Introduction

1. AASHTO LRFD 07 and 12 Steel Composite

1.1 Check List of AASHTO LRFD 07 and 12 Steel Composite

For AASHTO LRFD 07 and 12 Steel Composite Design, Limit State Design is applied. The criteria that Steel Composite Section must follow for Limit State Design is as follows.

(1) Cross-Section Proportion Limit State
Review on section properties, e.g. width-thickness ratio

(2) Strength Limit State
Review on flexure strength, shear strength and torsional strength

(3) Service Limit State
Review on permanent deformation

(4) Constructibility
Review on shear and flexure occurring from load combinations during construction stages

(5) Fatigue Limit State
Review on fatigue in steel and concrete materials in Steel Composite girder

1.2 Classification of Steel Composite

Steel Composite section can be categorized by the following classification groups.

(1) Section Shape Type
There are three main section shape types in midas Civil: I, Box and Tub shapes. In the case of box and tub sections, there are two more cases, single or multiple box section.

(2) Moment Type: Positive / Negative
For continuous beams, negative moments may occur around interior supports. Design code may apply different formulas for these cases.
(3) Bridge Type : Straight / Curved
Based on the horizontal alignment of a bridge, it can be classified as either straight or curved. The program recognizes curved bridges based on the input of the girder radius for each component.

(4) Compact Type : Compact / Noncompact / Slender

[Table 2.2] Steel Section Classification

<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compact</td>
<td>A composite section in positive flexure, which satisfies specific steel grade, web slenderness, and ductility requirements, is capable of developing a nominal resistance exceeding the moment at first yield, but not to exceed the plastic moment.</td>
</tr>
<tr>
<td>Noncompact</td>
<td>A composite section in positive flexure for which the nominal resistance is not permitted to exceed the moment at first yield.</td>
</tr>
<tr>
<td>Slender</td>
<td>Cross-Section of a Compression member composed of plate components of sufficient slenderness such that local buckling in the elastic range will occur.</td>
</tr>
</tbody>
</table>

1.3 Stiffeners of Steel Composite
The program considers transverse and longitudinal stiffeners.

[Table 2.3] Types of Stiffeners

<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Stiffeners</td>
<td>Transverse stiffeners are usually provided to increase shear resistance by tension field action. These work as anchors for the tension so that post buckling shear resistance can be developed.</td>
</tr>
<tr>
<td></td>
<td>It should be noted that elastic web shear buckling cannot be prevented by transverse stiffeners.</td>
</tr>
<tr>
<td>Longitudinal Stiffeners</td>
<td>Longitudinal stiffeners may be provided to increase flexural resistance by preventing local buckling. These work as restraining boundaries for compression elements so that inelastic flexural buckling stress can be developed in a web.</td>
</tr>
<tr>
<td></td>
<td>It consists of either a plate welded longitudinally to one side of the web, or a bolted angle.</td>
</tr>
</tbody>
</table>

[Fig.2.1] Longitudinal Stiffener and Transverse Stiffener
2. Considerations Steel Composite Design

2.1 Construction Stage for steel composite
During the construction of a steel composite bridge, the steel girder is constructed before the construction of the concrete deck of the upper part of the structure. The steel composite section is divided into three major steps.

<table>
<thead>
<tr>
<th>Construction stage for steel composite section</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Only Steel Girder (non-composite)</td>
<td>Only the steel girder has been constructed.</td>
</tr>
<tr>
<td>Steel girder and concrete deck as load</td>
<td>Although the concrete deck has been constructed, it has not hardened yet. Therefore, the weight of the wet concrete is applied as a load condition.</td>
</tr>
<tr>
<td>Steel girder and concrete deck as member (composite)</td>
<td>After concrete is hardened, the strength and stiffness are formed. Hereafter, the steel girder and concrete deck work as a complete composite section.</td>
</tr>
</tbody>
</table>

In order to find and portray the Steel Composite Section Design Process within the program, utilize the Construction Stage function.

2.2 Time Dependent Material
- Steel composite section is composed of steel and concrete. Concrete is a time dependent material and transforms due to creep and shrinkage. Also, the restraints imposed by the shear connectors cause additional stresses within the composite section. Therefore, time dependent characteristics (creep and shrinkage) must be taken into consideration.
- Modular ratio is the ratio of modulus of elasticity of steel to that of concrete. The short-term modular ratio "n" is used for transient loads in the program. Long-term modular ratio "3n" is used for permanent loads acting after composite action. For normal-weight concrete, AASHTO-LRFD 07 and 12 recommend the values of the short-term modular ratio.

3. Calculation of Plastic Moment and Yield Moment
- The plastic moment $M_p$ for a composite section is defined as the moment that causes yielding in steel section and reinforcement and uniform stress distribution of 0.85 in compression concrete slab. In positive flexure regions, the contribution of reinforcement in concrete slab is small and can be neglected.
- The yield moment, $M_y$, for a composite section is defined as the moment that causes the first yielding in one of the steel flanges or the moment at which an outer fiber first attains the yield stress. $M_y$ is the sum of the moments applied to the pre-composite steel section, the short-term composite concrete and steel section, and the long-term composite concrete and steel section.
3.1 Plastic Moment($M_p$), Yield Moment( $M_y$) in Positive Flexure

(1) Cross section proportions
I section and Box/Tub steel composite sections must satisfy the following criteria regarding cross section proportions. If the conditions have not been met after the design has been completed, it will be indicated as an “NG” on the design report generated.

1) Web Proportions
[Table 2.5] Web Proportions

<table>
<thead>
<tr>
<th>Case</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web with longitudinal stiffener</td>
<td>$\frac{D}{t_w} \leq 150$</td>
</tr>
<tr>
<td>Web without longitudinal stiffener</td>
<td>$\frac{D}{t_w} \leq 300$</td>
</tr>
</tbody>
</table>

2) Flange Proportions
[Table 2.6] Flange Proportions

<table>
<thead>
<tr>
<th>Section Type</th>
<th>I</th>
<th>Box / Tub</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\frac{b_f}{2t_f} \leq 12.0$</td>
<td>$\frac{b_f}{2t_f} \leq 12.0$</td>
</tr>
<tr>
<td></td>
<td>$b_f \geq \frac{D}{6}$</td>
<td>$b_f \geq \frac{D}{6}$</td>
</tr>
<tr>
<td></td>
<td>$t_f \geq 1.1t_{w}$</td>
<td>$t_f \geq 1.1t_{w}$</td>
</tr>
<tr>
<td></td>
<td>$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$</td>
<td>$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$</td>
</tr>
</tbody>
</table>

Where,
$I_{yc}$ : moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web
$I_{yt}$ : moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web

$$I_{yc} = \frac{t_f b_f 3}{12} , \quad I_{yt} = \frac{t_f b_f 3}{12}$$

(2.1)
(2) Section Classification

**Section Classification of Positive Flexure Moment**

6.10.6.2

- **Straight Bridge?**
  - Yes
  - Compact Section
  - Noncompact Section

- No
  - Curved Bridge

Min(\( \frac{F_{yw}}{F_{yc}}, \frac{F_{yw}}{F_{yc}} \)) \leq 70.0 ksi

\( \frac{d}{t_w} \leq 150 \)

\( \frac{D_{cp}}{f_{tw}} \leq 3.76 \sqrt{\frac{E_s}{F_{yc}}} \)

**Where,**

- \( D_{cp} \): depth of the web in compression at the plastic moment determined as per Article D6.3.2
- \( D_c \): Distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment
- \( D_t \): Total depth of the composite section

- The Section Classifications of I, Box, Tub are all the same.
- In a positive moment, the following ductility conditions must be met at all times. If not, the program will show NG.

\( D_p \leq 0.42 D_t \) \hfill (2.2)

**Where,**

- \( D_p \): Distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment

(3) Plastic Moment in Positive Moment (\( M_p \))

If the positive moment is applied on a compact section, \( M_p \) should be calculated as shown in Table 2.7.

**Fig. 2.2** Section Classification of Negative Positive Moment

**Fig. 2.3** Case of calculation of \( M_p \) in positive moment

[Table 2.7] Calculation of \( \overline{Y} \) and \( M_p \) for section in Positive Flexure

<table>
<thead>
<tr>
<th>Case</th>
<th>PNA</th>
<th>Condition</th>
<th>( \overline{Y} ) and ( M_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>In Web ( P_t + P_w \geq P_s + P_t + P_{rb} + P_{rt} )</td>
<td>( \overline{Y} = \left( \frac{D}{2} \right) \left[ \frac{P_t - P_r - P_{rt} - P_{rb} + 1}{P_w} \right] )</td>
<td>( M = \left( \frac{P_w}{2D} \right) \left[ \overline{Y}^2 + (t - \overline{Y})^2 \right] + \left{ P_r d_s + P_w d_{rt} + P_{rb} d_{rb} + P_w d_w + P_r d_r \right} )</td>
</tr>
<tr>
<td>II</td>
<td>In Top flange ( P_t + P_w + P_s \geq P_s + P_t + P_{rb} + P_{rt} )</td>
<td>( \overline{Y} = \left( \frac{t_s}{2} \right) \left[ \frac{P_w + P_t - P_{rt} - P_{rb}}{P_s} \right] + 1 )</td>
<td>( M = \left( \frac{P_w}{2t_s} \right) \left[ \overline{Y}^2 + (t - \overline{Y})^2 \right] + \left{ P_r d_s + P_w d_{rt} + P_{rb} d_{rb} + P_w d_w + P_r d_r \right} )</td>
</tr>
<tr>
<td>III</td>
<td>Concrete Deck, Below ( P_{rb} ) ( P_t + P_w + P_s \geq \left( \frac{C_{rb}}{t_s} \right) P_s + P_{rb} + P_{rt} )</td>
<td>( \overline{Y} = C_{rb} )</td>
<td>( M = \left( \frac{\overline{Y}^2 P_s}{2t_s} \right) + \left{ P_r d_{rt} + P_s d_{rb} + P_w d_c + P_w d_w + P_r d_r \right} )</td>
</tr>
<tr>
<td>IV</td>
<td>Concrete Deck, at ( P_{rb} ) ( P_t + P_w + P_s \geq \left( \frac{C_{rb}}{t_s} \right) P_s + P_{rb} )</td>
<td>( \overline{Y} = \left( t_s \right) \left[ \frac{P_s + P_w + P_t - P_{rb}}{P_s} \right] )</td>
<td>( M = \left( \frac{\overline{Y}^2 P_s}{2t_s} \right) + \left{ P_r d_{rt} + P_s d_{rb} + P_w d_c + P_w d_w + P_r d_r \right} )</td>
</tr>
<tr>
<td>V</td>
<td>Concrete Deck, Above ( P_{rb} ) ( P_t + P_w + P_s \geq \left( \frac{C_{rt}}{t_s} \right) P_s + P_{rt} )</td>
<td>( \overline{Y} = \left( t_s \right) \left[ \frac{P_s + P_w + P_t - P_{rb}}{P_s} \right] )</td>
<td>( M = \left( \frac{\overline{Y}^2 P_s}{2t_s} \right) + \left{ P_r d_{rt} + P_s d_{rb} + P_w d_c + P_w d_w + P_r d_r \right} )</td>
</tr>
<tr>
<td>VI</td>
<td>Concrete Deck, at ( P_{rt} ) ( P_t + P_w + P_s \geq \left( \frac{C_{rt}}{t_s} \right) P_s )</td>
<td>( \overline{Y} = C_{rt} )</td>
<td>( M = \left( \frac{\overline{Y}^2 P_s}{2t_s} \right) + \left{ P_r d_{rt} + P_s d_{rb} + P_w d_c + P_w d_w + P_r d_r \right} )</td>
</tr>
<tr>
<td>VII</td>
<td>Concrete Deck, Above ( P_r ) ( P_t + P_w + P_s \geq \left( \frac{C_{rt}}{t_s} \right) P_s )</td>
<td>( \overline{Y} = \left( t_s \right) \left[ \frac{P_{rb} + P_r + P_t + P_{rt}}{P_s} \right] )</td>
<td>( M = \left( \frac{\overline{Y}^2 P_s}{2t_s} \right) + \left{ P_r d_{rt} + P_s d_{rb} + P_w d_c + P_w d_w + P_r d_r \right} )</td>
</tr>
</tbody>
</table>

Where,

\( d_{rt} \): Distance from the plastic neutral axis to the centerline of the top layer of longitudinal concrete deck.

\( d_{rb} \): Distance from the plastic neutral axis to the centerline of the bottom layer of longitudinal concrete deck.

\( d_t \): Distance from the plastic neutral axis to the midthickness of the tension flange.

\( d_w \): Distance from the plastic neutral axis to middepth of the web.

\( d_c \): Distance from the plastic neutral axis to midthickness of the compression flange.

\( d_s \): Distance from the plastic neutral axis to the midthickness of the concrete deck.

\[ P_{rt} = F_{yr} A_{rt} \quad \text{(by reinforcement)} \]

\[ P_{rb} = F_{yr} A_{rb} \quad \text{(by reinforcement)} \]

\[ P_t = b_f t_f y_t \quad \text{(by steel girder)} \]

\[ P_w = D t_w y_w \quad \text{(by steel girder)} \]

\[ P_c = b_f t_f y_c \quad \text{(by steel girder)} \]

\[ P_s = 0.85 f_{ck} b_s d_s \quad \text{(by concrete slab)} \]

(4) Yield Moment in Positive Moment (\( M_y \))

When a positive moment is applied on a compact section, \( M_y \) is calculated as shown in Equation 2.3.

\[ M_y = \text{Min}(M_{yTop}, M_{yBot}) \quad (2.3) \]

Where,

\( M_{yTop} \): Yield Moment of Top Flange

\( M_{yBot} \): Yield Moment of Bottom Flange

\[ F_y = \frac{M_{D1}}{S_{Top}} + \frac{M_{D2}}{S_{Top}(3n)} + \frac{M_{AD}}{S_{Top}(n)} \quad (2.4) \]

\[ M_{yTop} = M_{D1} + M_{D2} + M_{AD} \]

\[ F_y = \frac{M_{D1}}{S_{Bot}} + \frac{M_{D2}}{S_{Bot}(3n)} + \frac{M_{AD}}{S_{Bot}(n)} \quad (2.5) \]

\[ M_{yBot} = M_{D1} + M_{D2} + M_{AD} \]

Where,

\( S \): Non-composite section modulus

\( S_{3n} \): Long-term composite section modulus

\( S_{n} \): Short-term composite section modulus

\( M_{D1} \): Moment of non-composite section

\( M_{D2} \): Moment of long-term composite section

\( M_{AD} \): Additional yield moment of short-term composite section

3.2 Plastic Moment (\( M_p \)), Yield Moment (\( M_y \)) in Negative Flexure

For I sections in negative flexure, \( M_p \) and \( M_y \) are calculated.

(1) Cross Section Proportions

For negative flexure, cross section proportions must meet the following requirements. If the program does not meet the requirements, NG will be reported after the design.
1) Web Proportions

[Table 2.8] Web Proportions

<table>
<thead>
<tr>
<th>Case</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web with longitudinal stiffeners</td>
<td>( \frac{D}{t_w} \leq 150 )</td>
</tr>
<tr>
<td>Web without longitudinal stiffeners</td>
<td>( \frac{D}{t_w} \leq 300 )</td>
</tr>
</tbody>
</table>

2) Flange proportions

[Table 2.9] Flange Proportions

Section Type: I / Box / Tub

\[
\frac{b_f}{2t_f} \leq 12.0 \\
\
\frac{b_f}{2t_f} \geq \frac{D}{6} \\
\
t_f \geq 1.1t_w \\
\
0.1 \leq \frac{I_w}{I_y^t} \leq 10
\]

(2) Section Classification

Section Classification of Negative Flexure Moment

6.10.6.2.3

Where,

\( D_c \): Depth of the web in compression in the elastic range.

\( I_{cy} \): moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web.

\( I_{yt} \): moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web.

[Fig.2.4] Section Classification of Negative Flexure Moment
Minimum Negative Flexure Concrete Deck Reinforcement
Under negative moment, concrete deck has to meet the minimum rebar ratio requirement. Once the requirements of Equation 2.6 are satisfied, the next design step can be taken.

\[ A_{rs} \geq 0.01A_{deck} \]  

(2.6)

(3) Plastic Moment in Negative Moment (\(M_p\))
Under negative moment, \(M_p\) is only calculated when Appendix A6 is used. \(M_p\) is calculated by either of the two following methods. Please refer to Table 2.10 for the equations.

\[ \bar{Y} = \left( \frac{D}{2} \right) \left[ \frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1 \right] \]

\[ M_p = \frac{P_w}{2D} \left[ \bar{Y}^2 + (D - \bar{Y})^2 \right] + \left[ P_{rt}d_{rt} + P_{rb}d_{rb} + P_t d_t + P_c d_c \right] \]

\[ \bar{Y} = \left( \frac{t_t}{2t} \right) \left[ P_w + P_c - P_{rt} - P_{rb} + 1 \right] \]

\[ M_p = \frac{P_t}{2t} \left[ \bar{Y}^2 + (t_t - \bar{Y})^2 \right] + \left[ P_{rt}d_{rt} + P_{rb}d_{rb} + P_w d_w + P_c d_c \right] \]

Where,
\[ P_{rt} = F_{yr} A_{rt} \quad \text{(by reinforcement)} \]
\[ P_{rb} = F_{yr} b_c t_c \quad \text{(by reinforcement)} \]
\[ P_c = F_{y} b_c t_c \quad \text{(by steel girder)} \]
\[ P_w = F_{yw} D t_w \quad \text{(by steel girder)} \]
\[ P_t = F_{yt} b_t t_t \quad \text{(by steel girder)} \]

(4) Yield Moment in Negative Moment (\(M_y\))
When Appendix A6 is used for negative flexure, \(M_y\) is calculated and utilized. \(M_y\) is calculated as shown below in Equation 2.7.

\[ M_y = \text{Min}(M_{yTop}, M_{yBot}) \]  

(2.7)

Where,
\[ M_{yTop} : \text{Yield Moment of Top Flange} \]
\[ M_{yBot} : \text{Yield Moment of Bottom Flange} \]
\[ F_y = \frac{M_{D1}}{S_{\text{Top}}} + \frac{M_{D2}}{S_{\text{Top}}(R)} + \frac{M_{AD}}{S_{\text{Top}}(R)} \]  

(2.8)

\[ M_{y\text{Top}} = M_{D1} + M_{D2} + M_{AD} \]  

(2.9)

\[ F_y = \frac{M_{D1}}{S_{\text{Bot}}} + \frac{M_{D2}}{S_{\text{Bot}}(R)} + \frac{M_{AD}}{S_{\text{Bot}}(R)} \]  

(2.10)

\[ M_{y\text{Bot}} = M_{D1} + M_{D2} + M_{AD} \]  

(2.11)

Where,

\( S_{R} \): Long-term composite section modulus with longitudinal reinforcements
1. Modeling Design Variables

In this chapter, the design variable values, the meaning behind the design requirements, and the design process for Steel Composite Design in midas Civil are explained.

