International Specialty Conference on Cold-Formed Steel Structures

(2014) - 22nd International Specialty Conference on Cold-Formed Steel Structures

Nov 6th, 12:00 AM - 12:00 AM

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Twenty-Second International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri, USA, November 5 & 6, 2014

Numerical Investigation of Cold-Formed Steel Top-Hat Purlins

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Abstract

This paper considers the use of cold-formed steel top-hat sections for purlins as an alternative to conventional zed-sections. The use of such top-hat sections may be viable for use in cold-formed steel portal framing systems, where both the frame spacing and purlin span may be smaller than in conventional hot-rolled steel portal frames. Furthermore, such sections are torsionally stiffer than zed-sections, and so have a greater resistance to lateral-torsional buckling. They also do not require the installation of anti-sag rods. The paper describes non-linear elasto plastic finite element analyses conducted on top-hat sections. The results of twenty-seven tests on four different top-hat sections are presented. Good agreement between experimental and finite element results is shown. The finite element model is then used for a parametric study to investigate the effect of different thicknesses and steel grades. Design recommendations are provided in the form of charts that can be used to assist designers when deciding which geometry of top-hat section to consider for further development. The use of the finite element method in this way exploits modern computational techniques for an otherwise difficult structural design problem and reduces the need for an expensive and time consuming full laboratory study, whilst maintaining realistic and safe coverage of the important structural design issues.

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Introduction

In the UK, single-storey steel portal frames account for approximately 50% of the constructional steel used each year, and 90% of all single-storey buildings. Such buildings typically use conventional hot-rolled steel sections for the primary column and rafter framing members, which in turn support the secondary cold-formed steel purlin and side rail members; these secondary members, in turn, support the cladding.

For portal frames of modest span (around 12 m), the introduction of higher strength grades of cold-formed steel in the past decade, has led to cold-formed steel sections being used for the both primary members as well as for the secondary members (see Figure 1). Such cold-formed steel portal framing systems are now a viable alternative to conventional hot-rolled steel portal framing systems.

Figure 1: Cold-formed steel portal framing system

However, unlike conventional hot-rolled steel portal frames, where the frame spacing is typically 6 m, there is some scope to vary the frame spacing in the design of cold-formed steel portal frames. This is because cold-formed steel sections are lighter than hot-rolled steel sections, so structural members can be bolted and erected on site by semi-skilled workers, without the need for an onsite crane; consequently, erection costs are much lower than in hot-rolled steel portal frames. A design optimization described by Phan et al. (2013) demonstrated that topology can have a significant effect on minimizing the cost of the primary members per meter square of the building. Furthermore, unlike conventional hot-rolled steel portal frames, where the purlin spacing is typically around 1.8 m, with cold-formed steel portal frames there is often a need to have a smaller spacing in order to provide more restraint to the column and rafter members.

Although purlins are secondary members, they can account for approximately 30% of the total cost of the building. For smaller buildings, having frame spacings (and therefore purlin spans) of around 4 m, and purlin spacings of 1 m, the specification of even the smallest zed-section available can result in an over-design by as much as 30%.

An alternative to the use of zed-sections for purlins is the top-hat section (see Figure 2). Such sections perform better than zed purlins against lateral torsional buckling. Furthermore, they are simple to install on site and, unlike zed purlins, do not require the installation of anti-sag rods or cleats. The authors have recently described and presented experimental and numerical investigation on cold-formed steel top-hat section under bending (Uzzaman et al., 2013).

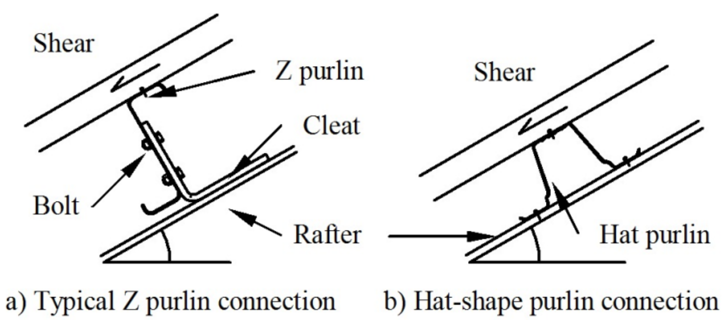
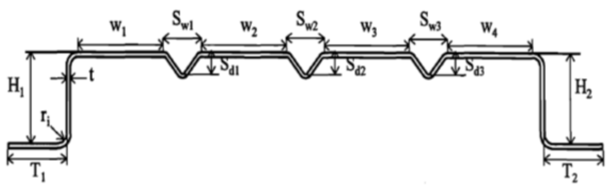


