# Classification of Structural Steel Sections

## *Summary*

Four Classes of rolled shapes characterize the shapes available for structural steel:

**Class 1: Plastic Design Sections**
Class 1 section can be subjected with a bending moment equal to plastic moment, and - given adequate cross-sectional stiffening – can rotate local, that means a plastic hinge is formed.

**Class 2: Compact Sections**
Class 2 section can be subjected with a bending moment equal to plastic, however, cannot undergo any local rotation.

**Class 3: Non-Compact Sections**
Class 3 section can be subjected with a bending moment equal to yield moment. The cross section starts buckling after the most outer fibres have yielded.

**Class 4: Slender Sections**
Class 4 section fails locally before yield moment can be obtained.

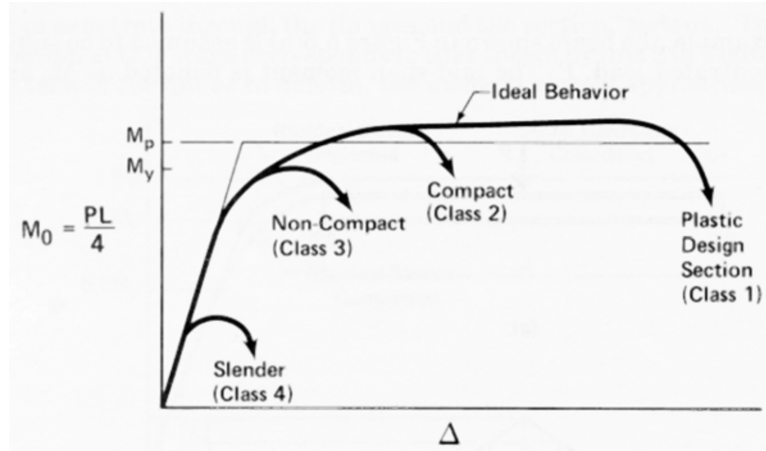
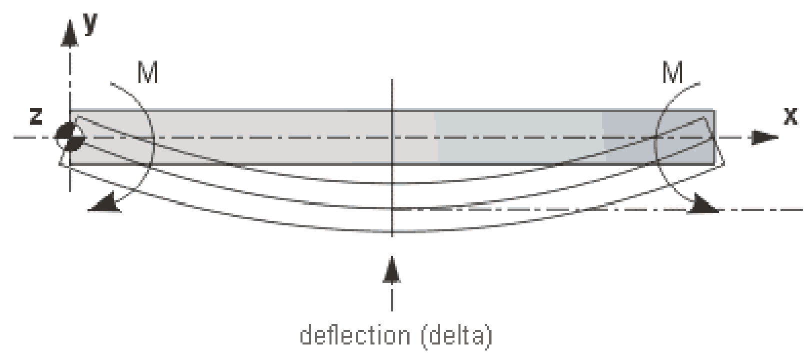



Fig. CS-1: Load-Deflection Relationships of Different Classes



## General

Structural steel section (rolled or welded shapes) under compressive loads are designated as Class 1, 2, 3, or 4 in order to simplify the designers' work. The classification enables the engineer to choose a particular shape with a distinct moment-deflection behaviour without going through an extensive failure analysis (buckling) for the individual elements of the shape such as webs, flanges, legs, etc.

## Stability of Shape Elements

Buckling is the stability failure mode for flat plates subjected to in-plane loads. The common way to solve this problem is to start from a general differential equation, which describes the equilibrium of a plate under loads.

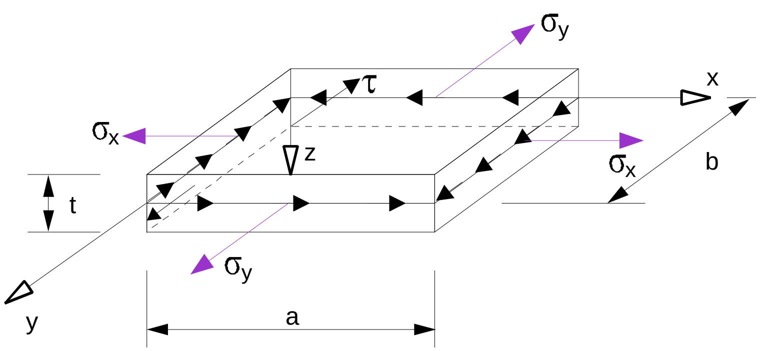



Fig. C-1: Flat Plate Element and Stress Notations

For the definition of tensile stresses being positive the equation is

$$\frac{EI}{1-\mu^2}\left(\frac{\delta^4\omega}{\delta x^4}+2\frac{\delta^4\omega}{\delta x^2\delta y^2}+\frac{\delta^4\omega}{\delta y^4}\right) \qquad \text{(C-1)}$$

