Design of Steel Plate and Box Girders

This design summary has been prepared by Devan Fitch in the framework of the graduate course CIVL510 “Behaviour of Steel Structures” by S.F. Stiemer.

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1. Introduction:

The references used in writing this report are mostly bridge related and hence this report is somewhat biased towards steel plate and box girder bridges. However the vast majority of observations, recommendations and equations are equally applicable to non-bridge related applications of steel plate and box girders.

Steel girders are typically rolled beams, steel plate, or box girders. A rolled beam is a steel girder which has been formed by hot-rolling. The most common type of rolled beam used is the I-type. Rolled beams have the advantage of quick erection, straight forward fabrication, and an overall simplicity of design.

To maintain an economy of material, rolled beams are sometimes equipped with a rectangular plate, or cover plate, at the bottom flange. The cover plate increases the ability of the stringer to resist flexure without having to use a large size rolled beam or plate girder.

A plate girder, like a rolled beam, has an I-type cross section. Rather than being hot-rolled, however, the girder is constructed from steel plate elements which are connected together with welds, bolts, or rivets. A greater economy of material can be obtained as the designer has the ability to specify the section properties of the stringer to accommodate the local forces (e.g. flange thickness).

Variation in plate sizes may represent a girder with the least weight; however, this may not be the most economical girder, due to the increase in fabrication costs which are associated with excessive variations in plate sizes. Plate girders gain an advantage over rolled beams as span lengths become great.

The box girder is a form of plate girder which utilizes four or five rather than three plates. Since box girders possess excellent torsional stiffness, they do not usually require secondary members to provide bracing.

Bridge superstructures are a common application of steel plate and box girders. Box girders while more expensive to fabricate than plate girders because of their complexity, have a number of significant advantages, particularly for longer spans. Firstly because of the shape of the box, the top flange itself can act as the decking without the need for a concrete deck. They can also be designed with an aerodynamic shape, again making them ideal for long spans.

This document outlines the CISC CAN/CSA S16.01 (referred to as the Standard for the remainder of this report) approach to the design of steel plate and box girders. Where appropriate the relevant clause number is listed next to any equations given in this text.
Advantages of plate girders versus box girders:

- Easier to fabricate
- Easier to handle in shop
- Jigs not required
- Fewer field splice bolts since bottom flange is narrower than for a box girder
- Lower unit price
- Lighter piece weight erection where crane capacity is a concern

Advantages of box girders versus plate girders:

- More efficient load distribution due to high torsional stiffness
- Efficient where girder depth must be minimized
- Efficient for curved alignments
- Less area exposed to airborne road salts
- Less horizontal surface onto which corrosion products can deposit
- Fewer bearings possible with multiple box girders
- Fewer pieces to erect
- Improved aesthetics
2. Buckling of Plates

The webs and flanges of both steel plate and box girders are comprised of flat steel plates. The design of these girders consists of assigning appropriate boundary conditions to each of these plate elements, ensuring that each plate element does not fail due to local buckling, yielding or an interaction of the two and that the girder does not fail due to global buckling.

It should be noted however that it is desirable for global buckling to be the first mode of failure reached as the loads on a girder are increased past the anticipated service loading. Global buckling is associated with large deformations which give warning of failure and is generally less brittle and sudden than local buckling.

In order to understand the design of steel plate and box girders, it is important to understand the principles behind plate buckling.

2.1 Buckling of Unstiffened plates:

The buckling stresses are obtained from the concept of bifurcation of an initially perfect structure. In practice, the response of the structure is continuous, due to the inevitable presence of initial imperfections.

For a plate to be considered slender, the in-plane dimensions, $a$, and $b$, need to be significantly greater than the plate thickness, $t$. The dimension, $b$, is usually taken as the direction transverse to the main direction of in-plane loading.

2.1.1 Uniaxial Uniform Compression:

Long rectangular plates:

The elastic critical stress of a long plate segment, $\sigma_c$, is determined by the plate width-to-thickness ratio, $b/t$, by the restraint conditions along the longitudinal boundaries, and by the elastic material properties. It is expressed as

$$
\sigma_c = k \frac{\pi^2 E}{12(1-\nu^2)(b/t)^2}
$$

(2.1)

![Figure 2.1 Uniform compression coefficients, k, for equation 1.1](image)

Figure 2.1 Uniform compression coefficients, $k$, for equation 1.1

The buckling coefficient, $k$, is determined by a theoretical critical load analysis, and is a function of plate geometry and boundary
conditions. The values given in Figure 2.1 are lower bounds, with the actual value depending on the plate aspect ratio, \( m = a/b \). This is because a perfect plate under in-plane compression will buckle into \( m \) square half waves if the plate aspect ratio is an integer, as this corresponds to the lowest energy mode and for non-integer ratios the plate will in theory have a higher buckling stress.

![Figure 2.2 Buckling of a plate with an aspect ratio of 3:1](image)

**Figure 2.2** Buckling of a plate with an aspect ratio of 3:1

Short plates:

When a plate is relatively short in the direction of the compressive stress (i.e. \( a/b << 1 \)), the critical stress may be conservatively estimated by assuming that a unit width of plate behaves like a column.

Postbuckling strength:

Local buckling causes a loss of stiffness and a redistribution of stresses. Membrane tensions are set up, which resist the growth of deflection and give the plate postbuckling strength.

Uniform edge compression in the longitudinal direction results in a non-uniform stress distribution after buckling, and the plate derives almost all of its stiffness from the longitudinal edge supports.

Elastic postbuckling stiffness is measured in terms of the apparent modulus of elasticity \( E^* \) (the ratio of the average stress carried by the plate to the average strain). For simply supported edges \( E^* = 0.5E \).

There is a decrease in stress at the center of the panel because of the reduction in in-plane stiffness along the center line of the plate caused by the lateral deflection. This reduction in stress due to buckling action gives rise to a semi-empirical method of estimating the maximum strength of plates by the use of the effective width concept.

![Figure 2.3 Definition of effective width](image)

**Figure 2.3** Definition of effective width

Here it is assumed that the maximum edge stress acts uniformly over two ‘strips’ of plate and the central region is unstressed as shown in Figure 2.3. This width is evaluated so that the total force carried by the plate is the same for the actual response.
2.1.2 Pure Bending:

Equation 2.1 is again used to calculate the critical buckling load, with substitution of the appropriate buckling coefficient, $k$. The buckling coefficient for a plate in bending is very significantly influenced by the fact that half (in linear response) of the load is applied in tension.

Unlike the case of edge compression, the buckling mode is composed of a combination of several waveforms, making the buckling analysis of shear more complex.

$$\tau_c = k_s \frac{\pi^2 E}{12(1-\nu^2)(b/t)^2}$$  \hspace{1cm} (2.2)

Shear buckling coefficients:

1. Plate simply supported on four edges:
   $$\alpha \leq 1: \quad k_s = 4.00 + \frac{5.34}{\alpha^2}$$  \hspace{1cm} (2.3)
   $$\alpha \geq 1: \quad k_s = 5.34 + \frac{4.00}{\alpha^2}$$  \hspace{1cm} (2.4)

2. Plate clamped on four edges:
   $$\alpha \leq 1: \quad k_s = 5.60 + \frac{8.98}{\alpha^2}$$  \hspace{1cm} (2.5)
   $$\alpha \geq 1: \quad k_s = 8.98 + \frac{5.60}{\alpha^2}$$  \hspace{1cm} (2.6)

Pure bending plate buckling coefficients:

- $k = 23.9$ edges simply supported
- $k = 39.6$ unloaded edges fixed
- $k = 0.85$ top edge free, bottom edge simply supported
- $k = 2.15$ top edge free, bottom edge fixed

2.1.3 Pure Shear:

In a plate subject to pure shear, there exists tension and compression stresses equal in magnitude to the shear stress and inclined at 45°. The destabilizing influence of compressive stresses is resisted by tensile stresses in the perpendicular direction. The critical stress can be obtained by substituting $\tau_c$ and $k_s$ for $\sigma_c$ and $k$ in equation 2.1.
3. Plate clamped on two opposite edges and simply supported on the other two edges:

\[
\alpha \leq 1: \quad k_s = 5.61 + \frac{8.98}{\alpha^2} - 1.99\alpha
\]
\[
\alpha \geq 1: \quad k_s = 5.61 + \frac{8.98}{\alpha^2} - 1.99\alpha
\]

\[(2.7)\]
\[(2.8)\]

**2.1.4 Combined Stresses:**

The Crane Code (CMAA Specifications #70 & #74, revised 2000) gives useful equations for the buckling coefficient for simply supported plates subject to combined in-plane bending and compression according to the cases shown in Figure 2.6.

<table>
<thead>
<tr>
<th>Case</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Compressive stresses, varying as a straight line, (0 &lt; \psi &lt; 1)</td>
</tr>
<tr>
<td>2</td>
<td>Compressive and tensile stresses; varying as a straight line and with the compression predominating, (-1 &lt; \psi &lt; 0)</td>
</tr>
<tr>
<td>3</td>
<td>Compressive and tensile stresses; varying as a straight line, with equal edge values, (\psi = -1) or with predominantly tensile stresses, (\psi &lt; -1)</td>
</tr>
</tbody>
</table>

Figure 2.6 Buckling coefficients for combined bending and compression

The critical stress is then calculated using the buckling coefficient, \(k\) (obtained using equations 2.9 – 2.13), in equation 2.1.

**Case 1:**

\[
\alpha \geq 1 \quad k = \frac{8.4}{\psi + 1.1}
\]
\[
\alpha < 1 \quad k = \left[\frac{\alpha + \frac{1}{\alpha}}{\psi + 1.1}\right]^2 - \psi
\]
\[
\psi = 2.1
\]
\[
(2.9)
\]
\[
(2.10)
\]

**Case 2:**

\[
k = \left[(1 + \psi)k'\right] - (\psi k'') + \left[10\psi (1 + \psi)\right]
\]
\[
(2.11)
\]

\(k'\) is the buckling coefficient for \(\psi = 0\) (case 1) and \(k''\) is the buckling coefficient for \(\psi = -1\) (case 3)

**Case 3:**

\[
\alpha \geq \frac{2}{3} \quad k = 23.9
\]
\[
\alpha < \frac{2}{3} \quad k = 15.87 + \frac{1.87}{\alpha^2} + 8.6\alpha^2
\]
\[
(2.12)
\]
\[
(2.13)
\]

With predominant tension replace the width of the plate, \(b\), by 2 times the width of the compression zone for calculation of \(\alpha\) and \(\sigma_c\).

The crane code also gives an interaction failure criteria for plates subject to in-plane bending, compression and shear. First the comparison stress, \(\sigma_{1k}\), is calculated.
\[
\sigma_{1k} = \frac{\sqrt{\sigma^2 + 3\tau^2}}{\left[1 + \psi\right] + \left[\frac{3 - \psi \sigma}{4 \sigma_c}\right] + \left[\frac{\tau}{\tau_c}\right]^2}
\]  
(2.14)

If the resulting critical stress is below the proportional limit, \(\sigma_p\), buckling is said to be elastic. If the resulting value is above the proportional limit, buckling is said to be inelastic. For inelastic buckling the comparison stress is reduced to \(\sigma_{1KR}\).

\[
\sigma_{1KR} = \frac{\sigma_y \sigma_{1k}^2}{0.1836 \sigma_y^2 + \sigma_{1k}^2}
\]  
(2.15)

\[
\sigma_p = \sigma_y/1.32
\]  
(2.16)

The comparison stress is then used to calculate a safety factor, \(\vartheta_B\), which is then compared with allowable design factor values, DFB, for each load combination.

Elastic buckling

\[
\vartheta_B = \frac{\sigma_{1k}}{\sqrt{\sigma^2 + 3\tau^2}} \geq DFB
\]  
(2.17)

Inelastic buckling

\[
\vartheta_B = \frac{\sigma_{1KR}}{\sqrt{\sigma^2 + 3\tau^2}} \geq DFB
\]  
(2.18)

Design factor DFB requirements:

Case 1  \(DFB = 1.7 + 0.175(\Psi - 1)\)  
(2.19)
Case 2  \(DFB = 1.5 + 0.125(\Psi - 1)\)  
(2.20)
Case 3  \(DFB = 1.35 + 0.05(\Psi - 1)\)  
(2.21)

2.2 Buckling of stiffened Plates:

Figure 2.7 shows the stress state in each sub panel for a stiffened plate subject to bending and shear (compressive stresses can easily included by modifying the ratio of longitudinal stresses at the edge of each panel). Each sub panel can be checked for local buckling subject to these stresses using the buckling coefficients given in section 2.1.4.

A second mode of failure also needs to be checked, which is local buckling of the plate as a whole including the stiffeners.

The third and final mode of failure involves the local buckling of elements of the stiffeners themselves. This is usually avoided through adhering to slenderness limits.