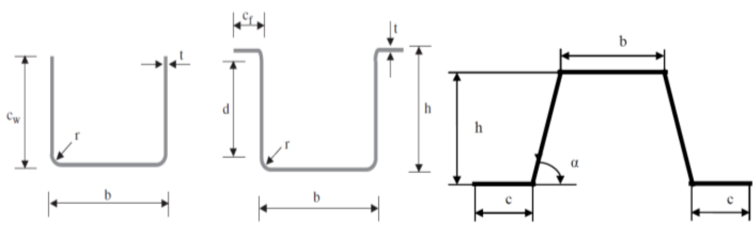
Figure 2: Typical Z purlin and hat-shape purlin connection

The behaviour of hat shaped sections has received limited attention in the literature. Figure 3 (a) shows the hat shaped sections tested by Acharya and

Schuster (1998). Pastor and Roure (2008, 2009) tested hat shaped sections (see Figure 3 (b)), considering the formation of the plastic hinge. A finite element analysis methodology was implemented to simulate the post collapse behaviour. Honfi (2006) considered the design optimization of hat shaped sections (see Figure 3 (c)) by use of a genetic algorithm.



(a) Hat shaped section tested by Acharya and Schuster (1998)



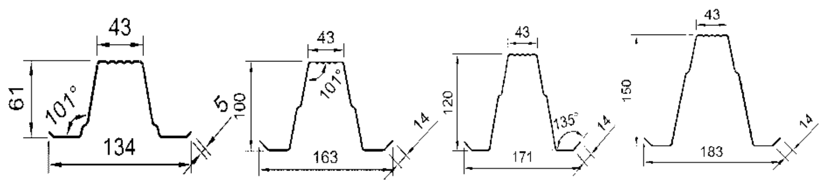
(b) Pastor and Roure (2008, 2009) hat shaped (c) Honfi (2006) hat shaped

Figure 3: Different hat shaped sections found in literature review

In this paper, a parametric study is undertaken. Design recommendations are provided in the form of bar charts that can be used to assist designers when deciding which top-hat section to consider for further development.

Experimental Investigation

Twenty-seven full-scale tests were conducted on the four different geometries of top-hat sections under four point bending, eleven tests in the under uplift and sixteen tests under gravity load. Full details of these full-scale tests can be found in Potter (2010) and Uzzaman et al. (2013). Two loading directions were considered: uplift (representing wind uplift load) and gravity load (representing vertical snow load). The nominal dimensions of the four types of top-hat sections are shown in Figure 4 (a, b, c and d). The nominal thickness of the top-hat sections was 1 mm.



(a) Top-hat 61 (b) Top-hat 100 (c) Top-hat 120 (d) Top-hat 150

Figure 4: Nominal dimensions of four types of top-hat section.

Numerical Investigation

The non-linear elasto-plastic general purpose finite element program ANSYS (2011) was used to simulate the top-hat sections subjected to pure bending. An accurate and reliable non-linear FEM for the top hat sections has been presented by Uzzaman et al. (2013). The details of the FEM are described in Uzzaman et al. (2013). In the finite element model, the measured cross-section dimensions and the material properties obtained from the tests were used. The model was based on the centreline dimensions of the cross-sections. The material non-linearity was incorporated in the finite element model by specifying 'true' values of stresses and strains. The plasticity of the material was determined by a mathematical model, known as the incremental plasticity model. Depending on the size of the section, the finite element mesh sizes ranged was 10×10 mm (length by width). Three elements were used around the inside corner radius that forms the bend. Along the length of the top-hat sections, the number of elements was chosen so that the aspect ratio of the elements was as close to one as possible. Mesh sensitivity analyses were performed to verify the number of elements.

Parametric Study

A parametric study comprising 32 models was conducted on Top-Hat 61 and Top-Hat 100 sections. Four different thicknesses of 1.0 mm, 1.2 mm, 1.4 mm and 1.6 mm are considered. Two different steel grades of 390 MPa and 450 MPa are also considered. It should be noted that Top-Hat 120 and Top-Hat 150 sections were excluded from the parametric study as they were found to be too susceptible to distortional buckling under uplift to be efficient when compared with the zed-sections. On specimens notation "U" represents loading under uplift direction and "G" represents loading under gravity direction. Second notation defines the nominal overall height dimension of the top- hat section in millimetres (61 = 61 mm). "M390" and T1.0 represents the grade of the material (M390= 390 MPa) and thickness of the top-hat section (T1.0= 1mm).