1.1. Composite Section Data

The steel composite section is mainly composed of steel girder and concrete slab. Stiffeners can be added to steel girder section while longitudinal reinforcement can be added to reinforce concrete slab. In this section, the input methods for these sections and the meaning and application of design variables are explained.

<table>
<thead>
<tr>
<th>Contents</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1.1 Composite Section</td>
<td>(1) Composite Section Data</td>
</tr>
<tr>
<td></td>
<td>◀ Properties &gt; Section &gt; Section Properties &gt; Add &gt; Composite Tab</td>
</tr>
</tbody>
</table>

For design, Girder Num must be inserted as 1. In such case, cross beams should be modelled to consider the transverse stiffness instead of increasing the girder number.

2) The value of $B_c$ for the slab is used as the effective width of the concrete deck.

3) Multiple Modulus of Elasticity Option

To design the steel composite section, the modulus of elasticity for short-term and long-term effect in creep and shrinkage can be input. The modulus of elasticity input here is applied for construction stage analysis of Steel Composite section as shown in [Fig.2.7].
(2) Section Stiffener
▶ Properties > Section > Section Properties > Add > Composite Tab > Stiffeners Button...

[Fig.2.8] Section Stiffener Dialog Box

1.1.2 Longitudinal Reinforcement
▶ Design > Composite Design > Longitudinal Reinforcement ...

[Fig.2.9] Longitudinal Reinforcement Dialog Box

(2) Section Stiffener (Longitudinal)
1) Types of longitudinal stiffeners that are useable are Flat, Tee, and U-Rib.
2) For I sections, stiffeners can be added on either side of the web. For Box/Tub sections, upper and lower flanges can be installed as well as the web panel.
3) When the check box under c column is checked, the stiffness value of the stiffener is considered in analysis. Otherwise, the value is not considered for analysis. Regardless of whether or not the check box is checked on or off, longitudinal stiffeners are considered in design.

Based on the assignment of longitudinal stiffener, Rb, web load shedding factor varies for stiffened web/unstiffened web. It is also required for classifying the interior panels in shear check as stiffener/unstiffened.

1.1.2 Longitudinal Reinforcement
In a steel composite section, the longitudinal reinforcements are arranged within the concrete deck. The strength is calculated as shown in Table 2.11.

[Table 2.11] Applicability of material under the calculation of strength

<table>
<thead>
<tr>
<th>Case</th>
<th>Positive Flexure</th>
<th>Negative Flexure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Slab</td>
<td>Applied</td>
<td>None</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>None</td>
<td>Applied</td>
</tr>
</tbody>
</table>
1.1.3 Transverse Stiffener

(1) Transverse Stiffener

- Design > Composite Design > Transverse Stiffener ...

Figure 2.10 shows the window in which users can arrange transverse stiffeners in steel composite section. When the transverse stiffeners are installed, the existence and spacing between stiffeners determine whether the web is stiffened or unstiffened under strength limit state. Tension field action in Shear check for Strength Limit State is considered only for stiffened interior panels.

1.1.3 Transverse Stiffener

Figure 2.10 shows the window in which users can arrange transverse stiffeners in steel composite section. When the transverse stiffeners are installed, the existence and spacing between stiffeners determine whether the web is stiffened or unstiffened under strength limit state. Tension field action in Shear check for Strength Limit State is considered only for stiffened interior panels.

(1) Stiffener Type

1) One / Two Stiffener Option Button
Choose between one or two stiffeners. The two stiffener option is available for I/Box/Tub sections.

2) Pitch (d_o)
Pitch refers to transverse stiffener spacing. At the strength limit state, this can be used to distinguish between stiffened and unstiffened webs or calculate shear strength of the web.
1.2. Design Material Data
For the design of steel composite section, construction stage and time dependent material properties of concrete must be defined. In this section, the input method for concrete’s time dependent properties and steel composite section material information is defined.

<table>
<thead>
<tr>
<th>Contents</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2.1 Time Dependent Material</td>
<td>1.2.1 Time Dependent Material</td>
</tr>
<tr>
<td>(1) Creep/Shrinkage</td>
<td>(1) Creep/Shrinkage</td>
</tr>
<tr>
<td>▶ Properties &gt; Time Dependent Material &gt; Creep/Shrinkage ...</td>
<td>The time dependent properties of concrete, such as creep and shrinkage, are defined. During construction stage analysis of bridges, these properties are utilized for concrete material. During analysis, they are reflected in the calculation of member forces but not reflected in the design of the steel composite section.</td>
</tr>
<tr>
<td>![Fig.2.13] Add/Modify Time Dependent Material Dialog Box (Creep/Shrinkage)</td>
<td></td>
</tr>
<tr>
<td>(2) Comp. Strength</td>
<td>(2) Comp. Strength</td>
</tr>
<tr>
<td>▶ Properties &gt; Time Dependent Material &gt; Comp. Strength ...</td>
<td>In order to reflect the change in the modulus of elasticity of the time dependent property of concrete, the change in compressive strength or modulus of elasticity is defined.</td>
</tr>
<tr>
<td>![Fig.2.14] Add/Modify Time Dependent Material Dialog Box (Compression Strength)</td>
<td>Aging effects may vary for each construction stage since concrete is poured at different locations. The varying aging effects are reflected in the calculation of the member force but not in the design of the composite sections.</td>
</tr>
<tr>
<td>1.2.2 Modify Composite Material</td>
<td>1.2.2 Modify Composite Material</td>
</tr>
<tr>
<td>(1) Modify Composite Material</td>
<td>The material utilized for steel composite sections are provided in the SRC material properties. The materials should be defined as SRC Type.</td>
</tr>
<tr>
<td>▶ Design &gt; Composite Design &gt; Design Material ...</td>
<td>(1) Modify Composite Material</td>
</tr>
<tr>
<td>Figure 2.15 shows the dialog box where users can type in material characteristics for the steel</td>
<td></td>
</tr>
</tbody>
</table>
composite section design. The material property values entered will have a priority over the values entered in Material Data dialog box.

1) Steel Girder Section - Steel
   Hybrid Factor
   Hybrid factor is considered in the case where flanges and web have different material properties.

2) Concrete of Concrete slab

3) Steel materials of Concrete slab

(2) Hybrid Factor

When the check box for Hybrid Factor is selected, icon on the right is activated. The different materials for the top and bottom flanges and web of the steel girder can be defined. Hybrid Factor \( R_h \) is determined based on these material information.
1.3. Design Parameters for Composite Section

<table>
<thead>
<tr>
<th>Contents</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1.3.1 Design Parameter</strong></td>
<td></td>
</tr>
<tr>
<td>Design &gt; Composite Design &gt; Design Parameters ...</td>
<td></td>
</tr>
<tr>
<td>![Fig.2.17](Composite Steel Girder Design Parameter Dialog Box)</td>
<td></td>
</tr>
</tbody>
</table>

**1.3.1 Design Parameter**

(1) Strength Resistance Factor

Strength Resistance Factor is defined.

By clicking [Update by Code], the resistance factors are automatically set to the default values defined in AASHTO LRFD 12. The values also may be modified or entered manually.

(2) Girder Type for Box/Tub Section

- Consider St. Venant Torsion and Distortion Stress
  - If the Multiple Box Sections option is selected, lateral bending stress is considered in accordance with St. Venant Torsion and Distortion Stress. If the Single Box Sections option is selected, the lateral bending stress is always considered.

(3) Options For Strength Limit State

- Appendix A6 for Negative Flexure Resistance in Web Compact/Noncompact Sections
  - If this option is checked, Appendix A6 is applied for the flexure strength of straight composite I-sections in negative flexure with compact/noncompact webs. Use of Appendix A6 is optional in accordance with the code as shown below.

![Fig.2.18](Negative Flexure Resistance in Web Compact/Noncompact Sections)

- $M < 1.3R_h M_{y}$ in Positive Flexure and Compact Sections (6.10.7.1.2-3)

Before deciding, whether to apply this check or not, following conditions need to be manually verified:
1.3.2 Unbraced Length

The span under consideration and all adjacent interior pier sections satisfy the requirements of Article B6.2.
- The appropriate value of $\theta_{RL}$ from Article B6.6.2 exceeds 0.009 radians at all adjacent interior-pier sections.
- In which case the nominal flexural resistance of the section is not subject to the limitation of Eq. 6.10.7.1.2-3.

If the above three conditions are not satisfied for the compact sections under positive flexure in a continuous span, the $M_n$ value is restricted to $1.3R_hM_y$.

- Post-buckling Tension-field Action for Shear Resistance (6.10.9.3.2)
  - If this option is checked, post buckling resistance due to tension field action is considered in the nominal shear resistance of an interior stiffened web panel. If not, $V_n$ is taken as $CV_p$.

  $C = \text{ratio of shear-buckling resistance to the shear yield strength}$
  $V_p = \text{plastic shear force}$.

(4) Design Parameters
Design and result outputs are generated for the limit states checked in the Design Parameters.

1.3.2 Unbraced Length

Unbraced length factor for steel composite section is considered. The value input here has higher priority than the value calculated from Span Group.

(1) $L_b$

Lateral Unbraced Length is used to calculate lateral torsional buckling resistance in compression flange of I Girder or top flange of Tub Girder. If the lateral unbraced length is not added, the program will use span lengths. If span lengths are not defined either, the lateral unbraced length is applied for the corresponding member length.
1.3.3 Shear Connectors

In this program, studs are used for shear connectors. The parameters used for calculation are shown below.

(1) Category
Category defined by 75yr-(ADTT)_SL equivalent to Infinite Life (Table 6.6.1.2.3-2)

(2) $F_u$
Shear Resistance of Shear Connector

(3) Shear Connector Parameters

(4) Length Between Maximum Moment and Zero Moment
The Length between Maximum Moment and Zero Moment needs to be inputted by users to verify pitch as per strength limit state.

(5) Nominal Shear Force Calculation (6.10.10.4.2)
One of the two conditions needs to be selected for the calculation of the nominal shear force, $P$ which is applied for the verification of pitch at the strength limit state.

1.3.4 Fatigue Parameter

(1) Category
Category defined by 75ye-(ADTT)_SL equivalent to infinite life (Table 6.6.1.2.3-2)

(2) (ADTT)_SL
Number of trucks per day in a single-lane averaged over the design life (3.6.1.4.2)
(ADTT)_SL can be manually calculated as per 3.6.1.4.2-1.

(3) $N$
Number of stress range cycles per truck passage
Value can be taken from Table 6.6.1.2.5-2.

(4) Longitudinal Warping Stress Range
For the verification of fatigue, flexure stress is
1.3.5 Span Information

▶ Structure > Wizard > Composite Bridge > Span Information ...

1.3.5 Span Information

The elements of composite sections are defined as one Span Group. The Span Group will serve the following functions.

(1) Finding the most critical parts of the group unit and providing the corresponding results in the Span Checking table. Refer to Chapter 7 of "Steel Composite Design Result" for more information.

(2) Calculation of Unbraced Length

When assigning a span group, support properties are considered for calculating the unbraced length. The unbraced length can also be manually inputted once the corresponding support conditions under the support column are selected. Using the span parameters inputted, the unbraced length can be calculated automatically. However, if the unbraced length is inputted in Section 1.3.2, this value will be applied as the unbraced length first.
1.3.6 Curved Bridge Information

▶ Design > Composite Design > Curved Bridge Info ...

![Curved Bridge Information Dialog Box](image)

[Fig.2.24] Curved Bridge Information Dialog Box

---

### Design Guide for midas Civil

### 1.3.6 Curved Bridge Information

(3) End web panels
For each element, location of support, if any, can be identified as i or j. The stiffened webs with supports are identified as end panels.
Also, the elements that are assigned with i or j for the support are considered as end panels. Tension field action is not considered for the end panel in Shear Check.

---

### 1.3.6 Curved Bridge Information

Once the girder radius value of the element units in the steel composite section is entered, the corresponding elements are categorized as curved bridges.

(1) Radius is used for the review of shear connectors’ pitch and the moment of inertia of area for the longitudinal stiffener attached to web.

The curve type needs to be determined as convex or concave so the program determines whether the longitudinal stiffener is on the side of the web away or toward from the center of the curvature.

Lateral bending stress due to curvature is obtained from the analysis results and not using V-Load equation.

(2) If convex, left stiffener is on the side of the web away from the center of curvature and right stiffener is on the side of the web toward the center of curvature. If concave, the opposite case of the convex is applied.

Please refer to the table below for the equations applied to each case.

### Table 2.12 Curvature Correction Factor for Longitudinal Stiffener

<table>
<thead>
<tr>
<th>Case</th>
<th>Equation</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left Stiffener</td>
<td>( \beta = \frac{Z}{6} + 1 )</td>
<td>(6.10.11.3.3-3)</td>
</tr>
<tr>
<td>Right Stiffener</td>
<td>( \beta = \frac{Z}{12} + 1 )</td>
<td>(6.10.11.3.3-4)</td>
</tr>
<tr>
<td>Left Stiffener</td>
<td>( \beta = \frac{Z}{12} + 1 )</td>
<td>(6.10.11.3.3-4)</td>
</tr>
<tr>
<td>Right Stiffener</td>
<td>( \beta = \frac{Z}{6} + 1 )</td>
<td>(6.10.11.3.3-3)</td>
</tr>
</tbody>
</table>

where,

\( \beta \) : Curvature correction factor for longitudinal stiffener rigidity

\( Z \) : Curvature Parameter

---

[Image 141x409 to 278x603]
1.3.7 Deck Overhang Loads

Design parameters for the Deck Overhang load can be entered. The \( f_l \) value obtained from \( F (\text{Distributed force}) \) and \( P (\text{Concentrated force}) \) is not applied to Box section, but only for I-section and top stiffener of Tub section. The \( f_l \) value for deck overhang is considered only for the constructibility limit state.

Distributed Force, \( F \)

\[
F_l = F \tan \alpha
\]

Concentrated Force, \( P \)

\[
P_l = P \tan \alpha
\]

Eccentricity of Overhang Loads, \( e \)

\[
\alpha = \tan^{-1} \left( \frac{e}{D} \right)
\]

The \( f_l \) value is generated by combining the values produced from the analysis and the value inputted in this dialog box. If this feature is not used, \( f_l \) value only from the analysis results will be used.

Lateral bending moment due to uniformly distributed lateral bracket force \( (F_l) \) is estimated as:

\[
M_l = \frac{F_l L_b^2}{12} \tag{c6.10.3.4-2}
\]

where,

- \( M_l \): flange lateral bending moment due to the eccentric loadings from the forming brackets
- \( F_l \): uniformly distributed lateral force
- \( L_b \): unbraced length

Lateral bending moment due to concentrated lateral bracket force \( (P_l) \) assumed to be placed at the middle of the unbraced length is estimated as:

\[
M_l = \frac{P_l L_b}{8} \tag{c6.10.3.4-3}
\]

where,

- \( M_l \): flange lateral bending moment due to the eccentric loadings from the forming brackets
- \( P_l \): concentrated lateral force
- \( L_b \): unbraced length

P and F are the dead loads and construction loads such as Deck Overhang Weight, Screed rail load, Railing load, Walkway load, Machine Load, etc. considered for the constructability check only.

The load coefficient applied to Erection (DC) Load
### 1.3.8 Design Force/Moment

- Design > Composite Design > Design Tables > Design Force/Moment...

Case is applied to calculate the load in this case.

### 1.3.8 Design Force/Moment

This feature displays design member forces (strong axis moment, $M_y$), weak-axis moment ($M_z$) and shear stress ($V_u$) for the local axis of elements under selected load combination of steel composite section for each construction stage. For explanation regarding design member forces under construction stages, please refer to Section 1.5 in this chapter.

![Design Force/Moment Dialog Box](Fig.2.27)
1.4 Load Combination for steel composite section

1.4.1 Application of load combination in midas Civil for AASHTO LRFD 12

The load combinations used for the review of each limit state as per Table 3.4.1-1, are shown below.