Methods for the evaluation of critical loads for stiffened plates are presented in section 5.5.
3. Beam Classification

3.1 Moment-Curvature (M-κ) relationships for beams:

From elementary strength of materials, an elastic beam subject to a bending moment will have a moment curvature relationship given by:

\[ \kappa = \frac{M}{EI} = \frac{\varepsilon}{y} \rightarrow \varepsilon = \frac{My}{EI} \quad (3.1) \]

From this it can be seen that the strain distribution will be linear, increasing from zero at the neutral axis to a maximum at the outermost fibre. If the strain does not exceed the yield strain of the material, the stress distribution will also be linear, given as the product of the stiffness multiplied by the strain at any given point, this is shown as stress state 1 in Figure 3.1.

As the moment in the section is increased, the strain in the outermost fibre increases and eventually reaches the yield strain \( \varepsilon_y \) at which point the moment is given by:

\[ M_y = \sigma_y S \quad (3.2) \]

Where \( S \) denotes the section modulus \( (S = I/\bar{y}) \). As the moment is increased further, the strain exceeds the yield strain of the material, \( \varepsilon_y \), the stress cannot increase above the yield strength (if work hardening is ignored), and hence yielding begins to penetrate from the extreme fibres towards the interior of the cross section, this is shown as stress state 2 in Figure 3.1.

The strains will continue to increase until the cross-section is fully plastified, at this stage the full plastic moment, \( M_p \), has been developed, this is shown as stress state 3 in Figure 3.1. The plastic moment is computed by summing up the moments caused by the forces on each fibre at a stress \( \sigma_y \) to obtain:

\[ M_p = \sigma_y Z \quad (3.3) \]

Where the plastic modulus \( Z \), represents the first moment of area of the tension and compression zones about the neutral axis. Theoretically an infinite strain is required for a section to reach \( M_p \), but experiments have shown that for most wide flange shapes, 98% of the plastic moment \( M_p \) is obtained at strains of twice \( \varepsilon_y \).
Figure 3.2 represents the moment curvature relationship of the beam element, as the moment is increased. For moment values below $M_y$, the relationship is elastic and the slope of the $M-\kappa$ curve is equal to $EI$. Above $M_y$, yielding begins to penetrate through the flanges and the section ‘softens’. As the section continues to deform, the moment closely approaches $M_p$.

Figure 3.2 Moment curvature relationship for beam sections

3.2 Load deflection relationships for beams:

3.2.1 Local buckling:

The moment-curvature relationship reflects only the behaviour of a short element of beam length. To predict the behaviour of a complete member, the curvatures corresponding to a given bending moment distribution must be integrated along the member to determine the slopes and deflections. The idealized load deflection relationship for a beam subject to a central concentrated load $P$ is shown as the solid line in Figure 3.3.

![Figure 3.3 Load deflection relationships for beams](image)

However the idealized load-deflection relationship is not attained for all beams, as local buckling of the component members making up the element may occur prior to the attainment of the full plastic moment, $M_p$. This leads to the categorization of beams into four primary classes according to what load-deflection relationship their component elements will allow as shown in Figure 3.3.

Class 1: Plastic design section

The component elements should be such that the beam has the capacity to both attain its full plastic moment, $M_p$, and undergo large inelastic rotations, enabling the complete structure to
redistribute bending moments and reach the load carrying capacity as anticipated on the basis of a plastic analysis.

\[ M_r = \phi M_p = \phi Z F_y \]  \hspace{1cm} (3.4)13.5a

**Class 2: Compact section**

The component elements should be such that the beam has the capacity to attain its full plastic moment, \( M_p \), but is not required to undergo large inelastic deformations.

\[ M_r = \phi M_p = \phi Z F_y \]  \hspace{1cm} (3.5)13.5a

**Class 3: Non compact section**

The component elements should be such that the beam is capable of developing a moment resistance just equal to the yield moment \( M_y \).

\[ M_r = \phi M_y = \phi S F_y \]  \hspace{1cm} (3.6)13.5b

**Class 4: Slender section**

The component elements are such that they buckle locally at a moment less than \( M_y \) and the moment resistance is a function of the width-to-thickness ratios of the component elements. This class is subdivided into three categories. The first category, (i), contains those sections having both flange and web plates falling within Class 4. The second category, (ii), contains those sections having flanges meeting the requirements of Class 3 but having Class 4 webs. The third category, (iii), contains sections having web plates meeting the Class 3 requirements, but with compression flanges exceeding Class 3 limits.

The Standard advises the use of CSA Standard S136 for calculation of the moment resistances of Class 4(i) and 4(iii) sections. However it allows the use of equation 3.7 for Class 4(iii) sections as an alternative to CSA S136.

\[ M_r = \phi S_e F_y \]  \hspace{1cm} (3.7)13.5c

Where \( S_e \) is the effective elastic section modulus determined using an effective flange width. The effective width is \( 670t/\sqrt{F_y} \) for flanges supported along two edges parallel to the direction of stress and \( 200t/\sqrt{F_y} \) for flanges supported along one edge parallel to the direction of stress. For flanges supported along one edge, in no case shall b/t exceed 60.

For Class 4(ii) sections, equation 3.6 is used to calculate the basic moment resistance, which is then reduced to account for the effects of local buckling (softening) of the slender web. Most plate girders fall into this class of section, hence this equation is given in Chapter 4 of this report (Eq. 4.7), which describes the design of plate girders.
Through consideration of flange buckling, and web buckling of the component elements of a section, the Standard gives slenderness limits for both flanges and webs for each class of section.

Through consideration of flange buckling, and web buckling of the component elements of a section, the Standard gives slenderness limits for both flanges and webs for each class of section.

<table>
<thead>
<tr>
<th>Class</th>
<th>Plate girder flange in compression</th>
<th>Web in compression</th>
<th>Box girder flange under compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 1</td>
<td>$b_0/t \leq 145/\sqrt{F_y}$</td>
<td>$h/w \leq 1100/\sqrt{F_y} \xi_1$</td>
<td>$b_0/t \leq 525/\sqrt{F_y}$</td>
</tr>
<tr>
<td>Class 2</td>
<td>$b_0/t \leq 170/\sqrt{F_y}$</td>
<td>$h/w \leq 1700/\sqrt{F_y} \xi_2$</td>
<td>$b_0/t \leq 525/\sqrt{F_y}$</td>
</tr>
<tr>
<td>Class 3</td>
<td>$b_0/t \leq 200/\sqrt{F_y}$</td>
<td>$h/w \leq 1900/\sqrt{F_y} \xi_3$</td>
<td>$b_0/t \leq 670/\sqrt{F_y}$</td>
</tr>
</tbody>
</table>

with

\[
\xi_1 = (1-0.39*{C_f}(\phi*{C_y})) \\
\xi_2 = (1-0.61*{C_f}(\phi*{C_y})) \\
\xi_3 = (1-0.65*{C_f}(\phi*{C_y}))
\]

Table 3.1 Width-to-thickness limits

3.2.2 Lateral Torsional Buckling

Beams subject to flexure have much greater strength and stiffness in the plane in which the loads are applied (major principal axis) than in the plane of the minor axis. It has been assumed thus far that the strength of the beam is determined by the capacity of its cross-section, and this, in turn, is dependent on the local buckling capacity of its plate elements.

However if the beam is laterally unsupported, the strength may be governed instead by lateral-torsional buckling of the complete member, as shown in Figure 3.4. Beams are especially prone to this type of buckling during the construction phase, where lateral bracing are either absent or different in type from their permanent ones.

![Figure 3.4 Lateral-Torsional buckling motion](image)

At a given stage of loading, the cross-section may twist and bend about its weak axis, reducing its ultimate moment capacity due to large deflections and yielding, as shown in Figure 3.5 (notice the similarity to Figure 3.3).

The principle variable affecting lateral-torsional buckling strength is the distance between lateral braces. Other variables are: the type
and position of the loads, the restraints at the ends and at intermediate locations, the type of cross sections, continuity at supports, the presence or absence of stiffening devices that restrain warping at critical locations, the material properties, the magnitude and distribution of the residual stresses, prestressing forces, initial imperfections of geometry and loading, discontinuities in the cross section, cross-sectional distortion, and interaction between local and overall buckling.

![Figure 3.5 M-Δ Relationships for laterally unbraced beams](image)

Beams can also be classified in terms of the effect of lateral-torsion buckling on the ultimate moment capacity attainable, as shown in Figure 3.6.

![Figure 3.6 Beam Failure modes](image)

A stocky beam is defined as a beam which is able to reach its local buckling capacity before lateral buckling occurs. The local buckling capacity of Class 1 or 2 sections is $M_p$ and for Class 3 sections, $M_y$. A slender beam buckles laterally before the member yields, and the resistance to lateral-torsion buckling is based on full elastic action. For the intermediate beam, the bending moment at the instant before lateral buckling is sufficient to cause portions of the section to yield, thus the resistance to both lateral and twisting motions is reduced.

The Standard provides an equation for calculating the elastic lateral buckling strength of doubly symmetric beams.

$$M_u = \frac{\omega_2 \pi}{L} \sqrt{EI_y GJ + (\pi E/L)^2 I_y C_w}$$  \hspace{1cm} (3.8)^{13.6}

$$\omega_2 = 1.75 + 1.05\kappa + 0.3\kappa^2 \leq 2.5$$  \hspace{1cm} (3.9)^{13.6}
where $\kappa$ is the ratio of the smaller bending moment to the larger bending moment at opposite ends of the unbraced length. This equation provides a reasonable estimate of the moment at which lateral buckling will occur, provided that the strains in the member are less than the yield strain at the instant before buckling. Thus, Equation 3.8 is accepted as the basis for the design of slender members.

Due to relatively large residual stresses in the flange tips, yielding will occur when the applied moment reaches approximately two-thirds of the buckling capacity of the member, $M_p$ for Class 1 or 2 sections, and $M_y$ for Class 3 or 4 sections. Equation 3.8 is thus valid until $M_u$ reaches two-thirds of $M_p$ for Class 1 or 2 sections, and $M_y$ for Class 3 or 4 sections. The following is valid for doubly symmetrical members:

for $M_u \leq 0.67M_p$ (slender members):

$$M_r = \phi M_u \text{ for Classes 1 or 2} \quad (3.10)$$

for $M_u > 0.67M_p$ (stocky members):

$$M_r = 1.15\phi M_p \left(1 - \frac{0.28M_p}{M_u}\right) \leq \phi M_y \text{ Class 1 or 2} \quad (3.11)$$

for $M_u \leq 0.67M_p$

$$M_r = \phi M_u \text{ for Classes 3 or 4} \quad (3.12)$$

Lateral torsional buckling can be avoided by properly spaced and designed lateral bracing, or by using cross sections which are torsionally stiff, such as box-shaped sections or open-section beam groups connected intermittently by triangulated lacing or by diaphragms or by ensuring that the required design moment does not exceed the lateral-torsional buckling capacity.

The Standard does not give equations for the calculation of monosymmetrical sections such as box girders or plate girders with flanges of differing width. The Standard advises the use of equations given in ‘Guide to stability Design Criteria for metal structures’; however these are in the general form.

CHBDC gives worked examples based on the same expressions listed in ‘Guide to Stability Design Criteria for metal structures’ for monosymmetrical plate girders and open top box girders, these equations are presented in Appendix B.

Lateral torsional buckling of conventional closed box girders is unlikely due to their high torsional stiffness; however if the webs are close enough lateral torsional buckling may govern. The crane code provides a conservative limit to how close box girder webs should be placed.

Lateral torsional buckling of beams is a complex problem which has been studied intensively since the mid 19th century and continues to be an area of intensive research.
4. Design of plate girders

There are a variety of factors which the designer must play off of each other in order to realize the greatest economy of materials. Questions such as; should a thicker web be used or a thinner one with fewer stiffeners? Should the flange thickness be varied or kept constant? Should the girder be haunched?

The web of the plate girder can fail due to buckling, yielding or a combination of the two in either shear or bending. The web dimensions and stiffener spacing are chosen in order to ensure that there is an adequate safety margin with respect to these failure modes. The flange of the plate girder is sized to prevent local buckling or yielding.

4.1 Preliminary sizing

A preliminary design of a plate girder takes the weight of steel and the amount of fabrication into main consideration. The optimum depth for moment resistance according to allowable stress design is used to obtain an expression for the web depth, h.

\[ h \approx 540 \left( \frac{M_L}{F_y} \right)^{1/3} \]  \hspace{1cm} (4.1)

Alternatively, the web depth can be chosen to accommodate depth limitations due to architectural, deflection, clearance etc… requirements. Once an approximate web depth is chosen, an approximate flange area \( A_f \) (area of one flange) can be calculated, assuming that lateral torsional buckling will not govern the design, the flange material will be able to reach yield, the flange areas are concentrated at the top and bottom of the web and the contribution of the web to bending resistance can be neglected.

\[ A_f \approx \frac{M_L}{F_y h} \]  \hspace{1cm} (4.2)

These assumptions are generally valid, as the girder cross-section will almost always fall in the class 4(ii) category due to the slender web, and it is also generally advantageous to provide lateral support at intervals close enough that lateral-torsional buckling will not govern the design. Once an approximate web depth is chosen, an assumption that the web carries the entire shear will enable the web thickness, w, to be determined.

\[ A_w = \frac{V_f}{\phi F_s} = wh \]  \hspace{1cm} (4.3)

To determine a preliminary web thickness, w, from equation 4.3 requires an estimation of the shear resistance, \( F_s \). As will be discussed later in this chapter, the shear resistance is a function of
the web slenderness, h/w, and whether there web is stiffened however the maximum strength is given by equation 4.4.

\[ F_s = 0.66F_y \] (4.4)

By considering the maximum allowable web slenderness, the minimum web thickness, \(w_{\text{min}}\), can be obtained. The minimum web thickness for corrosion protection is 4.5mm. By considering the minimum web thickness for no reduction in flange stress and hence no reduction in moment resistance due to web slenderness, \(w_2\), can be obtained. A value of web thickness can then be chosen between these two values as a starting point for the detailed design.

\[ w_{\text{min}} = \frac{F_y h}{83000} \] (4.5)

\[ w_2 = \frac{\sqrt{F_y h}}{1900} \] (4.6)

### 4.2 Design of cross section for flexure

A plate girder subject primarily to bending moment usually fails by lateral-torsional buckling, local buckling of the compression flange, or yielding of one or both flanges. Buckling of the compression flange into the web (vertical buckling) has been observed in many tests.