Table 1: Moment capacity obtained from FEA parametric study under both loading directions

Specimen	Thickness (t) mm	Area (A) cm^2	Moment of inertia (I) cm^4	Yield strength (σ_y) N/mm^2	Moment at Failure (M_u^{FEA})	Efficiency ratio (M_u^{FEA}/A) kNm/cm^2
U-61-M390-T1.0	1.0	2.25	11.21	390	1.29	0.57
U-61-M390-T1.2	1.2	2.70	13.45	390	1.60	0.59
U-61- M390-T1.4	1.4	3.15	15.45	390	1.99	0.63
U-61- M390-T1.6	1.6	3.60	17.94	390	2.38	0.66
U-61-M450-T1.0	1.0	2.25	11.21	450	1.39	0.62
U-61-M450-T1.2	1.2	2.70	13.45	450	1.74	0.64
U-61- M450-T1.4	1.4	3.15	15.45	450	2.19	0.70
U-61- M450-T1.6	1.6	3.60	17.94	450	2.64	0.73
U-100-M390-T1.0	1.0	3.06	39.82	390	2.13	0.70
U-100-M390-T1.2	1.2	3.67	47.79	390	3.06	0.83
U-100- M390-T1.4	1.4	4.29	55.75	390	3.88	0.91
U-100- M390-T1.6	1.6	4.90	63.72	390	4.73	0.97
U-100-M450-T1.0	1.0	3.06	39.82	450	2.25	0.73
U-100-M450-T1.2	1.2	3.67	47.79	450	3.28	0.89
U-100- M450-T1.4	1.4	4.29	55.75	450	4.20	0.98
U-100- M450-T1.6	1.6	4.90	63.72	450	5.16	1.05
G-61-M390-T1.0	1.0	2.25	11.21	390	1.42	0.63
G-61-M390-T1.2	1.2	2.70	13.45	390	1.80	0.67
G-61- M390-T1.4	1.4	3.15	15.45	390	2.20	0.70
G-61- M390-T1.6	1.6	3.60	17.94	390	2.68	0.74
G-61-M450-T1.0	1.0	2.25	11.21	450	1.70	0.76
G-61-M450-T1.2	1.2	2.70	13.45	450	2.11	0.78
G-61- M450-T1.4	1.4	3.15	15.45	450	2.58	0.82
G-61- M450-T1.6	1.6	3.60	17.94	450	3.03	0.84
G-100-M390-T1.0	1.0	3.06	39.82	390	3.31	1.08
G-100-M390-T1.2	1.2	3.67	47.79	390	4.10	1.12
G-100- M390-T1.4	1.4	4.29	55.75	390	5.07	1.18
G-100- M390-T1.6	1.6	4.90	63.72	390	6.03	1.23
G-100-M450-T1.0	1.0	3.06	39.82	450	3.75	1.22
G-100-M450-T1.2	1.2	3.67	47.79	450	4.46	1.21
G-100- M450-T1.4	1.4	4.29	55.75	450	5.40	1.26
G-100- M450-T1.6	1.6	4.90	63.72	450	6.35	1.30

Table 1 summarises the section properties and results of the parametric study. An efficiency ratio, defined as the ultimate bending capacity divided by the cross- sectional area (M_u^{FEA} / A) is also shown.

For comparison, the efficiency of the top-hat sections will be compared against those of a typical zed-section. Figure 5 shows the nominal dimensions of the smallest zed-section purlin available by Steadmans (2011, 2012), a UK manufacturer of purlins. The zed-section is available in thicknesses of 1.4 mm, 1.5 mm and 1.6 mm. It is only available in steel grade 390 MPa.

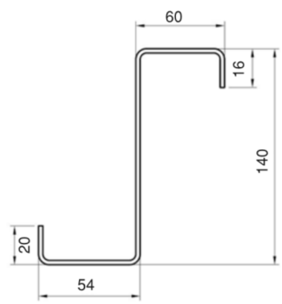


Figure 5: Nominal dimensions of Z140 section

Table 2 shows the same values for Z140 purlin sections. The values shown have been taken from load-span tables provided by SMAN (2011, 2012).

