SPECIFICATION

DIRECT STRENGTH METHOD
OF
COLD-FORMED STEEL DESIGN

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Appendix 1: Design of Cold-Formed Steel Structural Members by the Direct Strength Method

Preface

Users of the main Specification are aware of the complexity involved in current design calculations for cold-formed steel members. A new design method: Direct Strength, has been created that aims to alleviate the current complexity, ease calculation, provide a more robust and flexible design procedure, and integrate with available, established, numerical methods.

Advantages of the Direct Strength Method of design include

- everyday design improvements:
  - no effective properties for strength,
  - no element calculations,
  - no iteration for beams (webs),
  - gross properties of the section used for strength calculations,

- theoretical improvements:
  - interaction of elements in local buckling (e.g., web/flange interaction) is accounted for,
  - distortional buckling is explicitly treated in the design process,
  - reduction in systematic error in portions of the main Specification,

- improvements in applicability and scope:
  - is applicable to wider group of cross-sections than the main Specification,
  - provides rational analysis procedure for sections not previously covered,
  - allows and encourages greater cross-section optimization,

- important philosophical changes:
  - numerical methods and rational analysis brought to everyday design, and
  - integrates known behavior into a straightforward design procedure.

Use of the Direct Strength Method requires (1) determination of the elastic buckling behavior of the member and (2) using that information in a series of ultimate strength curves to predict the strength. The commentary to this Appendix provides full details of rational analysis methods, both traditional hand solutions and numerical solutions, that may be used to accurately calculate the elastic buckling behavior necessary for step 1. A freely available program, developed in part with AISI funds: CUFSM (www.ce.jhu.edu/bschafer/cufsm) complete with tutorials and examples is available to aid in the elastic buckling calculation. This appendix provides the ultimate strength curves along with appropriate safety and resistance factors necessary for the strength prediction of step 2.

The Direct Strength Method is verified with same data and care as the main Specification, and thus no loss of reliability is inherent in its use. The procedure employs the same underlying empirical assumptions as the effective width method used in the main Specification: ultimate strength is a function of elastic buckling stress (or load) and the yield stress (or load) of the material. The method has been extensively explored for beams and columns only. The applicability of the provided provisions is detailed in the General Provisions of this Appendix.
1.1 General Provisions

1.1.1 Applicability

The provisions of this Appendix are applicable for determination of the nominal axial ($P_n$) and flexural ($M_n$) strength of cold-formed steel members. Sections 1.2 and 1.3 present a method applicable to all cold-formed steel beams and columns consistent with the rational analysis clause of Section A1.1(b) of the main Specification. Those members meeting the geometric and material limitations of Section 1.1.1.1 for columns and 1.1.1.2 for beams have been pre-qualified for use, and calibrated $\Omega$ and $\phi$ factors apply.

Currently, the Direct Strength method provides no explicit provisions for members in: shear, combined bending and shear, web crippling, combined bending and web crippling, or combined axial load and bending (beam-columns). Further, no provisions are given for structural assemblies or connections and joints. As detailed in A1.1 of the main Specification, the provisions of the main Specification, when applicable, shall be used for all cases listed above.

For inapplicable members or situations, obvious extensions to the Direct Strength method may exist (e.g. in shear, in certain structural assemblies, etc.). Users who choose to employ such extensions to the Direct Strength method are subject to the same provisions as any other rational analysis procedure as detailed in Section A1.1(b) of the main Specification: (1) applicable provisions of the main Specification must be followed when they exist, (2) reduced $\phi$ and increased $\Omega$ factors are employed for strength when rational analysis is conducted.
### 1.1.1.1 Pre-qualified Columns

Unperforated columns that fall within the geometric limitations given below have been pre-qualified and may use the higher $\phi$ and $\Omega$ factors of section 1.2. Dimensional limitations listed in gray and italicized are not mandatory. All given dimensions are out-to-out.

