Sway and No-Sway Behaviour of Pitched Roof Portal Frames Composed of Cold-Formed Steel Sections

Ghada El-Mahdy Ph.D, P.Eng.

Maged Tawfick Hanna Ph.D

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G.M. El-Mahdy\textsuperscript{1} and M.T. Hanna\textsuperscript{2}

\textsuperscript{1}Professor, Civil Engineering Department, British University in Egypt (BUE), Cairo
\textsuperscript{2}Professor, Structures and Metallic Construction Research Institute, Housing and Building National Research Center (HBRC), Giza.

Abstract

Cold-formed sections buckle locally and distortionally in combination with overall buckling of the member. When these sections are incorporated into a portal frame structure, this combination of buckling modes affects the overall buckling behaviour of the portal frame. The objective of this paper is to study the effect of local and distortional buckling on the overall behaviour of portal frames. The portal frames studied have a span of 10 m and an eave height of 3 m with a roof pitch of 1:10. Two frames are studied the first with one intermediate interconnector at the midpoints of the column and rafter members and the second with two intermediate interconnectors at the third points of the column and rafter members. Three section thicknesses are studied for each frame. Both no-sway behaviour due to gravity loads and sway behaviour due to wind and gravity loads are studied. A nonlinear finite element analysis is performed for each frame. The critical frame section is designed according to the Egyptian Code of Practice (ECP) allowable stress design (ASD) and load and resistance factor design (LRFD), as well as, the American Iron and Steel Institute’s North American Specification for the Design of Cold Formed Structural Members (AISI S100). Unity factors are calculated and the factor of safety or reduction capacity factor as obtained by the finite element analysis is determined.

Keywords: cold-formed sections, portal frames, built-up sections, local plate buckling, distortional buckling, overall buckling, finite element analysis, codes.

1 Introduction

There has been a rapid increase in the use of cold-formed steel (CFS) in building construction recently. In low-rise commercial, light industrial and agricultural buildings previous studies have shown that portal frames with spans of up to 12 m can be constructed (Baigent and Hancock, 1982; Kirk, 1986; De Vos and Van Rensburg, 1997; J.B.P. Lim and Nethercot, 2004a and b). This has made cold-formed steel portal frames an attractive alternative to the use of
conventional hot rolled steel portal frames for this span range. Research has mainly concentrated on the semi-rigidity of the eaves and apex joints (Lim, 2001; Lim and Nethercot, 2002 and 2004), and little attention has been given to the design parameters of the frame components themselves. CFS can easily be bent into various shapes which have obvious design advantages. The most common of these shapes are channels, lipped channels, sigma sections, Z-sections, and hat sections. All these shapes are commonly used for roof purlins, but they can also be used in the built-up configuration as the column and rafter elements of portal frames. Lipped channels are probably the simplest shape section used and can either be configured back-to-back forming an I-section or face-to-face forming a box section, which has a greater torsional resistance. In the case of portal frame structures, the lateral-torsional buckling of the frame elements is easily prevented using purlins, side girts, and cladding so the traditional back-to-back I-section is again feasible. Using back-to-back lipped channel sections with rigid joints interconnected by plates at intervals along their length is the subject of this research.

In design, CFS structures differ from conventional steel structures in that there are more modes of buckling, such as local buckling and distortional buckling, that interact with the flexural action of the frame. Local buckling is accounted for in design by using an effective section in the design equations, but distortional buckling is often not taken into account when using the traditional effective width method.

The objective of the paper is to compare the capacity of CFS sections in portal frames found using a finite element analysis with the design according to the Egyptian Codes of Practice – Allowable Stress Design (ECP, 2001) and Load and Resistance Factored Design (ECP, 2007), as well as, according to the AISI Specification S100 (AISI, 2007). This paper is based upon El-Mahdy and Hanna (2013) which only studied the frame in the symmetrical mode under gravity loads (no-sway), but the current paper includes a study of the frame in the sway mode under gravity and wind loads together. A review of the effective length factors for the frame in the no-sway and sway modes according to the Julian and Lawrence alignment charts (Kavanagh, 1962) as well as from the finite element analysis is also given.