$$= t\left(\sigma_x\frac{\delta^2\omega}{\delta x^2}+2\tau\frac{\delta^2\omega}{\delta x\delta y}+\sigma_y\frac{\delta^2\omega}{\delta y^2}\right)$$

where

I = moment of inertia of cross sectional area of a unit strip of plate
t = thickness of plate
μ = Poisson's ratio
σ, τ = applied normal, shear stress
ϖ = deflection of a point in the middle plane of plate in z-direction

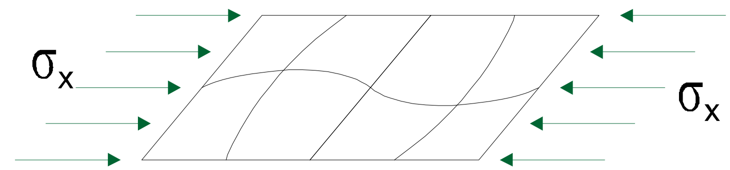



Fig.C-2: Buckling Shape of Flat Plate under Uniaxial Compressive Stress

This equation can be reduced by introducing boundary conditions along its edges and only uniaxial load in compression $\sigma_x$. For simple supports or hinges along all four edges it is

$$\frac{EI}{1-\mu^2}\left(\frac{\delta^4\omega}{\delta x^4}+2\frac{\delta^4\omega}{\delta x^2\delta y^2}+\frac{\delta^4\omega}{\delta y^4}\right)+\sigma_x t\frac{\delta^2\omega}{\delta x^2}=0 \qquad \text{(C-2)}$$

This equation can be satisfied by

$$\omega=\omega_{mn}\sin\frac{m\pi x}{a}\sin\frac{n\pi y}{b} \qquad \text{(C-3)}$$

with

m = 1, 2, 3, …
n = 1, 2, 3, …

This means we are assuming a sinusoidal shape for the deflected shape of the plate.

If we consider that a multiple-wave buckling will occur for arbitrary ratios of $a$ and $b$, we can solve the previous equation to

$$\sigma_x t = \frac{\pi^2 EI}{1-\mu^2}\left(\frac{m}{a}+\frac{n^2}{m}\frac{a}{b^2}\right)^2 \qquad \text{(C-4)}$$

The plate will buckle first for a value of $n = 1$, as this is the lowest value for which a buckled configuration exists. The buckled shape is one half wave transverse to the direction of loading. $m$ is number of half-waves in the direction of loading.

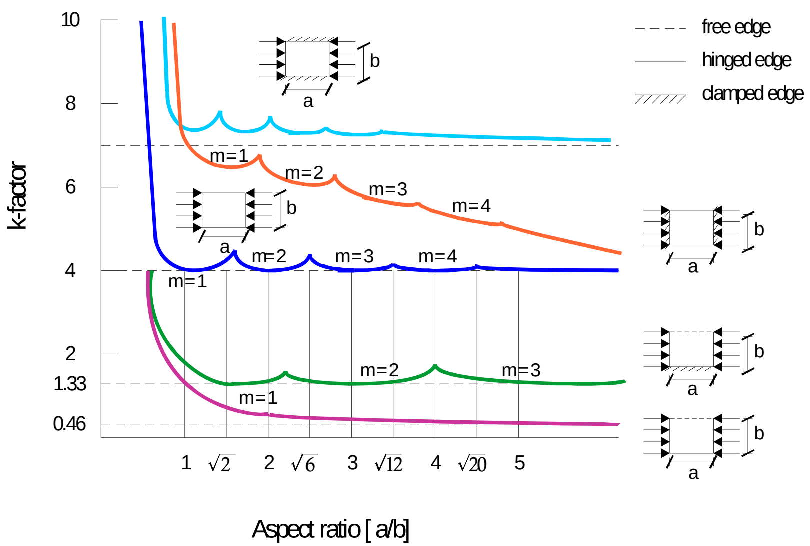



Fig. C-3: Plate Buckling Coefficients for Different Boundary Conditions

$$\sigma_x t = \frac{\pi^2 EI}{1-\mu^2 a^2}\left(m+\frac{1}{m}\frac{a^2}{b^2}\right)^2 \qquad \text{(C-5)}$$

