# Design of cold-formed portal frames

David Brown of the SCI discusses some of the issues to be considered when designing portal frames constructed from cold-formed steel.

## Background

Though apparently simple, all portal frames exhibit challenging forms of structural behaviour, including second-order effects and reversing combinations of loading.

Portal frames constructed from cold-formed steel members (typically back-to-back C sections, less than approximately 3 mm thickness) have additional effects from flexible connections at the eaves and apex. Many such frames are provided for the agricultural sector and are designed to BS 5502-22[1]. Frames designed to this Standard fall outside the Building Regulations and are not subject to any independent checks (the situation in Scotland may be different).

Cold-formed steelwork may be particularly attractive for modest span frames, being lightweight, accurately produced on numerically controlled machinery, requiring no welding and producing a cost-effective solution.  Some suppliers are able to provide designs, details, (including cladding, doors, windows etc), a complete material listing, manufacturing information and a final cost immediately a structural outline is conceived.

When cold-formed portal frames first made a significant appearance in the UK, the technology was largely imported from Australia, incorporating many of the cross-sectional profiles and details. Preliminary investigations by SCI in 1996 led to an article in *New Steel Construction*[2], pointing out the very significant concerns with merely transferring designs from Australia to the UK.

In the winter of 2009-2010, a number of structures in Scotland collapsed under snow loading. Confidential reports indicate over 400 agricultural buildings collapsed. OneEngineer with extensive experience in the design and construction of agricultural buildings commented:

> *"All of the snow affected buildings which I have seen have had problems of compression or lateral torsional buckling failure or distress of either the rafter, the rafter haunch bottom flange or the inside flange of the upper section of the Columns. Attention to detailing in these areas, particularly with respect to web/morris stiffeners, rafter and side rails stays, which give designed torsional buckling stability to the rafters and columns require highlighting to the agricultural steel frame supply industry."*

Public resources from 2011 reported that over the previous two winters, 4000 buildings collapsed under the weight of snow[3]. In 2013, the insurer, NFU Mutual, noted in April 2013 that the collapses were split about 50:50 between modern farm buildings and more traditional farm buildings.[4] NFU mutual also commented that "there is an ongoing concern that the specifications that are in place for farm roofs are good enough". There is nothing in these comments to indicate cold-formed portals are particularly badly designed; the same good engineering is required for all structures.

## Connection flexibility

Typically, eaves and apex connections in cold-formed portals are detailed with a steel plate sandwiched between the two back to-back channel sections. The channel sections are bolted through the plate, or screwed to the plate. Because the channel sections are thin, (and the plate between may be thin) there is considerable deformation of the material under load, which results in very significant connection flexibility.

Connection flexibility has been investigated by several researchers, including Lim and Nethercot in 2002[5]. Lim and Nethercot note that there will be considerable redistribution of bending moment around the frame as a result of the connection flexibility.

Lim and Nethercot provide a dramatic illustration of the effect of connection flexibility by providing results for the vertical deflection at the ridge, for the two frames they studied. Figure 1 is a representation of Figure 11 from Lim and Nethercot, showing apex deflection.

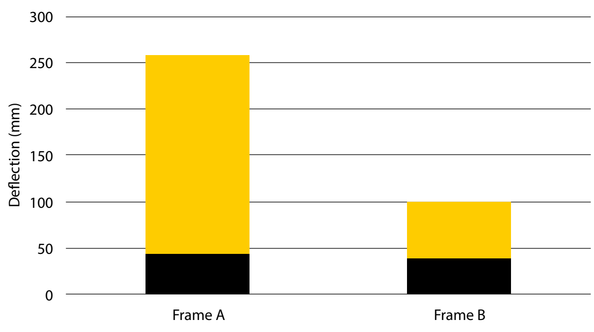


***Figure 1: Apex deflection***

In Figure 1, the deflection coloured black is the deflection assuming a rigid frame. The deflection coloured gold is the deflection due to bolt-hole deformation – in effect the flexibility of the connection. In Frame A, the apex deflection assuming rigid joints is approximately 45 mm. When the effects of connection flexibility are added, the deflection increases to approximately 260 mm.

In order to examine the effects of connection flexibility on the bending moments around the frame, and on in-plane stability, SCI modelled Frame A (tested by Lim and Nethercot), under the same loading, with identical member stiffness. The connection flexibility was also modelled and calibrated against the test results.

## Connection flexibility and bending moment

Modelling Frame A with rigid joints produces an apex deflection of 42 mm. This seems to correspond well with Figure 1.

With rigid joints, the bending moment diagram is shown in Figure 2.

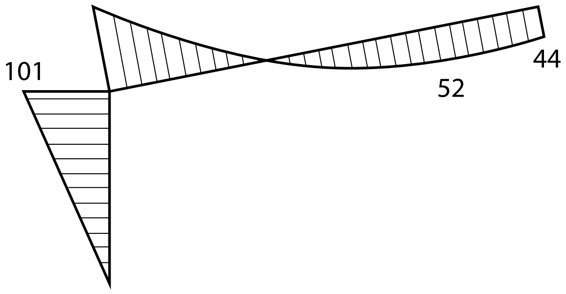


*Figure 2: Bending moment diagram with rigid joints (kNm)*

When the connection flexibility is allowed for, the deflection at the apex was 234 mm. This seems to correspond reasonably well with Figure 1.