2 Frames studied

The frames studied had a span of 10 m and an eave height of 3 m as shown in the schematic representation of the frames in Figure 1. The roof was given a slope of 1:10 making the apex height 3.5 m. The frames had hinged end conditions at the supports. Preliminary design determined that the suitable section for this frame consisted of two back-to-back lipped channel sections of dimensions 250x85 as shown in Figure 2. Three thicknesses were used to model and design the frames which were 1.5 mm, 2.0 mm, and 2.5 mm. This was to study whether the mode of failure would be local plate buckling, distortional
buckling, or overall frame buckling.

To achieve rigid connections at the eaves and apex a 6 mm connection plate was assumed having top and bottom flanges. The connections were such that 9 bolts were placed in the web portion of the channel sections and 2 rows of 5 bolts were placed in the top and bottom flanges of the channel sections as shown in Figures 1 and 3. Figure 3 shows the details of the eave and apex connectors, the dimensions shown are those used for the centreline of the connector as modelled in the finite element model. All dimensions are in mm. The web bolts transfer the moment by shear due to a torsional moment whereas the flange bolts transfer the moment by shear due to a moment connection. Bolt-hole elongation was considered negligible and hence the connections were assumed as rigid.

![Figure 1. Schematic representation of frames modelled](image)

The two channels were interconnected at the hinged support by 6 mm thick plates connected to the webs of the channel sections by 9 bolts. Interconnecting plates were also used along the lengths of the column and rafter. Two configurations were assumed the first with one interconnecting plate at the midpoint of the column and rafter and the second with interconnecting plates at the
Third points of the column and rafter as shown in Figure 1. These variations were made to see if the degree of interconnection had a significant effect on the strength of the CFS portal frame. The frames studied are designated by the number of interconnectors/thickness of section/NS or S to represent no-sway or sway frames (e.g., Model 1/1.5/NS, Model 2/2.0/S etc.).

![Figure 2. Details of cold formed channel sections](image)

Two cases of loading were studied to represent a no-sway and a sway frame. The no-sway frame was subjected to symmetrical gravity loads as a uniformly distributed load along the rafters. The sway frame was subjected to uniformly distributed gravity loads in combination with a concentrated lateral wind load at the left eave connection as shown in Figure 4. For the sway frame the maximum gravity loads were used (i.e., dead load and live load) on the rafter and only the wind load acting on the sides of building walls were considered neglecting the suction forces on the roof. This case of loading was chosen to give the maximum combination of bending moment and compression force in the right column at...
the eave connection as the purpose of this work is to study the interaction of local
and distortional buckling on the overall buckling of the frame. The design loads
used to design the frames, which had a spacing of 4 m, were assumed dead and
live loads as follows:

- Own weight of frame = 150 N/m²
- Purlins and cladding = 150 N/m²
- Live loads = 500 N/m²
- Wind loads = 700 N/m² with wind pressure coefficients of +0.8 and -
  0.5 on the windward and leeward sides of the building, respectively.

These loads gave a maximum bending moment of 21.4 kNm and compressive
force of 16 kN in the column at the eave connection for the no-sway frame, and
a maximum bending moment of 36.6 kNm and compressive force of 19.3 kN in
the right column at the eave connection for the sway frame. These straining
actions were used to design the double channel sections.

![Figure 4. Design loads on the no-sway and sway frames studied](image)

3 Finite element model

COSMOS/M 2.6 finite element package was used to model the portal frames.
A general 3-D shell element model was used. Due to the symmetry of the frame
and the vertical loading only half the frame was modelled for the no-sway frame
to save computing time, whereas the whole frame was modelled for the sway
frame. Four node thick shell elements (SHELL4T) were used to model the
channel elements to enable a material plasticity model to be used for the frame.
The channel section elements were given a thickness of 1.5 mm, 2.0 mm, or 2.5
mm and the connection plate elements were given a thickness of 6 mm. The
channel sections were stiffened at the ends to help maintain their configuration
by 6 mm thick stiffener plates. The material plasticity criterion used was a von
Mises isotropic model and large deflection analysis was prescribed. The material
properties used were linear elastic-perfectly plastic with a modulus of elasticity
of 210 GPa and a yield stress of 360 MPa. Hence reaching the yield stress was
the criteria for failure.
As the frame was designed as a two-hinge frame, the displacements in the plane of the frame were prevented at the hinges and rotation about the axis perpendicular to the plane of the frame was permitted. For the no-sway frame, the nodes of the mid-section of the apex connection were prevented from displacing in the direction of the span of the frame to model the symmetric boundary conditions. Out-of-plane displacements were constrained by suitable boundary conditions at the eave and apex connections to eliminate any out-of-plane deformations of the frame. The channel sections were free to displace out-of-plane in between the connections.