Table 2: Moment capacities of zed purlins (SMAN (2011, 2012))

Specimen	Thickness (t) mm	Area (A) cm^2	Moment of inertia (I) cm^4	Moment at Failure[1] (M_u^{SMAN})	Efficiency ratio (M_u^{SMAN}/A) kNm/cm^2
U-Z140-M390-T1.4	1.4	3.81	116.80	4.57	1.20
U-Z140-M390-T1.5	1.5	4.08	124.70	5.36	1.34
U-Z140-M390-T1.6	1.6	4.35	132.70	6.35	1.46
G-Z140-M390-T1.4	1.4	3.81	116.80	5.61	1.47
G-Z140-M390-T1.5	1.5	4.08	124.70	6.70	1.64
G-Z140-M390-T1.6	1.6	4.35	132.70	7.79	1.79

[1] Yield stress of 390 MPa assumed in SMAN load-span tables

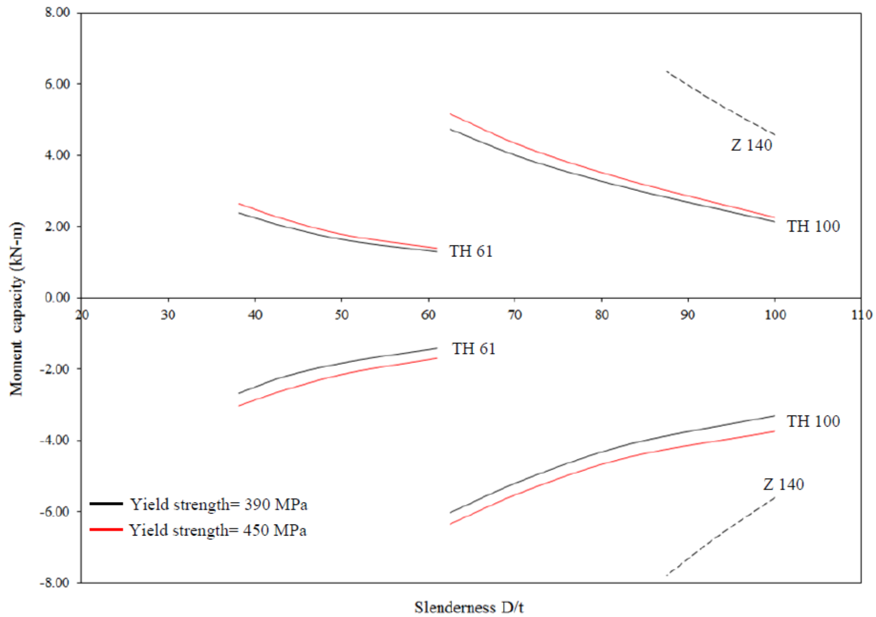


Figure 6: Effects of section slenderness and material grade on bending moment capacity.

Figure 6 shows the variation of moment capacity against slenderness (D/t), for the top-hat and zed-sections. As can be seen, the moment capacity increases as the slenderness decreases. It can be seen that use of 450 MPa grade steel is more beneficial for the top-hat sections with smaller values of slenderness. As distortional buckling governs the uplift moment capacity of the top hat, this mode of failure is less sensitive to the steel grade. Generally, increasing the grade of steel from 390 MPa to 450 MPa resulted in an average capacity increase of 17% under the gravity load case and only 8% under the uplift load case.

Application to practical purlin design

The design of the purlins is considered for the geometry of portal frame shown in Figure 7. As can be seen, a frame of span of 12 m, height of eaves of 3 m and roof pitch of 10^0 is adopted. Using this geometry of frame, frame spacings of 3 m, 4 m, 5 m, and 6 m are investigated. The length of the building is always assumed as being three times that of the frame spacing.

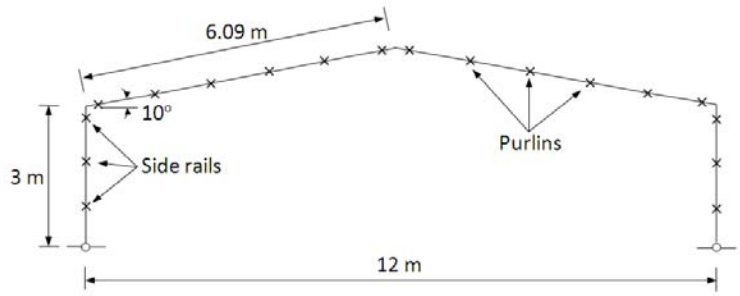


Figure 7: Geometry of Portal frame of building

The loads applied to the frame (and therefore to the pulins) were as follows.