It is convenient to write the equation in a different form using

$$I = \frac{t^3}{12} \qquad \text{(C-6)}$$

$$k = \left(m\frac{b}{a}+\frac{1}{m}\frac{a}{b}\right)^2 \qquad \text{(C-7)}$$

to the equation describing the critical value of the stress

$$\sigma_{crit} = \frac{k\pi^2 E}{12\left(1-\mu^2\right)\left(\frac{b}{t}\right)^2} \qquad \text{(C-8)}$$

The aspect ratio ($a$ / $b$) of the plate and the k-value are the governing parameter of the plate buckling behaviour. One can see the k has a constant minimum for the plates with loaded edges hinged. In case of clamping of the loaded edge the k value is approximating the value for the un-clamped version for longer plates. With an increase in plate length the influence of clamping gradually decreases.

Fig. C-4 shows the theoretical k-factors for elements of rolled steel sections. Note that the HSC uses in the shape tables a value of $b$ which determines the total flange width, while Table 1, HSC, Width-Thickness Ratios, uses only half the flange width, $b$ / 2, and calls this value $b$. In the following figure this value is denoted $b_o$.

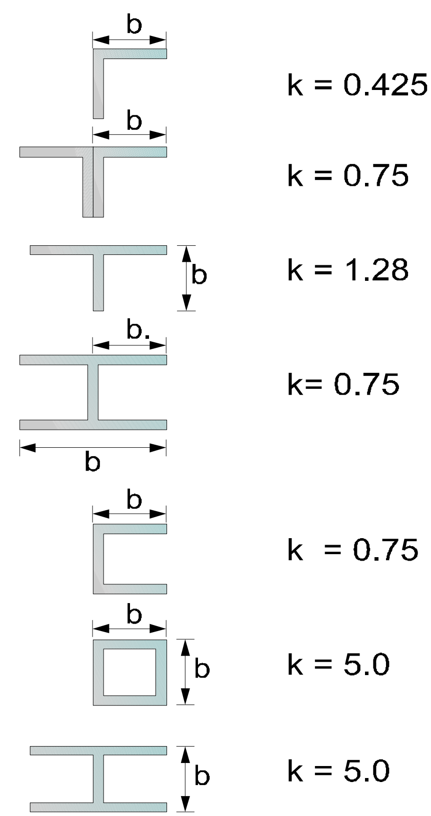



Fig. C-4: Theoretical K-Factors for Rolled Steel Sections

## Postbuckling Behaviour of Flat Plates

Under certain boundary conditions the load on a buckled plate can be increased even further. However, the parts of the plates which are heavily undulated (in the plate supported at the edges = inner part) cease to carry load. Nevertheless the resultant resistance might be higher in the post-buckled stage than in the elastic case. Now a so-called effective or reduced plate width is used for the analysis. The effective width is carrying the postbuckling stress resulting in a postbuckling behaviour. The postbuckling behaviour has little importance for the class specifications of rolled structural shapes, and will be discussed in a later chapter on plate girders.
The electronic textbook does not include the version of a shape classification template as shown in the following but a more condensed version. In order to show the complete selection process the following template is included in the text. It should be noted that both worksheets will yield the same result.

## General Section Classification

The following formulae are from Table 1 in the HSC. A section class is determined for each section or part of section. In the case of one or more classes existing for a section, the larger section class shall always govern.

