kNm.

It should be noted that unlike Chung (1998), the joint was formed through hot-rolled steel sections instead of back-to-back brackets. However, as the strength of the hot-rolled steel sections is much greater than that of the channel-sections, the behaviour of the joints is dominated by that of the channel-sections.



a) bolts on the web (KIS, KIP) b) bolts on the web and on the flange (KIS, KIP) c) bolts on the web (KSG)

Fig.7 Eaves joints after Dubina *at el* (2004), p.382: KIS- welded I section, KIP-welded I section with plate bisector, KSG-spaced gussets,

Dundu and Kemp (2006) conducted research on single channels connected back-to-back (Fig. 8). Such an arrangement is similar to that of Mills and LaBoube (2004). Dundu and Kemp were concerned with the development of a plastic hinge, and so concentrated on the ductility of the joints. A novel method for providing lateral restraint was introduced through an angle connection between the web of the rafter and purlin. It was demonstrated that this arrangement eliminated the lateral-torsional buckling failure mode, since both the top and bottom flanges were effectively restrained, reducing torsional instability.

Kwon *et al* (2006) reported research on applications of closed sections produced by a combination of cold-rolling and clinching techniques. The sections used for the tests were 150 mm deep, 40 mm wide and 0.8 mm thick. The local buckling moment calculated from the gross section modulus was 3.55 kNm. Connection brackets for the eaves and apex joints were constructed from mild steel plates 2.3 mm through combination of folding and welding, with four different connection types. The bracket of Connection Types 1 and 2 were produced by cutting the bottom flange of the C-shape bracket and welding lipped plate to build the haunch stiffener, with and without lip on the flange respectively. A similar shape of the bracket to Connection Type 2 is currently under investigation by authors. However the bracket was made by brake pressing cold-formed steel of thickness 3 mm. Figure 9a and 9b shows the general joint arrangement of Connection Type 3, with the lip on the flange. In Connection Type 4, the bracket of the same shape lip on the flange was not provided.

Rhodes and Burns (2006) conducted extensive component tests on the eaves joint of a cold-formed steel portal framing system. Figure 10 shows details of the joint. The columns and rafters were formed from back-to-back channel-sections having a moment capacity of 128.54 kNm and 76.68 kNm, respectively. As can be seen, the proposed eaves joint used knee-braces formed through back-to-back channel-sections bolted to the flanges of the column and rafter through a welded bracket. At the eaves, the joint was formed through a pair of angles sections; to avoid the failure of the flange under concentrated load a pair of angle stiffeners were introduced. As a means of comparison, single, flat, cold-formed 8mm thick gusset plate joints at the eaves and knee brace ends were also tested. This time the connection was formed by bolting though the web of the sections. Although the results were satisfactory, this joint was not investigated further as it would involved complicated erection issues when the section is split in order to place the gusset plates between them.

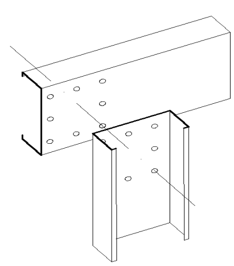
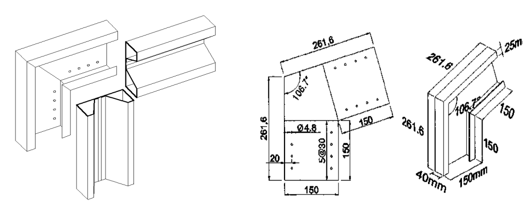


Fig. 8 Bolted joints after Dundu and Kemp (2006)



a) joint arrangement

b) self drilling screws configuration

Fig. 9 Connection type 3 after Kwon *et al* (2006)

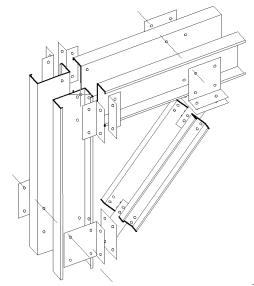


Fig. 10 Eaves joint after Rhodes and Burns (2006)

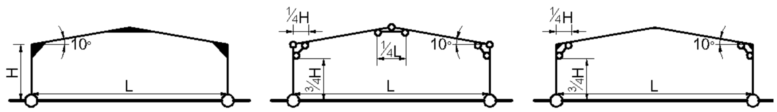
Parametric study

The joint arrangement tested by Rhodes and Burns removed the necessity of constructing expensive rigid joints by introducing knee brace. This arrangement allowed the joint to possess as much strength as the one described by Dubina and also significantly improved the overall sway of the frame.