#### 4.2.1 Lateral-torsional buckling (see chapter 3):

The Standard specifies for laterally supported members:

- Class 1,2: \(M_r = \phi M_p = \phi ZF_y\) (3.4)\(^{13.5a}\)
- Class 3,4(ii)*: \(M_r = \phi M_y = \phi SF_y\) (3.6)\(^{13.5b}\)
- Class 4(iii): \(M_r = \phi S_{\text{e}}F_y\) (3.7)\(^{13.5c}\)

Where \(S_{\text{e}}\) is the effective section modulus determined using an effective flange width of \(670*t/\sqrt{F_y}\) for flanges supported along two edges parallel to the direction of stress and an effective width of \(200*t/\sqrt{F_y}\) for flanges supported along one edge to the direction of stress. For flanges supported along one edge b/t should not exceed 60.

* For Class 4(ii) sections the moment resistance may need to be reduced using equation 4.7. The Standard specifies for laterally unsupported doubly symmetric members:

\[ M_u = \frac{\omega_2 \pi^2}{L} \sqrt{EI_y GJ + (\pi E/L)^2 I_y C_w} \] (3.8)\(^{13.6}\)

\[ \omega_2 = 1.75 + 1.05\kappa + 0.3\kappa^2 \leq 2.5 \] (3.9)\(^{13.6}\)

\[ M_u \leq 0.67M_p \] (Slender members):

\[ M_r = \phi M_u \text{ for Classes 1- 4} \] (3.10)\(^{13.6a}\)
4.2.2 Web buckling under pure flexure:

Experiments have shown that due to the initial out-of-flatness of the web, buckling of slender webs (Class 4) due to bending occurs at low stress levels. This does not exhaust the panel capacity, as stresses are redistributed in the post buckling range, allowing the girder to continue to take increasing bending moment. However the web does not resist the stresses given by conventional bending theory, but ‘throws off’ some of its load to the stiffer flange. Hence the web is less effective in resisting bending and the flange experiences a higher stress than that given by ETB resulting in a reduction in ultimate moment capacity of the girder.

This effect is accounted for in the Standard through the use of an effective width (as explained in chapter 2), by only considering 1/6 of the web area in the compression zone to be effective in resisting lateral or lateral-torsional buckling, as shown in Figure 4.1.

\[
M_u \geq 0.67M_p \quad \text{(Stocky members)}:
\]

\[
M_r = 1.15\phi M_p \left(1 - \frac{0.28M_p}{M_u}\right) \leq \phi M_p \quad \text{Class 1 or 2} \quad (3.11)^{\text{s13.6b}}
\]

\[
M_r = 1.15\phi M_y \left(1 - \frac{0.28M_y}{M_u}\right) \leq \phi M_y \quad \text{Class 3 or 4} \quad (3.12)^{\text{s13.6b}}
\]

In the Standard, it is assumed that the maximum moment that can be carried by Class 4(ii) sections is that which causes the extreme fiber in the compression zone to reach yield stress, as the thin web will not permit attainment of the theoretical plastic moment of the section.

A linear reduction to this maximum attainable value is then applied, which is a function of web slenderness, the relative proportions of the flange and web, and the buckling load of the web.

\[
M_r' = M_r \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{w} - \frac{1900}{\sqrt{M_r / \phi S}}\right)\right] \quad (4.7)^{\text{s14.3.4}}
\]

If a longitudinal stiffener is provided which will prevent the lateral deflection of the web, or if the web slenderness \( h/w \leq 1900/\sqrt{(M_r/\phi S)} \) then this reduction need not be applied. However if a reduction is required, then \( M_r' \) should be used in place of \( M_r \) in all subsequent calculations.
4.2.3 Vertical buckling of the web:

As the curvature which accompanies bending occurs, a vertical force is transmitted from the flanges into the web. This force may cause vertical buckling of the web on the compressive side as shown in figure 4.2.

By equating the buckling resistance of the web to the applied compressive force and by making some simplifying assumptions and substituting suitable values for $E$ and $\nu$ leads to a requirement on the web slenderness ($h/w$) in order to prevent vertical buckling of the web.

\[
\frac{h}{w} \leq \frac{83000}{F_y} \quad (4.8)
\]

4.3 Design of cross section for shear:

The Standard identifies 3 limiting states for determining the shear capacity of the web; shear buckling, shear yielding or a combination of the two. The mode of failure that will occur first is dependent on the web slenderness and stiffener spacing.

4.3.1 Unstiffened girder webs:

Shear yielding:

When steel is subject to a combined stress condition, the yield stress in shear, $F_y$, is normally approximated by the Von-Misses value, which is increased to allow for the strengthening effects of strain hardening:

\[
F_s = \lambda \frac{F_y}{\sqrt{3}} \rightarrow 0.66F_y \quad (4.4)
\]

Shear buckling:
The standard equation for buckling of a plate subject to pure shear (equation 2.2) is used to calculate the resistance to shear buckling after substituting the correct notation for dimensions of plate girder webs.

\[ \tau_{cr} = \frac{k \pi^2 E}{12(1 - \nu^2)(h/w)^2} \]  \hspace{1cm} (4.9)

For \( a/b \geq 1.0 \), for simply supported edges, it is found that:

\[ k = 5.34 + \frac{4.0}{(a/h)^2} \]  \hspace{1cm} (4.10)

The Standard assumes representative values for the terms in equation 4.9 (\( E = 200 \text{ GPa}, \quad \nu = 0.3, \quad k = k_y, \quad \tau_{cr} = F_s \)). With these values equation 4.9 reduces to:

\[ F_s \approx \frac{180000k_y}{(h/w)^2} \]  \hspace{1cm} (4.11)

Figure 4.3 below shows equations 4.4 and 4.11 plotted on the same chart, and clearly demonstrates that the failure mode is dependent on the slenderness, \( h/w \), of the web.

As mentioned before, there is a third mode of failure due to combined shear yielding and buckling, which creates a transition curve between the curves given by equations 4.4 and 4.11 shown in Figure 4.3. The equation for this curve is given in the Standard and was chosen mainly on the basis of experimental evidence.

1.  \( (h/w) \leq 439 \sqrt{k_y / F_y} \):

\[ F_s = 0.66F_y \]  \hspace{1cm} (4.4)\textsuperscript{13.4.1.1(a)}

with

\[ k_y = 4 + \frac{5.34}{(a/h)^2} \]  \hspace{1cm} (shear buckling coefficient)

or \( a/h \geq 1 \) then
k_v = 5.34 + \frac{4}{(a/h)^2} \quad \text{(shear buckling coefficient)}

and a/h aspect ratio (ratio of distance between end stiffeners to web depth)

2. \[ 439 \sqrt{\frac{k_v}{F_y}} \leq \left( \frac{h}{w} \right) \leq 621 \sqrt{\frac{k_v}{F_y}} \]

\[ F_s = F_{\text{cri}} = \frac{290 \sqrt{F_y k_v}}{(h/w)} \quad (4.12) \]

3. \[ (h/w) > 621 \sqrt{\frac{k_v}{F_y}} \]

\[ F_s = F_{\text{cre}} + k_a (0.5 * F_y - 8.66 * F_{\text{cre}}) \]

with

\[ F_{\text{cre}} = \frac{180000 k_v}{(h/w)^2} \quad (4.13) \]

and

\[ k_a = \frac{1}{\sqrt{1 + (a/h)^2}} \quad \text{(aspect coefficient)} \]

The Standard also imposes a limit on slenderness:

\[ \left( \frac{h}{w} \right) \leq \frac{83000}{F_y} \quad (4.8) \]

These equations are shown diagrammatically in Figure 4.4.

The capacity of the section is calculated by multiplying the ultimate shear stress for the web multiplied by a resistance factor \( \phi \) and the web area \( A_w \).

\[ V_t = \varphi A_w F_s \quad (4.14) \]

**4.3.2 Transversely stiffened Girder webs:**

The upper limit of the strength of a girder web stiffened by vertical supports will be the same as that of the unstiffened girder, that is, the strength corresponding to shear yielding. As for unstiffened webs, stiffened webs may fail due to shear buckling before shear yielding occurs, however in stiffened webs it is found that...
significant amounts of shear past the theoretical buckling load can be carried, due to ‘tension field action’. Subsequent to buckling the stress distribution in the web changes and considerable postbuckling strength may be realized because of the diagonal tension that develops. This is called the ‘tension field action’. Even without transverse stiffeners a plate girder can develop a shear stress significantly greater than the shear-buckling stress, although this is not recognized by the Standard.

Figure 4.5 shows the general distribution of the tension field that develops in the plate girder with transverse stiffeners. This tension field is anchored by the flanges and stiffeners.

![Figure 4.5 Tension field in stiffened girder web](image)

It is assumed that the tension field contribution to the shear capacity, $V_t$, is additive to the shear capacity as supplied by normal beam action, $V_b$. From Figure 4.5:

$$V_t = \sigma_t \frac{wh}{2\sqrt{1+\left(\frac{a}{h}\right)^2}}$$  \hspace{1cm} (4.15)

$$V_b = \tau_{cr} \frac{hw}{(h/w)} \approx \frac{180000k_\tau}{(h/w)}$$  \hspace{1cm} (4.16)

$$a/h > 1: k_\tau = 5.34 + \frac{4.0}{(a/h)^2} \text{ (shear buckling coefficient)} \hspace{1cm} (4.17)$$

$$a/h < 1: k_\tau = 4.0 + \frac{5.34}{(a/h)^2} \text{ (shear buckling coefficient)} \hspace{1cm} (4.18)$$

A simple stress interaction yield criterion is defined in order to relate $\sigma_t$ to $\tau_{cr}$.

$$\frac{\sigma_t}{\sigma_y} + \frac{\tau_{cr}}{\tau_y} = 1.0$$  \hspace{1cm} (4.19)

Substituting values for $\tau_y = \sigma_y \sqrt{3}$, $E = 200$ GPa, $\nu = 0.3$, an expression for the ultimate shear stress is obtained.

$$F_s = \frac{180760k_\tau}{(h/w)^2} + \frac{0.5F_{yy}}{\sqrt{1+\left(\frac{a}{h}\right)^2}} - \frac{156544k_\tau}{(h/w)^2\sqrt{1+(a/h)^2}}$$  \hspace{1cm} (4.20)
The Standard modifies this fundamental equation based on empirical data and similar to unstiffened webs gives multiple equations for the shear resistance of the web based on the web slenderness.

With

\[ F_{\text{cre}} = \frac{180000k_v}{(h/w)^2} \] (4.8) \( ^{\text{S13.4.1.3}} \)

\[ F_{\text{cri}} = \frac{290\sqrt{F_yk_v}}{(h/w)} \] (4.12) \( ^{\text{S13.4.1.1}} \)

\[ k_v = \frac{1}{\sqrt{1+(a/h)^2}} \] (aspect coefficient)

\[ V_s = \phi A_v F_s \] (shear resistance)

the Standard allows the following shear stresses:

(a) \( (h/w) \leq 439 \sqrt{k_v/F_y} \):

\[ F_s = 0.66F_y \] (4.8) \( ^{\text{S13.4.1.1(a)}} \)

(b) \( 439 \sqrt{k_v/F_y} \leq (h/w) \leq 502 \sqrt{k_v/F_y} \):

\[ F_s = F_{\text{cre}} = \frac{290\sqrt{F_yk_v}}{(h/w)} \] (4.8) \( ^{\text{S13.4.1.1(b)}} \)

(c) \( 502 \sqrt{k_v/F_y} \leq (h/w) \leq 621 \sqrt{k_v/F_y} \):

\[ F_s = F_{\text{cri}} + F_t \] (4.21) \( ^{\text{S13.4.1.1(c)}} \)

(d) \( (h/w) > 621 \sqrt{k_v/F_y} \):

\[ F_s = F_{\text{cre}} + F_t \] (4.23) \( ^{\text{S13.4.1.1(d)}} \)

\[ F_{\text{cre}} = \frac{180000k_v}{(h/w)^2} \] (4.13) \( ^{\text{S13.4.1.1}} \)

\[ F_t = \frac{0.50F_y - 0.866F_{\text{cre}}}{\sqrt{1+(a/h)^2}} \] (4.22)

\[ k_v \] is computed as before but the aspect ration \( a/h \) is now the ratio of distance between stiffeners to web depth.