<table>
<thead>
<tr>
<th>Load Combination Limit State</th>
<th>DC</th>
<th>DD</th>
<th>EWR</th>
<th>FY</th>
<th>ES</th>
<th>PS</th>
<th>CR</th>
<th>SH</th>
<th>Use One of These at a Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I (unless noted)</td>
<td>γp</td>
<td>1.75</td>
<td>1.00</td>
<td>—</td>
<td>—</td>
<td>1.00</td>
<td>0.50/1.20</td>
<td>γp</td>
<td>—</td>
</tr>
<tr>
<td>Strength II</td>
<td>γp</td>
<td>1.35</td>
<td>1.00</td>
<td>—</td>
<td>—</td>
<td>1.00</td>
<td>0.50/1.20</td>
<td>γp</td>
<td>—</td>
</tr>
<tr>
<td>Strength III</td>
<td>γp</td>
<td>—</td>
<td>1.00</td>
<td>1.4</td>
<td>0</td>
<td>1.00</td>
<td>0.50/1.20</td>
<td>γp</td>
<td>—</td>
</tr>
<tr>
<td>Strength IV</td>
<td>γp</td>
<td>—</td>
<td>1.00</td>
<td>—</td>
<td>—</td>
<td>1.00</td>
<td>0.50/1.20</td>
<td>γp</td>
<td>—</td>
</tr>
<tr>
<td>Strength V</td>
<td>γp</td>
<td>—</td>
<td>1.35</td>
<td>1.00</td>
<td>0.4</td>
<td>1.00</td>
<td>0.50/1.20</td>
<td>γp</td>
<td>—</td>
</tr>
<tr>
<td>Extreme Event 1</td>
<td>γp</td>
<td>—</td>
<td>1.00</td>
<td>—</td>
<td>—</td>
<td>1.00</td>
<td>—</td>
<td>—</td>
<td>1.00</td>
</tr>
<tr>
<td>Extreme Event II</td>
<td>γp</td>
<td>—</td>
<td>1.00</td>
<td>—</td>
<td>—</td>
<td>1.00</td>
<td>—</td>
<td>—</td>
<td>1.00</td>
</tr>
<tr>
<td>Service I</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.3</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00/1.20</td>
<td>γp</td>
<td>—</td>
</tr>
<tr>
<td>Service II</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>—</td>
<td>—</td>
<td>1.00</td>
<td>1.00/1.20</td>
<td>γp</td>
<td>—</td>
</tr>
<tr>
<td>Service III</td>
<td>1.00</td>
<td>0.80</td>
<td>1.00</td>
<td>—</td>
<td>—</td>
<td>1.00</td>
<td>1.00/1.20</td>
<td>γp</td>
<td>—</td>
</tr>
<tr>
<td>Service IV</td>
<td>1.00</td>
<td>—</td>
<td>1.00</td>
<td>0.7</td>
<td>0</td>
<td>1.00</td>
<td>1.00/1.20</td>
<td>γp</td>
<td>—</td>
</tr>
<tr>
<td>Fatigue I—LL, IM &amp; CE only</td>
<td>—</td>
<td>1.50</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Fatigue II—LL, IM &amp; CE only</td>
<td>—</td>
<td>0.75</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

Using the Auto Generation feature of the program, the load combinations regulated by the design code can be automatically generated. Load factors are considered for each load combinations in this program. Load factors are considered only within the program, and γp value can be designated by Auto Generation feature.

![Fig.2.28] Load Combinations and Load Factors

![Fig.2.29] Load Factors for Permanent Loads, γp
If a user wishes to review limit states based on the load combinations defined manually, it can be done by selecting the load combination of interest in Load Combination Type as in Section 1.4.2.

### Contents

1. **Auto Generation of Load Combinations**
   - Result > Combination > Load Combination > Composite Steel Girder Design > Auto Generation ...

### Explanation

1. **Auto Generation of Load Combinations**
   This feature automatically generates load combinations under provision of AASHTO LRFD 12.

   1. **Design Code**
   When load combinations are generated, they strictly follow the design code selected by the user.

   2. **Load Modifier ($\eta_i$)**
   Load modifier is a factor relating to ductility, redundancy, and operational classification. It is defined by the following equations.

   For loads for which a maximum value of $\gamma_i$ is appropriate:
   
   $$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$$

   For loads for which a minimum value of $\gamma_i$ is appropriate:
   
   $$\eta_i = 1/(\eta_D \eta_R \eta_I) \leq 1.0$$

   Where,
   
   $\eta_D$: a factor relating to ductility as per 1.3.3
   $\eta_R$: a factor relating to redundancy as per 1.3.4
   $\eta_I$: a factor relating to operational classification as per 1.3.5

3. **Load Factors for Permanent Loads ($\gamma_p$)**
   Load Factors for Permanent Loads are as per Table 3.4.1-2. Each option button for $\gamma_p$ value is activated when the corresponding static load case is defined.

![Automatic Generation of Load Combinations Dialog Box](image-url)
1.4.2 Used load combination for steel composite design
Load combinations used in the steel composite section design are defined under Load Combination Type.

<table>
<thead>
<tr>
<th>Contents</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Load Combination Type</td>
<td>(1) Load Combination Type</td>
</tr>
<tr>
<td>▶ Design &gt; Composite Design &gt; Load Combination Type...</td>
<td>1) Strength Limit State</td>
</tr>
<tr>
<td></td>
<td>Choose load combinations for use under review of strength limit state.</td>
</tr>
<tr>
<td></td>
<td>2) Service Limit State</td>
</tr>
<tr>
<td></td>
<td>Choose load combinations for review of usability limit state.</td>
</tr>
<tr>
<td></td>
<td>3) Fatigue 1 Limit State</td>
</tr>
<tr>
<td></td>
<td>Choose load combinations for review in fatigue limit state (Fatigue I Load Combination is for infinite life design; (ADTT)$<em>{sl}$ inputted in the software &gt; (ADTT)$</em>{sl}$, equivalent to infinite life as per Table 6.6.1.2.3-2).</td>
</tr>
<tr>
<td></td>
<td>4) Fatigue 2 Limit State</td>
</tr>
<tr>
<td></td>
<td>Similarly, choose load combinations for review in fatigue limit state (Conversely to Fatigue I, Fatigue II Load Combination is for finite life design).</td>
</tr>
</tbody>
</table>

![Load Combination Type Dialog Box](image)

1.5 Modeling Steel Composite Sections for Construction Staged Analysis
In this section, methods of construction stage modeling, implementation of concrete's time-dependent material properties in steel composite section and 3 types of design member forces applied to steel composite section design are explained. Construction stages of steel composite section can be implemented differently for case 1 to 3 as in table 2.13.

<table>
<thead>
<tr>
<th>Case</th>
<th>Construction Stage</th>
<th>Time Dependent Material(Creep / Shrinkage)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Defined</td>
<td>Defined</td>
</tr>
<tr>
<td>Case 2</td>
<td>Not Defined</td>
<td>Not Defined (Apply modular ratio of 3n)</td>
</tr>
<tr>
<td>Case 3</td>
<td>Not Defined</td>
<td>Not Defined (Apply modular ratio of 3n)</td>
</tr>
</tbody>
</table>

1.5.1 Member forces and stresses used in steel composite section design
(1) Member forces
For design of steel composite section, member forces per construction stage of steel composite section must be calculated. The program considers two main factors for design and review of construction stage of steel composite section.
• Construction stages of steel composite section
• Time dependent material properties of Concrete (Creep, Shrinkage and Compression Strength)

Design member forces used for design of steel composite section are divided into three main categories.
<table>
<thead>
<tr>
<th>Design Force/Moment</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead (Before)</td>
<td>Member forces before the concrete deck is activated. Only steel section properties are used.</td>
</tr>
<tr>
<td>Dead (After)</td>
<td>Member forces occurring due to erection load cases defined by user with the time dependent material properties (Creep &amp; Shrinkage) of concrete Long term section properties are used.</td>
</tr>
<tr>
<td>Short Term</td>
<td>Member forces from the post-construction state and load cases not included in the above categories. Short term section properties are used.</td>
</tr>
</tbody>
</table>

(2) Stress
Bending stress \( f_{bu} \) used for design of steel composite section is calculated as in equation 2.12.

\[
f_{bu} = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2} + M_{AD}}{S_{LT}}
\]  
(2.12)

Where,

- \( M_{D1} \): moment of non-composite section
- \( M_{D2} \): moment of long-term composite section
- \( M_{AD} \): additional yield moment of short-term composite section
- \( S_{NC} \): non-composite section modulus
- \( S_{LT} \): long-term section modulus
- \( S_{ST} \): short-term section modulus
- \( f_{bu} \): largest value of the flexural stress in the flanges at the section under consideration

On the other hand, lateral bending stress \( f_{l} \) is calculated as in equation 2.13.

\[
f_{l} = \frac{M_{uz} + M_{lat}}{S_{l}} \leq 0.6 F_{yf}
\]  
(2.13)

Where,

- \( f_{l} \): flange lateral bending stress
- \( S_{l} \): lateral section modulus of the flanges about z-axis
- \( M_{uz} \): flexural moment about z-axis
- \( M_{lat} \): lateral bending moment in the flange calculated from the overhang loads
- \( F_{yf} \): specified minimum yield strength of a flange

1.5.2 Case 1
In Case 1, construction stages and time dependent material properties of concrete (Creep/Shrinkage) are defined. Composite sections for Construction Stages function must be defined as well; otherwise, the sections shall be excluded from design. If time dependent material property information is inputted as well as long-term modulus of elasticity, long-term modulus of elasticity has higher priority in consideration of calculation.
### Define Composite Section for Construction Stage

<table>
<thead>
<tr>
<th>Contents</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Composite Section for Construction Stage</strong></td>
<td>For definition of construction stage, information in this window must be defined.</td>
</tr>
<tr>
<td>▶ Load &gt; Load Type &gt; Construction Stage &gt; Composite Section for C.S...</td>
<td></td>
</tr>
</tbody>
</table>

#### Add/Modify Composite Section for Construction Stage Dialog

- **Active Stage**: Construction stage where steel composite section should be activated is inserted.
- **Construction sequence**:  
  1. "Material Type" column  
     - By choosing Element, material property of the element is used.  
     - By selecting Material, material information chosen under "Material" Column is applied with higher priority.  
  2. Composite Stage column: Construction stages where steel girder and concrete slab should be activated are chosen.  
  3. Age column: Age information when each part is activated is input. Information in this column has higher priority over the age input during definition of construction stage.

#### (1) Member forces under Dead (Before composite)
Member forces before activation of Concrete Deck are applied. (Refer to Table 2.4 in "Introduction") For design purposes, Dead (Before) member forces are applied after multiplying the load factors applied in Dead Load (CS) in Load Combination dialog box.

#### (2) Member forces under Dead (After composite)
For the member forces under Dead (After), in post-composite stages, the long-term modulus of elasticity is determined by the time dependent material properties defined by users. Member forces under Dead (After) consist of static load cases and construction stage load cases. If Dead Load of Component and Attachments (DC2), Dead Load of Wearing Surfaces and Utilities (DW), Creep (CH), and Shrinkage (SH) are defined as erection loads, they are accounted for the Dead (After).
### Define Erection Load

<table>
<thead>
<tr>
<th>Contents</th>
<th>Explanation</th>
</tr>
</thead>
</table>
| (1) Define Erection Load  
▶ Analysis > Analysis Control > Construction Stage > Load Cases to be Distinguished from Dead Load for C.S Output > Add (Modify/delete)... | (1) Define Erection Load  
Erection Load is defined. |
| ![Fig.2.33] Define Erection Load Dialog |  
1) Load Type for C.S  
Determine the Load Type for the construction stages of the composite section. Load types are considered by the software for auto generation of load combinations.  
2) Assignment Load Cases  
Define Erection Load by selecting and moving the Load Cases desired from the List of Load Case panel to the Selected Load Case panel. |

(3) Calculation of the short-term member forces  
Short-term modulus of elasticity of the composite section is calculated based on the DB value inputted. All load cases are considered as the short-term loads except the ones defined as Dead (Before) and Dead (After).

#### 1.5.3 Case 2  
In Case 2, construction stages are defined without the time dependent material property (Creep/Shrinkage) information. Long term effects are considered using the long term modular ratio entered in the Section Data dialog box. Sections for different construction stages must be defined and differentiated using the Composite Section for Construction Stage definition. Otherwise, they will not be considered for the design check.

(1) Member forces under Dead (Before)  
Dead (Before) is applied before the concrete deck is activated. (Refer to Table 2.4 in the “Introduction”) For the design, the Dead (CS) multiplied by the load factor is applied as the member force under Dead (Before).

(2) Member forces under Dead (After)  
The effects of Creep/Shrinkage are reflected by applying the ratio of elastic modulus that is inputted in the Section Data (Refer to Section 1.1.1 (1)) for the long-term stage. In other words, the Creep/Shrinkage effects are reflected by using the section information with the ratio of elastic modulus that considers the time dependent material property for the analysis and design. These long term modular ratios defined for considering creep and shrinkage, auto generate Section Stiffness Scale Factors for the sections in which these are inputted. Section Stiffness Scale Factors need to be activated in the construction stages in accordance with the Composite Section for Construction Stage definition, i.e. the section stiffness scale factors are activated when the corresponding section becomes composite as per the definition of composite section for CS. If users compose construction stages and define Dead Load of Component and Attachments (DC2), Dead Load of Wearing Surfaces and Utilities (DW), Creep (CH), and Shrinkage (SH) as Erection Load, the load cases will be included in the Dead (After).

(3) Short term member forces  
The ratio of elastic modulus of the composite section is calculated using the DB value inputted. All the load cases which are not activated in the Construction Stage are considered as the short-term loads.
1.5.4 Case 3

In case the construction stages are not defined, users can model and define steel composite sections by using the Load Case for Pre-Composite Section function at

Load > Load Type > Settlement/Misc. > Misc. > Pre-composite Section.

For this case, short- and long-term ratios of elastic modulus defined in the section data (Refer to Section 1.1.1 (1)) are used. In this case, instead of member forces per construction stages, member forces under Dead (Before) is used to check the constructibility of the model.

(1) Member force under Dead (Before)
In the Load Cases for Pre-Composite Section dialog box, users can define which load cases to account for the member forces and apply as Dead (Before) in design. Since this is for pre-composite state, the steel only section properties are used (Refer to Section 1.1.1 (1)).

(2) Member forces under Dead (After)
Member forces under Dead (After) use the long term section properties. These loads should be separated from the short term member forces by the use of Analysis > Analysis Control > Boundary Change Assignment.

1) Data Selection
Check the box corresponding to Section Stiffness Scale Factor. As explained earlier, Section Stiffness Scale Factors are used for considering the long term section properties.

2) Boundary Group Combination
Create a boundary group combination considering the appropriate boundary groups from the boundary group list. The created boundary group combinations need to be selected for the post composite long term load cases. For the static load cases assigned with the section stiffness scale factor boundary groups, long term section property will be used.
(3) Short-term member forces
The ratio of elastic modulus from the database is used for the short-term loads of the composite section. All load cases are considered for the short-term loads except the ones considered for the Dead (Before) and Dead (After).
1. I Girder Section

1.1. Introduction

The program designs I-girder sections according to the orders in the flow chart below. This chapter demonstrates how the AASHTO LRFD 12 is applied in the program.

![Flow chart of Composite I-girder bridge](image)

Typical I-Sections have a cross section as shown below:

![I-Section in positive flexure](image)
1.2 Strength Limit State
The program checks the strength limit states for the flexure, shear, and ductility of the composite sections.

**Strength Limit States**

6.10.6

Check Ductility 6.10.7.3

Check flexural resistance 6.10.7 & 6.10.8

Check shear resistance 6.10.9

[Fig.2.38] Flow chart of Strength Limit States

### 1.2.1 Ductility
Ductility shall be checked to prevent premature crushing of concrete. For the verification of a web section that is under positive flexure, the ductility shall be verified as:

\[ D_p \leq 0.42D_t \quad (2.14) \]

*Where,*

- \( D_p \): distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment
- \( D_t \): total depth of the composite section

### 1.2.2 Flexural Resistance
There are four cases for checking flexural resistance of I Sections as shown below.

[Fig.2.39] Flow chart of flexural resistance
(1) Case 1: Compact Section in Positive flexural moment
The flexural resistance shall be checked according to the flow chart below if the section is under positive flexural moment, satisfies the ductility requirement and is a compact web. If the ductility requirement is not satisfied, the program will display NG in the design result page.

If a section is compact and under positive flexural moment, flexural resistance shall be checked according to the following equation:

\[ M_u + \frac{1}{3} f_l S_{x} \leq \phi_f M_n \]  \hspace{1cm} (2.15)

Where,
- \( f_l \): Flange lateral bending stress
- \( M_u \): Nominal flexural resistance of the section.
- \( M_s \): Bending moment about the major-axis of the cross-section.
- \( \phi_f \): Resistance factor for flexure.

1) Nominal Flexural Resistance\((M_n)\)
[Table 2.15] Calculation of Nominal Flexural Resistance\((M_n)\)

<table>
<thead>
<tr>
<th>Case</th>
<th>( M_n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D_p \leq 0.1D_f )</td>
<td>( M_n = M_p )</td>
</tr>
<tr>
<td>Otherwise</td>
<td>( M_n = M_p \left( 1.07 - 0.7 \frac{D_p}{D_f} \right) )</td>
</tr>
</tbody>
</table>
Where,

- \( M_p \): Plastic moment of the composite section determined as per Article D6.1.
- \( D_p \): Distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment
- \( D_t \): Total depth of the composite section

2) Strength Resistance Factor for flexure (\( \phi_f \))

The design code defines the flexural reduction factor as 1.00. However, the program primarily considers the factor that is inputted by users in the design parameters.

![Composite Steel Girder Design Parameter](Fig.2.41)

3) Especially, the following requirement regarding the nominal flexural resistance must be satisfied when "\( M_n \leq 1.3 R_y M_y \) in Positive Flexure and Compact Sections" is checked at ▶ Composite Steel Girder Design Parameters>Options for Strength Limit State. (Fig.2.41)

\[
M_n \leq 1.3 R_y M_y \tag{2.16}
\]
(2) Case 2: Positive flexural moment in noncompact section
The flexural resistance shall be checked according to the below flow chart if a section is under
positive flexural moment, satisfies the ductility requirement and is noncompact. Curved
bridges are considered as noncompact sections.

\[ \frac{f_{bu}}{f} \leq \phi \frac{F_{nc}}{F} \]
\[ F_{nc} = R_{c} R_{b} F_{wc} \] (2.17)
\[ F_{nc} = R_{c} R_{b} F_{wc} \] (2.18)

Where,
\( f_{bu} \): Flange stress calculated without consideration of flange lateral bending.
\( F_{nc} \): Nominal flexural resistance of the compression flange.

2) Compression flange
At the strength limit state, the compression flange shall satisfy the below criteria regarding
the flexure:
\[ f_{bu} \leq \phi \frac{F_{nc}}{F} \]
\[ F_{nc} = R_{c} R_{b} F_{wc} \] (2.17)
\[ F_{nc} = R_{c} R_{b} F_{wc} \] (2.18)

Where,
\( f_{bu} \): Flange stress calculated without consideration of flange lateral bending.
\( F_{nc} \): Nominal flexural resistance of the compression flange.

