COMPARISON of Chinese and Foreign Specifications on Bending Behavior of Z-Purlins under Wind Uplift Load

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Abstract. It is very difficult to calculate the bending bearing capacity of cold-formed purlins under wind uplift load precisely. The design requirements, which are different in Chinese and foreign specifications, such as GB 50018, CECS 102, EN 1993-1-3, AISI S100 and AS/NZS 4600, are summarized comprehensively. The differences of design methods, calculation models and considerations on restraint effects of lightweight steel roof systems among these specifications are also discussed. The bending bearing capacity of 18 rows of commonly used Z-purlins calculated according to these specifications is analyzed. The analysis shows that for purlins without sag-rod, the calculation results of GB50018 are conservative because the beneficial effects of restraint from lightweight steel roof systems are neglected. For purlins with two rows of sag-rods per span, the calculation results of GB50018 are nearly 20% greater than that of EC3, and the results of AS/NZS are the largest.

1. Introduction
Lightweight steel roof systems, which are commonly composed of profiled steel roof panels and cold-formed thin walled steel purlins, are widely used in industrial buildings and residential buildings all over the world due to their economy and convenience in fabrication and erection. However, because the self-weights of these roof structures are always lower than wind uplift loads, they are liable to destroy under high winds, especially in the coastal areas where wind load is much greater. Cold-formed thin-walled steel purlins are the main bearing members in the lightweight steel roof system, but they are weak in the lateral direction as well as in torsion, thus susceptible to suffer lateral torsional buckling under wind uplift load if not be braced sufficiently in the lateral direction and against twist. Fortunately roof panels which are connected to the upper flangers of purlins do provide some extent of lateral and torsional restraints by virtue of their shear rigidity and resistance to local bending at the connections, which, if be considered properly in design, will greatly improve the bending bearing capacity of purlins. However, the lateral and torsional restraints provided by roof panels are difficult to calculate accurately, because they are closely related to the connection between roof panels and purlins, which are diverse in configuration and sometimes very complex. In order to solve these problems, previous researchers have done a lot of work through theoretical analysis, experimental study and numerical simulation[1-2].

In this paper, the calculation requirements about bending bearing capacity of cold-formed thin walled purlins under wind uplift load in three different design specifications were summarized and compared. All of the design specifications are widely used in China and European countries, which are
2. Commonly used configurations connecting roof panels and purlins

2.1 Through-fastened by screws
In the early decades, lightweight steel roof systems composed of cold-formed steel purlins with the upper-flange through-fastened to trapezoidal steel roof panels by self-drilling screws were the mainly used roof systems in steel light-frame buildings. In these roof systems, full continuous lateral restraint may be supplied by trapezoidal steel roof panels with sufficient stiffness, thus the purlins can be regarded as laterally restrained. However, because the connection zone between purlin flange and roof panel is nearly only a point, rotational restraint provided by the trapezoidal steel roof panels may not be fully transmitted to the purlins. The purlin will twist slightly first when being loaded, and then as the other parts of purlin flange contact tightly with roof panels, the twist may be restrained by virtue of roof panel against local bending, but the rotational restraint may be very limited and usually effected by many uncertain influences, such as local bending rigidity of trapezoidal steel roof panels and the space between screw fasten point and the other contact point of purlin and roof panel, which is the flange-clip point for C-shape sections and the flange-web point for Z-shape sections.

2.2 Fastened by sliding clips
In recent years, standing seam roof systems composed of cold-formed steel purlins with the upper-flange fastened to standing seam roof panels by sliding clips are very prevalent and widely used in public buildings, such as airport terminals, gymnasiums, exhibition halls etc. However, conversely with the trapezoidal steel roof panels through-fastened by screws, standing seam roof panels, connecting with purlins by sliding clips, can slide horizontally relative to purlins, which means the lateral restraints to purlins provided by roof panels are very limited and sometimes even zero. But the length of the clips along the longitudinal direction of the roof panels is always longer than the width of purlin flanges, so that the clips may have sufficient shear rigidity to effectively transmit the rotational restraint to purlins.

3. Design requirements and distinctions of different Specifications
To calculate the bending bearing capacity of purlins under wind uplift load as accurately as possible, different calculation models and design methods, which considering the restraint effects in varying degrees, are adopted by the Chinese and foreign specifications as mentioned in section 1.