The eave and apex connections were modelled as rigid connections neglecting any bolt-hole elongation effects. This was done using rigid constraints between the connecting plate and the channel members. Nine connections were made to each channel web representing bolts and a row of 5 connections were made between the connecting plate and each flange of the channel sections. Nine connections were also made between the interconnecting plates and the web of each channel section. The configuration of these connections was as shown in the schematic representation of the frames shown in Figure 1.

The gravity load was applied to the rafter elements along the web-flange intersecting nodes. This was to minimize local bending deformations of the flanges due to the load being applied on the flanges. The design load intensity of 3.2 kN/m² was applied to the no-sway and sway frames and a lateral load of 10.92 kN was applied at the left eave connector for the sway frame. The linear response, as well as, the bifurcation buckling mode and the nonlinear response was obtained. For the nonlinear analysis an incremental-iterative procedure based on incremental load application was used. The load was permitted to exceed the design load to find the final failure load and hence the factor of safety of the design. Figure 5 shows the details of the finite element model of the sway frame with one interconnector in the column and rafter.

Figure 5. Finite element model of sway frame with one interconnector
4 Structural response of the frames

4.1 Response of no-sway frame

There was very little difference in the structural response of the two no-sway frames, the one with one interconnector and the one with two interconnectors, for each section thickness as shown in the load-deflection curves of the vertical apex deflection $\delta$ shown in Figure 6. Both the model with one interconnector and the model with two interconnectors took the same load path reaching a load of practically the same maximum load. This shows that the intermediate interconnectors played a very minor role in keeping the two channel sections together and reducing distortional buckling effects. The mode of failure and failure load did vary, however, with the change in thickness of the section.

![Figure 6: Load-deflection curves for apex vertical deflection of the no-sway frames](image)

For the models with 1.5 mm thick sections the mode of failure was that of local plate buckling in both the compression flange and the compression part of the web as can be seen by the finite element response in Figure 7. The apex vertical deflection showed very little nonlinear action for this thickness indicating that overall buckling of the frame was not the mode of failure.
For the models with 2.0 mm thick sections a kind of distortional buckling of the compression flange of the column section below the eave connection was detected as shown in the finite element response in Figure 8. However, the out-of-plane load-deflection curves in the web at the critical section a-a in the column just beneath the eave connection showed a distinct nonlinear behaviour, as shown in Figure 9, indicating that the mode of failure was also that of local buckling in the web at this section. There was also some nonlinear behaviour in the compression flange near the maximum load due to distortional buckling as shown in the relative load-deflection curves in Figure 10.

The models with 2.5 mm thick sections did not show any signs of local buckling.
at the critical section in the column below the eave connection, nor did they show
any signs of distortional buckling in the compression flange of this section. In
fact, from the vertical load-deflection curves at the apex of the frame and the
load-deflection curves of the horizontal deflection at the eave of the frame, shown
in Figure 6, the nonlinear response of the frame suggests that overall buckling
or yielding of the eave connector was the mode of failure. There is also a minor
load difference between the two models for this thickness which is possibly due
to a little distortional buckling in the flange of the rafter which was resisted by the
rafter interconnectors.