In this Section, a parametric study of sixteen frames is described, comparing a rigid-jointed frame to that of a knee-braced frame. The parametric study considers frames having spans between 8 m to 14 m and height to eaves between 3 m to 6 m. Table 3 shows the spans and heights of the analysed frames. The pitch of all frames is 10^{o}. The distance between adjacent bays is 4 m. The column base of all frames is pinned.

For each frame geometry considered, three types of joints are analysed: Joint A, Joint B, and Joint C. Figure 11 shows details of the three types of joint. As can be seen, in the case of Joint A, the eaves and apex joints are rigid. In the case of Joint B, the eaves and apex joints are pinned. Instead, the eaves joint is formed through a knee brace pinned to the column and rafter members, respectively, at a distance of H/4 from the top of the column, and a similar distance along the rafter. Similarly, the apex joint is formed through a knee brace pinned to the rafter members. The length of the apex knee-brace is a quarter of the span. Joint C is identical to Joint B except that the eaves and apex joints are rigid.

Table 3 also shows, for each frame geometry considered, the section sizes used in the frame design. A six digit designation is used to denote the section size of the channel-sections, which are used back-to-back. For the frame having a span of 8 m and height to eaves of 3 m, the back-to-back channel-sections used for the columns and rafter have a depth of 200 mm and a thickness or 2.5 mm.



Joint A Joint B Joint C

Fig. 11 Different joints arrangements

The unfactored vertical loads applied to the frames are as follows:

Dead load (DL) $= 0.2$ kN/m^2
Live load (LL) $= 0.6$ kN/m^2

The unfactored wind loads are calculated in accordance to BS 6399-2 for a site located in the country, assuming a wind speed of 24 m/s, and 10 km from the sea. For the frame having a span of 8 m and height to eaves of 3 m, this corresponds to a value of q_s of 0.77 kN/m^2. As q_s depends on the height and span of the frame, each frame is designed using a different value of q_s. This value of q_s is also shown in Table 3.

The frames are designed to the Ultimate Limit State (ULS) using the following load combinations:

LC1: 1.4 DL + 1.6 LL + NHL
LC2: 1.2 DL + 1.2 LL + (1.2 x 0.85) WT_{ULS}
LC3: 1.4 DL + (1.4 x 0.85) WT_{ULS}
LC4: 1.0 DL + (1.4 x 0.85) WT_{ULS}

The WT_{ULS} loads are calculated in accordance to BS 6399-2, assuming an internal pressure coefficient, C_{pi}, of -0.3 and pressure on the windward rafter.

For the ULS design, the frame is analysed using first-order frame analysis and designed in accordance with BS 5950-5 using a combined bending and axial force check. Out-of-plane member instability is assumed to be prevented by sufficient purlins and side rails. Second-order effects are ignored.

The frames are also designed to the Serviceability Limit State (SLS) using the following load combinations:

LC5: 1.0 LL
LC6: 1.0 WT_{SLS}

The WT_{SLS} loads are calculated using an internal pressure coefficient, C_{pi}, of 0 and pressure on the windward rafter.

For each frame geometry, the frame is designed three times, one for each type of Joint. The unity factors for ULS design are compared for each type of Joint and expressed as a percentage difference. In the case of vertical SLS design, the vertical deflections of the apex (from LC5) are compared for each type of Joint and again expressed as a percentage difference. Similarly, in the case of horizontal SLS design, the horizontal deflections at the eaves (from LC6) are compared for each type of Joint and expressed as a percentage difference. The results for each frame geometry are shown in Table 3.

From Table 3 it can be seen that the effect of changing from Joint A (rigid-jointed) to Joint B (knee-brace) is an average of 10% increase in load carrying capacity. In general, the benefit of having the knee-brace increases as the height decreases. With respect to vertical deflections, there is an average of 30% reduction in deflection for frames having a height of 3 m, as a result of changing from Joint A to Joint B. However, as the frame height increases, this reduction decreases and for some frames Joint A has smaller vertical deflections than Joint B. Vertical deflections, however, rarely control design.

Of more importance is the horizontal deflections. There is an average of 36% reduction in deflections as a result of changing from Joint A to Joint B.