Table 1 shows whether the restraints of roof panels are considered and which calculation model or design method is adopted in each specification.

| Specifications | Through-fastened by screws | Fastened by sliding clips |
|---------------|----------------------------|---------------------------|
|               | Considering the Restraint of roof | Design method | Considering the Restraint of roof | Design method |
| EC3           | Yes                        | beam-column on elastic foundation | No | - |
| AISI S100     | Yes                        | Reduction of strength | Yes | Reduction of strength |
| GB50018       | No                         | Lateral torsional buckling | No | Lateral torsional buckling |
| CECS 102      | Yes                        | beam-column on elastic foundation | No | Lateral torsional buckling |
3.1 Design requirements of EC3 and distinctions with CECS
The EC3 recommends, for trapezoidal steel sheeting or other profiled steel sheeting continuously connected to the flange of the purlin through the troughs of the sheets by screws, if the shear stiffness is sufficient, which means the condition expressed by the equation (1) is met, the purlin at the connection may be regarded as being laterally restrained in the plane of the sheeting:

\[
S \geq \left( EI_w \frac{n^2}{L^2} + GI_L + EI_z \frac{n^2}{L^2} 0.25h^2 \right) \frac{g}{h^2}
\]  

(1)

Where, \( S \) is the portion of the shear stiffness provided by the sheeting for the examined member connected to the sheeting at each rib; \( I_w \) is the warping constant of the purlin; \( I_t \) is the torsion constant of the purlin; \( I_z \) is the second moment of area of the cross-section about the minor axis of the cross-section of the purlin; \( L \) is the span of the purlin; \( h \) is the height of the purlin.

Then the behaviour of a laterally restrained purlin should be modelled as outlined in figure 1. The total deformation of purlin under wind uplift load can be split into two parts, which are in-plane bending and torsion and lateral bending. The in-plane bending can be analyzed using the simple flexure theory. The torsion and lateral bending should be analyzed through the use of an idealized analytical model. The model involves the assumption of a beam-column on elastic foundation, as shown in figure 3. The beam-column section consists of the compression flange and one fifth of the web. The connection of the purlin to the sheeting may be assumed to partially restrain the twisting of the purlin. This partial torsional restraint may be represented by a rotational spring with a spring stiffness \( C_D \), which is best determined by test through a test set-up shown in figure 2. The analytical formulas for calculating the spring stiffness \( C_D \) are also provided in EC3.

The stresses in the free flange, not directly connected to the sheeting, should then be calculated by superposing the effects of in-plane bending and the effects of torsion, including lateral bending due to cross-sectional distortion. The maximum stresses in the free flange of cross-section should satisfy the following equations:

Resistance of cross-sections:

\[
\sigma_{\text{max,Ed}} = \frac{M_{y,Ed}}{W_{\text{eff,y}}} + \frac{N_{Ed}}{A_{\text{eff}}} + \frac{M_{fz,Ed}}{W_{fz}} \leq f_y / \gamma_M
\]  

(2)

Buckling resistance of free flange:

\[
\frac{1}{\chi_{LT}} \left( \frac{M_{y,Ed}}{W_{\text{eff,y}}} + \frac{N_{Ed}}{A_{\text{eff}}} + \frac{M_{fz,Ed}}{W_{fz}} \right) \leq f_{yb} / \gamma_{M1}
\]  

(3)

Where, \( A_{\text{eff}} \) is the effective area of the cross-section for only uniform compression; \( f_y \) is the yield strength; \( M_{fz,Ed} \) is the bending moment in the free flange due to the lateral load \( q_{h,Ed} \); \( W_{\text{eff,y}} \) is the effective section modulus of the cross-section for only bending about the y - y axis; \( W_{fz} \) is the gross elastic section modulus of the free flange plus the contributing part of the web for bending about the z-z axis; \( \chi_{LT} \) is the reduction factor for lateral torsional buckling (flexural buckling of the free flange).
Figure 3. A beam-column on elastic foundation

3.2 Design requirements of GB50018

As shown in table 1, the favorable restraints provided by roof panels are not considered when calculating the bending bearing capacity of purlins under wind uplift load using the Chinese national standard GB50018. The purlin is treated as a biaxial flexural member, and the classical lateral torsional buckling theory of beam is adopted to calculate the stable bearing capacity. The following equations are adopted:

\[
\frac{M_x}{W_{ex}} + \frac{M_y}{W_{ey}} \leq f
\]  

(4)

\[
\frac{M_x}{\varphi_{bx} W_{ex}} + \frac{M_y}{\varphi_{by} W_{ey}} \leq f
\]  

(5)

Where, \(M_x\) is the bending moment about the major main axis; \(M_y\) is the bending moment about the minor main axis; \(W_{ex}\) is the section modulus of the effective cross-section for bending about the major main axis; \(W_{ey}\) is the section modulus of the effective cross-section for bending about the major main axis; \(\varphi_{bx}\) is the reduction factor for lateral torsional buckling.