Figure 10. Load-deflection curves for in-plane horizontal flange displacement of Model 2/2.0/NS (distortional buckling in flange)

Figure 11 shows the membrane normal stress distribution at the critical section
in one channel for the models with two interconnectors at the maximum loads
reached by the frames. The stress distribution for Model 2/1.5/NS, shown in
Figure 11(a), clearly shows the nonlinear stress distribution in the compression
flange and in the compression part of the web again indicating that local plate
buckling is the mode of failure. The stress distribution for Model 2/2.0/NS, shown
in Figure 11(b), does not clearly show any nonlinear stress distribution due to
local buckling in the compression part of the web, but the nonlinear out-of-plane
load-deflection curves of the web in compression suggest this is the mode of
failure. The stress distribution for Model 2/2.5/NS, shown in Figure 11(c), also
shows no nonlinear action in the compression part of the web and also reached
the highest membrane stresses as the terminating condition of the analysis was
not due to extra bending stresses at the faces of the section due to local buckling
or distortional buckling.

4.2 Response of sway frame
The response of the sway frame was similar to that of the no-sway frame
in that there was little difference between the frame with one interconnector and
the frame with two interconnectors as shown in lateral load-eave lateral
displacement curves in Figure 12. However, the thickness of the section again
had a significant effect on the performance of the frames.

Figure 11. Normal membrane stresses (MPa) in column beneath the eave connection of no-
sway frames at maximum load

(a) Model 2/1.5/NS \( w = 4.50 \text{ kN/m}^2 \)

(b) Model 2/2.0/NS \( w = 7.74 \text{ kN/m}^2 \)

(c) Model 2/2.5/NS \( w = 11.01 \text{ kN/m}^2 \)

Figure 11. Normal membrane stresses (MPa) in column beneath the eave connection of no-
sway frames at maximum load

Figure 12. Load-deflection curves for eave lateral deflection of the sway frames

Figure 13(a) shows the overall response of Model 1/1.5/S where the mode of
failure was that of local buckling in the right column just beneath the eave connection as illustrated in Figure 13(b). The nonlinear analysis reached a higher load than indicated by the elastic eigenvalue buckling analysis which suggests that post-buckling was also achieved.

![Stress distribution and response at failure of Model 1/1.5/S](image)

Figure 13. Stress distribution and response at failure of Model 1/1.5/S

For the sway frames with a section thickness of 2.0 mm the mode of failure in the nonlinear analysis was again that of local buckling in the web and distortional buckling in the compression flange of the critical section in the right column just beneath the eave connector. The mode of failure is similar to that shown in Figure 8 and non-linear load-deformation curves similar to those shown in Figures 9 and 10 can be plotted for the critical section. The elastic eigenvalue buckling analysis also indicated that the mode of failure was that of local buckling at the critical section at a similar load which suggests that no post-buckling occurred.

Finally, for the sway frames with a section thickness of 2.5 mm no local or distortional buckling occurred in the nonlinear analysis although local buckling and possible distortional buckling in the right column was the indicated mode of failure in the elastic eigenvalue buckling analysis at a much higher load. This leads to the deduction that some sort of yielding in the eave connector could be the mode of failure which caused the nonlinear behaviour of this section thickness shown in Figure 12.

5 \textit{K}-factors

To do the section calculations and find the unity factor as described in the next section the effective length factors (\textit{K}-factors) of the columns in the no-sway and sway frames need to be determined. This is either done using the Julian and Lawrence alignment charts (Kavanagh, 1962) or through an elastic eigenvalue buckling analysis of the frame. The alignment charts gave \textit{K}-factors of 0.93 for the no-sway frame and 2.25 for the sway frame. To check these values a finite element model was used. As the 3D shell element frame predicted buckling modes of local buckling or distortional buckling it was not possible to get the eigenvalue buckling values of the overall frame for the section thicknesses studied. So, a 3D beam element model was used using the inertias of the sections and assuming rigid joints. This model predicted \textit{K}-factors of 0.97 for the no-sway frame and 3.01 for the sway frame, which were used for the design of the critical sections. It should be noted that the stiffness of the eave and apex
connections would influence these values but this was neglected in the beam element model.