| **Description of element** | **Class 1 plastic design** | **Class 2 compact** | **Class 3 non-compact** |
|---|---|---|---|
| Legs of angles and elements supported along one edge, except as noted | - | - | $\frac{b}{t} \le \frac{200}{\sqrt{F_y}}$ |
| Angles in continuous contact with other elements; plate-girder stiffeners | - | - | $\frac{b}{t} \le \frac{200}{\sqrt{F_y}}$ |
| Stems of T-sections | $\frac{b}{t} \le \frac{145}{\sqrt{F_y}}$ ♣ | $\frac{b}{t} \le \frac{170}{\sqrt{F_y}}$ ♣ | $\frac{b}{t} \le \frac{340}{\sqrt{F_y}}$ |
| Flanges of I- or T-sections; plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact ♠ | $\frac{b}{t} \le \frac{145}{\sqrt{F_y}}$ | $\frac{b}{t} \le \frac{170}{\sqrt{F_y}}$ | $\frac{b}{t} \le \frac{200}{\sqrt{F_y}}$ |
| Flanges of channels | - | - | $\frac{b}{t} \le \frac{200}{\sqrt{F_y}}$ |
| Flanges of rectangular hollow structural sections | $\frac{b}{t} \le \frac{420}{\sqrt{F_y}}$ | $\frac{b}{t} \le \frac{525}{\sqrt{F_y}}$ | $\frac{b}{t} \le \frac{670}{\sqrt{F_y}}$ |
| Flanges of box sections, flange cover plates and diaphragm plates, between lines of fasteners or welds | $\frac{b}{t} \le \frac{525}{\sqrt{F_y}}$ | $\frac{b}{t} \le \frac{525}{\sqrt{F_y}}$ | $\frac{b}{t} \le \frac{670}{\sqrt{F_y}}$ |
| Perforated cover plates | - | - | $\frac{b}{t} \le \frac{840}{\sqrt{F_y}}$ |

♥ Class 4 includes all sections not otherwise specified.

♣ See Clause 11.1.3.

♠ Can be considered as Class 1 or Class 2 sections if angles are continuously connected by adequate mechanical fasteners or welds, and if there is an axis of symmetry in the plane of loading.

| | | | |
|---|---|---|---|
| Webs | $\frac{h}{\omega} \le \frac{1100}{\sqrt{F_y}}\left(1 - 0.39\frac{C_t}{C_y}\right)$ | $\frac{h}{\omega} \le \frac{1700}{\sqrt{F_y}}\left(1 - 0.61\frac{C_t}{C_y}\right)$ | $\frac{h}{\omega} \le \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_t}{C_y}\right)$ |
| Circular hollow sections, flexural | $D_t \le \frac{13000}{F_y}$ | $D_t \le \frac{18000}{F_y}$ | $D_t \le \frac{23000}{F_y}$ |

For the HSS, the type of member (square or circular) must be determined, and then the appropriate class used.

## Applicable CAN/CSA Clauses

11. Width-Thickness Ratios: Elements in Compression
    - 11.1 Classification of Sections
    - 11.2 Maximium Width-Thickness Ratios of Elements Subject to Compression
    - 11.3 Width and Thickness

## CISC Commentary

### 11.WIDTH-THICKNESS RATIOS: ELEMENTS IN COMPRESSION

Clause 11.1 identifies four categories of cross-sections, Class 1 through Class 4, based upon the width-thickness ratios of the elements of the cross-section in compression that are needed to develop the desired flexural behaviour. With the ratios given in Table 1 of Clause 11 for Classes 1, 2, or 3, the respective ultimate limit states will be attained prior to local buckling of the plate elements. These ultimate limit states are: Class 1---maintenance of the plastic moment capacity (beams), or the plastic moment capacity reduced for the presence of axial load (beam-columns), through sufficient rotation to fulfill the assumption of plastic analysis; Class 2---attainment of the plastic moment capacity for beams, and the reduced plastic moment capacity for beam-columns, but with no requirement for rotational capacity; Class 3Cattainment of the yield moment for beams, or the yield moment reduced for the presence of axial load for beam-columns. Class 4---have plate elements which buckle locally before the yield strength is reached.

### Elements in flexural compression

The requirements given in Figure 2-7 for elements of Class 1, 2, and 3 sections in flexural compression (and also for axial compression), particularly those for W-shapes are based on both experimental and theoretical studies. For example, the limits on flanges have both a theoretical basis (Kulak *et al.* 1990; ASCE 1971; Galambos 1988) and an extensive experimental background (Haaijer and Thurlimann 1958; Lay 1965; Lukey and Adams 1969). For webs in flexural compression the limits $1100/\sqrt{F_y}$ , $1700/\sqrt{F_y}$ , and $1900/\sqrt{F_y}$ for Class 1, 2 and 3 respectively when $C_f/C_y$ is zero come from both theory and tests on Class 1 sections (Haaijer and Thurlimann 1958) but mostly from test results for Class 2 and 3 sections (Holtz and Kulak 1973 and 1975).

For circular hollow sections in flexure, see Stelco (1973) for the requirements for Class 1 and Class 2 sections and Sherman and Tanavde (1984) for Class 3.