These equations are shown diagrammatically in Figure 4.6. Essentially the curve for the shear resistance of the unstiffened web (dotted line) is shifted upwards by the development of the tension field and associated increase in resistance, \( F_t \).
The capacity of the section is again calculated by multiplying the ultimate shear stress for the web by a resistance factor $\phi$ and the web area $A_w$.

$$V_t = \phi A_w F_s$$  \hspace{1cm} (4.14)\textsuperscript{13.4.1.1}

End Panels:

The tension field in a plate-girder panel is resisted by the flanges, the adjacent panels and transverse stiffeners. Since the panels adjacent to an interior panel are tension field designed, they can be counted on to furnish the necessary support. However, an end panel does not have such support and must be designed as a beam-shear panel unless the end stiffeners are designed to resist the bending effect of a tributary tension field.

The end stiffener spacing is usually selected to ensure the sub panel at the end of the girder has sufficient resistance to elastic buckling, and hence will undergo plastic buckling or yielding failure in order to guard against fatigue failure.

**Note:**

Longitudinal stiffeners prevent the attainment of tension fields in the sub panels of the divided web, and hence tension field contribution to shear resistance does not occur.

### 4.4 Design of stiffeners

Figure 4.7 shows the typical placement of longitudinal and transverse stiffeners on a plate girder.
4.4.1 Bearing stiffeners:

The Standard specifies that bearing stiffeners are required where factored concentrated loads or reactions exceed the factored compressive (vertical) resistances of the plate girder web. Failure due to concentrated loads can be as local buckling of the web in the region where it joins the flange, or overall buckling of the web throughout its depth.

The problem of local yielding leading to web crippling is addressed by assuming that the load or reaction is distributed uniformly over the length of the bearing plate, N, and then spreads out. This area resisting the load effect at the flange-web junction is different for interior and exterior concentrated loads; hence the compressive resistances are also different at these locations. The Standard defines an interior load as a load applied at a distance greater than a member depth away from the member end. The resistance of the web at the web-flange weld is given by:

To account for the possibility of buckling of the web, the Standard provides formulas for calculating the resistance based largely upon empirical evidence.

**Interior Loads:**

The smaller of

\[
B_i = \phi_{bi} w [N + 10 \times t] F_y
\]

and

\[
B_i = 1.45 \phi_{bi} w^2 \sqrt{F_y E}
\]

(4.27)$^{14.3.2(a)}$

**End Reactions:**

The smaller of

\[
B_e = \phi_{be} w [N + 4 \times t] F_y
\]

and

\[
B_e = 0.60 \phi_{be} w^2 \sqrt{F_y E}
\]

(4.28)$^{14.3.2(b)}$

where

- \(N\) = length of bearing
- \(\phi_{bi} = 0.80\)
- \(\phi_{be} = 0.75\)

If this resistance is not sufficient to resist the applied concentrated force, then the length of the bearing plate, N, should be increased, or a pair of bearing stiffeners provided.

In the case of extremely slender webs, failure may be accompanied by a buckling motion of the web. To guard against this possibility, the Standard requires that the unframed ends of girders having h/w ratios greater than 1100$\sqrt{F_y}$ must be provided with pairs of bearing stiffeners.
The bearing stiffeners are designed as axially loaded columns. Similarly to sections subject to flexure, columns can be categorized into different classes, depending on the slenderness. A short column is loosely defined as a member which can resist a load equal to the yield load $C_y = AF_y$. For longer columns, failure is accompanied by a rapid increase in lateral deflection due to buckling. Intermediate members are members where the maximum load occurs after some local yielding, whereas long fail due to buckling in the elastic range. As for flexural members, compression members are classified into 4 according to the slenderness ratios of the various members comprising the cross section.

The Standard gives an equation for the calculation of the compressive resistance of members, which is based on extensive experimental investigation into the relationship between the slenderness ratio, $\lambda$, and compression resistance, $C_r$, for steel columns.

$$C_r = \phi A F_y (1 + \lambda^{2n})^{1/n} \quad (4.29)^{\$13.3.1}$$

$$\lambda = \frac{K L}{r} \sqrt{\frac{F_y}{\pi^2 E}} \quad (4.30)^{\$13.3.1}$$

The effective column length (KL) is not to be taken as less than $\frac{3}{4}$ of the length of the stiffeners in calculating the ratio KL/r. The parameter, n, is dependant on the class of section and section type.

- $n = 1.34$ for W shapes of Class 1,2 and 3, fabricated I-shapes, fabricated box shapes, and Class C (cold-formed non-stress-relieved) hollow structural sections.
- $n = 2.24$ for WWF shapes with flange edges flame-cut and Class H (hot formed or cold-formed stress-relieved) hollow structural sections.

The effective column cross section consists of the pair of stiffeners and a centrally loaded strip of the web equal to not more than 25 times its thickness at interior stiffeners, or a strip equal to not more than 12 times its thickness when at exterior (end) stiffeners. With only the portion of the stiffeners outside of the angle fillet of the flange-to-web welds considered effective in bearing.
Stiffeners must be ground to bear against the flange which is subject to the concentrated load.

Stiffeners can be stopped short or bear against the flange opposite to the one which is subject to the concentrated load.

Section A-A, Area assumed to resist concentrated load

Figure 4.9 Bearing stiffeners

The bearing capacity of the area of stiffener outside the web-flange girder weld and the capacity of the gross area of the stiffeners must be checked using equation 4.31.

\[ B_r = 1.50\phi F_y A \]  

(4.31)$^{13,10}$

with \( A \) = contact area (machined, accurately sawn, or fitted)

Additional design considerations for bearing stiffeners include:

- bearing stiffeners should extend as far as practicable toward the edge of the flange.
- bearing stiffeners’ spacing must meet the web’s width-thickness ratio required to prevent local buckling.
- bearing stiffeners’ length can stop short of the flange opposite to the one through which loads are delivered.
- bearing stiffeners should be connected to the web so as to develop the full force required to be carried by the stiffener into and out of the web.

### 4.4.2 Longitudinal stiffeners

Longitudinal stiffeners can greatly increase the bending strength of plate girders. This additional strength can be attributed to control of the lateral deflection of the web which increases the flexural stress that the web can carry and also improves the bending resistance of the flange due to greater web restraint.

It has been proven that the optimum location for a longitudinal stiffener used to increase the flexural buckling resistance of panel is 0.22 times the web depth from the compression flange if the web is assumed to be fixed at the flanges and simply supported at all four edges.

Accordingly the Standard adopts 0.20 of the depth as the accepted location for a longitudinal stiffener. Tests show that an adequately proportioned stiffener at 0.2h from the compression flange eliminates the bend-buckling loss in girders with web slendernesses as large as 450, so that the ultimate moment as determined by compression-flange buckling strength is attained.

The increase in bending strength of a longitudinally stiffened thin-web girder is usually small because the web contribution to bending strength is small, however longitudinal stiffeners can be important in a girder subject to repeated loads because they reduce or eliminate the transverse bending of the web, which increases...
resistance to fatigue cracking at the web-to-flange juncture and allows more slender webs to be used.

The postbuckling strength of longitudinally stiffened girders has been evaluated either by assuming that each subpanel develops its own tension field after buckling or that the only one tension field is developed between the flanges and transverse stiffeners.

However there is argument that tension fields do not develop in girders which are longitudinally stiffened as the stress distribution on the subpanels of the divided web does not allow tension field action to develop. Hence for conservatism tension field action is not accounted for in longitudinally stiffened girders.

### 4.4.3 Intermediate transverse stiffeners

As discussed in chapter 2, transverse intermediate stiffeners increase the critical buckling stress of the web by providing anchorage for tension field action, and thus allow a reduced web thickness. As the number of transverse stiffeners is increased, the web thickness can be reduced further, however there is an optimum number of stiffeners before the fabrication cost of adding an extra stiffener outweighs the cost saving through a reduced web thickness.

The stiffeners can be sized once the stiffener spacing, $a$, has been decided. The Standard provides limits to the maximum allowable spacing according to the slenderness of the web (Table 5 of HSC 2007 ed.).

$$
\frac{h}{w} \leq 150 : \quad a/h \leq 3 \quad \text{(4.32)}^{14.5.2}
$$

$$
\frac{h}{w} > 150 : \quad a/h \leq \frac{67500}{(h/w)^2} \quad \text{(4.33)}^{14.5.2}
$$

The first restriction reflects the fact that when the stiffener spacing to web depth ratio exceeds about 3, the effectiveness of the tension field is minor. The second restriction is related to ease in handling and fabrication.

It is important to note that the end panel stiffener requires a different spacing. That is because at the ends of the girder, the horizontal force component of the tension field is either taken out of the member or resisted internally (see figure 4.5). It is generally not convenient to provide an external reaction and experience shows that, if the force is resisted by internal action only, considerable distortion of the girder occurs in the vicinity of the free end.

The end panel stiffener spacing is usually chosen so that the aspect ratio of the panel is such that it’s resistance to buckling is sufficient to resist the applied stresses, and vertical tension field action does
not develop. This is also advantageous from a fatigue standpoint and leads to the requirement that:

\[(h / w) \leq 439 \sqrt{\frac{k_v}{F_y}} : \]

\[F_s = 0.66F_y \geq f_s \quad (4.34) \text{S13.4.1.1.} \]

\[439 \sqrt{\frac{k_v}{F_y}} < \frac{(h / w) < 83000}{F_y} : \]

\[F_s = \frac{180000}{(h / w)^2} k_v \geq f_s \quad (4.35) \text{S13.4.1.1.} \]

Where \(f_s\) is the shear stress in the panel and \(F_s\) the shear resistance. If adequate anchorage is provided then tension field action can be accounted for in the design and the end panel can be designed as an interior panel. It should be noted that the treatment of panels with large openings is the same as end panels.

The intermediate transverse stiffeners are required to resist the vertical component, \(F_c\), of the tension field action over one panel width (See Figure 4.5).

\[F = \frac{\sigma_t h w}{2} \left[\frac{a}{h} - \frac{(a / h)^2}{\sqrt{1+(a / h)^2}}\right] \quad (4.36) \]

The Standard provides an equation for the stiffener area required to resist the compressive force, \(F\), derived by assuming that the stiffener will yield before buckling, the full ultimate shear force is attained and full tension field action is developed.

\[A_s \geq \frac{aw}{2} \left[1 - \frac{a / h}{\sqrt{1+(a / h)^2}}\right] C \frac{F_y}{F_{ys}} D \quad (4.37) \text{S14.5.3} \]

\[C = \left[1 - \frac{310000K_x}{F_y(h / w)^2}\right] \geq 0.1 \quad (4.38) \text{S14.5.3} \]

The stiffener factor, \(D\), is included to account for additional stresses induced in the stiffeners if they are not placed symmetrically about the web (eccentric loading).

\(D = 1.0\) stiffeners furnished in pairs
\(D = 1.8\) stiffeners composed of angles placed on one side of web only
\(D = 2.4\) stiffeners composed of plates placed on one side of web only

For girders or portions of girders where the moment predominates, the full ultimate shear force is not attained and the stiffener area can be reduced. The Standard conservatively allows a reduction in stiffener area by multiplying by the ratio \(V_f / V_r\). Where \(V_f\) is the factored shear force in the panel adjacent to the stiffener, and \(V_r\) is
the maximum factored shear resistance in the panel adjacent to the stiffener.

The stiffener is also required to be rigid enough so as to prevent lateral displacement of the web at its location. The Standard provides a requirement for the moment of inertia of the stiffener (single or pair) about an axis in the plane of the web, based on a combination of theory and experimental evidence.

\[ I_s \geq \left( \frac{h}{50} \right)^4 \] (4.39)\textsuperscript{514.5.3}

In order to prevent local buckling of the stiffener under the compressive force, \( F \), the slenderness ratio, \( (b/t) \), is limited.

\[ \frac{b}{t} \leq \frac{200}{\sqrt{F_y}} \] (4.40)\textsuperscript{511.2}

The stiffener serves to maintain the cross sectional shape as the loads are applied and the girder distorts. However studies have shown that since the tension flange is self-aligning, the stiffeners can be stopped short of the tension flange. This avoids expensive fabrication and minute cracks which would occur if the stiffener was fitted against both flanges and lightly welded, which can lead to brittle fatigue failure.