6 Design codes

The critical section in both the no-sway and sway frames is subjected to axial compression and bending moment and must be designed to resist local, distortional, and overall buckling. The classical method to do this is to use the effective section to account for local buckling and the interaction equations to account for combined axial compression and bending taking into account second order effects. The load which gives a unity factor of 1.0 from the interaction equation can then be compared with the ultimate load reached by the finite element model to predict a factor of safety, \( \Omega \), for allowable stress design or a reduction capacity factor, \( \phi \), for load and resistance factor design.

6.1 Egyptian Code of Practice – allowable stress design (ECP-ASD)

The approach of the ECP-ASD (ECP, 2001) is to use an effective section based on the yield stress and to neglect the distortional buckling of the member. The interaction equation according to ECP-ASD is

\[
\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 \leq 1.0 \quad \text{with} \quad A_1 = \left( \frac{C_{mx}}{1 - f_{ca}/F_c} \right)
\]

where \( f_{ca} \) and \( f_{bcx} \) are the actual compressive and flexural stresses in the section computed using service loads and the effective section properties for uniform compression and uniform bending, respectively. \( F_c \) and \( F_{bcx} \) are the allowable compressive and flexural stresses where flexural buckling of the column is included in \( F_c \) and lateral-torsional buckling is included in \( F_{bcx} \). \( A_1 \) is a magnification factor to account for the second order effects of the buckling of the frame, \( C_{mx} \) being a factor to account for sway and \( F_{Ex} \) being the Euler compressive resistance of the member. For members subject to small compressive loads (i.e., \( f_{ca}/F_c < 0.15 \)) \( A_1 \) can be taken as unity, which is the case for these frames. The number of interconnectors is taken into account in evaluating the allowable compressive stress \( F_c \) by using an equivalent slenderness ratio increased to account for local member buckling in between interconnectors as in the case of built-up columns. This equivalent slenderness ratio is taken as the square root of the squares of the slenderness ratios of the overall column length and the individual section (i.e., \( (KL/r)_eq = \sqrt{((KL/r)_o^2 + (a/r)^2)^{0.5}} \), where \( a \) is the distance between interconnectors and \( r \) is the minimum radius of gyration of one component member). Hence, the model with only one interconnector would have a greater equivalent slenderness ratio than the model with two interconnectors leading to a reduction in its allowable compressive strength. Table 1 shows the load \( w \) for a unity factor of 1.0 from the interactive equation (Equation (1)) as well as the factor of safety, \( \Omega \), for the models as predicted by the finite element analysis. The factors of safety for these frames are also plotted in Figures 14(a) and (b) for the no-sway frames and the sway frames, respectively.
6.2 Egyptian Code of Practice – load and resistance factor design (ECP-LRFD)

The approach of the ECP-LRFD (ECP, 2007) is to again use an effective section based on the yield stress and neglect the distorsional buckling of the member. The interaction equations according to ECP-LRFD for the limit state of in-plane buckling are:

\[
\frac{P_u}{\phi_c P_n} \geq 0.2 \quad \left(\frac{P_u}{\phi_c P_n} + \frac{8}{9} \frac{M_{ux}}{\phi_b M_{ax}} \right) \leq 1.0 \tag{2a}
\]

\[
\frac{P_u}{\phi_c P_n} < 0.2 \quad \frac{P_u}{2\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{ax}} \leq 1.0 \tag{2b}
\]

where \(P_u\) and \(M_{ux}\) are the applied factored axial compression and bending moment for strong axis bending determined from a second order elastic analysis to include \(P-\delta\) and \(P-\Delta\) effects. The load factors used are \(\alpha_D = 1.2\) for the dead load and \(\alpha_L = 1.6\) for the live load for the case of the no-sway frames subjected to gravity loads only, whereas these load factors were used in addition to \(\alpha_W = 0.8\) for the wind load for the sway frames subjected to gravity loads in combination with wind loads. \(P_n\) and \(M_{ax}\) are the nominal compressive strength and the nominal flexural strength about the X-axis computed using the effective section properties in compression and in flexure, respectively. \(\phi_c = 0.8\) is the resistance factor for compression and \(\phi_b = 0.85\) is the resistance factor for bending. In addition to the interaction equations given in Equations (2a) and (2b), the limit state for out-of-plane buckling should be checked using the interaction equation:

\[
\frac{P_u}{\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{ax}}\right)^2 \leq 1.0 \tag{3}
\]

where \(\phi_c P_{ny}\) is the available compressive strength for out-of-plane buckling and \(\phi_b M_{nx}\) is the available flexural-torsional strength for strong axis bending. Table 2 shows the factored load \(w_f\) at a unity factor of 1.0 from the interactive equations (Equations (2) and (3)) as well as the capacity reduction factor \(\phi\) for the models as predicted by the finite element analysis. The capacity reduction factor for these frames is also plotted in Figures 15(a) and (b) for the no-sway frames and the sway frames, respectively.