Dead Load (DL): Cladding and service loads on the slope and self-weight of columns, rafters, purlins, and side rails of 0.15 kN/m^2.

Live Load (LL): Snow load of 0.6 kN/m^2

The following site conditions were assumed, all considered as being typical in the UK.

Basic wind speed: 24m/s
Site altitude: 50m
Distance to the sea: 10km
Directional factor: 1
Seasonal factor: 1

In accordance with BS 6399 (1997), the design wind pressures (p) were calculated as follows.

$$p = q_s\left(C_{pe} - C_{pi}\right) \quad (1)$$

Where, Cpe is the external pressure coefficient
Cpi is the internal pressure coefficient.

For buildings of normal permeability, without dominant openings, C_{pi} has a minimum value of -0.3 for negative pressure, and a maximum value of +0.2 for positive pressure.

Two critical wind load cases were chosen for wind pressure (WP) and wind uplift (WU). The wind pressures for localised pressure zones were averaged into a conservative uniformly distributed load as described in SCI design guide for BS6399 (SCI, 2003).

The purlins were checked for the following four ultimate limit state load combinations (ULCs) (SCI, 2003)

$$\mathrm{ULC1} = 1.4\mathrm{DL} + 1.6\mathrm{LL} \tag{2a}$$
$$\mathrm{ULC2} = 1.2\mathrm{DL} + 1.2\mathrm{LL} + 1.2\mathrm{WP} \tag{2b}$$
$$\mathrm{ULC3} = 1.0\mathrm{DL} + 1.4\mathrm{WU} \tag{2c}$$

The purlins were also checked at the serviceability limit state for the following three serviceability load combinations (SLCs).

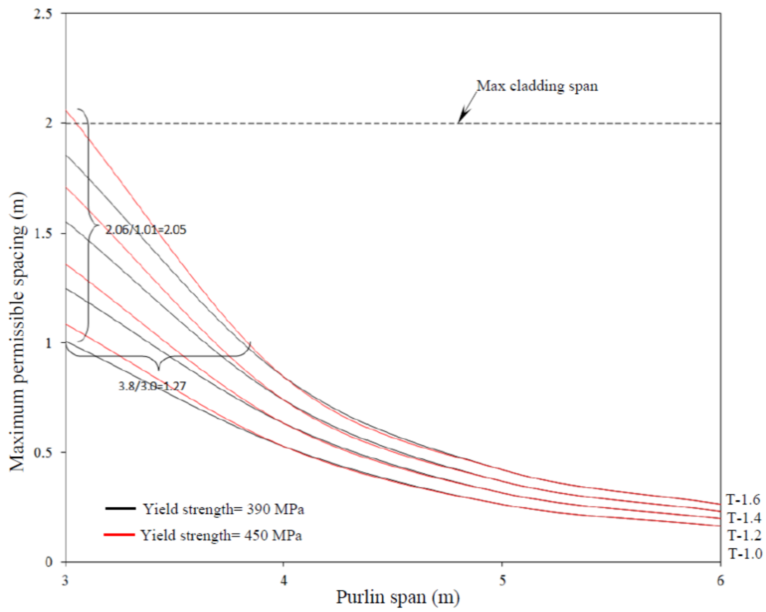
$$\mathrm{SLC1} = 1.0\mathrm{LL} \tag{2d}$$
$$\mathrm{SLC2} = 1.0\mathrm{WP} \tag{2e}$$
$$\mathrm{SLC3} = 1.0\mathrm{WU} \tag{2f}$$

The deflection limits adopted were the maximum of span /150 and 30 mm.