The Standard imposes a limit of six times the web thickness, on the amount that the stiffener can be set back from the tension flange in order to prevent local buckling and states that the gap should not be less than four times the web thickness in order to provide a reasonable strain gradient.

\[ f = \frac{1}{6} \text{w} + \frac{1}{4} \text{w} \]

\[ f = \frac{1}{4} \text{w} \]

Figure 4.10 Intermediate stiffener connections

Figures 4.11 and 4.12 show standard automatic plate welding equipment in use. Stiffeners should be clipped in order to clear the flange-web welds.
Figures 4.11 & 4.12 Automatic plate welding equipment

The Standard states that when a stiffener is placed on one side of the web only, a weld is required at the stiffener-flange junction in order to prevent tipping of the flange under loading. When the stiffener is furnished in pairs, there is no need for a weld at the join with the compression flange, provided the stiffener is not providing a connection for lateral bracing. The Standard provides the shear transfer, \( q \) (N/mm), which the welds must be designed for.

\[
q = 1 \times 10^{-4} h F_y^{1.5} \frac{V_f}{V_t} \tag{4.41}
\]

The Standard states that the total shear transfer calculated using equation 4.41 is not to be less than the value of any concentrated load or reaction required to be transmitted to the web through the stiffener. Fasteners connecting intermediate transverse stiffeners to the web shall be spaced not more than 300mm from center to center. If intermittent welds are used, the clear distance between welds shall not exceed 16 times the web thickness or 4 times the weld length.

### 4.5 Combined Loading Check

At certain locations, the beam may be subject to significant levels of shear and moment together, such as at the interior supports of a continuous beam. In such cases, the effect of the interaction between these two forces upon girder strength must be examined.

The Standard only requires this check to be performed if the girder is transversely stiffened and depends on tension-field-action to carry shear i.e. \( h/w > 502 \sqrt{(k_v/F_y)} \). As there is a limit state for the web yielding by the combined action of flexural stress and post buckling components of the tension field development in the web near the flange.

It is assumed in calculating the shear resistance of the girder, that only the web is effective in resisting shear. Hence for factored moments less than the yield moment of the flanges alone, there is no reduction in the maximum shear resistance due to moment i.e. \( V_f/V_t = 1.0 \) is the limit. Once the moment at the section exceeds the yield moment of the flanges alone, \( M_n \), contribution from the web is required to resist the moment and hence the shear force that the web can resist is reduced.

\[
M_n = \sigma_y d A_f \tag{4.42}
\]

As the moment increases further there is a continuing reduction in shear resistance until the moment, \( M_f \), is equal to the moment resistance of the entire section, \( M_t \), at which point the girder is unable to resist any shear. This is shown diagrammatically in figure 4.13.
The Standard applies a straight line to the interaction curve in order to simplify the equations.

\[ 0.727 \frac{M_f}{M_r} + 0.455 \frac{V_f}{V_r} \leq 1.0 \]  \hspace{1cm} (4.43)

**4.6 Formatted spreadsheet:**

A formatted spreadsheet was developed for the design of monosymmetric plate girders, including transverse, bearing and a longitudinal stiffener at 0.2 times the depth if used.

The design follows the prescriptive approach given by the Standard and presented in the preceding chapter. A guideline for the use of the spreadsheet is presented in the Appendix.
5. Design of box girders

Box girders are generally rectangular or trapezoidal in cross section, and can be either open or closed, as shown in Figure 5.1. Trapezoidal cross sections offer several advantages over rectangular cross sections. The narrower bottom flange allows steel savings in low bending areas and offers plate stockiness in compression zones of multi span bridges. A trapezoidal cross section is also more aesthetically pleasing.

The Standard does not give equations explicitly for the design of box girders and the equations given for the design of plate girders are heavily influenced by empirical data which has been gathered for plate girders.

Hence the Standard directs the designer to consult alternative reference material such as “Guide to stability design criteria for metal structures” for the design of box girders. The design of plate girders is quite prescriptive, however due to a lack of similar research and design equations for box girders, a more fundamental approach is required for the design of box girders.

Rather than a limit states approach, where the resistance of the section is calculated for several limit states and the lowest value used for comparison with the applied loading, a form of working stress design will be used. This involves the calculation of stresses due to the applied loading which are then used to check for buckling or yielding of any individual plate element or the cross section as a whole, accounting for the effects of stress interaction.

5.1 Design of cross section for flexure

A box girder subject primarily to bending will normally fail due to buckling of the compression flange. Unlike plate girders, lateral torsional buckling rarely governs for practical box girders.

Internal bracing and diaphragms of the type shown in Figure 5.2 are typically used to increase resistance to lateral torsional buckling and maintain the cross sectional shape. The section
shown in the bottom right hand corner of Figure 5.2 is the trend for box girders designed today.

Similarly to plate girders, a linear distribution of stress is assumed over the depth of the cross section, with the effects of buckling in the webs represented by using effective widths of web adjacent to the compression flange.

Failure is deemed to have occurred when the extreme fiber flange stress reaches the calculated ultimate stress of the compression flange, or the yield stress of the tension flange.

Traditionally box girder flanges were stiffened in both the longitudinal and transverse directions, however the current trend is towards the use of longitudinal stiffeners only. The unstiffened flanges of narrow box girders can be treated as plates, by using effective widths to account for the effects of buckling.

Influence of shear lag on Flange Buckling:

The phenomenon of shear lag arises because the direct stresses in a flange are introduced through shear along the web-flange boundary due to vertical bending of the web, this stress then has to be transferred across the width of the flange through the in-plane shear stiffness of the flange plate. The in-plane shear straining of a flange causes those parts most remote from a web to lag behind those in the vicinity of the web, as shown in Figure 5.3.

It is most marked in beams whose flanges are relatively wide compared to their length and therefore is of greater importance in box girder than in plate girder construction.

Shear lag results in a non-uniform distribution of longitudinal stresses across the flange width, with larger stresses occurring near the flange/web junction and smaller ones in the regions furthest removed from such junctions. The addition of stiffeners to a wide flange results in the non-uniformity becoming even more
pronounced, even though the maximum stresses may be reduced. This form of distribution can increase or increase the average stress, causing earlier buckling of the flange compared with the uniformly compressed case, depending on the stiffening of the plate.

The ‘Canadian Highway Bridge Design Code’ CAN/CSA S6-00 states that the effective width of bottom flange plates in tension shall be taken as not more than one fifth of the span for simply supported structures and not more than one-fifth of the distance between points of contraflexure under dead load for continuous structures.

Tests have shown, however, that for most practical purposes shear lag can be ignored in calculating the ultimate compressive strength of stiffened or unstiffened flanges. Hence there is no clause in the Standard for reducing the effective width of wide flanges.

5.2 Design of cross section for shear

The key difference between plate and box girders which may influence the shear strength of the webs is the use of relatively thin flanges in box girders at the boundaries of the webs. Caution is needed in applying available tension field models, derived and verified in the context of plate girders, to the design of the webs of box girders.

Of major concern is the relatively small amount of support against in-plane movement which may be afforded to the web by the thin flange of a box girder, compared with the restraint offered by the thicker and narrower flange of a corresponding plate girder. In the latter case the out-of-plane bending rigidity and in-plane extensional rigidity of the flange to resist movement perpendicular to and parallel to the flange/web junction, respectively, is more effectively mobilized than in the case of thin flanged box girders.

Furthermore, tension field action requires relatively large deflections, and most box girders are used in applications where large deflections are not desirable. Hence for conservatism, tension field action is not accounted for in the design of box girders.

5.3 Design of cross section for Torsion

In the design of webs, additional shear stresses caused by torsion may be added to those associated with bending when calculating the total stresses applied to the web.

Allowance can also be made for the effects of the shears induced by torsion in the flanges, by considering the effect on stability of the flanges.
Due to time limitations, no provision has been made for the calculation of additional stresses due to torsion in the formatted spreadsheet for the design of box girders which accompanies this report. However all equations needed to account combined loading on the flange and web plate elements are present, the only remaining aspect that needs to be included, is the calculation and inclusion of the additional stresses due to Torsion.

5.4 Design of stiffeners

The flanges of webs of box girders have traditionally been stiffened both longitudinally and transversely. However the design using three or more longitudinal stiffeners and designs using both longitudinal and transverse stiffeners are seldom economical. Hence the current trend for box girders is for the webs and flanges to be stiffened longitudinally only.

Methods for accounting for the effect of stiffeners on the plate behaviour of the webs and flanges are explained in section 5.5. To design against local buckling of the stiffeners themselves, slenderness limits given in the Standard are imposed on the stiffener elements.

5.5 Combined shear, moment and torsion

Each plate element of a box girder is typically longitudinally stiffened and may also be stiffened transversely. For detailed and accurate analysis finite element methods are utilized. However for an approximate and quicker solution, there are several structural idealizations that can be made to simplify the design. In increasing order of complexity, a stiffened plate element can be treated as:

1. A plate of uniform effective thickness
2. A series of disconnected struts
3. An orthotropic plate
4. A discretely stiffened plate

In each method, the sectional stresses in equilibrium with the applied loading are used to check for buckling or yielding of the idealized structure, taking account of combined stresses.

5.5.1 Plate effective thickness approach:

The basis of this approach is to smear the longitudinal stiffeners to create a flat plate of length equal to the transverse stiffener spacing and with an effective thickness, allowing the use of plate buckling formulas. The thickened plates are assumed to be simply supported between transverse stiffeners or internal diaphragms. The effective thickness is calculated to give the same moment of inertia as the stiffened plate.
The applied loads on the webs or flanges are used to check for buckling of the effective plates and sub panels (between longitudinal stiffeners) according to Chapter 2 of this report. Yielding of the section is also checked considering the effect of combined stresses. This is the simplest method and is adopted in the formatted design spreadsheets for the design of box girders prepared along with this report.

5.5.2 The strut approach:

The basis of this approach is to treat a plate stiffened by several equally spaced longitudinal stiffeners as a series of unconnected compression members or struts, each of which consists of a stiffener acting together with an associated width of plate that represents the plate between stiffeners, as shown in Figure 5.4. Knowing the cross-sectional properties of the strut, the calculation of the buckling strength of the strut can be obtained from a column buckling formula.

Where transverse stiffeners are present, they are designed to be sufficiently stiff to ensure that they provide nodal lines, acting as simple rotationally free supports to the ends of the longitudinal struts. Thus the equivalent buckling length of the longitudinal stiffeners is effectively the distance between transverse elements. Apart from the overall buckling of the longitudinal stiffeners between transverse stiffeners, allowance is made for reduction in effectiveness due to buckling of the plate between stiffeners. Limitations of stiffener cross sections are based on controlling the applied stresses under ultimate load to values that are fractions of the elastic critical buckling stresses.

Thus the design model relates to an orthogonally stiffened flange in which the controlling buckling mode envisaged is a buckling of the longitudinally stiffened plate between transverse stiffeners, which may or may not be accompanied by local plate buckling between stiffeners. However local buckling of the stiffener is suppressed, together with any participation of the transverse members in the overall buckling mode. To account for the effect of initial distortions and residual stresses caused by welding, an effective width approach can be used to account for both the reduced stiffness and strength of the plate.

Figure 5.4 effective strut
5.5.3 Orthotropic plate approach:

An orthotropic plate is one whose material properties are orthogonally anisotropic. A uniformly stiffened plate is reduced to this case by effectively ‘smearing’ the stiffness characteristics of its stiffeners over the domain of the plate. Clearly, the theory is best applicable when the spacing of the stiffeners is small. Possible modes of buckling of orthotropic plates are shown in Figure 5.5.

![Figure 5.5 various forms of local buckling of orthotropic plates](image)

The potential advantage of this approach is that the inherent plate action, ignored by the strut approach, can be mobilized. This has a particular advantage in the postbuckling range when transverse tensile membrane stresses in the plate restrain the rate of growth of out-of-plane deflections in the stiffeners. As the equations describing postbuckling behaviour are non-linear, the solutions generally involve an iterative procedure to produce the ultimate strength, although in some cases this has been greatly aided by the provision of design charts.

5.5.4 Discretely stiffened plate approach:

Analytical studied have been made of stiffened panels in which account has been taken of the discrete nature of the stiffening and which incorporate non-linear geometric and material effects. Both finite difference and finite element numerical formulations have been used of this purpose.

5.6 Formatted spreadsheet:

For the design of closed box girders, a formatted spreadsheet was developed which checks for local buckling or yielding of a monosymmetric box girder stiffened longitudinally and transversely, with webs and flanges each stiffened longitudinally with two stiffeners. The equations used in this spreadsheet are described in this section.