<table>
<thead>
<tr>
<th>Column Type</th>
<th>Limitations</th>
</tr>
</thead>
</table>
| **Lipped Channel**           | $34 < h/t < 472$  
|                              | $18 < b/t < 159$  
|                              | $4 < d/t < 33$  
|                              | $0.7 < h/b < 5.0$  
|                              | $0.05 < d/b < 0.41$  
|                              | $0 \sim 90$ deg.  
|                              | $340 < E/F_y < 1020$  
|                              | $(29$ ksi $< F_y < 86$ ksi$) or (200$ MPa $< F_y < 593$ MPa)$            |
| **Lipped Channel with Web Stiffener(s)** | $105 < h/t < 489$  
|                              | $68 < b/t < 160$  
|                              | $6 < d/t < 33$  
|                              | $1.3 < h/b < 2.7$  
|                              | $d/b \sim 0.20$  
|                              | $0.4 < d_1/d_2 < 2.8$ (one or two intermediate stiffeners)  
|                              | $340 < E/F_y < 820$  
|                              | $(36$ ksi $< F_y < 86$ ksi$) or (248$ MPa $< F_y < 593$ MPa)$            |
| **Zee (Zed)**                | $76 < h/t < 137$  
|                              | $30 < b/t < 56$  
|                              | $0 < d/t < 36$  
|                              | $1.5 < h/b < 2.7$  
|                              | $0.00 < d/b < 0.73$  
|                              | $0 \sim 50$ deg.  
|                              | $E/F_y \sim 590$  
|                              | $(F_y \sim 50$ ksi$) or (F_y \sim 345$ MPa)$                           |
| **Rack Upright**             | $34 < h/t < 51$  
|                              | $12 < b/t < 22$  
|                              | $5 < d/t < 8$  
|                              | $2.1 < h/b < 2.9$  
|                              | $1.6 < b_2/d < 2.0$ (b_2=small outstand parallel to b)  
|                              | $d_2/d \sim 0.3$ (d_2=second lip parallel to d)  
|                              | $E/F_y \sim 343$  
|                              | $(F_y \sim 86$ ksi$) or (F_y \sim 593$ MPa)$                           |
| **Hat**                      | $33 < h/t < 50$  
|                              | $12 < b/t < 20$  
|                              | $4 < d/t < 6$  
|                              | $1.0 < h/b < 1.2$  
|                              | $d/b \sim 0.13$  
|                              | $428 < E/F_y < 952$  
|                              | $(31$ ksi $< F_y < 69$ ksi$) or (214$ MPa $< F_y < 476$ MPa)$            |
1.1.1.2 Pre-qualified Beams

Unperforated beams that fall within the geometric limitations given below have been pre-qualified and may use the higher $\phi$ and $\Omega$ factors of section 1.3. Dimensional limitations listed in gray and italicized are not mandatory. All dimensions are out-to-out. All beams are bent about a horizontal axis as shown in the figures.

<table>
<thead>
<tr>
<th>Type</th>
<th>Limitations</th>
</tr>
</thead>
</table>
| Channels              | $43 < h/t < 321$  
12 $< b/t < 75$  
0 $< d/t < 34$  
1.5 $< h/b < 17.0$  
0.0 $< d/b < 0.70$  
44 deg. $< \theta < 90$ deg.  
421 $< E/F_y < 983$  
(30 ksi $< F_y < 70$ ksi) or (207 MPa $< F_y < 483$ MPa) |
| Lipped Channels with Web Stiffener | $200 < h/t < 358$  
29 $< b/t < 58$  
14 $< d/t < 17$  
5.5 $< h/b < 11.7$  
0.27 $< d/b < 0.56$  
$\theta \sim 90$ deg.  
578 $< E/F_y < 670$  
(44 ksi $< F_y < 51$ ksi) or (303 MPa $< F_y < 352$ MPa) |
| Zee (Zed)             | $114 < h/t < 183$  
45 $< b/t < 71$  
10 $< d/t < 16$  
2.5 $< h/b < 4.1$  
0.15 $< d/b < 0.34$  
36 deg. $< \theta < 90$ deg.  
440 $< E/F_y < 921$  
(32 ksi $< F_y < 67$ ksi) or (220 MPa $< F_y < 462$ MPa) |
| Hats (Decks)          | $37 < h/t < 97$  
92 $< b_{c}/t < 467$  
0 $< d_{c}/t < 26$  
0.14 $< h/b_{c} < 0.87$  
0.44 $< b_{c}/2b_{t} < 2.7$  
0 $< n < 4$  
492 $< E/F_y < 656$  
(45 ksi $< F_y < 60$ ksi) or (310 MPa $< F_y < 414$ MPa) |
| Trapezoids (Decks)    | $51 < h/t < 203$  
61 $< b_{c}/t < 231$  
0.42 $< (h/sin\theta_{b}c)/b_{c} < 1.91$  
0.55 $< b_{c}/2b_{t} < 1.69$  
0 $< n_{c} < 2$ (compression flange stiffener)  
0 $< n_{w} < 2$ (web stiffener/fold)  
0 $< n_{t} < 2$(tension flange stiffener)  
52deg. $< \theta_{h} < 84$deg.  
310 $< E/F_y < 686$  
(43 ksi $< F_y < 95$ ksi) or (296 MPa $< F_y < 655$ MPa) |
1.1.2 Elastic Buckling