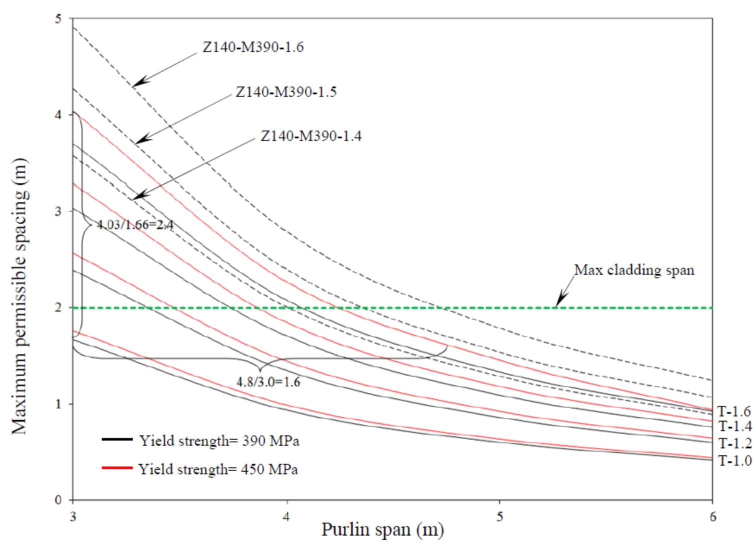
Figure 8(a) shows the variation of maximum permissible purlin spacing against frame spacing for the Top-hat 61. The horizontal line at 2 m indicates the maximum spanning capability of the cladding (Steadmans, 2011). Therefore, even if the maximum purlin spacing can be greater than 2 m, the purlin spacing needs to be reduced to 2 m in order to accommodate the design of the cladding. It can be seen from Figure 8(a) that the effect of the higher steel grade of 450 MPa is only beneficial for purlin spans less than 4 m; this indicates that for spans greater than 4 m, the design is controlled by serviceability.

Figure 8(b) shows the variation of maximum permissible purlin spacing against frame spacing for the Top-hat 100. The same results for the Z-140 are also shown. As mentioned previously, Z-140 is the smallest zed section available in the manufactures' catalogue (Steadmans, 2012). It can be seen that if zed sections are used for purlin spans less than 4 m that the purlins will be over designed.

Figure 9(a) shows, for the case of a purlin span of 3 m, the purlin weight per square meter (on plan). The maximum permissible spacing is shown above each of the bars. As can be seen, the weight of TH61-T1.6 and TH100-T1.0 are competitive compared with the zed-sections. However, this does not take into account the fact that the cost of the 1.0 mm steel by volume is likely to be cheaper than that of the zed-sections. It also does not take into account the fact that the top-hat sections are easier to install on site.

