LIMIT STATES DESIGN IN STRUCTURAL STEEL

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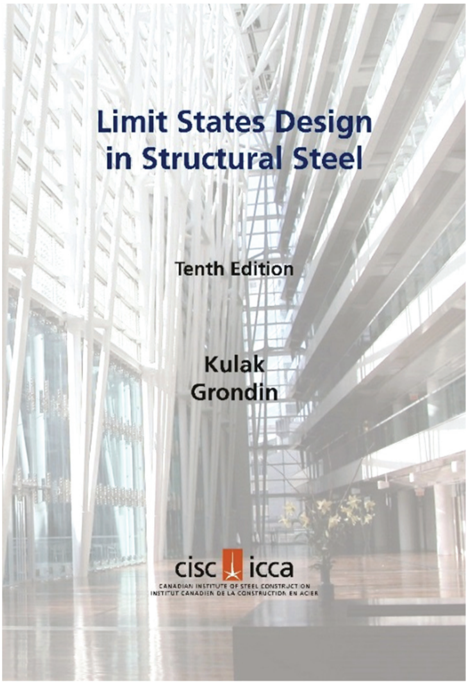
Revisions and updates incorporated into the 10th Edition, 2nd Revised Printing (2018) of *Limit States Design in Structural Steel* are highlighted on the following pages. Minor editorial corrections are not shown.



Since live load deflections are usually checked under specified live load due to use and occupancy, the serviceability limit state load factor applied to the specified live load is 1.0.

The load factor α_i in Equation 1.1 accounts for the variability in loading as well as the probability of having loads of different sources acting simultaneously on the structure. In this respect, loads are separated into two groups: the principal loads and the companion loads. Five load cases are considered, as outlined in Table 1.3. Since dead loads are always part of the loads applied on a structure, they are always considered to be a principal load. With the exception of dead loads, the principal loads are factored by a factor greater than or equal to 1.0. When dead loads counteract the effect of the other applied loads, such as for overturning, uplift, sliding, or stress reversal, the load factor is taken as 0.9. The load factor applied to the companion load is less than or equal to 1.0 in order to account for the reduced probability of having the companion load at its maximum value when the principal load is at its maximum value. When the live load is used as a companion load (load cases 3, 4 and 5 in Table 1.3), the live load factor shall be increased to 1.0 for structures or parts of structures used for storage, housing of equipment, or consist of service rooms. The load factor on dead load and earthquake load is 1.0 when earthquake loading is considered. This reflects the level of conservatism used in the definition of the earthquake load in the National Building Code [1.1].

In addition to the load combinations in Tables 1.3 and 1.4, the effect of factored loads 1.5H, 1.0P, and 1.25T shall be included in the load combinations presented in Table 3 when any of these loads affects structural safety. The load H is the permanent load due to lateral earth pressure, P is the prestress load, and T represents the load effects due to contraction, expansion, or deflection.

Table 1.3 – Load combinations without crane loads for ultimate limit states

Case	Load combination	
	Principal loads	Companion loads
1	1.4D	—
2	(1.25D or 0.9D) + 1.5L	1.0S or 0.4W
3	(1.25D or 0.9D) + 1.5S	1.0L or 0.4W
4	(1.25D or 0.9D) + 1.4W	0.5L or 0.5S
5	1.0D + 1.0E	0.5L + 0.25S

The load combinations in Table 1.4 are used for industrial structures where crane loads must be considered as part of the loading. The load C is the live load exclusive to crane load. The load C_d consists of the self-weight of all the cranes positioned to create the maximum effect and C_7 is the crane bumper impact load. This is a horizontal force applied in the direction of the crane runway and it is created by a moving crane bridge striking the end stop. The load L_{xc} is the live load other than the crane loads. More information about crane loading and design of crane-supporting steel structures can be found in reference [1.11].

Table 1.4 – Load combinations with crane loads for ultimate limit states

Case	Load Combination	
	Principal Loads	Companion Loads
1	(1.25D or 0.9D) + (1.5C+1.0 L_{xc})	1.0S or 0.4W
2	(1.25D or 0.9D)+(1.5 L_{xc} +1.0 C)	1.0S or 0.4W
3	(1.25D or 0.9D) + 1.5S	1.0C + 1.0 L_{xc}
4	(1.25D or 0.9D) + 1.4W	1.0C + 0.5 L_{xc}
5	(1.25D or 0.9D) + C_7	—
6	1.0D + 1.0E	1.0 C_d + 0.5 L_{xc} +0.25S

As an example of the use of specified loads and factored loads, Figure 1.2 illustrates the basic design checks required for a beam. The deflection, Δ_s , when the beam is subjected to bending moment, M_s , computed using specified loads, must be within the limits specified in CSA–S16–14. When the bending moment reaches M_f , computed using factored loads, the beam would fail or be on the verge of failure. The value of the deflections at this load level is not usually of interest.

