** 2003 New Guide Specifications

Section 2  Limit States
Section 3  Loads
Section 4  Structural Analysis
Section 5  Flanges with One Web
Section 6  Webs
Section 7  Shear Connectors
Section 8  Bearings
Section 9  I –Girders
Section 11  Splices and Connections
Section 12  Deflections
Section 13  Constructibility
** 2003 New Guide Specifications Section 2

2.2  STRENGTH LIMIT STATE  
(same as 2002; LFD only)

2.3  FATIGUE LIMIT STATE  
(use modified AASHTO LRFD Art. 6.6.1 with load factor of 0.75 instead; Uncracked section is used as per Art. 9.6.1)

2.4  SERVICEABILITY LIMIT STATE  
(same as 2002; Deflection and Concrete Crack Control)

2.5  CONSTRUCTIBILITY LIMIT STATE  
(This limit state considers deflection, strength of steel and concrete and stability during critical stage of construction)
** 2003 New Guide Specifications Section 9

** Deflection

- Max. Preferable Span-to-Depth Ratio

\[
\frac{L_{as}}{d} = 25 \sqrt{\frac{50}{F_y}}
\]

- Live Load Deflections

\[D_{LL+I} < \frac{L}{800}\]

\[D_{LL+I+sidewalk} < \frac{L}{1000}\]

** Effective for the full width and full length of the concrete deck
2.3 LOADS

- **Uplift** must be investigated, without reduction for multiple lane loading and during concrete placement

  AASHTO 3.17 Check $D + 2 \times (L + I)$
  and $1.5 \times (D + L + I)$

  AASHTO 10.29.6 Anchor bolts

- **Impact** same as current AASHTO

  $(I = 50 / (L + 125) \leq 0.3 \quad \text{AASHTO Eq. 3.1})**$

** Revised in 2003.
Impact

Art. 3.5.6.1 I-Girders.

<table>
<thead>
<tr>
<th>Load Effect</th>
<th>Impact Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reactions, shear, cross frame and diaphragm actions</td>
<td>0.25</td>
</tr>
<tr>
<td>Girder bending moment, torsion and deflections</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Art. 3.5.6.2 Box Girders.

<table>
<thead>
<tr>
<th>Load Effect</th>
<th>Impact Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reactions, shear, cross frame and diaphragm actions</td>
<td>0.35</td>
</tr>
<tr>
<td>Girder bending moment, torsion and deflections</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Art. 3.5.6.3 Fatigue - 15%
2.3 LOADS (cont.)

- **Centrifugal Force**
  same as current AASHTO
  
  \[
  C = 0.00117 \times S^2 \quad D = 6.68 \times \frac{S^2}{R} \quad \text{AASHTO Eq. 3.2}
  \]
  
  \[
  S = \text{the design speed in miles/hr}
  \]

- **Thermal Forces**
  do not have to be considered if temperature movements are allowed to occur (more in Article 1.6 & 1.7)

- **Superelevation**
  redistribution of wheel load
2.4 LOAD COMBINATIONS AND LOAD FACTORS

(A) Construction Loads (Construction Staging)
1.3 \((D_p + C)\)

(B) Service Loads (Stress range values and live load deflections)
\[ D + L_r \]
where
\[ L_r = L + I + CF \]
\[ L = \text{Basic live load} \]
\[ I = \text{Impact loads} \]
\[ CF = \text{Centrifugal force} \]

(C) Overloads
\[ D + \frac{5}{3} L_r \]
2.4 LOAD COMBINATIONS AND LOAD FACTORS (cont.)