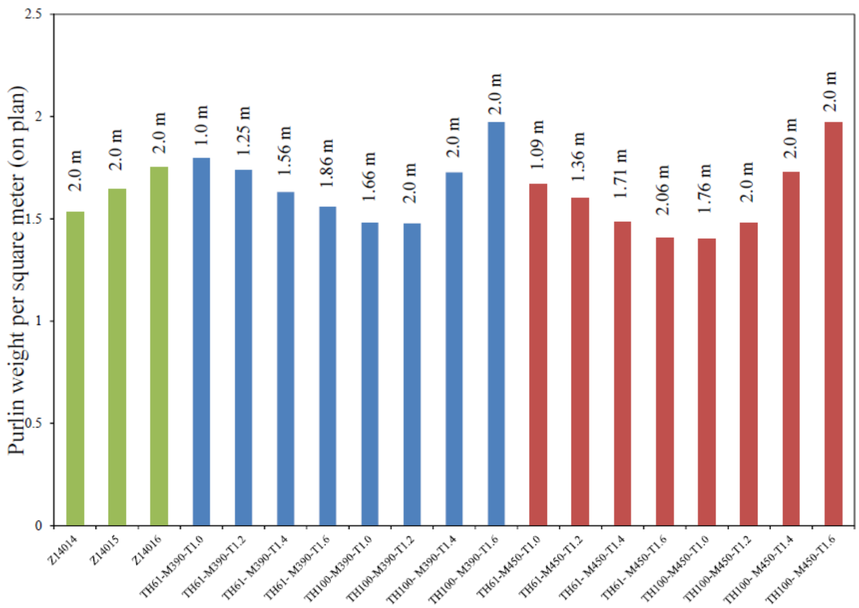


(a) Top-hat 61 purlin

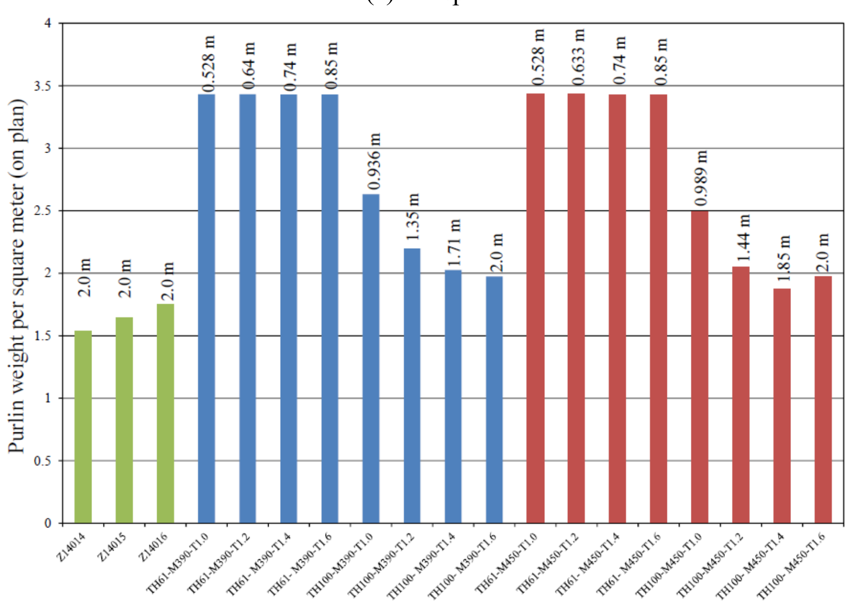


(b) Top-hat 100 purlins

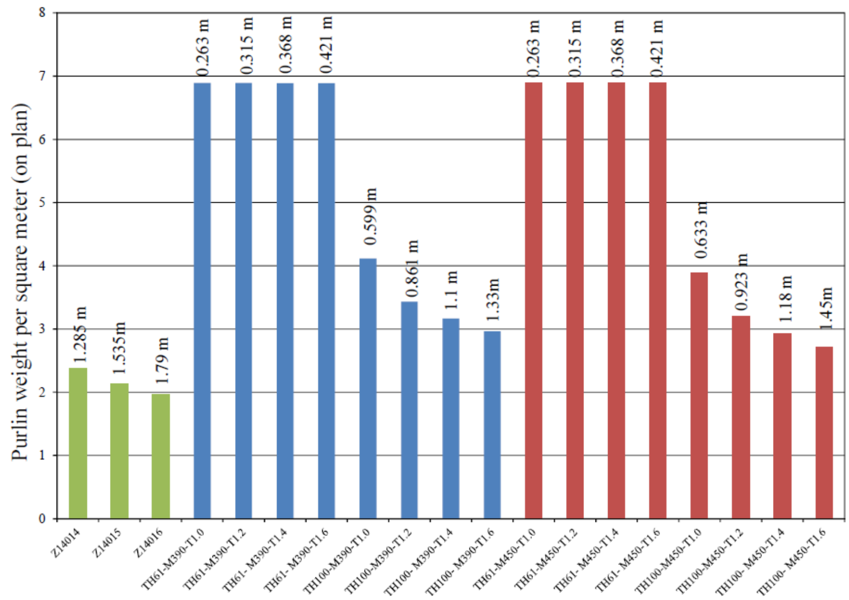
Figure 8: Variation of maximum permissible spacing against span



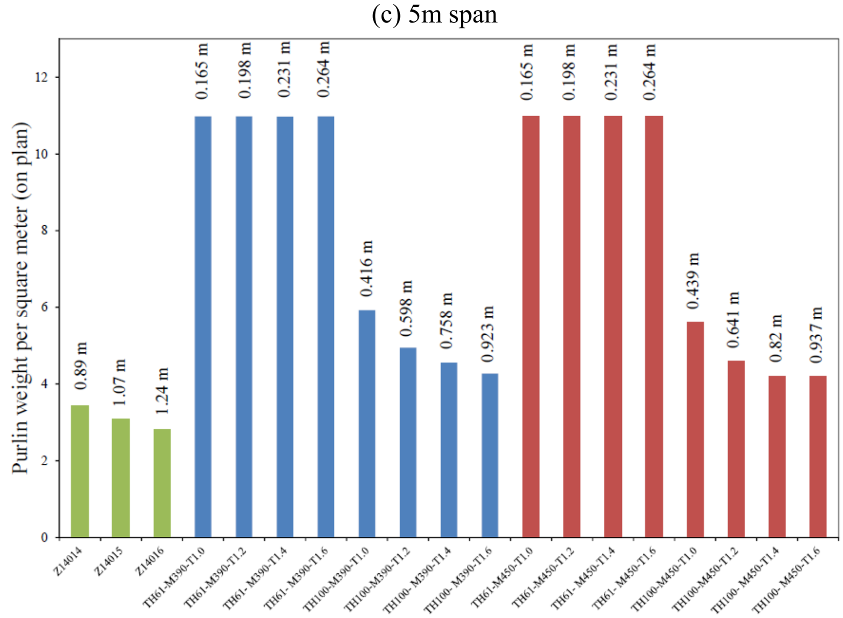
(a) 3m span



(b) 4m span



(c) 5m span



(d) 6m span

Figure 9: Purlin weight per square meter (on plan)

Figure 9(b) shows the same results for the case of a purlin span of 4 m. As can be seen, the TH100-T1.4 is the most competitive top-hat purlin, with a weight approximately only 20% higher than that of the zed-sections.