3.3 Convention for member axes

The most widely used cold-formed thin walled purlins are C-sections and Z-sections. As shown in figure 4a), the wind uplift load is always perpendicular to the plane of roof panel, which is perpendicular to the plane of purlin web. And the major main axis of the C-section is also perpendicular to the plane of purlin web. So under the wind uplift load, the C-section purlin can be treated as an uniaxial flexural member. However, the major main axis of the Z-section is not perpendicular or parallel to the plane of purlin web. So under the wind uplift load, the Z-section purlin should be treated as a biaxial flexural member. Considering that the roof panels can provide some extent of torsional restraint to prevent purlin from twisting sharply, the axes perpendicular and parallel to the purlin web are adopted as the axis convention by EC3, AISI S100 and AS/NZS. But the Chinese Specifications, GB50018 and CECS, still use the main axes as the axis convention. Thus, as shown in figure 4b), for Z-section purlin under wind uplift load, the distribution of normal stress on the cross section calculated according to the Chinese specification may be a little different from which being calculated using other specifications.
4. Comparison of bending bearing capacity calculated using different specifications

In order to understand the influence to the calculation results caused by the aforementioned differences among those different specifications, the bending bearing capacity of 18 rows of commonly used Z-purlins calculated according to these specifications are analyzed using the programming software Matlab. The purlin is assumed simply supported and with the upper flange through-fastened to trapezoidal steel sheeting by self-drilling screws. The thickness of the steel sheeting is 0.6mm, and the space between screws is 200mm. The purlin spans are 6m, 7.5m and 9m. Three different lateral bracing arrangements are considered, which are setting zero and two rows of sag rods per span. The calculation results are shown in table 2 and 3.

4.1 Bending bearing capacity of Z-purlins without sag-rods

The calculation results of Z-purlins without sag-rods are listed in table 2. Through the comparison of the calculation results, it is clear that although different design methods are adopted by EC3 and AISI to consider the lateral and torsional restraints of roof panels, the results are nearly the same. But the calculation results of GB50018 are much lower than the others because it doesn’t consider any useful restraint of roof panels at all. The results of AS/NZS are greater than that of AISI, because the $R$ values of AS/NZS are also a little higher.

| Purlin size | Span length | GB | EC3 | AISI | AS | GB | AISI | AS |
|-------------|-------------|----|-----|------|----|----|------|----|
| Z140x2.0    | 6           | 1.14 | 5.16 | 5.25 | 5.62 | 0.221 | 1.017 | 1.090 |
| Z140x2.2    | 6           | 1.33 | 5.63 | 5.75 | 6.16 | 0.236 | 1.021 | 1.094 |
| Z140x2.5    | 6           | 1.63 | 6.30 | 6.50 | 6.96 | 0.259 | 1.032 | 1.106 |
| Z160x2.0    | 6           | 1.69 | 5.94 | 6.09 | 6.52 | 0.284 | 1.026 | 1.099 |
| Z160x2.2    | 6           | 1.94 | 6.49 | 7.01 | 7.51 | 0.300 | 1.080 | 1.157 |
| Z160x2.5    | 6           | 2.36 | 7.25 | 8.28 | 8.88 | 0.325 | 1.143 | 1.225 |
| Z180x2.0    | 7.5         | 1.73 | 7.20 | 6.72 | 7.75 | 0.240 | 0.933 | 1.076 |
| Z180x2.2    | 7.5         | 1.99 | 8.07 | 7.47 | 8.62 | 0.247 | 0.926 | 1.068 |
| Z180x2.5    | 7.5         | 2.42 | 9.11 | 8.96 | 10.34 | 0.266 | 0.984 | 1.135 |
| Z200x2.0    | 7.5         | 1.88 | 7.14 | 7.76 | 8.96 | 0.264 | 1.088 | 1.255 |
| Z200x2.2    | 7.5         | 2.17 | 8.16 | 8.63 | 9.96 | 0.265 | 1.058 | 1.220 |
| Z200x2.5    | 7.5         | 2.62 | 9.18 | 10.33 | 11.92 | 0.286 | 1.125 | 1.299 |
| Z220x2.0    | 9           | 1.73 | 7.77 | 6.92 | 10.38 | 0.223 | 0.891 | 1.337 |
| Z220x2.2    | 9           | 1.99 | 9.07 | 7.70 | 11.54 | 0.220 | 0.848 | 1.273 |
| Z220x2.5    | 9           | 2.42 | 10.64 | 9.02 | 13.53 | 0.227 | 0.848 | 1.272 |
| Z250x2.0    | 9           | 1.93 | 7.47 | 8.11 | 12.17 | 0.259 | 1.086 | 1.628 |
| Z250x2.2    | 9           | 2.22 | 8.69 | 9.21 | 13.82 | 0.255 | 1.061 | 1.591 |
| Z250x2.5    | 9           | 2.68 | 10.37 | 10.78 | 16.17 | 0.259 | 1.040 | 1.560 |
| Average value | -          | -    | -    | -    | -    | 0.258 | 1.011 | 1.249 |
| Mean square error | -      | -    | -    | -    | -    | 0.028 | 0.089 | 0.179 |