For all loadings less than H20
Group IA: $1.3 \left[D + 2.2 L_r\right]$ (Single lane)
Group II: $1.3 \left[D + W + F + SF + B + S + T\right]$  
  (EQ may replace W and ICE may replace SF)
Group III: $1.3 \left[D + L_r + 0.3W + WL + F + LF\right]$  

(D) Maximum Design Loads
Group I: $1.3 \left[D + 5/3 L_r\right]$  

For uplifting reaction:
$1.3 \left[0.9D + 5/3 L_r\right]$
2.5 DESIGN THEORY

General

- Analysis should be of entire structure
- However, primary bending moment curvature effects may be neglected if the following conditions exist:

<table>
<thead>
<tr>
<th>No. of Girders</th>
<th>Limiting Central Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 Span</td>
</tr>
<tr>
<td>2</td>
<td>2°</td>
</tr>
<tr>
<td>3 or 4</td>
<td>3°</td>
</tr>
<tr>
<td>5 or more</td>
<td>4°</td>
</tr>
</tbody>
</table>
2.5 DESIGN THEORY

- **Torsion**: Diaphragms must be provided and designed as primary members

- **Nonuniform Torsion**: otherwise known as “warping” or “lateral flange bending,” this effect always must be taken into account

- **Composite Design**: may be used, but shear connectors must be designed using procedures that will be discussed at a later point
The effects of curvature may be ignored if

- Girders are concentric,
- Bearing lines are not skewed more than 10 degrees from radial, and
- The arc span divided by the girder radius is less than 0.06 radians where the arc span, $L_{as}$, shall be taken as follows:

For simple spans:

$$L_{as} = \text{arc length of the girder},$$

For end of spans of continuous members:

$$L_{as} = 0.9 \times \text{arc length of the girder},$$

For interior spans of continuous members:

$$L_{as} = 0.8 \times \text{arc length of the girder}.$$
Simplified formula for the lateral bending moment in I-girder flanges due to curvature.

\[ M_{\text{lat}} = \frac{6M \ell^2}{5RD} \]  

Eq. (4-1)

Where:

\( M_{\text{lat}} \) = lateral flange bending moment (k-ft)
\( M \) = vertical bending moment (k-ft)
\( \ell \) = unbraced length (ft)
\( R \) = Girder radius (ft)
\( D \) = web depth (in)
Methods (Art. 4.3)

- **Approximate Methods:**
  - V-Load method for I-girder bridges
  - M/R Method for Box-girder bridges

- **Refined Methods:**
  - Finite Strip method.
  - Finite Element method.
  (including gird model considering torsional warping)
DESCUS Alternate Solution for Torsion and Warping Differential Equations

Equation & Solution for Concentrated Torsion M

\[ \frac{M}{EC_\omega} = \frac{1}{a^2} \phi' - \phi'' \]

\[ \phi = A + B \cosh \frac{Z}{a} + C \sinh \frac{Z}{a} + \frac{MZ}{GJ} \]

Equation & Solution for Uniform Torsion m

\[ \frac{1}{a^2} \phi'' - \phi''' = \frac{-m}{EC_\omega} \]

\[ \phi = A + BZ + C \cosh \frac{Z}{a} + D \sinh \frac{Z}{a} - \frac{mZ^2}{2GJ} \]

Equation & Solution for Linear Varying Torsion m

\[ \frac{1}{a^2} \phi'' - \phi''' = \frac{-mZ}{LEC_\omega} \]

\[ \phi = A + BZ + C \cosh \frac{Z}{a} + D \sinh \frac{Z}{a} - \frac{mZ^3}{6GJL} \]
DESCUS Alternate Solution for Torsion and Warping Differential Equations

Fixed-Fixed End Solution for Uniform and Linear Varying Torsion $m$

CASE 7

$$\phi = \frac{mLa}{2GJ} \left[ \left( 1 + \cosh \frac{L}{a} \right) \left( \cosh \frac{Z}{a} - 1.0 \right) + \frac{Z}{a} \left( 1 - \frac{Z}{L} \right) \left( \sinh \frac{Z}{a} \right) \right]$$

CASE 8

$$\phi = \frac{mL^2}{GJ} \left\{ \left[ \frac{a}{2L \sinh \frac{L}{a}} - S \cdot \tanh \frac{L}{2a} \right] \left( \cosh \frac{Z}{a} - 1.0 \right) + S \cdot \left( \sinh \frac{Z}{a} - \frac{Z}{a} \right) - \frac{Z^3}{6L^3} \right\}$$