2) Tension flange
The tension flange shall satisfy the below criteria regarding the flexure:
\[ f_{bu} + \frac{1}{3} f_{l} \leq \phi \frac{F_{nt}}{F} \] (2.19)
\[ F_{nt} = R_{c} F_{yt} \] (2.20)

Where,
\( f_{l} \): Flange lateral bending stress, \( f_{l} \leq 0.6 F_{w} \)
\( F_{nt} \): Nominal flexural resistance of the tension flange.
\( R_{c} \): Web load-shedding factor.
(3) Case 3: Negative flexural moment in composite section and noncomposite section

The flexural resistance shall be checked according to the below flow chart if a section is under negative flexural moment and is one of the following cases:

- Curved bridge
- Straight Bridge but slender section
- Straight Bridge and compact or noncompact, but Appendix A6 is not applied

![Case 3: Check flexural resistance of Negative Flexure Moment](image)

1) Discretely Braced Compression Flange

For a compression flange, the following requirement shall be satisfied at the strength limit state:

\[
f_{hu} + \frac{1}{3} f_y \leq \phi_f F_{nc}
\]  

(2.21)

Where,

\[
F_{nc} = \text{Min}(F_{nc(FLB)}, F_{nc(LTB)})
\]  

(2.22)

Where,

\( F_{nc(FLB)} \) : Local Buckling Resistance based on Discretely Braced Compression Flange
**Chapter 2: Steel Composite Design - AASHTO LRFD 4th and 6th (2007/2012)**

Page Dimensions: 595.2x841.9

**Table 2.16 Calculation of \( F_{nc(FLB)} \)**

<table>
<thead>
<tr>
<th>Case</th>
<th>( F_{nc(FLB)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \lambda_f \leq \lambda_{pf} )</td>
<td>( F_{nc(FLB)} = R_b R_y F_{yc} )</td>
</tr>
<tr>
<td>( \lambda_f &gt; \lambda_{pf} )</td>
<td>( F_{nc(FLB)} = \left[ 1 - \left( \frac{F_{yr}}{R_y F_{yc}} \left( \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right) \right] R_b R_y F_{yc} )</td>
</tr>
</tbody>
</table>

In which:

- \( \lambda_f \): Slenderness ratio for the compression flange
- \( \lambda_{pf} \): Limiting slenderness ratio for a noncompact flange
- \( R_b \): Web load-shedding factor determined as specified in Article 6.10.10.2
- \( R_c \): Hybrid factor determined as specified in Article 6.10.10.1

\[
\lambda_f = \frac{b_{fc}}{2i_{fc}} \quad \text{(2.23)}
\]

\[
\lambda_{pf} = 0.38 \sqrt{\frac{E}{F_{yc}}} \quad \text{(2.24)}
\]

\[
\lambda_{rf} = 0.56 \sqrt{\frac{E}{F_{yr}}} \quad \text{(2.25)}
\]

\( F_{yr} \): Compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects, but not including compression-flange lateral bending, taken as the smaller of 0.7\( F_{yc} \) and \( F_{yw} \), but not less than 0.5\( F_{yc} \).

**Table 2.17 Calculation of \( F_{nc(LTB)} \)**

<table>
<thead>
<tr>
<th>Case</th>
<th>( F_{nc(LTB)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L_p &lt; L_b \leq L_r )</td>
<td>( F_{nc(LTB)} = C_b \left[ 1 - \left( \frac{F_{yr}}{R_y F_{yc}} \left( \frac{L_p - L_b}{L_r - L_p} \right) \right) \right] R_b R_y F_{yc} )</td>
</tr>
<tr>
<td>( L_b &gt; L_r )</td>
<td>( F_{nc(LTB)} = F_{cr} \leq R_b R_y F_{yc} )</td>
</tr>
</tbody>
</table>

Where,

- \( C_b \): Moment gradient modified

**Table 2.18 Calculation of \( C_b \)**

<table>
<thead>
<tr>
<th>Case</th>
<th>( C_b )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unbraced cantilevers and for members where ( f_{mid}/f_2 &gt; 1 ) or ( f_2 = 0 )</td>
<td>1.0</td>
</tr>
<tr>
<td>For all other cases</td>
<td>( 1.75 - 1.05 \left( \frac{f_1}{f_2} \right) + 0.3 \left( \frac{f_1}{f_2} \right)^2 \leq 2.3 )</td>
</tr>
</tbody>
</table>
Where,

\( L_b \): Unbraced length.

\( L_p \): Limiting unbraced length to achieve the nominal flexural resistance of \( R_b R_f F_{hc} \) under uniform bending.

\[
L_p = 1.0 r_t \frac{E}{F_{yc}}
\]  

(2.26)

\[
L_r = \pi r_t \frac{E}{F_{yr}}
\]  

(2.27)

\[
F_{cr} = \frac{C_2 R_f \pi^2 E}{(L_n / r_t)^2}
\]  

(2.28)

\[
r_t = \frac{b_k}{\sqrt{12(1 + 1/3)} \frac{D_t}{t_c} \frac{F_{hc}}{F_{yc}}}
\]  

(2.29)

\( L_r \): Limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression flange residual stress effect (in).

\( F_{cr} \): Elastic lateral torsional buckling stress.

\( r_t \): effective radius of gyration for lateral torsional buckling

\( F_{yr} \): compression-flange stress at the onset of nominal yielding within the cross-section, including

\( D_t \): depth of the web in compression in the elastic range determined as per D6.3.1

\( f_{mid} \): Stress without consideration of lateral bending at the middle of the unbraced length of the flange under consideration, calculated from the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression

2) Continuously braced Tension Flange

At the strength limit state, the following requirement shall be satisfied for the continuously braced tension flange:

\[
f_{bu} \leq \phi_f R_h F_{yl}
\]  

(2.30)

(4) Case 4: Flexural resistance of Negative Flexure Moment by using Appendix A6

The optional provisions of Appendix A6 shall apply to the sections in negative flexural and straight bridges and compact and noncompact web I-sections according to the flow chart below.
If Appendix A6 is applied at the strength limit state, the following four requirements regarding flexure shall be satisfied. The design verification is done for the compression and tension flanges.

[Fig.2.44] Case 4: Flow chart of flexural resistance of Negative Flexure Moment by using Appendix A6.
### [Table 2.19] Limit State defined by Appendix A6

<table>
<thead>
<tr>
<th>Case</th>
<th>Limit State</th>
</tr>
</thead>
<tbody>
<tr>
<td>Discretely-Braced Flange Section</td>
<td></td>
</tr>
<tr>
<td>Compression</td>
<td>[ M_u + \frac{1}{3} f_1 S_{xc} \leq \phi , M_{nc} ]</td>
</tr>
<tr>
<td>Tension</td>
<td>[ M_u + \frac{1}{3} f_1 S_{st} \leq \phi , M_{nt} ]</td>
</tr>
<tr>
<td>Continuously-Braced Flange Section</td>
<td></td>
</tr>
<tr>
<td>Compression</td>
<td>[ M_u \leq \phi , R_{pc} , M_{sy} ]</td>
</tr>
<tr>
<td>Tension</td>
<td>[ M_u \leq \phi , R_{pt} , M_{sy} ]</td>
</tr>
</tbody>
</table>

Where,

\( f_1 \): Resistance factor for flexure.

\( f_1 \): Flange lateral bending stress, \( f_1 \leq 0.6 \, F_{pc} \)

\( M_{nc} \): Nominal flexural resistance based on the compression flange.

\( M_c \): Bending moment about the major-axis of the cross-section.

\( M_{nt} \): Yield moment with respect to the compression flange.

\( M_u \): Nominal flexural resistance based on the tension flange.

\( M_u \): Yield moment with respect to the tension flange.

\( S_{xc} \): Elastic section modulus about the major axis of the section to the compression flange taken as \( M_c/F_{yc} \)

\( R_{pc} \): Web plastification factor for the compression flange.

\( R_{pt} \): Web plastification factor for the tension flange.

### [Table 2.20] Calculation of \( R_{pc} \) and \( R_{pt} \)

<table>
<thead>
<tr>
<th>Case</th>
<th>Web Plastification Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compact web</td>
<td>( R_{pc} = \frac{M_p}{M_{pcw}} ) ( R_{pt} = \frac{M_p}{M_{ptw}} )</td>
</tr>
<tr>
<td>Noncompact web</td>
<td>( R_{pc} = 1 - \frac{R_s M_{pcw}}{M_p} ) ( R_{pt} = 1 - \frac{R_s M_{ptw}}{M_p} )</td>
</tr>
</tbody>
</table>

in which:

\( M_p \): Plastic moment

\( D_c \): Depth of the web in compression determined as per D6.3.1.

\( D_{cp} \): Depth of the web in compression in the plastic moment.

\( M_y \): Yield moment taken as the smaller of \( M_{cp} \) and \( M_{pt} \).

\( \lambda_{pw(D_c)} \): Limiting slenderness ratio for a noncompact web

\[ \lambda_{pw(D_c)} = 5.7 \sqrt{\frac{E}{F_{yc}}} \tag{2.31} \]

\( \lambda_w \): Slenderness ratio for the web based on the elastic moment.

\[ \lambda_w = \frac{2D_c}{t_w} \tag{2.32} \]

\( \lambda_{pw(D_c)} \): Limiting slenderness ratio for a compact web corresponding to \( 2D_{cp}/t_w \)
1) Discretely braced Compression Flange

For the discretely braced compression flanges, the minimum of the local buckling resistance and lateral torsional buckling resistance is used to perform the design check as:

\[
M_{nc} = \text{Min}[M_{nc(FLB)}, M_{nc(LTB)}]
\]  

(2.34)

① Local buckling Resistance (\(M_{nc(FLB)}\))

The local buckling resistance shall be calculated as shown in the following table:

<table>
<thead>
<tr>
<th>Case</th>
<th>(k_c)</th>
<th>(M_{nc(FLB)})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\lambda_f \leq \lambda_{pf}) (Compact flange)</td>
<td>-</td>
<td>(M_{nc(FLB)} = R_{pc}M_{yc})</td>
</tr>
<tr>
<td>(\lambda_f &gt; \lambda_{pf}) (Noncompact flange)</td>
<td>(k_c = 0.76)</td>
<td>(M_{nc(FLB)} = 4/\sqrt{D_{cp}})</td>
</tr>
</tbody>
</table>

Where,

\[
\lambda_f = \frac{b_{fc}}{2t_{fc}}
\]  

(2.35)

\(\lambda_{pf}\): Limiting slenderness ratio for a compact flange.

\[
\lambda_{pf} = 0.38 \sqrt{\frac{E}{F_{yc}}}
\]  

(2.36)

\(\lambda_{rf}\): Limiting slenderness ratio for a noncompact flange.

\[
\lambda_{rf} = 0.95 \sqrt{\frac{E_k}{F_{yr}}}
\]  

(2.37)

\(k_c\): Flange local buckling coefficient determined as per A6.3.2-6 for built-up sections and 0.76 for rolled shapes.

\(F_{yc}\): compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects, but not including compression-flange lateral bending, taken as the smaller of 0.7\(F_{yc}\), \(R_{hy}\) \(S_{xt}/S_{xc}\) and \(F_{yw}\), but not less than 0.5\(F_{yc}\)

\(S_{ec}\): Elastic section modulus about the major axis of the section to the compression flange taken as \(M_{yc}/F_{yc}\)

\(S_{ec}\): Elastic section modulus about the major axis of the section to the tension flange taken as \(M_{yt}/F_{yt}\)

② Lateral Torsional Buckling Resistance (\(M_{nc(LTB)}\))

The lateral torsional buckling resistance is calculated as shown in the following table:
Where,

- \( L_{p} \): Limiting unbraced length to achieve the nominal flexure resistance \( R_{pc}M_{yc} \) under uniform bending

\[
L_{p} = 1.0r_{b} \sqrt{ \frac{E}{F_{yc}} } \tag{2.38}
\]

- \( L_{r} \): Limiting unbraced length to achieve the nominal onset of yielding in either flange under uniform bending with consideration of compression flange residual stress effects

\[
L_{r} = 1.95r_{b} \sqrt{ \frac{J}{F_{yc}E} } \left( 1 + \frac{1}{2} \frac{F_{yc}S_{w}h}{E} \right) \tag{2.39}
\]

- \( C_{k} \): moment gradient modifier, is divided into two cases and calculated according to either A6.3.3-6 or A6.3.3.3-7 of AASHTO LRFD 12. For the detailed calculations, please refer to the section "3.2 Strength Limit State > (1) Flexural Resistance > Case 3".

- \( F_{cr} \): Elastic lateral torsional buckling stress

\[
F_{cr} = \frac{C_{k} \pi^{2} E}{J} \left( \frac{L_{0}}{r_{b}} \right)^{2} \left( 1 + 0.078 \frac{J}{S_{w}h} \right) \tag{2.40}
\]

- \( J \): St. Venant torsional constant

\[
J = \frac{Dt_{p}^{3}}{3} + \frac{b_{p}t_{p}^{3}}{3} (1 - 0.63 \frac{t_{p}}{b_{p}}) + \frac{b_{p}t_{p}^{3}}{3} (1 - 0.63 \frac{t_{p}}{b_{p}}) \tag{2.41}
\]

- \( r_{b} \): Effective radius of gyration for lateral torsional buckling

\[
r_{b} = \frac{b_{p}}{\sqrt{ \frac{12}{1 + \frac{1}{3} \frac{D_{t}}{b_{p}t_{p}}} } } \tag{2.42}
\]

Where,

- \( F_{yw} \): compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects, but not including compression-flange lateral bending, taken as the smaller of 0.7\( F_{yc} \), \( R_{nf}F_{yc} \), \( S_{xc}S_{xe} \) and \( F_{yw} \), but not less than 0.5 \( F_{yc} \).
- \( h \): Depth between the centerline of the flanges.
- \( M_{mid} \): Major-axis bending moment at the middle of the unbraced length, calculated from the moment envelop value that produces the largest compression at this point in the flange under consideration, or the smallest tension if this point is never in compression. \( M_{mid} \) shall be due to the factored loads and shall be taken as positive when it causes compression and negative when it causes tension in the flange under consideration.
- \( M_{0} \): moment at the brace point opposite to the one corresponding to \( M_{2} \), calculated from the moment envelope value that produces the largest compression at this point in the flange under consideration, or the smallest tension if this point is never in compression (kip-in). \( M_{0} \) shall be...
due to the factored loads and shall be taken as positive when it causes compression and negative when it cause tension in the flange under consideration.

\[ M_1 = M_0 \]  

- Otherwise 

\[ M_1 = 2M_{mid} - M_2 \geq M_0 \]  

\( M_2 \): Except as noted below, largest major-axis bending moment at either end of the unbraced length causing compression in the flange under consideration, calculated from the critical moment envelope value.  
\( M_2 \) shall be taken as positive. If the moment is zero or cause tension in the flange under consideration at both ends of the unbraced length, \( M_2 \) shall be taken as zero. 

\( M_{yc} \): Yield moment with respect to the compression flange. 

\( M_{yt} \): Yield moment with respect to the tension flange.

1.2.3 Shear resistance

Shear resistance of an I-web Steel Composite Section is checked as shown in the flow chart below.

![Flow chart of shear resistance](figure)

[Fig.2.45] Flow chart of shear resistance

Shear resistance AASHTO LRFD 12 (6.10.9)
The program distinguishes Unstiffened and Stiffened webs according to the following criteria:

<table>
<thead>
<tr>
<th>Case</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without a longitudinal stiffener and with transverse stiffener spacing not exceeding 3D</td>
<td>Stiffened web</td>
</tr>
<tr>
<td>With one or more longitudinal stiffeners and with a transverse stiffener spacing not exceeding 1.5D</td>
<td>Otherwise Unstiffened web</td>
</tr>
</tbody>
</table>

However, even stiffened webs are classified as unstiffened web if the check box is not checked at Composite Steel Girder Design Parameters > Options for Strength Limit State > Post-buckling Tension-field Action for Shear Resistance. (Fig.2.41)

(1) Shear Resistance Check
Shear resistance shall be checked as:

\[ V_u \leq \phi_v V_n \]  
(2.45)

Where,
\( \phi_v \): Resistance factor for shear.
\( V_n \): Nominal shear resistance.
\( V_u \): Shear in the web at the section under consideration due to the factored loads

1) Unstiffened Webs
The nominal shear resistance of unstiffened webs shall be taken as:

\[ V_n = V_{cr} = CV_p \]  
(2.46)

\[ V_p = 0.58 F_{yw} Dt_w \]  
(2.47)

Where,
\( V_{cr} \): Shear-buckling resistance
\( V_p \): Plastic shear force
\( C \): Ratio of shear-buckling resistance to shear yield strength

[Table2.24] Calculation of Ratio of shear-buckling resistance to shear yield strength, C

<table>
<thead>
<tr>
<th>Case</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>[ \frac{D}{t_w} \leq 1.12 \sqrt{\frac{E_k}{F_{yw}}} ]</td>
<td>( C = 1.0 )</td>
</tr>
<tr>
<td>1.12 [ \sqrt{\frac{E_k}{F_{yw}}} ] \leq \frac{D}{t_w} \leq 1.40 \sqrt{\frac{E_k}{F_{yw}}}</td>
<td>( C = \frac{1.12}{D/t_w} \sqrt{\frac{E_k}{F_{yw}}} )</td>
</tr>
<tr>
<td>1.40 [ \sqrt{\frac{E_k}{F_{yw}}} ] \leq \frac{D}{t_w} ]</td>
<td>( C = \frac{1.57}{D/t_w} \left( \frac{E_k}{F_{yw}} \right) )</td>
</tr>
</tbody>
</table>

Where,
\( k \): Shear-buckling coefficient

\[ k = 5 + \frac{5}{\left( \frac{d_o}{D} \right)^2} \]  
(2.48)
2) Stiffened Webs

The nominal shear resistance is calculated differently for the two types of stiffened webs: interior web panels and end web panels. All webs with a support assigned on its i or j node in the Span Information (Fig.2.22) are considered as end panels and the others are considered as interior web panels.