6.3 American Iron and Steel Institute (AISI S100-2007) – effective width method allowable stress design

The AISI S100-2007 – effective width method uses the effective section to
calculate the nominal resistances in compression and bending and also checks these nominal resistances with the resistance to distortional buckling taking into consideration the least resistance. The calculation of the effective section in this specification is not based on the yield stress but on the nominal stress. The interactive equations for the allowable stress design are as follows:

\[
\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{nx} M_x}{M_{nx} \alpha_x} \leq 1.0 \quad (4a)
\]

\[
\frac{\Omega_x P}{P_{no}} + \frac{\Omega_x M_x}{M_{xx}} \leq 1.0 \quad (4b)
\]

where the ASD safety factors \( \Omega_c = 1.80 \) and \( \Omega_b = 1.67 \); \( P \) and \( M_x \) are the required compressive axial strength and the required flexural strength about the X-axis, respectively, computed using service loads and a first-order elastic analysis; \( P_n \) and \( M_{nx} \) are the nominal axial strength and nominal flexural strength, respectively, taking distortional buckling into consideration; \( P_{no} \) is the nominal axial strength taking the nominal stress equal to the yield stress to calculate the effective section properties as well as to calculate the strength; \( C_{mx} \) is a coefficient based on the type of loading and lateral restraint conditions of the frame; \( \alpha_x \) is a moment magnification factor \( = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \) where \( P_{Ex} \) is the Euler load about the X-axis. When \( \Omega_c P/P_n \leq 0.15 \) the following equation is permitted in lieu of Equations (4a) and (4b):

\[
\frac{\Omega_c P}{P_n} + \frac{\Omega_c M_x}{M_{xx}} \leq 1.0 \quad (5)
\]

In general, the AISI specification predicted a higher load for a unity factor of 1.0 and hence a lower factor of safety, \( \Omega \), than the ECP as shown in Table 1 and plotted in Figures 14(a) and (b) for the no-sway frames and sway frames, respectively. For the section with a thickness of 1.5 mm, distortional buckling governed for flexure which led to a lower nominal resistance in flexure and consequently a break towards a higher factor of safety.

6.4 American Iron and Steel Institute (AISI S100-2007) – effective width method load and resistance factor design

The design approach of the AISI S100-2007 – load and resistance factor design is similar to that of the allowable stress design given in Section 5.3 with the exception that factored loads and resistances are used. The interactive equations for the load and resistance factor design are as follows:

\[
\frac{P_u}{\phi \phi_u P_n} + \frac{C_{mx} M_{xx}}{\phi_x \phi_x M_{nx} \alpha_x} \leq 1.0 \quad (6a)
\]
where $P_u$ and $M_{ux}$ are the required axial strength and flexural strength about the X-axis, respectively, calculated using factored loads ($\alpha_D = 1.2$, $\alpha_L = 1.6$, $\alpha_W = 0.8$) and a first order elastic analysis; and the resistance factors $\phi_c = 0.85$ and $\phi_b = 0.9$, and $\alpha_x$ is a moment magnification factor $= 1 - \frac{P_u}{P_{Ex}} > 0$ where $P_{Ex}$ is the Euler load about the X-axis. When $P_u/\phi_c P_n \leq 0.15$ the following equation is permitted in lieu of Equations (6a) and (6b):

$$\frac{P_u}{\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{ux}} \leq 1.0$$

In general, the AISI specification predicted a higher factored load for a unity factor of 1.0 and hence a higher capacity reduction factor, $\phi$, than the ECP as shown in Table 2 and plotted in Figure 15(a) and (b) for no-sway frames and sway frames, respectively. For the section with a thickness of 1.5 mm, distortional buckling governed for flexure which led to a lower nominal resistance in flexure and consequently a break towards a lower capacity reduction factor.