Where:

$$S = \left[ \frac{\left( \cosh \frac{L}{a} - 1.0 \right) \cdot \frac{a}{2L} - \frac{\sinh \frac{L}{a}}{6.0}} {\left( \frac{L}{a} \sinh \frac{L}{a} + 2.0 - 2 \cosh \frac{L}{a} \right)} \right]$$

Reference: “Torsion Analysis” published by Bethlehem Steel
DESCUS Alternate Solution for Torsion and Warping Differential Equations

Fixed-Hinged End Solution for Uniform and Linear Varying Torsion

\[ \phi = \frac{ma^2}{GJ} \left\{ \left[ -\frac{5}{6} \frac{L^3}{a^2} - \left( \frac{a}{L} - \frac{L}{2a} \right) \cdot \tanh \frac{L}{a} + 1.0 \right] + \left[ -\frac{Z}{L} + \frac{ZL}{a^2} \right] + \left[ \frac{a}{L} - \frac{L}{2a} \right] \right\} \]

\[ \phi = \frac{ma^2}{GJ} \left[ H \cdot \left( \tanh \frac{L}{a} - \frac{Z}{a} - \tanh \frac{L}{a} \cdot \cosh \frac{Z}{a} + \sinh \frac{Z}{a} \right) + \frac{\cosh \frac{Z}{a}}{\cosh \frac{L}{a}} - \frac{1}{\cosh \frac{L}{a}} - \frac{Z^2}{2a^2} \right] \]

Where:

\[ H = \left\{ \frac{L^2}{2a^2} - 1.0 + \frac{1}{\cosh \frac{L}{a}} \right\} \cdot \left( \tanh \frac{L}{a} - \frac{L}{a} \right) \]
DESCUS Alternate Solution
for Torsion and Warping
Differential Equations

Stresses induced by Torsion & Warping

Formula 1
\[ \tau_t = Gt \phi' \]

Formula 2
\[ \tau_{\omega_s} = -\frac{ES_{\omega_s} \phi''}{t} \]

Formula 3
\[ \sigma_{\omega_s} = EW_{ns} \phi'' \]
DESCUS Alternate Solution for Torsion and Warping Differential Equations

Stresses induced by Bending

Formula 4

$$\sigma_b = \frac{M_b y}{I}$$

Formula 5

$$\tau_b = \frac{VQ}{It}$$
2.5 DESIGN THEORY (cont.)

- Overload **

\[
(f_b + f_w)_{\text{overload}} < 0.8 \, F_y \text{ noncomposite}
\]
\[
< 0.95 \, F_y \text{ composite}
\]

** Modified in 2003 New Specifications Art. 9.5 Permanent Deflections as

\[
(f_b)_{\text{overload}} < 0.95 \, F_y \text{ Continuously braced flanges & partially braced tension flanges}
\]

\[
(f_b)_{\text{overload}} < F_{cr1} = F_{bs} \, \rho_w \, \rho_s < 0.95 \, F_y
\]

Partially braced compression flange

\[
F_{cr} = \frac{0.9 \, E_k}{\left( \frac{D}{t_w} \right)^2} \leq F_y
\]
2.6 FATIGUE

- Same as Straight Bridge except for the contribution of torsional stress

- **AASHTO Standard Spec.**
  - Allowable Fatigue Stress Range
    (Redundant or nonredundant; stress cycles & stress category)
  - Stress cycles
  - Stress category

- **AASHTO LRFD (** 2003 New Guide Specs.)**
  - Infinite fatigue life (ADTT single-lane ≥ 2000 Trucks/day)
    \[ F_n = (\Delta F)_{\text{threshold}} \]
  - Finite fatigue life (ADTT single-lane < 2000)
    \[ F_n = \Delta F = (A/N)^{1/3} \]
**2003 New Guide Specifications Section 4**

**Fatigue Load**

Design truck only with constant 30' between 32-kip axles.