![Classification of End Panel and Interior Panel](image)

---

### End panels

The nominal shear resistance, $V_n$, of a web end panel shall be taken as:

$$ V_n = V_{cr} = CV_p $$

$V_p = 0.58F_{yw}Dt_w $  \hspace{1cm} (2.49)

### Interior web panels

There are two cases of an interior web panel as shown in the following table:

<table>
<thead>
<tr>
<th>Case</th>
<th>$V_n$</th>
<th>$V_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\frac{2Dt_w}{(b_{jc}t_{fc}+b_{jc}t_{j})} \leq 2.5$</td>
<td>$V_n = V_p \left[ C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}} \right]$</td>
<td>$V_p = 0.58F_{yw}Dt_w$</td>
</tr>
<tr>
<td>Otherwise</td>
<td>$V_n = V_p \left[ C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}} + \frac{d_o}{D} \right]$</td>
<td>$V_p = 0.58F_{yw}Dt_w$</td>
</tr>
</tbody>
</table>

Where,

- $d_o$ : Transverse stiffener spacing
- $V_n$ : Nominal shear resistance of the panel
③ User’s option

Users need to specify that the web is stiffened by checking the check box at:
▶ Composite Steel Girder Design Parameters > Options for Strength Limit State > 'Post-buckling Tension - Field Action for Shear Resistance (6.10.9.3.2)'. Depending on the user’s verification, the calculation will differ as shown in the following table:

<table>
<thead>
<tr>
<th>Check</th>
<th>[V_n, V_p]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 ( \frac{2Dt_w}{(b_f t_{fc} + b_p t_{fb})} ) \leq 2.5</td>
<td>( V_n = V_p = C \left( \frac{0.87(1-C)}{1+\left(\frac{d_n}{D}\right)^2} \right) )</td>
</tr>
<tr>
<td>On</td>
<td>( V_p = 0.58 F_{yw} D t_w )</td>
</tr>
<tr>
<td>Otherwise</td>
<td>( V_n = V_p = 0.58 F_{yw} D t_w )</td>
</tr>
<tr>
<td>Off</td>
<td>( V_n = V_p = C V_p )</td>
</tr>
</tbody>
</table>

\[\begin{align*}
AASHTO LRFD 12 & \text{ (Eq.6.10.9.3.2-2)}
\end{align*}\]

\[\begin{align*}
AASHTO LRFD 12 & \text{ (Eq.6.10.9.3.2-8)}
\end{align*}\]

\[\begin{align*}
\text{Service Limit State} & \text{ (6.10.4.2)}
\end{align*}\]

### 1.3 Service Limit State

Flange stress for permanent deformation and web bend-buckling are verified at the service limit state.

The program does not check elastic deformation. Elastic deformation can be reviewed manually after moving load analysis at: ▶ Results > Deformation

At the completion stage of the construction, the program applies Service II load combination, specified in AASHTO LRFD 12 Article 6.10.4.2, and reviews the permanent deformation. Therefore, the permanent deformation is reviewed only for the composite section since the section cannot be non-composite in the completed state. But, the software can assume the concrete deck in the composite section to be ineffective as per 6.10.4.2.1, which states that the concrete deck may be assumed to be ineffective for both positive and negative flexure, provided that the maximum tensile stresses in concrete deck at the section under consideration caused by Service II loads are greater than 2\( f_r \). Software performs this check and determines whether to consider the concrete deck to be effective or not.
The service limit state is reviewed as shown in the flow chart follows:

1.3.1 Flexure

Flange shall satisfy the following requirements at the service limit state for the top and bottom flanges of the composite sections:

(1) Top Flange
The top steel flange of composite section shall satisfy the following requirement.

\[ f_f \leq 0.95 R_f F_{sf} \]  \hspace{1cm} (2.51)

(2) Bottom Flange
The bottom steel flange of composite section shall satisfy the following requirement.

\[ f_f + \frac{f_{l}}{2} \leq 0.95 R_f F_{sf} \]  \hspace{1cm} (2.52)

Where,

- \( f_f \) : Flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending
- \( f_{l} \) : Flange lateral bending stress at the section under consideration due to the Service II loads determined; \( f_1 \leq 0.6 F_{lw} \)
- \( F_{sf} \) : Specified minimum yield strength of a flange

Top Flange
AASHTO LRFD 12  
(Eq.6.10.4.2.2-1)

Bottom Flange
AASHTO LRFD 12  
(Eq.6.10.4.2.2-2)
1.3.2 Nominal Bend-buckling Resistance for webs
If composite section is in positive flexure and the web section property satisfies \( \frac{D}{t_w} \leq 150 \),
use the service limit state shall be verified according to:

\[
f_c \leq F_{crw}
\]

Where,

\( f_c \) : Compression-flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending

\( F_{crw} \) : Nominal bending-buckling resistance for webs with or without longitudinal stiffeners

\[
F_{crw} = \frac{0.9 E_k \left( \frac{D}{t_w} \right)}{k} \leq \text{Min}(R_y F_{yw}, F_{yw} / 0.7)
\]

Where,

\( k \) : bend-buckling coefficient

\[
k = \frac{9}{(D_y / D_t)^2}
\]

1.3.3 Concrete Deck
The program verifies the stress of the concrete deck for shored construction cases in positive flexure as per Article 6.10.1.7.

\[
f_{deck} \leq \Phi f_r
\]

Where,

\( f_{deck} \) : longitudinal flexure stresses in the concrete deck with short-term modular ratio, \( n \)

\( \Phi f_r \) : \( \Phi \) shall be taken as 0.9 and \( f_r \) shall be taken as the modulus of rupture of the concrete, 0.24 \( \sqrt{f'_c} \) as per Article 6.10.1.7

1.4 Check Constructibility
Constructibility shall be verified for the three categories as shown in the following chart:

[Fig 2.48] Flow chart of Constructibility limit stage

The constructibility is checked based on the design member forces under Dead (Before).

1.4.1 Flexure
The program shall verify lateral bending stress in discretely braced compression and tension flanges during the construction stages, for when slabs are not deflected yet. Therefore, the program considers all flanges as discretely braced flanges for the design check. Constructibility is verified in terms of flexural resistance according to the following flow chart:
(1) Section classification

[Table 2.27] Section classification

<table>
<thead>
<tr>
<th>Case</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{2D_{c}}{t_{w}} \leq 5.7 \sqrt{\frac{E}{F_{cc}}} )</td>
<td>Compact or non-compact Web</td>
</tr>
<tr>
<td>( \frac{2D_{c}}{t_{w}} &gt; 5.7 \sqrt{\frac{E}{F_{cc}}} )</td>
<td>Slender Web</td>
</tr>
</tbody>
</table>

(2) Discretely braced flanges in Compression

Discretely braced flanges in compression are verified according to the following three equations.

1) Check flange nominal flexure yielding

For the critical stages of construction, the following equation shall be satisfied. However, the requirement does not need to be checked if a section has slender web and its \( f_{i} \) is equal to 0.
\[ f_y + f_i \leq \phi_f R_y F_{yfc} \]  

(2.57)

2) Check local buckling and lateral torsional buckling as per Article 6.10.8.2.2 and Article 6.10.8.2.3 respectively

\[ f_y + \frac{1}{3} f_i \leq \phi_f F_{nc} \]  

(2.58)

3) Check web bend buckling as per Article 6.10.1.9

Only for the sections with slender webs, the following equation shall be checked.

\[ f_{bw} \leq \phi_f F_{crw} \]  

(2.59)

Where,

\[ \phi_f \]: resistance factor for flexure specified in 6.5.4.2

\[ f_{bw} \]: flange stress calculated without consideration of flange lateral bending.

\[ f_i \]: flange lateral bending stress, \( f_i \leq 0.6 F_w \)

\[ F_{crw} \]: nominal bending-buckling resistance for webs.

\[ F_{nc} \]: nominal flexure resistance of the flange.

(3) Discretely braced flanges in Tension

The following equation shall be checked for discretely braced tension flanges.

\[ f_f + f_i \leq \phi_f R_y F_{my} \]  

(2.60)

1.4.2 Concrete Deck

If the longitudinal tensile stress in concrete deck determined as per Article 6.10.1.1.1d, exceeds \( \Phi f_t \), then the minimum one percent longitudinal reinforcement determined as per Article 6.10.1.7 is required at the section. Code recommends that the minimum reinforcement should be No. 6 bars or smaller spaced at not more than 12 inches.

The total tensile force in the concrete deck is transmitted from the deck through the shear connectors to the top flange. Software assumes the shear connectors to be sufficiently present at this location to resist the force and prevent potential crushing of concrete. Software doesn’t calculate the length over which this force must be transmitted. Shear connector pitch calculations are as per Fatigue and Strength Limit State only.

\[ F_{deck} \leq \phi_f \]  

(2.61)

Where,

\[ f_t = 0.24 \sqrt{f_y} \]  

modulus of rupture of the normal-weight concrete

\[ \phi : 0.9 \]

\[ F_{deck} \]: Longitudinal tensile stress in the concrete deck

\[ F_{deck} = \frac{My}{In} \]  

(2.62)

Where,

\[ n = \frac{E_s}{E_c} \]
1.4.3 Shear

The program shall use the load combinations defined in the Load Combination Type (Refer to Section 1.4.2 in this chapter) for the verification of the shear strength. Webs shall satisfy the following requirement during critical stages of construction.

\[ V_u \leq \phi V_{cr} \]  
(2.63)

Where,

\( V_u \): shear in the web at the section under consideration due to the factored loads
\( \phi \): resistance factor for shear, \( \phi_v = 1.0 \) (Fig. 2.41)
\( V_{cr} \): shear buckling resistance determined from Eq. 6.10.9.3.3

The program checks the nominal resistance for unstiffened webs and stiffened webs with the same formula as the tension field action is not considered for Constructibility check.

1) Unstiffened/Stiffened web

\[ V_u = V_{cr} = CV_r \]  
(2.64)

\[ V_r = 0.58F_{sw} D t_w \]  
(2.65)

2) Calculation of Ratio of shear-buckling resistance to shear yield strength, C

Please refer to Section 1.2.2 in this chapter for the calculation of C.

1.5 Fatigue Limit State

For horizontally curved I-girder bridges, the range of fatigue stress due to major-axis bending plus lateral bending shall be investigated. Article 6.10.5 also mentions the requirements for Fracture. But Fracture Limit State is not considered in midas Civil. Code specifies the fatigue live load in Article 3.6.1.4 for the Fatigue check. But in the software, fatigue check is performed only for the moving load defined for the analysis.

For considering the fatigue live load as specified in code, user will have to define a user defined vehicle and then manually edit the auto generated load combinations, so that the fatigue moving vehicle is the only vehicle considered for fatigue check and is only included in fatigue combination.

For fatigue limit state, software assumes the shear connector to be provided along the entire length of the girder, ensuring composite action. Therefore, the concrete deck is assumed to be effective in computing all stresses and stress ranges applied to the composite section in the subsequent fatigue calculations.

1.5.1 Load Combinations Used for Fatigue Limit State Check

For this part of design check, AASHTO LRFD 07 and 12 are applied differently in the program. Please refer to Section 5.1 in this chapter for more information. Fatigue limit shall be verified according to the two paths. Fatigue limit shall be verified according to Section 1.5.3(1) for the load combinations that are inputted as Fatigue 1 Limit State Load Combination Type (Section 1.4.2 in Chapter "Modeling and Design Variables"). For the load combinations that are inputted as Fatigue 2 Limit State, Section 1.5.3(2) shall be followed.

The program verifies the load combinations defined in the Load Combination Type. If users define \( (ADTT)_{SL} \leq 75 \text{ year} \) \((ADDTT)_{SL}\)' Equivalent to Infinite Life, the verification shall consider the Fatigue II Load Combination. Otherwise, this combination of fatigue limit state shall be skipped and Fatigue I Load Combination shall be considered for verification.
1.5.2 Fatigue Limit State

For the compression flange, compressive stress due to unfactored dead load is compared with the tensile stress due to factored live load before performing the fatigue check. If two times the tensile stress due to factored live load is greater than the compressive stress due to unfactored dead load, then only the fatigue check is performed.

For the tension flange, fatigue check is always performed.

(1) The fatigue limit state shall be verified according to the following.

\[
\gamma (\Delta F) \leq (\Delta F)_{n}
\]  

(2.68)

Where,
\( \gamma \) : Load factor for the fatigue load combination.
\( (\Delta F) \) : Force effect, live load stress range due to the passage of the fatigue load.
\( (\Delta F)_{n} \) : Nominal fatigue resistance.

(2) The load factor, \( \gamma \), specified in the table below, shall be applied for the fatigue load combination. These factors are automatically considered by the software, while auto generating the load combinations.

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>DC, DD, DW, EH, EV, ES, EL, PS, CR, SH</th>
<th>LL, IM, CE, BR, PL, LS</th>
<th>WA</th>
<th>WS</th>
<th>WL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fatigue I - LL, IM &amp; CE only</td>
<td>-</td>
<td>1.50</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Fatigue II - LL, IM &amp; CE only</td>
<td>-</td>
<td>0.75</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

1.5.3 Nominal Fatigue Resistance

The nominal fatigue resistance is calculated differently for the load combinations in the Service 1 Limit State and the Service 2 Limit State.
(1) Nominal Fatigue Resistance Due to the Load Combinations for Fatigue I Limit State
The program shall calculate the nominal fatigue resistance according to the input categories made in the fatigue dialog box (Fig. 2.22).

\[(\Delta F)_n = (\Delta F)_{TH} \quad \text{(2.69)}\]

The program shall apply the nominal fatigue resistance according to Categories A, B, B', C, C', D, E, and E', specified in the table below. For all other cases, the nominal fatigue resistance shall be considered as 24.0 ksi (165.0 MPa).

<table>
<thead>
<tr>
<th>Detail Category</th>
<th>Threshold</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>US Unit(ksi)</td>
</tr>
<tr>
<td>A</td>
<td>24.0</td>
</tr>
<tr>
<td>B</td>
<td>16.0</td>
</tr>
<tr>
<td>B'</td>
<td>12.0</td>
</tr>
<tr>
<td>C</td>
<td>10.0</td>
</tr>
<tr>
<td>C'</td>
<td>12.0</td>
</tr>
<tr>
<td>D</td>
<td>7.0</td>
</tr>
<tr>
<td>E</td>
<td>4.5</td>
</tr>
<tr>
<td>E'</td>
<td>2.6</td>
</tr>
</tbody>
</table>

(2) Nominal Fatigue Resistance due to the Load Combinations for Fatigue II Limit State
If Fatigue Resistance is verified for Fatigue Load Combination 2, the below equation shall be used. For the verification, the program uses the design parameter values inputted by users in the Fatigue dialog box (Fig. 2.22).

\[(\Delta F)_n = \left(\frac{A}{N}\right)^{1/3} \quad \text{(2.70)}\]

\[N = (365)(75)(ADTT)_{SL} \quad \text{(2.71)}\]

Where,
\[A : \text{Constant taken from Table 2.30}\]
\[n : \text{Number of stress range cycles per truck passage taken from Table 2.31}\]
Table 2.31 Cycles per Truck Passage, n

<table>
<thead>
<tr>
<th>Longitudinal Members</th>
<th>Span Length</th>
<th>&gt;40.0 ft</th>
<th>≤40.0 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple span Girders</td>
<td></td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Continuous Girders</td>
<td>Near interior support</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>Elsewhere</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Cantilever Girders</td>
<td></td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Orthotropic Deck plate Connections</td>
<td></td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Trusses</td>
<td></td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Transverse Members</td>
<td>Spacing</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 20.0 ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤20.0 ft</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The n value inputted in the Fatigue Parameter dialog box (Fig. 2.22) according to Table 2.31 is used for the calculation.

\[(ADTT)_{SL}\] : ADTT for single lane

1.5.4 Special Fatigue Requirement for Webs

The fatigue limit state shall be verified in terms of shear buckling resistance as:

\[V_u \leq V_{cr}\] \hspace{1cm} (2.72)

Where,

\[V_u : \text{shear in the web at the section under consideration due to the unfactored permanent loads plus the factored fatigue load}\]

\[V_{cr} = CV_p\] \hspace{1cm} (2.73)

\[V_p = 0.58 F_{yw} D t_w\] \hspace{1cm} (2.74)
2. Box / Tub Girder Section

2.1 Introduction
Design of Box/Tub steel composite sections follow the same procedure as for I-Girders.

2.2 Strength Limit State
The program checks the strength limit states for the flexure, shear and ductility of the composite sections.

2.2.1 Ductility
Ductility shall be checked to prevent premature crushing of concrete. If a section is under positive flexure, ductility shall be verified as:

\[ D_r \leq 0.42 D_t \]  \hspace{1cm} (2.75)

2.2.2 Flexure
(1) Classification of Composite Section for Flexure
There are four cases for checking flexural resistance of Box/Tub composite sections as shown below.

The webs that are under \textbf{positive flexure} and satisfy the following requirements shall be
considered as compact sections. Otherwise, they shall be considered as non-compact sections for the positive flexure design check. Sections of a curved bridge are considered to be non-compact.

- Flange and web yield strength do not exceed 70 ksi (485 MPa)
- Web satisfies the requirements in Article 6.11.2.1 as shown below.
  
  Webs without longitudinal stiffeners: \( \frac{D}{t_w} \leq 150 \)
  
  Webs with longitudinal stiffeners: \( \frac{D}{t_w} \leq 300 \)
- Web slenderness limit satisfies the requirements in Article 6.11.6.2.2

\[ 2D_{cp}/t_w \leq 3.76\sqrt{(E/F_{yc})} \]

The classification of the section under negative flexure, as compact /noncompact /slender is not required for the design checks.

(2) Case 1: Positive Flexural Moment in Compact Section

\[
\begin{align*}
\text{Case 1: Check flexural resistance of Positive Flexure Moment in Compact Section} \\
6.11.7.1
\end{align*}
\]

If \( D_p \leq 0.1D_t \)

Calculate \( M_n \)

\[ M_n = M_p \left( 1.07 - 0.7 \frac{D_p}{D_t} \right) \]

6.10.7.1.2-2

Check Flexural Resistance

\[ M_u \leq \phi_f M_n \]

6.10.7.1.1-1

End

[Fig.2.53] Case 1: Flow Chart of Flexural resistance for Compact Section in Positive Flexure Moment

For compact sections, flexure at the strength limit state shall be verified as:

\[ M_u \leq \phi_f M_n \] (2.76)

Where,

1) Bending moment about the major-axis( \( M_u \))
   \( M_u \) is the bending moment about the major axis due to the factored loads. The maximum bending moment from the load combinations, applied to Strength Limit State in the Load Combination Type (Refer to Chapter "Modeling Design Variable" Section 1.4.2) is applied as \( M_u \).