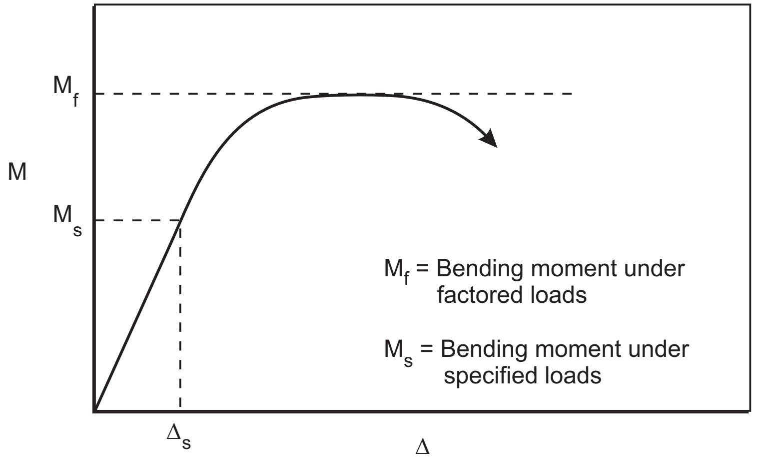


Figure 1.2 – Moment vs. Deflection Curve

More information on limit states design is widely available and reference [1.8] can be used as a convenient starting point. Chapters 3 to 11 inclusive contain more detailed information on the design of members using limit states design procedures. The design function is facilitated in practice by the use of design aids such as handbooks, manuals [1.4] and computer programs.

Example 1.1

Given

The loading conditions for a roof beam in a school building are to be determined using limit states design. The specified loads are:

References

1.1 National Research Council of Canada, "National Building Code of Canada 2015," Ottawa, Ontario.

1.2 Canadian Standards Association, CSA–S16–14, "Design of Steel Structures," Mississauga, Ontario, 2014.

1.3 Canadian Standards Association, CSA-G40.20–13/G40.21–13, "General Requirements for Rolled or Welded Structural Quality Steel / Structural Quality Steel," Mississauga, Ontario, 2013.

1.4 Canadian Institute of Steel Construction, "Handbook of Steel Construction," 11th Edition, Markham, Ontario, 2016.

1.5 National Research Council of Canada, "The National Building Code of Canada 2015," Structural Commentaries (Part 4 of Division B), Ottawa, Ontario.

1.6 Canadian Standards Association, CSA–S6–14, "Canadian Highway Bridge Design Code," Mississauga, Ontario, 2014.

1.7 American Railway Engineering and Maintenance-of-Way Association, "2013 Manual for Railway Engineering, Volume 2 – Structures," Landover, MD, 2013.

1.8 Segui, W.T., "Steel Design," Thomson Canada Ltd, 2007.

1.9 Canadian Standards Association, CAN3–Z234.1–00, "Metric Practice Guide," Mississauga, Ontario, 2000.

1.10 Gewain, R.G., N.R. Iwankiw, F. Alfawakhiri and G. Frater, "Fire Facts for Steel Buildings," CISC/AISC, 2006.

1.11 MacCrimmon, R.A., “Crane-Supporting Steel Structure – Design Guide,” third edition, Canadian Institute of Steel Construction, 2015.