Figure 9(c) and (d) show the same results for the case of purlin spans of 5 m and 6 m, respectively. As can be seen, the zed-sections are more competitive.

Conclusions

This paper has considered the viability of using top-hat sections for purlins in cold-formed steel portal frames. For such frames, the optimal building may have a frame spacing less than the 6 m used typically in hot-rolled steel construction. Furthermore, in cold-formed steel portal frames, the purlin spacing may need to be smaller in order to provide more lateral stability to the primary column and rafter members.

The finite element model was used to undertake a parametric study comprising different thicknesses and strengths of the top-hat sections. The results were then used to construct bar charts showing the efficiency of the top-hat sections compared with the zed-section in terms of weight of steel of purlin required per square meter on the roof. While the zed-sections were shown to be more efficient for all cases, the comparison showed that top-hat sections performed similarly for frame spacings of 3 m and 4 m. For frame spacings of 5 m and 6 m, use of top-hat sections would not be efficient.

However, this comparison in terms of weight ignores some of the advantages of the top-hat sections in terms of ease of installation on site, as well as beneficial effects such as stressed-skin action. Furthermore, a comparison in terms of cost would be more favourable for the top-hat section of thinner gauge.

The complete study demonstrates how modern numerical analysis techniques of the sort that are now readily available to the research community may be used to develop design guidance for complex structural components. Such an approach greatly reduces the need for expensive and time consuming laboratory study, whilst maintaining realistic and safe coverage of all important structural issues.

Acknowledgements

The Authors would like to acknowledge the Capital Steel Buildings for providing all test specimens. The experimental work was carried out by Angus Potter, as part of his undergraduate research project. The authors also wish to thank Mr Chris Cameron and Mr Andrew Crockett for their assistance in preparing the specimens and carrying out the experimental testing.

References

ANSYS 2011. “Release 11- User’s Manuals”, ANSYS Inc.

Acharya , V.V. & Schuste, R. M. 1998. “Bending tests of hat sections with multiple longitudinal stiffeners”, *Fourteenth international specialty conference on cold-formed steel structures*, St. Louis, Missouri.

BS 6399-2. 1997. “Loading for buildings. Code of practice for wind loads ”. London, British Standards Institution.

Honfi, D. 2006. “Neural network based genetic algorithm optimization of hat-shaped beams”, *Proceedings of the Fifth International Conference on Engineering Computational Technology*, Civil-Comp Press, Paper 52, Stirlingshire.

Pastor, M.M. & Roure , F. 2008. “Open cross-section beams under pure bending I. Experimental investigations”, *Thin-Walled Structures*, Volume 46, Pages 476–483.

Pastor, M.M. & Roure , F. 2009. “Open cross-section beams under pure bending II. Finite element simulation”, *Thin-Walled Structures*, Volume 47, Pages 514–521.

Phan, D.T., Lim ,J.B.P., Sha, W., Siew , C.S.Y., Tanyimboh T.T., Issa, H. & Mohammad, F.A. 2013. “Optimization of cold-formed steel portal frame topography using real-coded genetic algorithm”, Engineering *Optimization*, Vol. 45, No. 4, Pages 415-433.

Potter, A. 2010. “A comparison between physical testing results and numerical calculation methods pertaining to the determination of moment capacity and deflection”, Final year project thesis, University of Strathclyde, Glasgow.

SCI. 2003 “Guide to evaluating design wind load to BS 6399-2.1997’ The Steel Construction Institute, 2003.

Steadmans 2012. “Purlins, rails and eaves beams design guide” *A Steadman & Son*.

Steadmans 2011. “Purlins, rails and eaves beams load tables” *A Steadman & Son*.

Uzzaman, A., Wrzesien , Hamilton , A.M., R., Lim , J.B.P & Nash , D. 2013. “Investigation of Cold-Formed Steel Top-Hat Sections Under Bending”, *Proceedings of the Fourteenth International Conference on Civil, Structural and Environmental Engineering Computing*, Civil-Comp Press, Stirlingshire, UK, Paper 15, doi:10.4203/ccp.102.15..