2) Nominal flexure resistance(\( M_n \))

<table>
<thead>
<tr>
<th>Case</th>
<th>( M_n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D_p \leq 0.1D_t )</td>
<td>( M_n = M_p )</td>
</tr>
</tbody>
</table>
If a section is under positive flexure, plastic moment is calculated for the location of the plastic neutral axis. For more information, please refer to Chapter "Introduction" Section 3.2.

3) \( \phi_f \)

Flexural resistance factor are taken as 1.00 in AASHTO LRFD 12. However, if the factor is defined by users in the design parameter dialog box, the user defined value is utilized as a priority.

(3) Case 2 : Non-compact Section in Positive Moment

For non-compact sections, flexural strength limit state is verified as shown in the flow chart follows. Webs of a curved bridge is considered to be non-compact sections.

![Flow Chart of Flexural Resistance for Non-compact Section in Positive Flexure Moment](image)

1) Compression Flange
At the strength limit state, compression flanges shall satisfy the following in terms of flexure.

\[
f_{bu} \leq \phi_f F_{nc} \quad (2.77)
\]

The nominal flexural resistance of the compression flange, \( F_{nc} \), is taken differently for box and tub sections as:
### Table 2.33 Calculation of $F_{nc}$

<table>
<thead>
<tr>
<th>Section Type</th>
<th>$F_{nc}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Box</td>
<td>$F_{nc} = F_p R_h F_{yc} \Delta$</td>
</tr>
<tr>
<td>Tub</td>
<td>$F_{nc} = F_p R_h F_{yc}$</td>
</tr>
</tbody>
</table>

Where,

$$\Delta = \left[1 - 3 \left( \frac{f_v}{F_{yc}} \right)^2 \right]$$

in which:

$$f_v = \frac{T}{2A_f \lambda}$$

(2.78)

$\Delta$: a factor dependent on St. Venant torsional shear stress in the bottom flange of the tub section.

$R_h$: Web load shedding factor.

### Table 2.34 Calculation of $R_b$

<table>
<thead>
<tr>
<th>Case</th>
<th>$R_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constructibility Limit State is reviewed</td>
<td>1.0</td>
</tr>
<tr>
<td>Composite web under positive flexure satisfies Article 6.10.2.1.1 &amp; 6.11.2.1.2</td>
<td></td>
</tr>
<tr>
<td>One or more longitudinal stiffener &amp;</td>
<td></td>
</tr>
<tr>
<td>$\frac{D}{t_w} \leq 0.95 \sqrt{\frac{E_k}{F_{yc}}}$</td>
<td></td>
</tr>
<tr>
<td>$2D_c \leq \lambda_{ty}$</td>
<td></td>
</tr>
<tr>
<td>Otherwise,</td>
<td></td>
</tr>
<tr>
<td>$R_b = 1 - \left( \frac{a_{wc}}{1200 + 300a_{wc}} \right) \left( \frac{2D_c - \lambda_{ty}}{t_w} \right) \leq 1.0$</td>
<td></td>
</tr>
</tbody>
</table>

$R_b$: Hybrid Factor.

### Table 2.35 Calculation of $R_h$

<table>
<thead>
<tr>
<th>Case</th>
<th>$R_h$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hybrid Section</td>
<td>$R_h = \frac{12 + \beta(3\rho - \rho^2)}{12 + 2\beta}$ in which: $\beta = \frac{2D_t}{A_{y_k}}$</td>
</tr>
<tr>
<td>Non-Hybrid or Web strength &gt; flange strength</td>
<td>1.0</td>
</tr>
</tbody>
</table>
2) Tension Flange
At the strength limit state, tension flanges shall satisfy:

\[ f_{bu} \leq \phi f_{nt} \]  

(2.79)

For both box and tub type composite sections, the nominal flexure resistance of tension flange, \( F_{nt} \) shall be calculated as:

\[ F_{nt} = R_h F_{y} \Delta \]  

(2.80)

Where,

\[ \Delta = \sqrt{1 - 3 \left( \frac{f_v}{f_{nt}} \right)^2} \]  

in which: \[ f_v = \frac{T}{2A_f f_{p}} \]  

(2.81)

If \( 1 - 3 \left( \frac{f_v}{f_{y,c}} \right)^2 < 0 \), consider \( \Delta = 0 \) so that \( F_{nt} = 0 \)

(4) Case 3 & Case 4: Negative Flexure
Flexural resistance of negative flexure moment shall be verified as shown in the flow chart below.

[Fig.2.55] Case 3 & Case 4: Flow Chart of Flexural Resistance for Negative Flexural Moment

(5) Case 3: Compression Flange in Negative Flexural Moment
For this part of design check, AASHTO LRFD 07 and 12 are applied differently in the program. Please refer to Section 5.4 in this chapter for more information.
The program shall distinguish unstiffened and longitudinally stiffened elements depending on whether the longitudinal stiffener is applied on the compression flanges in the section property dialog box. At the strength limit state, the following requirement shall be satisfied in terms of flexure:

\[ f_{bu} \leq \phi_f F_{nc} \tag{2.82} \]

1) Unstiffened Flange
For unstiffened flanges, the following requirement shall be satisfied:
\[ F_{nc} = F_{cb} \sqrt{1 - \left( \frac{f_y}{\phi_y F_{cv}} \right)^2} \]  \hfill (2.83)

<table>
<thead>
<tr>
<th>Case</th>
<th>( F_{cb} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \lambda_f \leq \lambda_p )</td>
<td>( F_{cb} = R_h R_F F_{ch} \Delta )</td>
</tr>
<tr>
<td>( \lambda_p &lt; \lambda_f \leq \lambda_r )</td>
<td>( F_{cb} = R_h R_F F_{ch} \left[ \Delta - \left( \frac{\Delta - 0.3}{R_h} \right) \left( \lambda_f - \lambda_p \right) \right] )</td>
</tr>
<tr>
<td>( \lambda_r &lt; \lambda_f )</td>
<td>( F_{cb} = \frac{0.9 ER_h^2}{\lambda_f^2} )</td>
</tr>
</tbody>
</table>

Where,
\( \lambda_f \) : slenderness ratio for the compression flange

\[ \lambda_f = \frac{b_f}{t_f c} \text{, } \lambda_p = 0.57 \sqrt{\frac{E_k}{F_{cv} \Delta}} \text{ and } \lambda_r = 0.95 \sqrt{\frac{E_k}{F_{cy}}} \]  \hfill (2.84)

For unstiffened flanges, \( k = 4.0 \text{ and } k_s = 5.34 \).

\[ \Delta = \sqrt{1 - 3 \left( \frac{f_s}{F_{sy}} \right)^2} \text{ in which } f_s = \frac{T}{2 A_f t_f} \]  \hfill (2.85)

\( F_{sy} \) : smaller of the compression-flange stress at the onset of nominal yielding, with consideration of residual stress effects, or the specified minimum yield strength of the web

\[ F_{sy} = (\Delta - 0.3) F_{sy} \leq F_{sy} \]  \hfill (2.86)

<table>
<thead>
<tr>
<th>Case</th>
<th>( F_{cv} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \lambda_f \leq 1.12 )</td>
<td>( F_{cv} = 0.85 F_{sy} )</td>
</tr>
<tr>
<td>1.12 ( \sqrt{\frac{E_k}{F_{sy}}} &lt; \lambda_f \leq 1.40 \sqrt{\frac{E_k}{F_{sy}}} )</td>
<td>( F_{cv} = \frac{0.65 \sqrt{F_{sy} E_k}}{\lambda_f} )</td>
</tr>
<tr>
<td>1.40 ( \sqrt{\frac{E_k}{F_{sy}}} &lt; \lambda_f )</td>
<td>( F_{cv} = \frac{0.9 E_k}{\lambda_f^2} )</td>
</tr>
</tbody>
</table>

2) Longitudinally Stiffened Flange

Also for longitudinally stiffened flanges, the following requirement shall be satisfied as for unstiffened flanges. However, the plate-buckling coefficients, \( k \text{ and } k_s \), shall no longer be constant but calculated to account for \( F_{nc} \).

\[ F_{nc} = F_{cb} \sqrt{1 - \left( \frac{f_y}{\phi_y F_{cv}} \right)^2} \]  \hfill (2.87)

For longitudinally stiffened compression flanges, \( k \text{ and } k_s \) are determined depending on the number and location of stiffeners applied to the flanges.

1) Plate-Buckling Coefficient for Uniform Normal Stress \( k \)

Depending on the number of uniformly spaced stiffeners, \( k \) shall be taken as:
### Calculation of k

<table>
<thead>
<tr>
<th>Case</th>
<th>( n = 1 )</th>
<th>( n \geq 2 )</th>
</tr>
</thead>
</table>
| \( k \) | \[
  k = \left( \frac{8I_s}{wF_{tc}^3} \right)^{\frac{1}{3}}
\]
| \( k \) | \[
  k = \left( \frac{0.894I_s}{wF_{tc}^3} \right)^{\frac{1}{3}}
\]

\[ 1.0 \leq k \leq 4.0 \]

#### 2 Plate-Buckling Coefficient for Shear Stress (\( k_s \))

\[
k_s = \frac{5.34 + 2.84 \left( \frac{I_s}{wF_{tc}^3} \right)^{\frac{1}{3}}}{(n+1)^2} \leq 5.34 \quad (2.88)
\]

Where,
- \( I_s \): moment of inertia of a single longitudinal flange stiffener about an axis parallel to the flange and taken at the base of the stiffener
- \( n \): number of equally spaced longitudinal flange stiffeners
- \( w \): larger of the width of the flange between longitudinal flange stiffeners or the distance from a web to the nearest longitudinal flange stiffener

![Definition of w](image)

#### 6 Case 4: Tension Flange in Negative Flexural Moment

For tension flanges, flexural resistance limit state shall be verified as shown in the flow chart:

The flexural resistance of negative flexure moment and tension flange will be checked by the process indicated in the flow chart below.

![Flow Chart of Flexural Resistance for Tension Flange in Negative Moment](image)

The tension flanges shall be verified according to:

\[
 f_{tu} \leq \phi_f F_{nt} \quad (2.89)
\]
F_{nt} shall be taken as:

\[
F_{nt} = \begin{cases} 
R_n F_{yt} & \text{for Tub} \\
R_n F_{yt} \Delta & \text{for Closed-Box}
\end{cases}
\]

\[
\Delta = \sqrt{1 - \left( \frac{f_s}{F_{yt}} \right)^2} \quad \text{in which} \quad f_s = \frac{T}{2A_f f_y}
\]

### 2.2.3 Shear

Box and tube type steel composite sections shall be verified for its shear strength as shown in the flow chart:

![Flow Chart of Shear Resistance](image)

The program classifies stiffened and unstiffened webs as shown in the table below:

<table>
<thead>
<tr>
<th>Case</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without a longitudinal stiffener and with transverse stiffener spacing not exceeding 3D</td>
<td>Stiffened web</td>
</tr>
<tr>
<td>With one or more longitudinal stiffeners and with a transverse stiffener spacing not exceeding 1.5D</td>
<td>Stiffened web</td>
</tr>
<tr>
<td>Otherwise</td>
<td>Unstiffened web</td>
</tr>
</tbody>
</table>
(1) Shear Strength Verification
Shear strength shall be verified as shown in the flow chart:

Shear strength shall be verified as:

\[ V_u \leq \phi_V V_n \]  \hspace{1cm} (2.90)

Where,

\( \phi_V \): resistance factor for shear

\( V_u \): shear in the web at the section under consideration due to the factored loads

1) Unstiffened web
For unstiffened webs, the nominal shear resistance \((V_n)\) shall be taken as:

\[ V_n = V_{cr} = CV_p \]  \hspace{1cm} (2.91)

in which:

\[ V_p = 0.58 F_{yw} Dt_w \]  \hspace{1cm} (2.92)

Where,

\( C \): ratio of the shear-buckling resistance to the shear yield strength

\( V_p \): plastic shear force
2) Stiffened Web Shear Strength
Program shall determine whether a stiffened web belongs to an end panel or interior panel depending on whether its nodes are supported or not in the span information. The web shall be first identified as an end panel or an interior panel and, then, its shear strength shall be verified. If a web is supported at its nodes, the web belongs to an end panel; if not supported, it belongs to an interior panel.

① End panels
For end panel webs, the nominal shear resistance shall be taken as:

\[ V_n = V_{cr} = CV_p \]  
(2.93)

in which:

\[ V_p = 0.58F_{yw}Dt_w \]  
(2.94)

② Interior panels
For interior panels, the nominal shear resistance shall be taken as:

*Table 2.41* Calculation of \( V_n \) for Interior Panel

<table>
<thead>
<tr>
<th>Case</th>
<th>Nominal shear resistance ( (V_n) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>[ \frac{2Dt_w}{(b_\beta t_\beta + b_\mu t_\mu)} \leq 2.5 ]</td>
<td>[ V_n = V_p \left[ C + \frac{0.87(1-C)}{\sqrt{1 + \left( \frac{d_w}{D} \right)^2}} \right] ]</td>
</tr>
</tbody>
</table>

Otherwise,

\[ V_n = V_p \left[ C + \frac{0.87(1-C)}{\sqrt{1 + \left( \frac{d_w}{D} \right)^2 + \frac{d_o}{D}}} \right] \]

Where,

*Table 2.42* Calculation of Ratio of the shear buckling resistance to the shear yield strength, \( C \)

<table>
<thead>
<tr>
<th>Case</th>
<th>( C )</th>
</tr>
</thead>
<tbody>
<tr>
<td>[ \frac{D}{t_w} \leq 1.12 \frac{E_k}{F_{yw}} ]</td>
<td>( C = 1.0 )</td>
</tr>
<tr>
<td>[ 1.12 \frac{E_k}{F_{yw}} &lt; \frac{D}{t_w} \leq 1.40 \frac{E_k}{F_{yw}} ]</td>
<td>( C = \frac{1.12}{\frac{D}{t_w}} ) ( \sqrt{\frac{E_k}{F_{yw}}} )</td>
</tr>
<tr>
<td>[ 1.40 \frac{E_k}{F_{yw}} &lt; \frac{D}{t_w} ]</td>
<td>( C = 1.57 \left( \frac{D}{t_w} \right)^2 \left( \frac{E_k}{F_{yw}} \right) )</td>
</tr>
</tbody>
</table>

Where,

\[ k: \text{shear-buckling coefficient} \]
\[ k = 5 + \frac{5}{\left( \frac{d_o}{D} \right)^2} \]  
(2.95)

End panels
AASHTO LRFD 12
(Eq.6.10.9.3.3-1)

AASHTO LRFD 12
(Eq.6.10.9.3.3-2)

Interior panels
AASHTO LRFD 12
(Eq.6.10.9.3.2-2)

AASHTO LRFD 12
(Eq.6.10.9.3.2-8)

C
AASHTO LRFD 12
(Eq.6.10.9.3.2-4)

AASHTO LRFD 12
(Eq.6.10.9.3.2-5)

AASHTO LRFD 12
(Eq.6.10.9.3.2-6)

k
AASHTO LRFD 12
(Eq.6.10.9.3.2-7)
(2) Check for Inclination

For box and tube composite sections, inclination of webs shall be considered. Shear force on each section shall be evenly applied to its two webs after the consideration of the incline angle as:

$$V_{ui} = \frac{V_u}{\cos \theta}$$

(2.96)

Where,

- $V_{ui}$: shear on each web due to the factored loads
- $V_u$: vertical shear due to the factored loads on one inclined web
- $\theta$: the angle of inclination of the web plate to the vertical (degrees)

### 2.3 Service Limit State

For box and tub composite sections, flexure and web bend-buckling at the service limit state are verified as shown in the flow chart below. The program shall verify service limit state for the composite sections at the completion stage of construction. Load combinations defined in the Load Combination Type (Please Refer to Chapter "Modeling Design Variable" Section 1.4.2) shall be used for the verification of the service limit state.

[Fig.2.62] Flow Chart of Service Limit State
2.3.1. Flexure

Flexure shall be verified at top and bottom flanges. As per Article C6.11.4, Eq. 6.10.4.2.2-1 and 6.10.4.2.2-2 are checked only for compact sections in positive flexure. Thus in midas Civil, these equations are not checked for negative flexure and noncompact sections in positive flexure.

(1) Verification of Top steel flange of composite sections for flexure
Serviceability of top steel flanges shall be verified by comparing the stress as:

\[ f_y \leq 0.95 R_y F_{sf} \]  \hspace{1cm} \text{(2.97)}

(2) Verification of Bottom steel flange of composite sections for flexure
Serviceability of bottom steel flanges shall be verified by examining flexure as shown in the equation below. If a web is under positive flexure and satisfies the requirements in AASHTO LRFD Article 6.11.2.1.2, its strength shall be determined to be satisfactory and verification shall be skipped. For box and tub composite sections, flange lateral bending stress shall be assumed as 0 for the design check.

\[ f_y + \frac{f_f}{2} \leq 0.95 R_y F_{sf} \quad \text{where,} \quad f_f = 0 \]  \hspace{1cm} \text{(2.98)}

2.3.2. Web Bend Buckling

Except for sections in positive flexure in which the web satisfies the requirement of Article 6.11.2.1.2, all sections shall satisfy Eq.6.10.4.2.2-4 shown below.

Webs shall be verified in terms of bend-buckling as:

\[ f_i \leq F_{crw} \]  \hspace{1cm} \text{(2.99)}

Where,

- \( f_i \): compression-flange stress
- \( F_{crw} \): nominal bend-buckling resistance for webs

\[ F_{crw} = \frac{0.9 E_k}{D} \leq \text{Min}(R_y F_{fw}, F_{fw} / 0.7) \]  \hspace{1cm} \text{(2.100)}

in which:

- \( k \): bend-buckling coefficient

\[ k = \frac{9}{(D_i / D)^2} \]  \hspace{1cm} \text{(2.101)}

Where,

- \( D_i \): Depth of the web in compression in the elastic range

2.4 Check Constructibility

For box and tub composite sections, constructibility shall be verified in terms of flexure and shear. Member force under Dead (Before) shall be used as the design member force for the verification of constructibility limit strength.