5.6.1 Sectional Properties:

The neutral axis height, $\bar{y}$, and moment of inertia of the box girder section, $I_{xtot}$, are easily calculated using equations 5.1 and 5.2.

$$\bar{y} = \frac{\sum y_i A_i}{A_{tot}} \quad (5.1)$$

$$I_{xtot} = \sum I_{xi} + \sum A_i (y_i - \bar{y})^2 \quad (5.2)$$

For the calculation of effective thickness of flanges or webs with the longitudinal stiffeners smeared, equations 5.3 and 5.4 are used.

$$I_{xstiffenedplate} = I_{xplate} + \sum I_{xstiffeners} + \sum A(y_i - \bar{y}_{stiffenedplate})^2 \quad (5.3)$$

$$t_{eff} = \left( \frac{12I_{xstiffenedplate}}{I_{plate}} \right)^{1/3} \quad (5.4)$$

5.6.2 Stresses:

The factored moment and previously calculated sectional properties allow the calculation of the longitudinal stresses at any point in the section (assuming that plane sections remain plane).

$$\sigma = \frac{M_f y}{I_{xtot}} \quad (5.5)$$

The factored shear (assumed to be applied along the line of symmetry) gives rise to a shear flow around the section, $q$, which is calculated using equation 5.6. The general distribution of shear around the section is given in Figure 5.7.

$$q = \frac{V_f}{I_{xtot}} D_x + q_o \quad (5.6)$$
By taking a cut along the vertical line of symmetry of the bottom flange, the constant shear flow term, \( q_0 \), is zero and the shear flow, \( q \), can be evaluated traveling anti-clockwise around the section by evaluating \( D_x \).

\[
D_x = -\int_0^s t y ds - \sum y_{si} A_{si} \tag{5.7}
\]

\[
\tau_{xy} = \frac{q}{t} \tag{5.8}
\]

Once the longitudinal (normal) and shear stresses are known, the principal stresses, \( \sigma_1 \) and \( \sigma_2 \), can be calculated (the maximum and minimum normal stresses in a plane, always perpendicular to each other and oriented in directions for which the shear stresses are zero).

\[
\sigma_1, \sigma_2 = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x + \sigma_y}{2}\right)^2 + \tau_{xy}^2} \tag{5.9}
\]

The principal stresses are calculated at several discrete points; the intersection of the flanges and web, the web stiffener locations, and at the neutral axis.

### 5.6.3 Yielding failure:

These principal stresses are used to check for local yielding failure of the cross section using the Von Mises failure criterion for plane stress, which is given in equation 5.10 (this assumes that failure occurs when the energy of distortion reaches the same energy for yield/failure in uniaxial tension).

\[
\sigma_1^2 - \sigma_1 \sigma_2 + \sigma_2^2 \leq F_y^2 \tag{5.10}
\]

The Von Mises yield criterion was chosen over other yield failure criteria such as the Tresca criterion, as it is more conservative.

### 5.6.4 Buckling failure:

Equations 2.9 through 2.21 (section 2.1.4) are used to check for local buckling of the sub panels between longitudinal stiffeners.
and also the wider panels with the longitudinal stiffeners smeared to create a plate with increased thickness.

Lateral-torsional buckling is not evaluated for the closed cell box girder, as it is unlikely that this failure mode will dominate for typical box girders. However if the height to width ratio of the box is relatively large, lateral-torsional buckling may dominate and needs to be evaluated.

5.6.5 Stiffeners:

No provision for design of stiffeners has been made in the box girder formatted spreadsheet, buckling of the longitudinal stiffeners can be avoided through conformance with slenderness limits given in the Standard.

6. Protecting Steel

Corrosion of steel is an electrochemical process. When two different metals are placed in an electrolyte, an electrical current is created between the two metals. An example of an electrolyte would be salt water. In essence, one metal acts as the anode and the other as the cathode with deterioration being the end result. This process of deterioration is known as galvanic corrosion.

The corrosion of a steel structure is accelerated by:

- The presence of an electrolyte like salt water, due to deicing chemicals or a marine environment,
- The presence of carbon dioxide,
- The presence of hydrogen sulfide.
- Extremely high temperatures.

This corrosion can cause significant loss of section to structural members and impact the integrity of the structure as a whole.

There are several measures that can be taken to limit corrosion of steel:

- Reduce the contact between dissimilar metals.
- Galvanize the surface of the metal.
- Separate surfaces by a different material such as rubber or paint.
- Use of weathering steel.

6.1 Weathering Steel:

Atmospheric Corrosion steel (CSA G40.21 Grades 350A and 350AT) commonly referred to as weathering steel, is now the norm for bridges in Canada. Painted steels are used in environments not considered acceptable for the weathering process, such as continued wetness due to climate and precipitation, proximity to airborne chlorides e.g. near the sea coast or above a high traffic
volume expressway, and exposure to harsh industrial environments. Weathering steel’s strength to cost ratio and ability to form a superior base for a paint system means that it may still be the chosen material even when a coating is specified.

If weathering steel is used, protective measures such as drip tabs, drip pans, etc… must be taken to prevent staining of the substructure.

To prevent galvanic corrosion, avoid contact between weathering steel and galvanized steel, such as galvanized anchor bolts, rigid metal conduit, etc… Zinc or cadmium coated bolts should not come into contact with weathering steel, the thin sacrificial coatings reportedly corrode quickly when in contact with weathering steel.

An adequate protective coating should be applied to weathering steel that will be embedded in soil or gravel pockets or subject to standing water.

Weathering steel is not recommended if:

- The atmosphere contains concentrated corrosive industrial or chemical fumes
- The steel is subject to heavy salt-water spray or salt-laden fog.
- The steel is in direct contact with timber, because timber retains moisture and may have been treated with corrosive preservatives.
- The location has very high rainfall and humidity or there is constant wetness.
- There is low clearance (less than 8 to 10 feet) over stagnant or slow moving water.
- The steel is used for a low urban-area bridge/overpass that will create a tunnel like configuration over a road on which deicing salt is used. In these situations, road spray from traffic under the bridge causes salt to accumulate on the steel.

6.2 Protective Coatings:

When using a protective coating system, corrosion of steel member is controlled through use of one of three products:

- Inhibitive primers
- Sacrificial primers
- Barrier coatings

The coatings used can come in a variety of forms, the major types being metallic, organic, and inorganic in composition.

It should be noted that while the coating material is itself expensive, the surface preparation and eventual application of the protective coating is often several times the cost of that for the basic materials.
Another important aspect of applying protective coatings, is the containment and disposal of paint waste. The preparation of steel and the subsequent coating methods used can lead to the production of toxic by-products. This waste must be collected and disposed of to the satisfaction of environmental regulations. This can lead to severe increases in cost and in extreme cases may make replacement a viable alternative to recoating.

6.2.1 Inhibitive Primers

A primer is the initial coat of paint which is applied to the virgin surface of a steel member. The quality of a primer is indicated by its ability to adhere to the surface of the steel. An inhibitive primer functions through use of a coating which stops corrosion through a process of chemical or mechanical inhibition. This inhibition is designed to prevent deterioration caused by moisture and oxygen.

The pigment of inhibitive primers gives coatings their color, hardness, and corrosion resisting properties and can be either organic or inorganic.

6.2.2 Sacrificial Primers

Like an anode in a cathodic protection system, a sacrificial or galvanic primer protects the underlying steel surface by creating a surface which is electrochemically negative in relation to the steel.

Zinc is the most common material used to make the primer act as an anode. The zinc is dispersed through the paint film as a pigment and is applied directly to the surface.

A limitation of inorganic, sacrificial primers is that the surface must receive a near white blast cleaning prior to application, and the difficulty in determining which areas of the structure are the base metal and which have been newly primed.

6.2.3 Barrier Coatings

A barrier system is designed to prevent water, oxygen, and ionic material from coming in contact with the underlying steel surface. A barrier system is typically composed of multiple layers of essentially the same substance. Types of barrier system coatings are:

- Coal-tar enamels
- Low-build vinyl lacquers
- Epoxy and aliphatic urethanes
- Coal-tar epoxies
The thickness of barrier coatings is usually insufficient in preventing moisture and oxygen from precipitating the cathodic reaction which causes deterioration. However the ionic impermeability of the barrier systems usually offers high enough electrical resistance that the cathodic reaction is minimal.

6.2.4 Surface Preparation

The bond between coating and the base steel is of paramount importance and adequate surface preparation is required in order to create a sound and secure bond. Proper surface preparation increases the bonding action and also creates a uniform surface upon which to place the coating system and thereby minimizes the amount of foreign particles which could potentially accelerate the cathodic reaction such as chloride or sulfate ions.

The type of surface preparation required depends on the type of protective coating used. Inhibitive primers generally require less surface preparation than sacrificial and barrier systems.

<table>
<thead>
<tr>
<th>Cleaning methodology</th>
<th>Normalization of substrate</th>
<th>Removal of interface material</th>
<th>Increase in surface area</th>
<th>Removal of soluble salts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hand tool</td>
<td>Poor</td>
<td>Poor-fair</td>
<td>Poor-fair</td>
<td>Poor</td>
</tr>
<tr>
<td>Power tool</td>
<td>Fair</td>
<td>Fair</td>
<td>Fair</td>
<td>Poor-fair</td>
</tr>
<tr>
<td>Brush-blast</td>
<td>Fair</td>
<td>Fair</td>
<td>Good</td>
<td>Poor-fair</td>
</tr>
<tr>
<td>Commercial blast</td>
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<td>Good</td>
<td>Excellent</td>
<td>Good</td>
</tr>
<tr>
<td>Near white blast</td>
<td>Very good</td>
<td>Very good</td>
<td>Excellent</td>
<td>Very good</td>
</tr>
<tr>
<td>White blast</td>
<td>Excellent</td>
<td>Excellent</td>
<td>Excellent</td>
<td>Very good</td>
</tr>
<tr>
<td>Water blasting</td>
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<td>Good</td>
<td>Poor</td>
<td>Fair</td>
</tr>
<tr>
<td>High pressure water blast</td>
<td>Very good</td>
<td>Good-very good</td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td>Wet abrasive blasting</td>
<td>Very good - excellent</td>
<td>Very good - excellent</td>
<td>Excellent</td>
<td>excellent</td>
</tr>
</tbody>
</table>

Table 6.1 Surface preparation methods

The use of power or hand tools by themselves is generally insufficient and does not satisfy any of the major requirements of surface preparation. The use of hand and power tools is typically confined to the removal of heavy rust scale, loose paint etc… after which a more robust method of surface preparation is used. An adverse effect of power and hand tools is their tendency to force corrodent back into the steel surface itself.

Dry abrasive blasting cleans the steel surface by blasting small abrasive particles at the steel surface which strips off the layers of paint, rust etc... above the bare steel. Zinc based particles can also be used which offers cathodic protection to the steel.

The main advantages of water blasting, is that it is good at removing chloride contaminants from the steel surface and that the water does not scarify (scratch) the surface of the steel as mechanical methods are prone to do.
Due to fund limitations it is not always possible to completely clean and recoat a structure. In these cases spot cleaning is used where only the deteriorated areas are prepared and then coated.

7. Fabrication considerations

Evaluation of the economics of design often includes a perception that least weight and least cost are synonymous. Although cost of a structure is related to the weight of steel material, there are numerous other considerations in purchasing, fabricating, shipping, and erection and effective use of material locally which may override the decision to aim for a least weight structure. Some of these considerations require familiarity with purchasing, fabricating and erecting processes.

Various fabricators have their own processes, and it is difficult for a designer to produce a design to satisfy everyone. Fabricators should be allowed flexibility in detailing, with designer approval, to make adjustments to the number and location of splices.

Material content is only one element in the cost equation and will represent about 20% to 30% of the total ‘in place’ cost in fairly standard bridges.

The total rate per tonne of steel depends on several factors, including:

- Complexity of details
- Quality control requirements
- Amount of welding, including grinding, type and amount of inspection etc…
The amount of repetition and reuse of assembly jigs
Size and number of individual pieces to be fabricated
Other demands on shop space, particularly when large box girders are involved
The access for erection
Number of girder field splices

The allowable fabrication tolerances are defined in W59, clauses 5.8 and 5.9. The tolerances in the individual pieces that make up a continuous span will be additive.

7.1 Materials:

Weathering steel is now the norm for bridges in Canada. Painted steels are used in environments not considered acceptable for the weathering process, such as continued wetness due to climate and precipitation, proximity to airborne chlorides e.g. near the sea coast or above a high traffic volume expressway, and exposure to harsh industrial environments. In many cases weathering steel is selected even when a paint system is to be applied due its lower strength to cost ratio and its ability to form a superior base for paint systems.

A designer needs to be aware of the plate sizes available so that splices in webs and flanges are kept to a minimum, particularly longitudinal splices which should be avoided. The maximum length of plate that may be supplied is dependent on the thickness of the plate and the material type, and will vary from mill to mill and hence local fabricators should be consulted.

The designer should also be aware of other factors which influence material cost such as:

- There is a small premium on plates longer than 18 meters (about 4%)
- Plates less than 9 mm thick and more than 25mm thick attract a premium of from 4% - 8%
- Small orders also incur mill extras and small quantities of any one plate thickness should be avoided.

As a general guide the maximum piece-weight for handling in the shop is of the order of 50 tonnes, and the optimum length for the shop is about 27m although these values are increasing.

7.2 Proportioning of spans:

When there is choice in the positioning of piers for a continuous bridge. The end spans should be approximately 75% of the length of the main span, this will permit balancing of dead and live load moments, reduce the potential for uplift at the abutments, and permit the most economical design when proportioning the girder.
7.3 Selection of a girder cross section:

For compositely designed continuous spans, the designer should start with a main span to girder depth ratio of approximately 28 for box girders and 26 for I girders. On bridges where there are no pedestrians the bridge may be made more slender due to the reduced deflection requirements, ratios of 30 to 34 may be used successfully.

7.4 Webs:

The optimum web thickness and subsequent number of transverse stiffeners depends on the depth of the web and should be considered. For example, it is economically advantageous to have an unstiffened web if the girder is 1200 mm deep or less. The economics of unstiffened webs decreases as web depth increases. Sometimes the minimum web thickness is dictated by the method of construction such as launching, in this case thicker webs will usually be the economical solution because of local bearing, buckling, and crippling considerations, as well as overall stability.