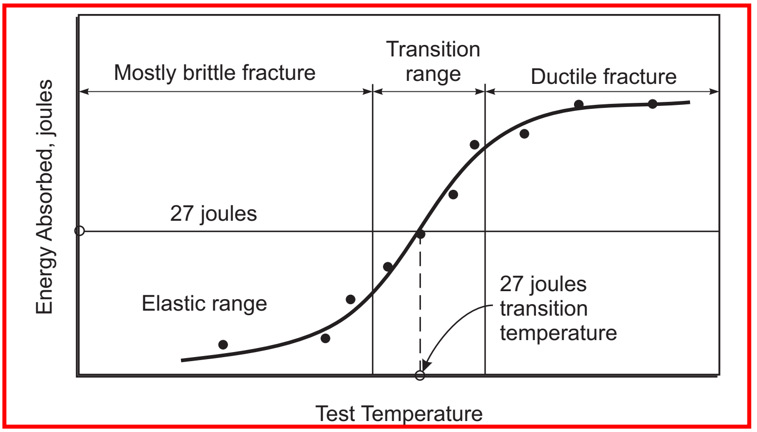


Figure 2.4 – Energy vs. Temperature Transition Curve

Structural steels differ greatly in toughness. A fully killed, fine-grain steel with a suitable chemical composition or a specially heat-treated steel will exhibit considerable toughness. Several tough steels have been developed for use in Canada, where low ambient temperatures are an inescapable design condition for exposed structures. Under low temperature conditions, a structural member with a notch or other stress raiser, subject to a significant tensile stress and high loading rate, may be susceptible to brittle fracture. Selection of a tough steel for this design condition will help to minimize the possibility of brittle fracture. Guidance in the selection of a suitable steel is provided in Annex L of S16 and in references [2.3] and [2.6].

Most steel structures are fabricated and erected with the aid of welding. Many structures are assembled with welds and bolts—but seldom with bolts only. Thus, the weldability of the structural steel to be used is usually a design consideration. All structural steels are weldable in the sense that two pieces can be connected with a weld. However, the ease with which the welding can be accomplished, the cost, and the quality of the welds differ from steel to steel. Standard specifications [2.1, 2.7] provide information on those structural steels considered readily weldable, while other sources [2.3, 2.8] provide more details on the weldability of steel.

Atmospheric corrosion resistance may be an important design consideration when structures are exposed to the weather. Special steels have been developed that eliminate the need for paint or other protective coatings under most atmospheric conditions [2.1, 2.3].

The modulus of elasticity (Young's modulus) for structural steel is practically constant, and the value usually used is 200 000 MPa. Poisson's ratio (ratio of transverse to longitudinal strain) is taken as 0.30 in the elastic range. The density of steel is $7850\ \mathrm{kg/m^3}$ and the coefficient of thermal expansion, at atmospheric temperatures, is approximately $11.7\times10^{-6}\ /\ ^{\circ}\mathrm{C}$. The coefficient of thermal expansion corresponds to the change in length per unit of length for a change of one degree of temperature.

Table 8.1– Width-Thickness Ratios for Beam-Columns of I-Sections

Element	Section Classification		
	Class 1	Class 2	Class 3
Axial compression and bending about the strong (major) axis			
Flanges	$\frac{b_{el}}{t} \le \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{200}{\sqrt{F_y}}$
Webs	$\frac{h}{w} \le \frac{1100}{\sqrt{F_y}}\left(1 - 0.39\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1700}{\sqrt{F_y}}\left(1 - 0.61\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$
Axial compression and bending about the weak (minor) axis			
Flanges	$\frac{b_{el}}{t} \le \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \le \frac{340}{\sqrt{F_y}}$
Webs and $C_f > 0.4\,\phi\,C_y$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$
Webs and $C_f \le 0.4\,\phi\,C_y$	$\frac{h}{w} \le \frac{1100}{\sqrt{F_y}}\left(1 - 1.31\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1700}{\sqrt{F_y}}\left(1 - 1.73\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$
Axial compression and bi-axial bending			
$M_{fy}/S_y > 0.9\,M_{fx}/S_x$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$
$M_{fy}/S_y \le 0.9\,M_{fx}/S_x$	Use above limits for combined axial compression and strong axis bending		

The slenderness provisions for the web element of the cross-section are not so simple because the amount of the web plate that is under compression will depend upon the magnitude of the axial compressive force present. If the axial force approaches zero, the beam-column is really just a beam and the web slenderness limit should be the same as that for a beam. In this case, only one-half of the web plate is in compression. On the other hand, if the axial force is very high, the beam-column approaches the behaviour of a column, that is, almost the entire web is in compression. Obviously, in this case the slenderness limit for the beam-column web should be the same as that for the web of a column. Table 8.1 lists the web slenderness limits for these limiting cases. As developed in Section 5.4, the h/w ratios for beam webs are limited to $1100/\sqrt{F_y}$, $1700/\sqrt{F_y}$, or $1900/\sqrt{F_y}$, for Class 1, Class 2, or Class 3, respectively. For webs of I-sections bent about their weak axis, the limit for classes 1 and 2 sections is more stringent than for a column since the web could be subjected to uniform compression and expected to maintain its yield capacity until the flanges reach their full yield resistance. Therefore, the h/w for the web of beam-columns bent about their weak axis or in bi-axial bending is lower than $670/\sqrt{F_y}$ (Section 4.1).

The rules for the slenderness of the webs of beam-columns must lie between the limiting cases of beams (negligible axial force) and columns (negligible bending moment). In order to reflect the level of axial force present, the S16–14 standard provides the slenderness limitations listed in Table 8.1.