2.4.1 Flexure

The program shall verify flexural strength by assuming that concrete hardening has not occurred yet and all section are discretely braced. The flexural verification shall be done in three cases as shown in the figure follows.
(1) Open Flange (top flange of tub section) in Compression and Tension

1) Open flange in compression

For tub composite sections, compression top flanges shall be verified for yielding, flexure and bend buckling of webs, as shown in the equation below. If \( f_i = 0 \) for slender webs, AASHTO LRFD 12 Eq.6.10.3.2.1-1 shall not be verified.

\[
f_{bu} + \frac{f_i}{3} \leq \phi_f R_f F_{wc} \quad \text{and} \quad f_{bu} + \frac{f_i}{3} \leq \phi_f F_{nc}
\]  

(2.102)

For slender webs, bend-buckling shall be verified as:

\[
f_{bu} \leq \phi_f F_{crw}
\]  

(2.103)

2) Open flange in tension

For tub composite sections, tension top flanges shall satisfy the requirement of Eq. 6.10.3.2.2-1 which is same as that for I girder.

(2) Noncomposite box flange (top flange of box section and bottom flange of tub or box section) in Compression and Tension (for constructability check, the top flange of box section is designed as a noncomposite box flange)

1) Noncomposite box flange in compression

For box flanges in compression, constructability shall be examined based on the compressive stress due to flexure and bend buckling on webs. For sections with compact or noncompact webs, Eq. 6.11.3.2-2 shall not be checked as per Article 6.11.3.2.

- Verification of compression stress due to flexure: \( f_{bu} \leq \phi_f F_{nc} \)  
  (2.104)
- Verification of bend buckling on webs: \( f_{bu} \leq \phi_f F_{crw} \)  
  (2.105)

2) Noncomposite box flange in tension and continuously braced box flange in tension or compression shall satisfy the following requirement:

\[
f_{bu} \leq \phi_f R_f F_{crw} \Delta
\]  

(2.106)

Where,

\[
\Delta = \sqrt{1 - \frac{1}{2} \left( \frac{f_v}{F_{crw}} \right)^2}
\]

in which: \( f_v = \frac{T}{2A_t f} \)
2.4.2 Shear
For the verification of constructibility, shear shall be verified to prevent shear buckling at webs according to the following requirement. The program shall distinguish end panel and interior panel for the verification of shear-buckling resistance.

\[ V_u \leq \phi V_{cr} \]  \hspace{1cm} (2.108)

Where,

\[ V_{cr} = CV_p \] in which: \[ V_p = 0.58 F_y w D_t w \]  \hspace{1cm} (2.109)

2.4.3 Concrete Deck
Constructibility of concrete deck shall not be verified for the box and tub steel composite sections.

2.5 Fatigue Limit State
2.5.1 Load combinations of Fatigue Limit State
In this section, AASHTO LRFD 07 and 12 are applied differently. For more information about the 07 edition, please refer to Section 5.1 in this chapter. For more information on basic considerations and assumptions for Fatigue Limit State, please refer to Section 1.5 in this chapter. Fatigue limit state shall be verified as shown in the flow chart:

The verification of fatigue resistance shall follow Section 2.5.3(1) for the load combinations of Fatigue 1 Limit State in Load Combination Type (Chapter "Modeling Design Variables" Section 1.4.2) and Section 2.5.3(2) for the load combinations of Fatigue 2 Limit State. However, if \('(ADTT)_{SL} \leq 75\text{year (ADTT)}'\) is inputted, Fatigue II Load Combination is verified. Otherwise, the verification needs not to be done.

2.5.2 Fatigue Limit State
As per Article 6.11.5, one additional requirement specified particularly for tub girders sections...
is in regard to longitudinal warping and transverse bending stresses. When tub girders are subjected to torsion, their cross-sections become distorted, resulting in secondary bending stresses. Therefore, longitudinal warping stresses and transverse bending stresses due to cross-section distortion shall be considered for:

- Single tub girder in straight or horizontally curved bridges
- Multiple tub girders in straight bridges that do not satisfy requirements of Article 6.11.2.3
- Multiple tub girders in horizontally curved bridges
- Any single or multiple tub girder with a tub flange that is not fully effective according to the provisions of Article 6.11.1.1.

For consideration of these distortion stresses in the software, Longitudinal Warping Stress Range input is required in the fatigue parameters dialog box. (Fig.2.21)

Fatigue limit state shall be verified per stress unit as:

\[ \gamma (\Delta f) \leq (\Delta F)_n \]  \hspace{1cm} (2.110)

Where,
\( \gamma \) : load factor for fatigue load combination
\( (\Delta f) \) : force effect, live load stress range due to the passage of the fatigue load
\( (\Delta F)_n \) : nominal fatigue resistance

### 2.5.3 Nominal Fatigue Resistance
The program's calculation of Nominal Fatigue Resistance will be different based on whether the load combinations are entered into Fatigue I Limit State or Fatigue II Limit State. Between the two values, the lower value will be applied and reviewed.

(1) The Nominal Fatigue Resistance of Fatigue I Limit State due to load combinations
The program will calculate the Nominal Fatigue Resistance based on the category selected in the Fatigue dialog window.

\[ (\Delta F)_n = (\Delta F)_{TH} \]  \hspace{1cm} (2.111)

Within the program, categories of Nominal Fatigue Resistance, such as A, B, B', C, C', D, E, and E' are applied as shown in [Table2.29].

(2) The Nominal Fatigue Resistance of Fatigue II Limit State due to load combinations
If fatigue review is performed with consideration to fatigue load combination 2, the following equation is used to calculate the resistance value of fatigue.

\[ (\Delta F)_n = \left( \frac{A}{N} \right)^{1/3} \]  \hspace{1cm} in which: \( N = (365)(75)n(ADTT)_{SL} \)  \hspace{1cm} (2.112)

Where,
\( A \) : Constant taken from Table 6.6.1.2.5-1
\( n \) : Number of stress range cycles per truck passage taken from Table 6.6.1.2.5-2
\( (ADTT)_{SL} \) : ADTT for single lane

The value of the Detail Category Constant \((A)\) and 75-yr \((ADTT)_{SL}\) Equivalent to Infinite Life \((n,\) truck per day) are each respectively applied in [Table2.30] and [Table2.31]. If, the \(n\) value is entered into the Fatigue Parameter, this value will be applied first.

### 2.5.4 Special Fatigue Requirement for Webs
The program will perform the review of the fatigue due to the shear buckling of the web.
\[ V_{cr} = CV_p \]  \hspace{1cm} (2.113)

Where,

\[ V_{cr} : \text{shear in the web at the section under consideration due to the unfactored permanent loads plus the factored fatigue load} \]

\[ V_{cr} = CV_p \quad \text{in which:} \quad V_p = 0.58F_{yw}Dt_w \]  \hspace{1cm} (2.114)

3. Shear Connector

When the shear connector is defined in the steel composite sections, the review of the shear connectors is performed. The shear connector performs review of Pitch, Transverse spacing, Cover and Penetration, Fatigue, Special Requirement for point, and strength limit state.

3.1 Section Proportion

For the ratio of height to diameter of the stud type shear connector, following equation is used.

\[ \frac{h}{d} \geq 4.0 \]  \hspace{1cm} (2.115)

3.2 Pitch

The pitch is reviewed using the below equation.

\[ p \leq \frac{nZ_r}{V_{sr}} \]  \hspace{1cm} (2.116)

Where,

\[ Z_r : \text{shear fatigue resistance of an individual shear connector determined as per Article 6.10.10.2} \]
\[ n : \text{number of shear connector in a cross section} \]
\[ V_{sr} : \text{horizontal fatigue shear range per unit length} \]

Also, the program checks if \( p \geq 6 \times \text{Stud Diameter} \) and \( p \leq 24 \text{ inches} \) are satisfied as well as Equation 2.116.

\[ V_{sr} = \sqrt{(V_{fat})^2 + (F_{far})^2} \]  \hspace{1cm} (2.117)

in which:

\[ V_{fat} : \text{longitudinal fatigue shear range per unit length} \]
\[ V_{fat} = \frac{V_JQ}{I} \]  \hspace{1cm} (2.118)

\[ F_{far} : \text{radial fatigue shear range per unit length taken as the largest of either} \]

\[ F_{far1} = \frac{A_{bot} \sigma_{flg} l}{wR} \quad \text{or} \quad F_{far2} = \frac{F_{rc}}{w} \]  \hspace{1cm} (2.119)

in which:

\[ \sigma_{flg} : \text{range of longitudinal fatigue stress in the bottom flange without consideration of flange lateral bending} \]
\[ A_{bot} : \text{area of the bottom flange} \]
\[ F_{rc} : \text{net range of cross-frame of diaphragm force at the top flange} \]
\[ l : \text{distance between brace point} \]
\[ R : \text{minimum girder radius within the panel} \]
\[ w : \text{effective length of deck (in.) taken as 48.0 in. except at end supports where w may be taken as 24.0 in. effective length of deck distance} \]

- If it is straight members, the value of \( F_{far1} \) is 0.
- If it is a Box/Tub section, regardless of whether it is straight or curved, the value of \( F_{far1} \) is 0.
• The program will consider the value of $F_{fat2}$ as 0.
• The center-to-center distance of the shear connectors cannot exceed 24 inches and 6 times the diameter of the stud.

### 3.3 Transverse spacing

1. The transverse spacing of the shear connector must be more than 4 times the diameter of the stud.
2. The shear connectors must be located 1 inch inwards from the edge.

### 3.4 Cover and penetration

The following conditions must be met for the cover and penetration of the shear connector.
1. The clear depth of concrete cover over the tops of the shear connector must not be at least 2.0 inches.
2. The shear connector must penetrate at least 2.0 inches into the concrete slab.

### 3.5 Fatigue Shear Resistance, $Z_r$

This part is applied differently in the AASHTO LRFD 07 and 12. For the 07 conditions, follow Section 5.2 of this chapter. The fatigue shear resistance of the shear connector is calculated as shown in the following table.

<table>
<thead>
<tr>
<th>Shear Connector Type</th>
<th>Case</th>
<th>Fatigue shear resistance ($Z_r$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stud</td>
<td></td>
<td>$Z_r = ad^2$</td>
</tr>
<tr>
<td></td>
<td>$75 - \text{year} (ADTT)_{SL} \leq 960$</td>
<td>Where, $\alpha$</td>
</tr>
<tr>
<td></td>
<td>$N = 0$</td>
<td>34.5</td>
</tr>
<tr>
<td></td>
<td>$N &gt; 0$</td>
<td>$34.5 - 4.28 \log N$</td>
</tr>
<tr>
<td></td>
<td>$75 - \text{year} (ADTT)_{SL} &gt; 960$</td>
<td>$Z_r = 5.5d^2$</td>
</tr>
</tbody>
</table>

### 3.6 Strength Limit State

1. **Strength Limit State**
   After the strength limit state is calculated, the minimum number of shear connector (n) is calculated as shown in the equation below.
   $$ n = \frac{P}{Q_r} $$  \hspace{1cm} (2.120)

   Where,
   $P$ : total nominal shear force

2. **Factored shear resistance of a single shear connector**
   The resistance of the shear connector is calculated as shown in the equation below.
\[ Q_r = \phi_{sc} Q_n \quad (2.121) \]

Where,
- \( Q_n \): nominal shear resistance of a single shear connector determined as in Article 6.10.10.4.3
- \( \phi_{sc} \): resistance factor for shear connectors inputted by the user in Composite Steel Design Parameter (Fig.2.17)

### (3) Total Nominal Shear Force, \( P \)

1) Calculate the Total Nominal Shear Force, \( P \), for the verification of the shear connectors under positive moment.

\[ P = \sqrt{P_p^2 + F_p^2} \quad (2.122) \]

Where,
- \( P_p \): total longitudinal force in the concrete deck

\[ P_p = \text{Max}(P_{1p}, P_{2p}) \quad (2.123) \]

in which:
\[ P_{1p} = 0.85 f_y b_n t_s \quad (2.124) \]
\[ P_{2p} = F_{yw} D_{tw} + F_{yH} b_n t_f t_f + F_{yc} b_c t_f \]

- \( F_p \): total radial force in the concrete deck

\[ I_{r1} > I_{r2} \quad (2.125) \]

in which:
- \( L_p \): arc length between an end of the girder and an adjacent point of maximum positive live load plus impact moment

For straight bridges, the value of \( F_p \) is calculated as 0.

2) Calculate \( P \) when the shear connector experiences a negative moment.

\[ P = \sqrt{P_t^2 + F_t^2} \quad (2.126) \]

Where,
- \( P_t \): total longitudinal force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support

\[ P_t = P_p + P_n \quad (2.127) \]

in which:
- \( P_n \): total longitudinal force in the concrete deck over an interior support taken as:

\[ P_n = \text{Min}(P_{1n}, P_{2n}) \quad (2.128) \]

in which:
\[ P_{1n} = F_{yw} D_{tn} + F_{yH} b_n t_f + F_{yc} b_c t_f \]
\[ P_{2n} = 0.45 f_c b_t \quad (2.129) \]

- \( F_t \): total radial force in the concrete deck between the point of maximum positive live load plus impact...
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moment and the centerline of an adjacent interior support taken as:

\[ F_T = P_T \frac{L_n}{R} \tag{2.130} \]

in which:

\( L_n \): arc length between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support inputted by the user in shear connector dialog box (Fig.2.19)

For straight bridges, the value of \( F_p \) is calculated as 0.

(4) Nominal shear resistance, \( Q_n \)

\[ Q_n = 0.5A_{sc}\sqrt{\frac{f_t}{E_c}} \leq A_{sc}F_u \]

Where,

\( A_{sc} \): cross-sectional area of a stud shear connector
\( E_c \): modulus of elasticity of the deck concrete
\( F_u \): specified minimum tensile strength of a stud shear connector

4. Stiffener

The Stiffener calculates the transverse/longitudinal stiffener attached to the web and the longitudinal stiffener attached to the compression flange.

[Fig.2.6] Flow Chart of Stiffener

4.1 Web Transverse Stiffener

(1) Projecting Width

Projecting width of transverse stiffener attached to web panel shall satisfy following two conditions:

\[ 16t_p \geq b \geq b / 4 \]

\[ 16t_p \geq b \]

[Table 2.46] Projecting Width Conditions of Web Transverse Stiffener

<table>
<thead>
<tr>
<th>Check List</th>
<th>I Section</th>
<th>Tub Section</th>
<th>Closed-Box Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Condition 1</td>
<td>( \geq b / 4 )</td>
<td>16t_p \geq b</td>
<td>16t_p \geq b</td>
</tr>
</tbody>
</table>
Where,

\( t_p \): thickness of the projecting stiffener element

\( b_f \): for I-sections, full width of the widest compression flange; for tub section, full width of the widest top flange. For closed box section, the limit of \( b_f/4 \) does not apply.

<table>
<thead>
<tr>
<th>Table 2.47</th>
<th>Define ( b_f ) according to Section Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section Type</td>
<td>( b_f )</td>
</tr>
<tr>
<td>I</td>
<td>Full width of the widest compression flange with in the field section under consideration</td>
</tr>
<tr>
<td>Tub</td>
<td>Full width of the widest top flange within the field section under consideration</td>
</tr>
</tbody>
</table>

(2) Moment of Inertia Check
This part is applied differently in the AASHTO LRFD 07 and 12. For the 07 conditions, follow the section 5.3 of this chapter. The program will perform the calculation of the vertical stiffeners attached to the web.

1) \( V_u > V_n \)

\[ I_t \geq \text{Min} \left( I_{t1}, I_{t2} \right) \]  
(2.131)

Where,  

\( I_t \): moment of inertia of transverse stiffener

[Table 2.48] Calculation of Moment of Inertia of the transverse stiffener for I girder section, \( I_t \)

<table>
<thead>
<tr>
<th>Case</th>
<th>( I_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single-sided vertical stiffeners</td>
<td>( I_t = t_p \frac{b_f^3}{3} )</td>
</tr>
<tr>
<td>Double-sided vertical stiffeners</td>
<td>( I_t = 2 \left( t_p \frac{b_f^3}{12} + b_f t_p \left( 0.5b_f + 0.5t_w \right)^2 \right) )</td>
</tr>
</tbody>
</table>

\[ I_{t1} = b t_w^3 J \]

\[ I_{t2} = \frac{D^2 \rho_t^{1.3} \left( F_{yw} \right)^{1.5}}{E} \]  
(2.132)

\[ J = \frac{2.5}{(d_e / D)^2} - 2.0 \geq 0.5 \]

Where,

\( J \): stiffener bending rigidity parameter

\( \rho_t = \text{Max}(F_{yw} / F_{crs}, 1.0) \)  
(2.133)

\( F_{crs} \): local buckling stress for the stiffener

\[ F_{crs} = 0.31E \left( \frac{h_s}{t_p} \right)^2 \leq F_{ys} \]  
(2.134)
\[ F_{ys} : \text{specified minimum yield strength of the stiffener} \]
\[ d_o : \text{the smaller of the adjacent web panel widths} \]
\[ b : \text{the smaller of } d_o \text{ and } D \]
\[ C : \text{ratio of the shear-buckling resistance} \]

2) \( V_u \leq V_n \)

Table 2.49: Check for Transverse Stiffener when \( V_u \leq V_n \)

<table>
<thead>
<tr>
<th>Case</th>
<th>Verifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>( I_1 &gt; I_2 )</td>
<td>( V_u &gt; V_{cr} )</td>
</tr>
<tr>
<td>( V_u &gt; V_{cr} )</td>
<td>( I_1 \geq I_{II} + (I_{I2} - I_{II}) \left( \frac{V_u - \phi V_{cr}}{\phi V_{cr} - \phi V_{cr}} \right) )</td>
</tr>
<tr>
<td>Otherwise</td>
<td>( I_1 &gt; I_{I2} )</td>
</tr>
<tr>
<td>Otherwise</td>
<td>( I_1 &gt; I_{I2} )</td>
</tr>
</tbody>
</table>

3) The following is calculated when the transverse and longitudinal stiffeners attach to the web at the same time.

\[ I_\ell > \left( \frac{b_2}{b_1} \right) \left( \frac{D}{3.0 d_o} \right) I_1 \]  \hspace{1cm} (2.135)

Where,
\[ b_2 : \text{projecting width of the transverse stiffener} \]
\[ b_1 : \text{projecting width of the longitudinal stiffener} \]
\[ I_\ell : \text{moment of inertia of the longitudinal stiffener} \]

4.2 Web Longitudinal Stiffener

(1) Strength limit state

The longitudinal stiffener attached to the web is calculated as shown in the following equation.

\[ f_\ell \leq \phi_f R_h F_{ys} \]  \hspace{1cm} (2.136)

Where,
\[ f_\ell : \text{the flexural stress in the longitudinal stiffener} \]
\[ R_h : \text{specifed minimum yield strength of the stiffener} \]

(2) Projecting width

The projecting width of the longitudinal stiffener is limited as per the following equation. As per Article C6.11.11.2, for the structural tees, \( b_1 \) should be taken as one half the width of the flange.

\[ b_1 \leq 0.48 t_s \sqrt{\frac{E}{F_{ys}}} \]  \hspace{1cm} (2.137)

Where,
\[ t_s : \text{thickness of the stiffener} \]

(3) Moment of inertia and radius of gyration

Moment of inertia and radius of gyration are calculated using the dimensions inputted in the Section Stiffener dialog box (Fig.2.8). The moment of inertia and the radius of gyration of the longitudinal stiffener shall satisfy:
\[
I_i \geq Dt_w^3 \left[ 2.4 \left( \frac{d_o}{D} \right)^2 - 0.13 \right] \beta \quad \text{and} \quad r \geq \frac{0.16d_o}{\sqrt{\frac{F_{ye}}{E}} \sqrt{1 - 0.6 \frac{F_{ye}}{R_A F_{ye}}}}
\]

(2.138)

Where,
- \(d_o\): transverse stiffener spacing
- \(R\): minimum girder radius in the panel
- \(r\): radius of gyration of the longitudinal stiffener including an effective width of the web equal to 18*tw taken about the neutral axis of the combined section
- \(I\): moment of inertia of the longitudinal stiffener including an effective width of the web equal to 18*tw taken about the neutral axis of the combined section
- \(\beta\): curvature correction factor for longitudinal stiffener rigidity

\[Z = \frac{0.95d_o^2}{Rt_w} \leq 10\]

(2.139)

4.3 Longitudinal Compression Flange Stiffener (for box compression flange)

(1) The strength of the stiffeners must be greater than the yield strength of the compression flanges.