### Table 1. The allowable design load and factor of safety according to various codes ASD for the no-sway and sway frames

| Model 1/1.5/NS | ECP - ASD |  | AISI – ASD |  |
|----------------|-----------|  |-----------|  |
| $w$ (kN/m')    | $\Omega$  |  | $w$ (kN/m') | $\Omega$ |
| 1.800          | 2.486     |  | 1.998     | 2.240 |
| 1.829          | 2.460     |  | 2.024     | 2.223 |
| 2.717          | 2.883     |  | 3.361     | 2.289 |
| 2.761          | 2.803     |  | 3.412     | 2.268 |
| 3.730          | 2.916     |  | 4.313     | 2.522 |
| 3.792          | 2.904     |  | 4.378     | 2.516 |
| 1.080          | 2.519     |  | 1.201     | 2.265 |
| 1.080          | 2.529     |  | 1.210     | 2.257 |
| 1.630          | 2.839     |  | 2.027     | 2.284 |
| 1.630          | 2.868     |  | 2.041     | 2.291 |
| 2.238          | 2.834     |  | 2.599     | 2.440 |
| 2.238          | 2.833     |  | 2.621     | 2.419 |
Table 2. The ultimate factored design load and reduction capacity factor $\phi$ according to various codes LRFD for the no-sway and sway frames.

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<thead>
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<th>ECP – LRFD</th>
<th>AISI – LRFD</th>
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<td></td>
<td>$w_f$ (kN/m')</td>
<td>$\phi$</td>
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<td>Model 2/2.5/S</td>
<td>3.419</td>
<td>0.539</td>
</tr>
</tbody>
</table>

Figure 14. Factor of safety $\Omega$ for ASD

It can be seen from Figures 14(a) and (b) that the factors of safety for the ASD codes compare relative well for the sway frame and the no-sway frame. However, from Figures 15(a) and (b) there is a variance in the load reduction factors for the LRFD codes for the sway frame and the no-sway frame which is probably due to the use of different load combination factors in the design.
6.5 American Iron and Steel Institute (AISI S100-2007) – direct strength method

The AISI S100 specification only accounts for the design of beams and columns using the direct strength method, beam-columns are not accounted for so the comparison will not be made.

7 Conclusions

Loads and factored loads due to unity factors of 1.0 from the compression-flexure interaction equations given in the ECP and AISI codes both ASD and LRFD are calculated for the critical frame section of no-sway and sway frames. The factor of safety of the frame for ASD or the capacity reduction factor for LRFD as obtained from the nonlinear finite element analysis is determined and compared. The results of the finite element analysis show that the number of interconnectors has little effect on the strength of the frame. The finite element model showed that the mode of failure at the critical column section depended on the thickness of the section and was one of local flange and web buckling for the 1.5 mm section, a combination of local web buckling and flange distortional buckling for the 2.0 mm section, and overall frame buckling or yielding of the eave connector for the 2.5 mm section. The factor of safety calculated using the ECP-ASD was higher than that calculated using the AISI-ASD and the capacity reduction factor calculated using the ECP-LRFD was lower than that calculated using the AISI-LRFD. Taking distortional buckling into account in the design when it governs as given in the AISI can increase the factor of safety for ASD and reduce the capacity reduction factor for LRFD. The factors of safety in the ASD for the sway and no-sway frames compared relatively well, whereas there was a little variance in the reduction capacity factor for LRFD between the no-sway and sway frames due to the use of different load combination factors.
References

AISI, (2007) North American Specification for the Design of Cold-Formed Steel Structural Members (AISI S100), American Iron and Steel Institute, USA.


Kirk, P. (1986) “Design of cold-formed section portal frame building system”, Proceedings of the 8th International Speciality Conference on Cold-Formed Steel Structures, St. Louis, University of Missouri-Rolla, 295-310.