Accounting for connection flexibility, the bending moment diagram is shown in Figure 3.

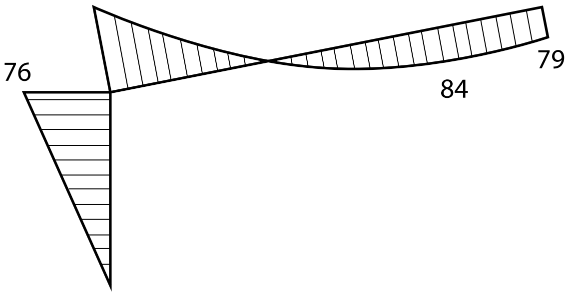


*Figure 3: Bending moment diagram with flexible joints (kNm)*

With flexible joints, the maximum sagging moment has increased by 60%. This is not the final design moment, because as will be seen in the next section, second-order effects become significant and the bending moments shown in Figure 3 must be amplified to allow for this effect.

## Connection flexibility and in-plane stability

The assessment of in-plane stability followed the procedure given in *Eurocode 3 and the in-plane stability of portal frames*[6].

### Rigid joints

With rigid joints, the lateral deflection at the top of the column was 0.74 mm

$$\text{Thus} \quad \alpha_{cr} = \frac{3000}{200 \times 0.74} = 20.2$$

No base stiffness was considered in this calculation, as the typical details are considered to be pinned.

$N_{cr}$ was calculated to be 772 kN. From the analysis, $N_{ed}$ = 40 kN.

$$\text{Thus} \quad \alpha_{cr,est} = 0.8\left[1 - \frac{40}{772}\right] \times 20.2 = 15.3$$

Because $\alpha_{cr}$ is greater than 10, second-order effects are small enough to be ignored; no amplification is necessary.

### Flexible joints

With flexible joints, the lateral deflection at the top of the column was 2.65 mm

$$\text{Thus} \quad \alpha_{cr} = \frac{3000}{200 \times 2.65} = 5.66$$

$$\text{Thus} \quad \alpha_{cr,est} = 0.8\left[1 - \frac{40}{772}\right] \times 5.66 = 4.29$$

Because $\alpha_{cr}$ is less than 10, second-order effects must be allowed for.

$$\text{The amplification} = \frac{4.29}{4.29 - 1} = 1.3$$

Applying this amplifier to the bending moments shown in Figure 3, the design bending moments are shown in Figure 4.

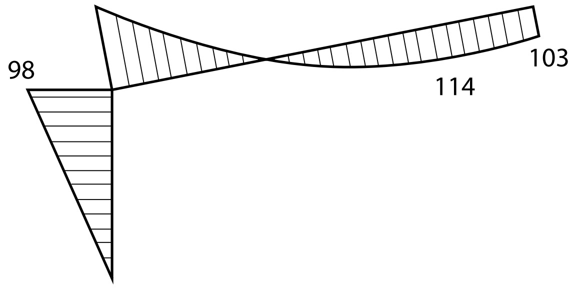


*Figure 4: Amplified bending moment diagram with flexible joints (kNm)*

Allowing for connection flexibility has increased the design moment in the sagging zone from 52 kNm to 110 kNm. Allowing for the connection flexibility has resulted in a frame where second-order effects are very significant; these are small enough to be ignored if rigid joints are assumed.

## Member verification

All member verifications are carried out between restraints. Unless these are specifically provided for, it cannot be assumed that the inside flange is restrained. Purlin and side rail positions on the outside flange cannot be assumed to provide restraint to the inside flange.

## Conclusions

1. Connection flexibility must be allowed for in design. The bending moment diagram changes dramatically, with the potential that members are verified for only 50% of the design moment. The apex joint is likely to experience a moment around twice that predicted by a rigid frame analysis. In the Lim and Nethercot study, the frames failed at the apex joint, which had a resistance higher than required for a rigid-jointed frame.
2. Connection flexibility must be allowed for when assessing in-plane stability. Connection flexibility will reduce the in-plane stability significantly. In the frame considered in this article, the assessment of the frame changed from apparently very stiff (αcr = 15.3) to one where second-order effects are very significant (αcr = 4.29)
3. The member verification checks must be compatible with the actual details of restraint. It is not appropriate to assume that every purlin or rail provides restraint to the inside flange, unless such restraints (straps, braces or other detail) are provided and installed.

## References

1 BS5502-22
Buildings and strictures for agriculture – Part 22: Code of practice of design, construction and loading. BSI, 2013

2 Brown, D. G.
Cold-rolled portal frames
New Steel Construction, May 2006

3 http://www.stackyard.com/news/2011/04/buildings/01_snow_collapse.html

4 http://www.yorkshirepost.co.uk/news/rural/farming/insurer-counting-the-cost-of-farm-building-roof-collapses-1-5586285

5 Lim, J. B. P. and Nethercot, D. A.
Design and development of a general cold-formed steel portal framing system
The Structural Engineer, November 2002

6 Lim, J. B. P. *et al*
Eurocode 3 and the in-plane stability of portal frames
The Structural Engineer, November 2005