For the case of comparing Joint A (rigid-jointed) to Joint C (rigid-jointed with knee-brace), the average benefit of introducing the knee-brace is 14%, 37% and 38% for ULS design, vertical deflections, and horizontal deflections, respectively. This compares with 10%, 5% and 36%, respectively for the case of comparing Joint A (rigid-jointed) to Joint B (knee-brace). Since vertical deflections rarely control design, taking into account the potential semi-rigidity of the eaves and apex joint in frames having knee braces would appear to offer little benefit.

The same conclusion can be drawn from the comparison of Joint B (knee-brace) to Joint C (rigid-jointed with knee-brace).

Table 3. Portal frame comparison study

L x H	q_s	Column section	Rafter section	Eaves/apex brace	Percentage load increase Joint A - Joint B			Percentage load increase Joint B - Joint C			Percentage load increase Joint A - Joint C		
(m x m)	(kN/m^2)				$ULS_{c,r}$	SLS_v	SLS_h	$ULS_{c,r}$	SLS_v	SLS_h	$ULS_{c,r}$	SLS_v	SLS_h
8 x 3	0.77	C20025	C20025	C15014	13	26	39	0	17	0	13	47	39
8 x 4	0.85	C30025	C30025	C15014	8	-16	40	9	58	2	18	33	43
8 x 5	0.89	C30030	C30030	C15016	8	-17	52	2	64	2	10	36	55
8 x 6	0.93	C40025	C40025	C15018	0	-29	55	3	75	0	3	25	55
10 x 3	0.78	C25020	C25020	C15018	13	38	32	-1	4	0	12	43	32
10 x 4	0.86	C30025	C30025	C15018	10	6	37	4	32	0	14	40	37
10 x 5	0.9	C40025	C40025	C15018	9	-17	37	9	64	2	19	36	40
10 x 6	0.94	C40030	C40030	C15020	6	-14	47	4	62	2	10	38	50
12 x 3	0.79	C25030	C25025	C20016	17	38	23	-1	0	0	16	38	23
12 x 4	0.87	C30025	C30025	C20016	11	19	32	3	17	2	14	39	34
12 x 5	0.91	C40025	C40025	C20016	9	-5	34	7	42	2	17	35	37
12 x 6	0.95	C40030	C40030	C20018	12	-3	45	4	48	2	16	43	48
14 x 3	0.81	C30025	C30025	C20018	13	20	8	1	6	9	15	27	17
14 x 4	0.87	C30030	C30030	C20018	11	26	28	1	9	0	13	38	28
14 x 5	0.91	C40030	C40030	C20018	9	2	31	7	32	0	17	35	31
14 x 6	0.95	C40030	C40030	C20020	13	12	43	2	28	0	15	44	43
Average					10	5	36	3	35	1	14	37	38

Conclusions

A number of different arrangements for the eaves and apex joint of cold-formed steel portal frames have been reviewed. Whilst cold-formed steel joints that function close to rigid can be fabricated, this is often at great expense. On the other hand, while joints that function as semi-rigid can be cheaper to be fabricated, but will result in larger frame deflections.

A knee-braced joint arrangement, tested by Rhodes and Burns, has been shown to be distinctive from other joint arrangements described in the literature, as rigid-joints are formed inexpensively through the use of knee braces.

A parametric study comparing the design of portal frames in accordance with the British Standards, has led to conclusions pertaining to the most efficient joints for different geometries of the frame with and without knee braces. It has been seen that use of a knee-braced frame results in a 10% increase in load carrying capacity, and a 36% reduction in horizontal deflections. This, however, is at the expense of losing clear height to the eaves, which can be problematic, when large openings in the gable are required.

Appendix. – References

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Appendix. – Notation

b	width of the flange
d	depth of the section
H	height to the eaves of portal frame
L	span of the portal frame
l_e	length of the eaves bracket
M_c	section moment capacity reported in the literature
q_s	dynamic wind pressure calculated to BS 6399-2
SLS_h	comparison factor according to serviceability limited state criterion for horizontal deflection of the frame
SLS_v	comparison factor according to serviceability limited state criterion for vertical deflection of the frame
t	thickness of the section
t_b	thickness of the bracket
$ULS_{c,r}$	comparison factor according to ultimate limited state criterion for column or rafter design
σ_y	yield strength of the steel used for members
σ_{yb}	yield strength of the steel used for brackets