Elastic buckling of cold-formed steel members under flexure or axial load may involve three distinct buckling modes: local, distortional, and overall. Rational analysis is required for determination of the elastic buckling loads and or moments used in the Direct Strength method, namely, for columns $P_{cr}$, $P_{crd}$, and $P_{cre}$ of 1.2, and for beams $M_{cr}$, $M_{crd}$, and $M_{cre}$ of 1.3. For a given beam or column all three modes may not exist. In this case, the non-existent mode should be ignored in the calculations of sections 1.2 and 1.3.

The rational analysis method(s) employed for determination of the elastic buckling loads and or moments shall provide the solution for the member as a whole. Compatibility and equilibrium between the elements that comprise a member must be included, or if neglected in part or in full, must be done so in a conservative fashion for the entire member. The rational analysis model employed should be able to reproduce classic benchmark problems (at a minimum a simply supported plate in pure compression and pure bending) with no more than a 1% over-estimation of the exact buckling solution. The same methods used to produce benchmark solutions (e.g., the element sizes and density for the finite element or finite strip method) must also be employed in actual solutions on members.

The commentary to this Appendix discusses a variety of applicable rational analysis methods for elastic buckling prediction. Two methods which meet the criteria of this section are discussed in full detail: (1) a conservative extension of conventional hand solutions, and (2) a numerical implementation of the finite strip method. Complete formulas for the former method are provided in the Appendix. For the latter method a free program complete with tutorials and examples is referenced and discussed in the Appendix. Elastic buckling is a well-defined and repeatable calculation; a variety of other methods: finite element, boundary element, generalized beam theory, can provide a reliable solution. Any methods meeting the criteria of this section are acceptable as a rational analysis method for elastic buckling prediction.

1.1.3 Deflection Determination

The bending deflection at any moment ($M_a$) due to service loads, may be determined by reducing the gross moment of inertia, $I_g$, to an effective moment of inertia for deflection, per:

$$I_{eff} = I_g(M_{na}/M_a) \quad \text{(Eq. 1.1.3-1)}$$

where: $M_{na} = M_n$ of section 1.3, determined with $M_y$ replaced by $M_a$

$M_a = \text{moment due to service loads on member of interest} \quad (M_a \leq M_y)$

The axial deflection at any service load ($P_a$) may be determined by reducing the gross area, $A_g$, to an effective area for deflection, per:

$$A_{eff} = A_g(P_{na}/P_a) \quad \text{(Eq. 1.1.3-2)}$$

where: $P_{na} = P_n$ of section 1.2, determined with $P_y$ replaced by $P_a$

$P_a = \text{service load of interest} \quad (P_a \leq P_y)$
1.2 Columns

The nominal axial strength, $P_n$, is the minimum of $P_{ne}$, $P_{nl}$, and $P_{nd}$ as given below.

For columns meeting the geometric criteria of section 1.1.1.1, $\phi$ and $\Omega$ are as follows:

<table>
<thead>
<tr>
<th></th>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_c$ (ASD)</td>
<td>1.80</td>
<td>2.00</td>
</tr>
<tr>
<td>$\phi_c$ (LRFD)</td>
<td>0.85</td>
<td>0.75</td>
</tr>
<tr>
<td>$\phi_c$ (LSD)</td>
<td>0.80</td>
<td>0.70</td>
</tr>
</tbody>
</table>

For all other columns, standard rational analysis values are used: $0.9\phi$ and $1.1\Omega$, as follows:

<table>
<thead>
<tr>
<th></th>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_c$ (ASD)</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>$\phi_c$ (LRFD)</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td>$\phi_c$ (LSD)</td>
<td>0.70</td>
<td>0.70</td>
</tr>
</tbody>
</table>

1.2.1 Flexural, Torsional, or Flexural-Torsional Buckling

The nominal axial strength, $P_{ne}$, for flexural, torsional, or flexural-torsional buckling is

$$P_{ne} = \begin{cases} (0.658\lambda_c^2)P_y & \text{for } \lambda_c \leq 1.5 \\ (0.877\lambda_c^2)P_y & \text{for } \lambda_c > 1.5 \end{cases}$$  \hspace{1cm} (Eq. 1.2.1-1)

where $\lambda_c = \sqrt{\frac{P_y}{P_{cre}}}$ \hspace{1cm} (Eq. 1.2.1-3)