### Elements in axial compression

The distinction between classes based on moment capacity does not apply to axially loaded members as the plate elements need only reach a strain sufficient for the plate elements to develop the yield stress. This strain is affected by the presence of residual stresses but there is no strain gradient across the cross-section as there is for members subject to flexure. Thus for webs, in Table 1 for each of Classes 1, 2 and 3 when $C_f/C_y = 1.0$ the limit on h/w is the same value of about $670/\sqrt{F_y}$ as given in Figure 2-7. The width-thickness limit for the flanges of axially loaded columns, based on the same argument, is the same as for Class 3 beam flanges, i.e., $/\sqrt{F_y}$ (Dawe and Kulak 1984). As well the limit on the D/t ratio of $23\,000/F_y$ (Winter 1970) for circular hollow sections in axial compression is the same irrespective of the Class.

### Elements in compression due to bending and axial load

In Figure 2-8, the requirements for webs in compression ranging from compression due to pure bending to that due to pure compression are plotted. Because the amount of web under compression varies from complete (columns) to one-half (beams), the depth-to-thickness limits vary as a function of the amount of axial load. The results presented here reflect the latest research results, particularly those of Dawe and Kulak (1986), and are significantly more liberal than previous limits (Perlynn and Kulak 1974; Nask and Kulak 1976).

## Class 4 Sections

Sections used for columns, beams, or beam-columns may be composed of elements whose width-to-thickness ratios exceed those prescribed for Class 3 provided that the resistance equations are adjusted accordingly. These sections, called Class 4, should be evaluated according to the rules given in Clause 13.3 or 13.5 as applicable.

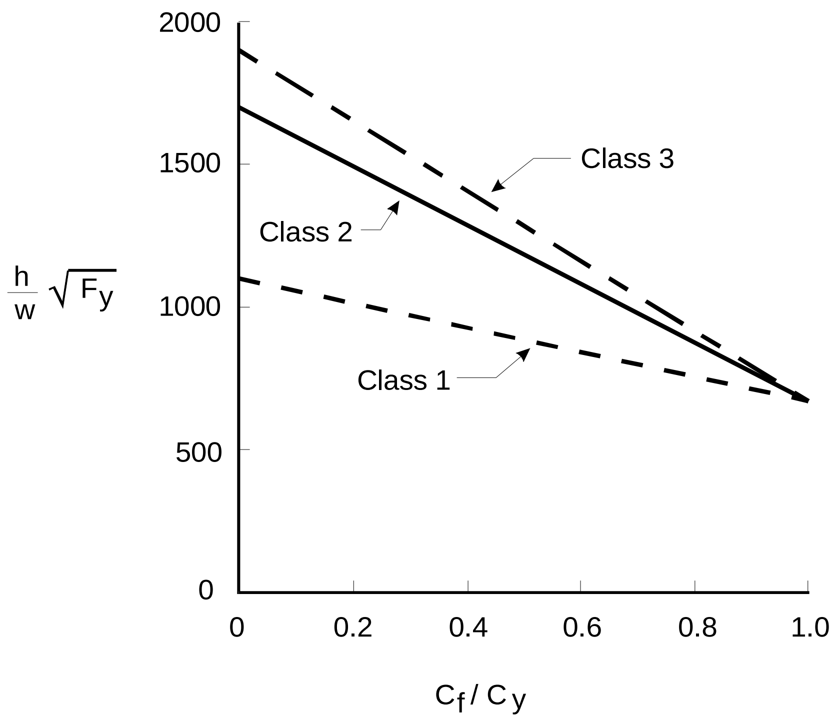



Fig. 2-8: Width-Thickness Ratios for Webs

## *Gimme Five*

A Class 4 section will carry a moment equal to:

| | |
|---|---|
| plastic moment | [_] |
| yield moment | [_] |
| less than yield moment | [_] |

Classes are existing to ensure that:

| | |
|---|---|
| the designer selects an appropriate section according to applied analysis | [_] |
| to confuse the young engineer | [_] |
| to complicate life | [_] |

The web width-thickness ratio limit requires consideration of :

| | |
|---|---|
| thickness and width | [_] |
| width and stress ratio | [_] |
| thickness, width and compressive load | [_] |

The flanges of box section width-thickness ratio limit requires consideration of:

| | |
|---|---|
| thickness and width | [_] |
| width and stress ratio | [_] |
| thickness, width and compressive load | [_] |

When is double symmetry of a section required:

| | |
|---|---|
| Class 1 in flexure | [_] |
| Class 1 in compression | [_] |
| Class 2 in tension | [_] |