(2) Projecting Width

The Projecting Width (b) of the Longitudinal Compression Flange Stiffener is calculated as shown in the following equation.

\[
b_l \leq 0.48t_c \sqrt{\frac{E}{F_{ye}}} \]

(2.140)

Where,
- \(t_c\): thickness of the projecting longitudinal stiffener element

(3) Moment of inertia

Each Moment of inertia of the Longitudinal Compression Flange Stiffener is calculated as shown in the following equation.

\[
I_i \geq \psi w t_{f_c}^3
\]

(2.141)

Where,
- \(w\): larger of the width of the flange between longitudinal flange stiffeners or the distance from a web to the nearest longitudinal flange stiffener

[Table 2.51] Calculation of \(\psi\)

<p>| Number of the longitudinal stiffener attached to | (\psi) |</p>
<table>
<thead>
<tr>
<th>compression flange(n)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(n = 1)</td>
<td>0.125k^3</td>
</tr>
</tbody>
</table>


5.1 Fatigue Limit State

In both standards, the fatigue resistance is calculated differently.

<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>The calculation only considers the Fatigue 2 Load Combination out of the user load combinations.</td>
<td>Based on the conditions, the calculation considers the Fatigue 1 or 2 Load Combination.</td>
</tr>
</tbody>
</table>

Fatigue Resistance ($\Delta F$), Calculation
Fatigue 1 Load Case Combination Is not used in the calculation.

When using the Fatigue 2 Load Case Combination, the value of $\Delta F$ is calculated as such:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^\frac{1}{3} \geq \frac{1}{2} (\Delta F)_{TH}$$

in which: $$N = (365)(75)n(ADTT)_{SL}$$

5.2 Fatigue Limit State for Shear Connector

In both standards, Fatigue resistance for Shear Connector ($Z_r$) is calculated differently.

<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>The Fatigue resistance($Z_r$) of the stud type for the Shear Connector is calculated as such: $$Z_r = \alpha d^2 \geq \frac{38.0d^2}{2} \quad \text{(in SI Unit)}$$ $$\alpha = 238 - 29.5 \log N \quad \text{(in SI Unit)}$$</td>
<td>The Fatigue resistance($Z_r$) of the stud type for the Shear Connector is calculated as such: $$Z_r = \alpha d^2 \quad \text{(in US Unit)}$$ $$\alpha = 34.5 - 4.28 \log N \quad \text{(in US Unit)}$$</td>
</tr>
</tbody>
</table>
5.3 Transverse Stiffener
In both standards, Transverse Stiffener is calculated differently

<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculation of the Stiffener bending rigidity parameter ($J$)</td>
<td>Calculation of the Stiffener bending rigidity parameter ($J$)</td>
</tr>
<tr>
<td>$J = 2.5 \left( \frac{D}{d_o / D} \right)^2 - 2.0 \geq 0.5$</td>
<td>$J = \frac{2.5}{(d_o / D)^2} - 2.0 \geq 0.5$</td>
</tr>
</tbody>
</table>

When the Web post buckling or tension-field resistance is considered, the following is calculated.

$I_t > I_{t2}$

When the Web post buckling or tension-field resistance is considered, the following is calculated.

1) $V_n > V_{cr}$

$I_t \geq I_{t1} + (I_{t2} - I_{t1}) \left( \frac{V_n - \phi V_{cr}}{\phi_n V_n - \phi V_{cr}} \right)$

2) Other conditions

$I_t > I_{t2}$

(2) $I_{t1} \leq I_{t2}$

$I_t > I_{t2}$
### 5.4 Flexure Resistance of Box Flange in compression under Unstiffened condition

In both standards, the Flexure Resistance of Box Flange in compression under Unstiffened condition is calculated differently.

<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>(1) F_{nc}</strong></td>
<td><strong>(1) F_{nc}</strong></td>
</tr>
<tr>
<td><strong>1) ( \lambda_f \leq R_1 \frac{kE}{F_{yc}} )</strong></td>
<td><strong>1) ( F_{nc} = \phi_f F_{cb} \sqrt{1 - \left( \frac{f_c}{\phi_c F_{cv}} \right)^2} )</strong></td>
</tr>
<tr>
<td>[ F_{nc} = R_y R_b F_{yc} \Delta ]</td>
<td><strong>2) F_{cb}</strong></td>
</tr>
<tr>
<td><strong>2) ( R_1 \frac{kE}{F_{yc}} &lt; \lambda_f \leq R_2 \frac{kE}{F_{yc}} )</strong></td>
<td><strong>2) F_{cb}</strong></td>
</tr>
<tr>
<td>[ F_{cb} = R_y R_b R_y F_{yc} \times \left[ \Delta - \frac{F_{yc}}{R_b F_{yc}} \left( 1 - \sin \left( \frac{\pi}{2} \frac{R_y - b_{fc}}{R_y - R_b} \right) \right) \right] ]</td>
<td><strong>3) ( \lambda_y &lt; \lambda_f )</strong></td>
</tr>
<tr>
<td><strong>3) ( \lambda_f &gt; R_2 \frac{kE}{F_{yc}} )</strong></td>
<td><strong>3) ( \lambda_y &lt; \lambda_f )</strong></td>
</tr>
<tr>
<td>[ F_{nc} = \frac{0.9 E R_b k}{b_{fc} t_{fc}} - \frac{R_y f_{yc}^2 k}{0.9 E k_{fc}^2 t_{fc}} ]</td>
<td>[ F_{cb} = 0.9 E R_y k_{fc} \frac{F_{yc}}{\lambda_f^2} ]</td>
</tr>
<tr>
<td>Where,</td>
<td>Where,</td>
</tr>
<tr>
<td>( R_1 ) : constant which when multiplied by ( \sqrt{kE/F_{yc}} ) yields the slenderness ratio equal to 0.6 times the slenderness ratio for which ( F_{nc} ) from Eq.3 is equal to ( R_y R_b F_{yc} \Delta )</td>
<td>( \lambda_p = 0.57 \sqrt{\frac{E k}{F_{yc} \Delta}} )</td>
</tr>
<tr>
<td>[ R_1 = \sqrt{\frac{0.57}{\Delta + \sqrt{\Delta^2 + 4 \left( \frac{f_{yc}}{F_{yc}} \right)^2 \left( \frac{k}{k_f} \right)^2}}} ]</td>
<td>( \lambda_y = 0.595 \sqrt{\frac{E k}{F_{yc}}} )</td>
</tr>
<tr>
<td>( R_2 ) : constant which when multiplied by ( \sqrt{kE/F_{yc}} ) yields the slenderness ratio for which ( F_{nc} ) from Eq.3 is equal to ( R_y R_b F_{yc} \Delta )</td>
<td></td>
</tr>
</tbody>
</table>

Steel Composite Design Result

1. Strength Limit State Result

1.1 Flexure

(1) by Result Table

As shown in the table below, the results can be checked in the result table.

▶ Design > Composite Design > Design Result Tables > Strength Limit State (flexure)...

<table>
<thead>
<tr>
<th>Condition</th>
<th>Output Items</th>
</tr>
</thead>
<tbody>
<tr>
<td>My</td>
<td>Mp</td>
</tr>
<tr>
<td>compact</td>
<td>O</td>
</tr>
<tr>
<td>non-compact</td>
<td>-</td>
</tr>
<tr>
<td>Appendix A6</td>
<td>O</td>
</tr>
</tbody>
</table>

Where,

- $M_y$: yield moment
- $M_p$: plastic moment
- $M_u$: moment due to the factored loads
- $\phi \mu_n$: nominal flexural resistance of a section multiplied by resistance factor, $\phi$, for flexure
- $f_{bu}$: largest value of the compressive stress throughout the unbraced length in the flange under condition, calculated without consideration of flange lateral bending
- $\phi \mu_f$: nominal flexure resistance of a flange
- $D_p$: distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment
- $D_t$: total depth of the composite section

Based on the different search conditions, the result values which appear will vary, as shown in the table below.

[Table 2.52] Result Case Table for Strength Limit State of Flexure
(2) by Excel Report
The results can be viewed in an Excel Report as shown below.

1) Positive Flexure

![Image of a table and calculations related to positive flexure.]

![Fig.2.67 Excel Report for Strength Limit State of Positive Moment]
2) Negative Flexure

![Fig.2.68] Excel Report for Strength Limit State of Negative Moment

1.2 Shear

(1) Result Table

As shown in the table below, the results can be checked in the result table.

![Fig.2.69] Result Table for Strength Limit State of Shear

Where,

\( V_u \): shear due to the factored load

\( \phi V_n \): nominal shear resistance multiplied by resistance factor, \( \phi \), for shear

\( b_{\text{lim}1} \): projecting width limit for transverse stiffener, \( 2.0 + (D/30) \), as per Eq. 6.10.11.1.2-1

\( b_{\text{lim}2} \): projecting width limit for transverse stiffener, \( 16t_p \), as per Eq. 6.10.11.1.2-2

\( b_{\text{lim}3} \): projecting width limit for transverse stiffener, \( b/4 \), as per Eq. 6.10.11.1.2-2

\( b \): projected width of transverse stiffener as per Article 6.10.11.1.2

\( I_{\text{lim}} \): limiting moment of inertia of transverse stiffener as per Eq. 6.10.11.1.3-3&4

\( I \): Moment of Inertia of transverse stiffener as per Article 6.10.11.1.3
(2) by Excel Report
The results can be viewed in an Excel Report as shown below.

![Excel Report for Strength Limit State of Shear](image)

2. Service Limit State Result

(1) by Result Table
The results can be viewed in an Excel Report as shown below.

![Result Table for Service Limit State](image)

Where,
- \( f_c \): compression-flange stress
- \( f_{crw} \): nominal bending buckling resistance for webs as per Eq. 6.10.11.1-1
- \( f_c \): compression-flange stress
- \( f_{cf,lim} \): limit of compression-flange stress
- \( f_{tf} \): tension-flange stress
- \( f_{tf,lim} \): limit of tension-flange stress
The results can be viewed in an Excel Report as shown below.

![Excel Report or Strength Limit State of Shear](Fig.2.7)

3. Constructibility Result

3.1 Flexure

(1) by Result Table

The results can be viewed in a result table as shown below.

![Result Table for Constructibility Limit State of flexure](Fig.2.73)

Where,

\(f_{sw}\) : flange stress calculated without consideration of flange lateral bending

\(\phi_{sw}\) : nominal bend-buckling resistance for webs

\(f_{swc}\) : compression-flange stress with consideration of flange lateral stress

\(\phi_{swc}\) : limit of compression-flange stress

\(f_{swt}\) : tension-flange stress with consideration of flange lateral stress

\(\phi_{swt}\) : limit of tension-flange stress

\(f_{d}\) : longitudinal tensile stress in a composite section deck

\(\phi_{fr}\) : limit of concrete deck tensile stress. \(fr\) shall be taken as the modulus of rupture as per the Article 6.10.1.7
The results can be viewed in an Excel Report as shown below.

![Excel Report for Constructibility of Positive Moment](image)
2) Negative Flexure

VII. Constructibility

1. Flexure

- Positive moment

1) Design Forces and Stresses

<table>
<thead>
<tr>
<th>Component</th>
<th>M (kips-in)</th>
<th>f (ksi)</th>
<th>T (kips-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Force</td>
<td>0.000</td>
<td></td>
<td>50.973</td>
</tr>
<tr>
<td>Stress</td>
<td>Top 0.000</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bot 0.000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2) Check slenderness of web (AASHTO LRFD Bridge, 2012, 6.10.6.2.3-1):

\[ \frac{D_t}{t_w} = \frac{157.213}{61.895} = 2.57 \text{ ksi} \geq 161.779 \text{ ksi} \quad \text{Compact or noncompact Web} \]

Where:

\[ D_t = \frac{D_t}{t_w} \]

3) Discretely Braced Flanges in Compression (AASHTO LRFD Bridge, 2012, 6.10.3.2.1)

2) Check flange nominal yielding (6.11.3.2-3):

\[ f_{nu} = 0.000 \leq \frac{f}{f_y} \leq \frac{F_{cr}}{\phi} \leq 35.998 \text{ kips} \quad \text{... OK} \]

Where:

\[ f_{nu} = \frac{f}{f_y} \]

\[ f = 1.000 \]

\[ f_y = 0.107 \text{ ksi} \]

\[ \phi = 1.000 \text{ ksi} \]

3.2 Shear

(1) by Result Table

The results can be viewed in a result table as shown below.

▶ Design > Composite Design > Design Result Tables > Constructibility (shear)...

Where,

\[ V_u : \text{shear in the web due to the factored load} \]

\[ \phi V_{cr} : \text{shear-buckling resistance multiplied by resistance factor, } \phi, \text{ for shear} \]
The results can be viewed in an Excel Report as shown below.

![Excel Report for Constructibility of Shear](image)

4. Fatigue Limit State Result

The results can be viewed in a result table as shown below.

![Result Table for Fatigue Limit State](image)

Where,

\( \gamma(\Delta f) \) : Range of Fatigue Limit State

\( (\Delta F)^n \) : Nominal Fatigue Resistance

\( L_{com} \) : Load combinations used in the calculation

\( V_u \) : shear in the web due to the unfactored permanent load plus the factored fatigue load

\( V_{cr} \) : shear buckling resistance as per Eq. 6.10.9.3-1
(2) by Excel Report
The results can be viewed in an Excel Report as shown below.

![Excel Report for Fatigue Limit State]

5. Shear Connector Result

(1) by Result Table
The results can be viewed in a result table as shown below.

![Result Table for Shear Connector]

Where,

- \( H/D \): height to diameter ratio
- \( (H/D)_{\text{lim}} \): limit value of height to diameter ratio (=4.0)
- \( p \): pitch of shear connectors specified by the user
- \( p_{\text{lim1}} \): pitch limit value, \( nZI/(Vsr) \), as per Eq. 6.10.10.1.2-1
- \( p_{\text{lim2}} \): pitch limit value, \( 6d \)
- \( s \): transverse spacing of shear connectors spacing (Transverse Cross Section)
- \( \text{edge} \): distance of the top compression flange edge \( \text{edge}_{\text{lim}} \) (\( =1.0 \) in)
- \( \text{Cover} \): clear depth of concrete cover over the tops of the shear connectors (\( > 2.0 \) in)
- \( \text{Penetration} \): depth of penetration of the shear connector (\( >2.0 \) in)
- \( n \): number of shear connectors entered in transverse direction
- \( n_{\text{Req}} \): required number of shear connectors
The results can be viewed in an Excel Report as shown below.

![Excel Report for Shear Connector](image)

### 6. Stiffener Result

(1) by Result Table

The results can be viewed in a result table as shown below.

![Result Table for Stiffener](image)

Where,

- \( b_l \) : projecting width
- \( b_l \_lim \) : limit of projecting width as per Eq. 6.10.11.3.2-1
- \( I \) : Moment of inertia of cross-section
- \( I \_lim \) : limit of moment of inertia of cross-section as per Eq. 6.10.11.3.3-1
- \( r \) : radius of gyration
- \( r \_lim \) : limit of radius of gyration as per Eq. 6.10.11.3.3-2
- \( f_s \) : flexure stress of longitudinal stiffener
- \( \phi \_Rh \_Fys \) : limit of flexure stress as per Eq. 6.10.11.3.1-1
(2) by Excel Report
The results can be viewed in an Excel Report as shown below.

![Excel Report for Stiffener](image)

7. Span Checking

(1) by Result Table
▶ Design > Composite Design > Design Result Table...

Most critical member results in each span can be viewed in a result table as shown below.

![Result Table for Span Group](image)
(2) by Span Result Graph

Design > Composite Design > Design Result Diagram...

The results of the span group defined by the span information can be checked here. The flexure and shear results based on distance or node can be checked here. The current applied member force or elasticity is marked in red while the strength or elasticity is marked in green.

8. Total Checking

(1) by Result Table

Design > Composite Design > Design Result Table...

Summary results for each member can be viewed in a result table as shown below.

![Result Table for Total Checking](image)