4.2 Bending bearing capacity of purlin with two rows of sag-rods per span

The calculation results of Z-purlin with two rows of sag-rods per span are listed in table 3. Through the comparison of the calculation results, it is clear that the calculation results of GB50018 are nearly 20% greater than that of EC3, though the restraints of roof panels are not considered by GB50018. This is because the lateral calculation length of purlin is very short when setting two rows of sag-rods per span, hence the reduction factor for lateral torsional buckling, which is mainly determined by the lateral calculation length, is very large, sometimes even close to 1.0. The results of AS/NZS are still the largest, because the $R$ value is 1.0 when two rows of sag-rods are set per span, which means the bending bearing capacity of purlin is equal to the yielding strength of the section.
Table 3. Bending bearing capacity of Z-purlins with two rows of sag-rods

| Purlin size | Span length | GB (EC3) | EC3 | CECS (EC3) | AS (EC3) |
|-------------|-------------|----------|-----|------------|----------|
| Z140x2.0   | 6           | 6.29     | 4.91| 4.60       | 7.50     |
| Z140x2.2   | 6           | 7.12     | 5.40| 5.07       | 8.22     |
| Z140x2.5   | 6           | 8.28     | 6.13| 5.60       | 9.28     |
| Z160x2.0   | 6           | 8.22     | 6.87| 5.79       | 8.70     |
| Z160x2.2   | 6           | 9.30     | 7.61| 6.44       | 10.01    |
| Z160x2.5   | 6           | 10.97    | 8.09| 7.36       | 11.83    |
| Z180x2.0   | 7.5         | 9.55     | 7.22| 6.65       | 10.33    |
| Z180x2.2   | 7.5         | 10.81    | 8.75| 7.46       | 11.50    |
| Z180x2.5   | 7.5         | 12.77    | 10.05| 8.57      | 13.78    |
| Z200x2.0   | 7.5         | 11.40    | 8.24| 7.57       | 13.84    |
| Z200x2.2   | 7.5         | 12.93    | 9.72| 8.52       | 15.39    |
| Z200x2.5   | 7.5         | 14.27    | 11.25| 9.18      | 15.89    |
| Z220x2.0   | 9           | 13.19    | 9.03| 8.18       | 16.22    |
| Z220x2.2   | 9           | 14.95    | 10.63| 9.22     | 18.43    |
| Z220x2.5   | 9           | 17.70    | 13.04| 10.71    | 21.56    |
| Z250x2.0   | 9           | 13.19    | 9.03| 8.18       | 16.22    |
| Z250x2.2   | 9           | 14.95    | 10.63| 9.22     | 18.43    |
| Z250x2.5   | 9           | 17.70    | 13.04| 10.71    | 21.56    |

**Average value**

| Mean square error | - | - | - | - | - | 0.071 | 0.040 | 0.158 |

5. Conclusions

Focusing on the bending bearing capacity of purlins under wind uplift load, through the comparison of design requirements and calculation results of five different specifications, the distinctions of design methods, calculation models and considerations on restraint effects of roof panels among these specifications are discussed. The results show that for purlins without sag-rods, the bending bearing capacity calculated according to GB50018 are very conservative, because the beneficial effects of restraint from steel roof panels are totally neglected. However, the calculation results of EC3 and AISI are nearly the same, though different design methods are adopted by these two specifications to consider the lateral and torsional restraints provided by roof panels. For purlins with two rows of sag-rods, the calculation results of GB50018 are nearly 20% greater than that of EC3.

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