Changes in web thickness should coincide with either a field splice or a maximum length of mill material available for the thickness and depth of web being considered. In addition, it will usually be found satisfactory to avoid grinding or to use only nominal grinding to touch up the profile of full penetration butt welds in the web when using the submerged arc process. In many cases it will be found economical to maintain a constant thickness of web throughout the girder, varying the spacing of intermediate transverse stiffeners according to the shear diagram and possible eliminating transverse stiffeners in the areas of low shear.

The recommended minimum web thickness is ½ inch as thinner plate is subject to excessive distortion from welding. Web thickness increments should be 1/16 inch up to a plate thickness of ⅜ inch, use 1/8 increments up to 1 inch, if the web plate needs to be thicker than 1 inch, use ¼ inch increments.

For web splices use the submerged arc process and avoid grinding if possible, or use only nominal grinding to touch up the profile of full penetration butt welds in the web. Web shop splices should be at least 10 feet apart and at least 6 inches away from a flange splice or transverse stiffener, in order to facilitate testing of the weld.

7.5 Stiffeners:

Welding of bearing stiffeners to the bottom flange should be specified as fillet welds, use of full penetration welds is costly and can cause distortion of the bottom flange, thus making it difficult to achieve the desired flatness for the sole plate or bearing.
For composite bridge girders, stiffeners welded to the top flange throughout do not alter the fatigue category of the flange (already Class ‘C’ because of the studs).

The use of both transverse and longitudinal stiffeners is difficult to avoid on deeper girders. However every effort should be made to place longitudinal stiffeners on one side of the web, with transverse stiffeners on the other so that interferences occur only where the longitudinal stiffener meets the double sided web stiffeners used on I girders for connection of cross frames.

Fabricators have indicated that flat bars are typically more economical than plates for stiffeners.

The clear distance between longitudinal stiffeners should be no less than 24 inches, to accommodate automated welding equipment.

It is highly preferable not to have several stiffener sizes for a girder. Bearing stiffener thickness that matches the flange thickness is suggested. Bearing stiffeners should be thick enough to preclude the need for multiple bearing stiffeners at any given bearing, as multiple stiffeners present fabrication difficulties and usually are not needed.

It is very important that the width be sufficient to provide clearance for field welding of diaphragm members to the stiffener.

Four inches or more of clearance between the web face and a vertical weld on a gusset plate/diaphragm member is required for good welding access. Three inches or more of clearance is needed between a gusset plate/diaphragm member and a flange.

For box girders, the current trend is to longitudinally stiffen the webs and flanges and use internal diaphragms, without the requirement for transverse stiffeners.

### 7.6 Flanges:

When deciding how to fit the flange sizes of a girder to the moment envelope, the designer must consider the cost implications as well as technical factors. The trade-off to be considered here is the cost of the material saved by reducing the flange size versus the cost of the full penetration but welded splice in the material including; material preparation, fitting up, welding, gouging, grinding, inspection and possibly repairs and so can involve a considerable number of man-hours.

In I girders and open top box girders, the designer may change the flange width or thickness or both. It is usually more economical to produce several flange splices simultaneously, this process involves butting two thicknesses of plate, wide enough to produce 2, 4 or 6 flanges, producing one butt weld across them and then flame cutting (stripping or ripping) the flanges longitudinally. Thus
by making the flanges a constant width between field splices, the costly procedure of butt welding individual plates is avoided, although this is not always possible e.g. a constant width may require a flange which is locally beyond a practical thickness. If flange widths are varied, it is best to change the width at field splices only.

Top and bottom flanges should be the same width. Girders in positive bending that are composite with a slab can have a top flange narrower than the bottom flange, but the weight savings achieved are typically not worth the reduced lateral stability prior to hardening of the deck. Also, if continuous construction is used, the top flange width would normally have to be increased for the negative moment sections, which creates slab-forming difficulties.

The desirable maximum flange thickness is 3 inches. Grade 50 and HPS70W steels are not available in thicknesses greater than 4 inches. Weld time is disproportionately increased when splicing plates thicker than 3 inches.

A 10-foot minimum length should be used for any given flange segment on a girder. It is only economical to introduce a flange splice if it is possible to save about 800 – 1000 pounds, these numbers are approximate and are a function of the current cost of steel plate.

Flange thickness increments should be $\frac{1}{8}$ inch for thicknesses from $\frac{3}{4}$ to 1 inch, $\frac{1}{4}$ inch from 1 to 3 inches, and $\frac{1}{2}$ inch from 3 to 4 inches. A change in thickness should be made at a slope of 1 in 2½.

Flange thicknesses should be sufficient to preclude the need for lateral bracing. Lateral bracing is to be avoided because it creates fatigue-sensitive details and is costly to fabricate and install.

Flange splices should be located at least 6 inches away from a web splice or transverse stiffener, in order to facilitate testing of the weld. Splices should be at least 10 feet apart. Field splices are good locations to change flange sizes.

Top flanges for open box girders should follow the suggestions for plate girder flanges, except for the stability criteria. Top and bottom flanges of closed box girders and bottom flanges of open box girders should extend past the centerline of each web a minimum of 2 inches to allow for automated welding equipment. Flange width is somewhat dependent on the need for enough room inside the box girder to allow the passage of inspection personnel. Provision must be made for entrance to the box girder by inspection personnel, typically a hatch-type, lockable door at each end of the box is sufficient.

For wide bottom flanges of box girders, plate distortion during fabrication and erection can be a problem. Designer should check with fabricators when using bottom tension flange plates less than
1-inch thick to determine whether practical stiffness needs are met. In no case should bottom tension flanges be less than ½ inch thick.

7.7 Field splices:

The gap between girder ends should be made large enough to accommodate normal shop tolerances. A dimension of 10mm is commonly used, smaller values would be difficult to work to, and unnecessarily expensive. Designer should use one bolt diameter throughout a structure, if practical, and ensure that it is physically possible to install bolts in their specified locations.

When welded field splices are specified, usually it is because aesthetics are paramount and a bolted splice is deemed unattractive. They have several disadvantages compared with bolted and are rarely seen. It can be difficult to detail an all welded splice to have acceptable fatigue and fracture performance, not to mention the problems of welding (including possible repairs), grinding and inspecting the welds in the field. Temporary connections are required to hold the parts in alignment during welding, and the accuracy of fabrication and fit up is more critical than with a bolted splice.

7.8 Fatigue Details

Flanges with welded shear studs and a web with welded transverse stiffeners both fall into Category ‘C’. Grinding is expensive and if carried out improperly can be detrimental to the fatigue life of the structure. Each tension flange butt weld should be radiographed, compression flange butt splices should be radiographed randomly (form 10% to 25%) and only butt splices in webs in critical tensile areas (e.g. 20% of the web adjacent to a tension flange) should be radiographed. A radius should be provided at the end of the gusset to eliminate a sharp notch, reduce the stiffness at the tip and minimize longitudinal stresses at the tip of the attachment.
8. Erection considerations

A well conceived economical steel bridge requires consideration of its erection at two stages in the design process. Firstly, erection must be considered at the concept stage because it typically represents about 30% of the superstructure cost and therefore the most economical arrangement cannot evolve without its consideration. Truss versus girder, curved versus parallel chords and flanges, continuity, main member dimensions, drop in spans, pier arrangements, etc… all have significance at this stage.

Secondly, erection must be considered at the detail stage. Details of splices, diaphragms, bracing and pier members are very significant contributors to erection cost.

Those elements which are in the control of the designer should be designed to facilitate construction wherever possible. Field labor is very expensive, therefore keep things simple. Realistic tolerances must be built into the system wherever shop fabricated elements meet field construction. Access to splices, anchor bolts and bearings and adequate space to install jacks is very necessary for proper installation, inspection and future maintenance.

Constant depth or Curved chords:

Strictly from an erection point of view, constant depth girders have the advantage. Pier sections of haunched girders frequently require extraordinary effort in shipping, handling and turning because of their increased bulk. Constant depth girders are much easier to ship, to turn and to lift and block.

Plate girder or box girder:

From an erection point of view, the box girders are usually preferable to the plate girders because there are fewer pieces of girder and less bracing. Box girders are reasonably stable in shipping, handling and free cantilever, whereas plate girders, particularly if slender in flange width, can pose stability problems in shipping, and handling and frequently require top chord stiffening trusses in cantilever erection.

Plate girders can often be nested during shipping whereas the internal diaphragms present in common box girders prevent nesting.

The particular configuration of box selected has a very significant effect on the erection cost of the bridge. Unless circumstances dictate boxes larger than about 3.5 m in width should be avoided because they will cause excessive shipping and handling problems and, in the limit, will require a longitudinal splice.

Box girders having more than two webs should be avoided except for special situations such as an axial girder cable stayed bridge.
Flange width has an impact on the stability of the girder during handling and erection. According to an industry rule of thumb, I-girders will be stable if their length is less than or equal to 60 times the flange width. If this is exceeded the erector and fabricator may need to use temporary bracing to handle and erect the girder.

Splicing:

The maximum economy will result if the fabricator/erector is permitted freedom to choose the splice locations that best suit his equipment. If the strength requirements of the splices are spelled out in general terms in the drawings and specifications, then the Contractor can detail the bridge with his preferred splice locations for the Engineer’s approval.

It is common to have all holes drilled or punched sub-size and then reamed to full size in full girder assembly of not fewer than three girder sections, laid on blocking corresponding to the cambered shape. If this method is performed accurately, all components should fit precisely in the field and the required bridge geometry will be attained. This method has the advantage of minimum time spent on field fitting and rework, as well as optimum quality in the connection. However it should be noted that the large assemblies tie up a lot of shop space and reaming is very time consuming.

Diaphragms and bracing:

While angle bracing and diaphragms are very cheap to fabricate, they are generally very expensive to erect, due to the cost of the crane and labor for erection. In order to minimize these costs, the designer should not use bracing and diaphragms indiscriminately, but only were strictly necessary.
9. Conclusions:

This report has outlined the CAN/CSA S16.1 approach to the design of doubly symmetric plate girders. The Standard gives a prescriptive method which was easily transferable to formatted spreadsheets, this report expands on the clause equations to give some explanation of their derivation and why they are applied. In order to expand on the applicability of the spreadsheet, equations for analyzing the lateral-torsional buckling resistance of monosymmetric plate girders was obtained from CAN/CSA-S6-00 (Canadian Highway Bridge Design Code). A second spreadsheet can easily be created which will allow the design of monosymmetric bathtub girders (open top box girders), as these are essentially plate girders with two webs and a wider bottom flange, with substitution of the lateral-torsional buckling equations taken from CAN/CSA-S6-00 into the existing sheet along with some other minor modifications. There is a high level of confidence in using the plate girder spreadsheet, as it is based entirely on CSA standards equations and clauses, although some more extensive validation of the spreadsheet may be required before their use for actual structures (to check for errors in equations etc.).

The Standard does not give methods for the design of closed cell box girders, and hence a more fundamental approach was required. This approach is influenced by equations given in the Crane Code (CMAA Specifications #70 & #74), which allows plate elements of box girders subject to combined linearly varying normal edge stress and shear stress to be checked for buckling. A formatted spreadsheet was created for the design of closed cell box girders, however unlike the spreadsheets for the design of plate girders which are based on limit states design, this spreadsheet is based on allowable or working stress design. This spreadsheet, unlike the plate girder spreadsheets is very simple and is not heavily influenced by conclusions drawn form years of experimental research and hence there is a significantly lower confidence in the use of this spreadsheet, and hence the equations and methods used are quite conservative. In the opinion of the writer, this spreadsheet should only be used for very preliminary sizing of box girders, with detailed FE analysis used to perform the detailed design.

This report also includes a discussion of economical and practical aspects associated with the design, fabrication and erection of steel plate and box girders. The general considerations given are true across North America, and will continue to be true for the foreseeable future. However the reader should bear in mind that the exact values given will vary from one geographical location to the next and also with time. In any case it is of paramount importance to collaborate with and receive input from local fabricators when designing plate and box girders, in order to ensure a practical and economical design.
References:

Due to the nature of this report, I can not claim originality for any of the work presented. This entire report is a synthesis of several references, where my input has been to digest the relevant discrete content, to expand on it and present it as a flowing report. Through the use of references which overlap in their content, I have attempted to create a report which can stand on its own and which explains the CISC Handbook of Steel Construction CAN/CSA S16.1 approach to the design of steel plate I girders and the design of box girders from a more fundamental approach. In order to preserve the flow of the text and to keep the report readable, I have omitted listing references in the text, they are listed on a per chapter basis below.