$P_y = A_gF_y$ \hspace{1cm} (Eq. 1.2.1-4)

$P_{cre}$ = Minimum of the critical elastic column buckling load in flexural, torsional, or flexural-torsional buckling

1.2.2 Local Buckling

The nominal axial strength, $P_{nl}$, for local buckling is

$$P_{nl} = \begin{cases} P_{ne} & \text{for } \lambda_l \leq 0.776 \\ 4.0\left(\frac{P_{cre}}{P_{ne}}\right)^{0.4} + 4.0\left(\frac{P_{ne}}{P_{cre}}\right)^{0.4}P_{ne} & \text{for } \lambda_l > 0.776 \end{cases}$$  \hspace{1cm} (Eq. 1.2.2-2)

where $\lambda_l = \sqrt{\frac{P_{ne}}{P_{cre}}}$ \hspace{1cm} (Eq. 1.2.2-3)

$P_{cre}$ = Critical elastic local column buckling load

1.2.3 Distortional Buckling

The nominal axial strength, $P_{nd}$, for distortional buckling is

$$P_{nd} = \begin{cases} P_y & \text{for } \lambda_d \leq 0.561 \\ 25.01\left(\frac{P_{ne}}{P_y}\right)^{0.6}P_y & \text{for } \lambda_d > 0.561 \end{cases}$$  \hspace{1cm} (Eq. 1.2.3-2)

where $\lambda_d = \sqrt{\frac{P_y}{P_{crd}}}$ \hspace{1cm} (Eq. 1.2.3-3)

$P_{crd}$ = Critical elastic distortional column buckling load
1.3 Beams

The nominal flexural strength, $M_n$, is the minimum of $M_{ne}$, $M_{nl}$ and $M_{nd}$ as given below.

For beams meeting the geometric criteria of section 1.1.1.2, $\phi$ and $\Omega$ are as follows:

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_b$ (ASD)</td>
<td>$\phi_b$ (LRFD)</td>
</tr>
<tr>
<td>1.67</td>
<td>0.90</td>
</tr>
</tbody>
</table>

For all other beams, standard rational analysis values are used: $0.9 \phi$ and $1.1 \Omega$, as follows:

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_b$ (ASD)</td>
<td>$\phi_b$ (LRFD)</td>
</tr>
<tr>
<td>1.85</td>
<td>0.80</td>
</tr>
</tbody>
</table>

1.3.1 Lateral-Torsional Buckling

The nominal flexural strength, $M_{ne}$, for lateral-torsional buckling is

- for $M_{cre} < 0.56M_y$, $M_{ne} = M_{cre}$ (Eq. 1.3.1-1)
- for $2.78M_y > M_{cre} > 0.56M_y$, $M_{ne} = \frac{10}{9} M_y \left(1 - \frac{10M_{cre}}{36M_{ne}}\right)$ (Eq. 1.3.1-2)
- for $M_{cre} > 2.78M_y$, $M_{ne} = M_y$ (Eq. 1.3.1-3)

where $M_y = S_g F_y$, where $S_g$ is referenced to the extreme fiber in first yield

$M_{cre}$ = Critical elastic lateral-torsional buckling moment

1.3.2 Local Buckling

The nominal flexural strength, $M_{nl}$, for local buckling is

- for $\lambda_l \leq 0.776$, $M_{nl} = M_{ne}$ (Eq. 1.3.2-1)
- for $\lambda_l > 0.776$, $M_{nl} = \left(1 - 0.15 \left(\frac{M_{crf}}{M_{ne}}\right)^{0.4} \left(\frac{M_{cre}}{M_{ne}}\right)^{0.4}\right) M_{ne}$ (Eq. 1.3.2-2)

where $\lambda_l = \sqrt{\frac{M_{ne}}{M_{crf}}} = $ Critical elastic local buckling moment

1.3.3 Distortional Buckling

The nominal flexural strength, $M_{nd}$, for distortional buckling is

- for $\lambda_d \leq 0.673$, $M_{nd} = M_y$ (Eq. 1.3.3-1)
- for $\lambda_d > 0.673$, $M_{nd} = \left(1 - 0.22 \left(\frac{M_{crd}}{M_y}\right)^{0.5} \left(\frac{M_{cre}}{M_y}\right)^{0.5}\right) M_y$ (Eq. 1.3.3-2)

where $\lambda_d = \sqrt{\frac{M_y}{M_{crd}}}$ = Critical elastic distortional buckling moment

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