Chapter 2:
3. Crane Code (CMAA Specifications #70 & #74, revised 2000, Material Handling Industry)

Chapters 3, 4 & 5:
3. Steel Bridges, Design, Fabrication, Construction, ‘Notes and References’, Canadian Institute of Steel Construction
4. CAN/CSA-S6-00, Canadian Highway Bridge Design Code

Chapter 6:
1. ‘Preferred Practices for Steel Bridge Design, Fabrication and Erection’, November, 2000, Texas Steel Quality Council, Texas Department of Transportation (TxDOT)
2. ‘Economic factors in steel bridge design, fabrication and erection’, JK Ritchie, P Wells, AF Wong

Chapter 7:
1. ‘Preferred Practices for Steel Bridge Design, Fabrication and Erection’, November, 2000, Texas Steel Quality Council, Texas Department of Transportation (TxDOT)
2. ‘Economic factors in steel bridge design, fabrication and erection’, JK Ritchie, P Wells, AF Wong

Chapter 8:
Appendix A: List of Symbols

Symbols are listed on a per chapter basis as some symbols are used to denote different variables depending on the chapter. In order to avoid repetition, only new symbols for new variables not previously used or symbols re-used to denote a different variable are listed for each chapter.

**Chapter 2:**

- $\sigma_c$: plate critical elastic buckling stress
- $E$: Young’s Modulus (200000 GPa for steel)
- $E^*$: Post buckling apparent Young’s Modulus
- $\nu$: Poisson’s ratio (0.3 for steel)
- $a$: Plate width parallel to the direction of loading
- $b$: Plate width perpendicular to the direction of loading
- $t$: plate thickness
- $m$, $\alpha$: plate aspect ratio ($a/b$)
- $k, k', k'', k_v, k_s$: buckling coefficients
- $\sigma_e$: plate buckling effective stress
- $b_e$: plate buckling effective width
- $k_s$: plate shear buckling coefficient
- $\tau_c$: plate critical shear buckling stress
- $\psi$: plate loading stress ratio
- $\sigma_{1k}$: combined loading critical comparison stress
- $\sigma_{1kR}$: reduced combined loading critical comparison stress
- $\sigma_p$: proportional limit
- $\sigma_y$: yield stress
- $\theta_B$: safety factor
- $\sigma$, $\tau$: applied stresses
- DFB: Allowable design factor values

**Chapter 3:**

- $\kappa$: curvature
- $M$: Moment
- $E$: Young’s Modulus
- $I_x$: Strong axis moment of inertia
- $\varepsilon$: strain
- $y$: local co-ordinate axis
- $M_y$: Yield moment
- $\sigma_y$, $F_y$: yield stress
- $S$: elastic section modulus ($I_y / \bar{y}$)
- $\bar{y}$: distance from the outermost fibre to the neutral axis
- $M_p$: plastic moment
- $Z$: Plastic modulus, represents the first moment of area of the tension and compression zones about the neutral axis.
- $M_r$: Factored moment resistance
- $\phi$: resistance factor
- $S_e$: effective elastic section modulus
- $t$: flange thickness
- $b_0$: width of one half of flange
- $h$: clear depth of web between flanges
- $w$: web thickness
- $\Delta$: Beam deflection
- $M_a$: elastic lateral torsional buckling resistance
- $L$: Unbraced length
- $I_y$: Weak axis moment of inertia
- $C_w$: Warping torsional constant
\( \omega_2 \) coefficient to account for increased moment resistance of a laterally unsupported beam segment when subject to a moment gradient

\( \zeta \) ratio of the smaller factored moment to the larger factored moment at opposite ends of the unbraced length, positive for double curvature and negative for single curvature

\( C_1, C_2, C_3, K \) constants depending on conditions of loading and support of the beam

\( G \) shear modulus (77 000 MPa for steel)

\( J \) St. Venant torsion constant

\( \beta_x \) coefficient of monosymmetry

**Chapter 4:**

\( M_f \) maximum factored moment

\( M_r' \) reduced moment resistance due to soft web

\( A_f \) flange area

\( A_w \) web area

\( f_s \) actual shear stress in the panel

\( F_s \) shear stress resistance

\( D \) stiffener factor

\( V_f \) factored applied shear force

\( V_r \) factored shear force resistance

\( M_{fl} \) Flange moment

\( F_{cri} \) inelastic shear buckling stress

\( F_{cre} \) elastic shear buckling stress

\( F_t \) Tension field addition to shear stress resistance

\( a \) transverse stiffener spacing

\( B_r \) bearing resistance

**Chapter 5:**

\( I_{xi} \) moment of inertia of plate elements or stiffeners

\( A_i \) area of plate elements or stiffeners

\( y_i \) neutral axis height of plate elements or stiffeners

\( \bar{y} \) neutral axis height of entire cross section

\( \bar{y}_{tot} \) total cross sectional area

\( I_{x_{tot}} \) moment of inertia of entire cross section

\( I_{x_{stiffenedplate}} \) moment of inertia of stiffened plate

\( t_{eff} \) thickness of equivalent uniform thickness plate

\( q \) shear flow per unit length

\( \sigma_1, \sigma_2 \) principal stresses

\( \tau_{xy} \) shear stress

\( \sigma_{x}, \sigma_{y} \) normal stresses

N bearing seat length

d girder depth

k flange-web weld depth

t flange thickness

\( \lambda \) slenderness ratio

\( C_r \) compression resistance

\( K \) effective length factor

\( r \) radius of gyration

\( n \) compression parameter

\( L \) column length

\( A \) column cross sectional area

\( C \) compression factor

\( F_{ys} \) transverse stiffener yield strength

\( I_s \) stiffener moment of inertia

q shear transfer (N/mm)
Appendix B: Lateral-torsional buckling of monosymmetric beams

The general expression given in the CHBDC CAN/CSA-S6-00 for $M_u$: ($10.10.2.3$)

$$M_u = C_i \frac{\pi^2 EI_y}{(KL)^2} \left[ C_2 g + C_3 k + \gamma \sqrt{(C_2 g + C_3 k)^2 + \frac{C_w}{I_y}} \right]$$

$$\gamma = 1 + C_1 \frac{GJ(KL)^2}{\pi^2 EC_w}$$

$$k = \beta_x/2$$

$C_1$, $C_2$, $C_3$ and $K$ depend on conditions of loading and support of the beam. Two common conditions are:

Uniform moment: $C_1 = 1.0$  $C_2 = 0.0$  $C_3 = 1.0$  $K = 1.0$
Uniform load:  $C_1 = 1.13$  $C_2 = 0.45$  $C_3 = 0.5$  $K = 1.0$

These values assume unrestrained weak axis bending and unrestrained warping at the ends of the member. For the values of the coefficients for other conditions of loading and support, refer to the paper by Clark and Hill (1960). Note that $C_1$ is equivalent to the moment factor $\omega_2$.

$\beta_x$ is the coefficient of monosymmetry

e is the distance from the shear center to the centroid of the section

g is the distance from the shear center to the point of application of transverse load (i.e. top flange, midheight loading, bottom flange loading). g is positive (corresponding to an increase in $M_u$) when the load is below the shear center.

**Monosymmetric I-girder:**

![Monosymmetric I section plate-girder](image)

$$e = d_1 \frac{b_1^2 t_1}{b_1^2 t_1 + b_2^2 t_2} - y_2$$

$$C_w = \frac{d_1^2}{12} \frac{b_1^3 t_1 b_2^3 t_2}{(b_1^2 t_1 + b_2^2 t_2)}$$

$$J = \frac{1}{3} \left( b_1 t_1^3 + b_2 t_2^3 + dw^3 \right)$$
\[ \beta_x = 0.9d_1 \left[ \frac{1}{2} \frac{I_y}{I_x} - 1 \right] \left[ 1 - \left( \frac{I_y}{I_x} \right)^2 \right] \]

**Open top box girder with sloping or vertical webs:**

\[ y' = \left( b_2 d \left( A_n \frac{b_1}{2} + \frac{A_w}{6} \left( b t + \frac{b b}{2} \right) \right) - \frac{d^2 w}{6} A_n \right) / I_{yy} \]

\[ e = (e' + y_2) + t_2 \]

\[ J = \frac{1}{3} \left( 2b_1 t_1^3 + 2d_w w^3 + b_2 t_2^3 \right) \]

\[ C_w = \frac{2}{3} \left( k_1^2 \frac{A_{r_2}}{2} + \left( k_2^2 + k_2^2 - k_1 k_2 \right) A_w + \left( 4k_2^2 + (k_2 - k_3)^2 + (k_2 + k_3)^2 \right) \frac{A_n}{2} \right) \]

\[ k_1 = \frac{e' b_b}{2} \]

\[ k_2 = \frac{b_b}{2} - s(e' + d + t_2) - k_i \]

\[ k_3 = (e' + d + t_2) \frac{b_1}{2} \]

\[ k = \frac{\beta_x}{2} \]

\[ \beta_x = \frac{1}{I_{xx}} \left( y_2 A_{r_2} \left( \frac{b_2^2}{12} + y^2_2 \right) - y_1 A_{n} \left( \frac{b t^2}{2} + \frac{b^2}{6} + 2y_1^2 \right) \right) + \]

\[ w \left[ e'(y^2_2 - y_1^2) - \frac{4c s}{3d} (y^2_2 + y_1^2) + \frac{1}{2} \left( 1 + \left( \frac{s}{d} \right)^2 \right) (y^4_2 - y^4_1) \right] \]

\[ C = \frac{b_2}{2} + \frac{y^2_2}{d} \]
Appendix C: Notes for accompanying spreadsheets

Monosymmetric steel plate girders:

Suitable analysis should be used to determine the shear and moment envelope for the beam being designed. The cross section at any point along the beam can then be sized to give adequate resistance to these local forces using this spreadsheet.

This process may be performed at several different locations along the beam, rather than designing the entire beam for the largest factored forces, in order to increase the efficiency of the beam. However when changing cross section dimensions please refer to Chapters 7 and 8, which give guidance as to how often to change the dimensions and how to do so in order to achieve an economical design.

The designer is advised to perform the design by completing the sheets in the following order.

Sheet 0: Preliminary Sizing

This sheet is used to determine a preliminary girder size. Several simplifying assumptions are used to yield a quick sizing. The girder is sized as a doubly-symmetric girder at this stage. Based on the loading a recommended web depth and thickness and flange width and thickness is calculated. The user can then input specific dimensions which may be the same as the recommended values or modified to suit other criteria, these dimensions are then checked for slenderness limits.

Sheet 1: Moment

The design process starts by determining a web depth, h, web thickness, w, flange width, b, and flange thickness, t, in order to give the desired moment resistance.

The moment resistance depends on; the class of the section (1-4) which in turn depends on the slenderness of the component elements, and on the resistance to lateral torsional buckling. A preliminary cross section is obtained from this sheet, however to keep the calculations simple, the preliminary cross section developed in this sheet is doubly symmetric.

Sheet 2: Shear

Once a web depth and thickness have been selected, the shear resistance of the girder can be calculated.
The shear resistance depends on the presence of transverse stiffeners, the stiffener spacing and web thickness. If stiffeners are used, it is assumed in this sheet that they are adequately sized to anchor the tension field.

In end panels the user must declare if adequate anchorage is provided at the end of the girder in the input section, if adequate anchorage is not provided, then it is assumed that there is no contribution from tension field action, and the panel is sized in order to prevent elastic buckling.

For the purposes of this sheet, if the panel is adjacent to a large opening, the panel should be treated as an end panel.

The user may adjust the web thickness, web height or stiffener spacing to reach an adequate shear resistance. If the web dimensions are changed, these new values should be entered back into the moment sheet to check the moment resistance, this may allow a reduction in flange dimensions or require an increase.

A finalized cross section is obtained from this sheet.

Sheet 3: Combined Shear and Moment

This is a very simple sheet, used to ensure that the girder has sufficient capacity to resist the combined action of the applied shear and moment.

If the resistance is insufficient, the user must increase the girder size to provide an increased moment or shear resistance, and hence must return to the previous sheets.

Sheet 4: Transverse stiffeners

If no stiffeners were used to increase the shear resistance of the web, then this sheet does not need to be used. If a stiffener spacing, \( a \), was selected to increase the shear resistance of the web, then this sheet must be used to size the stiffeners based on this spacing.

This sheet calculates the required stiffener dimensions in order to resist the tension field action. The user may then select a stiffener type from the CISC handbook to meet the area and inertia requirements. If the required stiffener dimensions are excessive, then the user may change the stiffener spacing. However this will require the designer to input this new spacing in the shear spreadsheet and resize the web, requiring the user to check these new dimensions in the moment calculation.

Sheet 5: Bearing stiffeners

This sheet is used to check for the requirements for bearing stiffeners at points of application of concentrated loads, such as at the points of support of the beam.
It is assumed that the bearing stiffeners will be furnished in pairs and that they will be simple plates of a given width and thickness.

The user may increase the bearing length, $N$, or add/increase the size of stiffeners in order to achieve the required bearing resistance. The web and flange dimensions may also be changed, however this will require the new values to be inputted into the previous spreadsheets.

**Monosymmetric steel closed box girders:**

There is only one sheet for this design, as the design follows a working stress rather than limit states approach. The user inputs the geometry of the section, including all thicknesses, widths and position and properties of stiffeners. The user must also input the applied loading, to allow the stresses on any given panel to be calculated and each panel checked for buckling or yielding using the methods given in section 5.6 of this report.