(LRFD Art. 3.6.1.4 with a load factor of 0.75)

\[
\text{ADTT}_{\text{single-lane}} = p \times \text{ADTT}
\]

where \( p \)

- = 1.0 for one-lane bridge
- = 0.85 for two-lane bridge
- = 0.80 for three-lane or more bridge

<table>
<thead>
<tr>
<th>Class of Highway</th>
<th>ADTT/ADT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural Interstate</td>
<td>0.2</td>
</tr>
<tr>
<td>Urban Interstate</td>
<td>0.15</td>
</tr>
<tr>
<td>Other Rural</td>
<td>0.15</td>
</tr>
<tr>
<td>Other Urban</td>
<td>0.10</td>
</tr>
</tbody>
</table>
Fatigue Limit State

\( \gamma (\Delta f) \leq (\Delta F)_n \)  \hspace{1cm} \text{(LRFD Eq. 6.6.1.2.2-1)}

Infinite Fatigue Life \((\text{ADTT}_{\text{single-lane}} \geq 2000 \text{ trucks/day})\)
High traffic volume

\[ F_n = (\Delta F)_{\text{threshold}} = \begin{array}{c|c|c|c|c|c} 
\text{ksi for Category} & A & B & B' & C & C' & D & E & E' \\
16 & 12 & 10 & 12 & 7 & 4.5 & 2.6 \\
\end{array} \]
Fatigue Limit State (cont.)

Finite Fatigue Life \( (ADTT_{\text{single-lane}} < 2000 \text{ trucks/day}) \)

\[
N = (365)(75)(n)(ADTT)_{\text{single-lane}} \\
Fn = \Delta F = \left( \frac{A}{N} \right)^{\frac{1}{3}} \geq \frac{1}{2} (\Delta F)_{TH}
\]

<table>
<thead>
<tr>
<th>Category</th>
<th>A \times 10^9</th>
<th>(\Delta F)_{TH} (ksi)</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2.5 \times 10^{10}</td>
<td>24</td>
<td>( \ell &gt; 40' )</td>
</tr>
<tr>
<td>B</td>
<td>1.2 \times 10^{10}</td>
<td>16 Simple-span</td>
<td>1.0</td>
</tr>
<tr>
<td>B'</td>
<td>6.1 \times 10^9</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>4.4 \times 10^9</td>
<td>10 Continuous</td>
<td></td>
</tr>
<tr>
<td>C'</td>
<td>4.4 \times 10^9</td>
<td>12 (1) Interior</td>
<td>1.5</td>
</tr>
<tr>
<td>D</td>
<td>2.2 \times 10^9</td>
<td>7 Support</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>1.1 \times 10^9</td>
<td>4.5 (2) Elsewhere</td>
<td>1.0</td>
</tr>
<tr>
<td>E'</td>
<td>3.9 \times 10^8</td>
<td>2.6</td>
<td></td>
</tr>
</tbody>
</table>
2.7 EXPANSION AND CONTRACTION

Must allow thermal movements to take place in directions radiating from fixed supports

2.8 BEARINGS

- Must permit horizontal movements in directions radiating from fixed supports
- Must allow angular rotation in a tangential vertical plane
- Must hold down in cases of uplift
2.9 DIAPHRAGMS, CROSS FRAMES, AND LATERAL BRACING

Same as current AASHTO except as follows:

- Diaphragms/Crossframes shall be provided at each support and at intervals between Diaphragms/Crossframes shall extend in a single plane across full width of bridge.

- Diaphragms/Crossframes need not be located along skew at interior supports.

- Diaphragms/Crossframes should be full depth design as main structural element.

- Diaphragms/Crossframes/Lateral Bracing should be framed to transfer forces to flanges and webs, as necessary.
## 2.9 DIAPHRAGMS, CROSS FRAMES, AND LATERAL BRACING (cont.)

**Suggested Diaphragm/Crossframe spacing**

<table>
<thead>
<tr>
<th>Centerline Radius of Bridge (feet)</th>
<th>Suggested Max. Spacing (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Below 200</td>
<td>15</td>
</tr>
<tr>
<td>200 to 500</td>
<td>17</td>
</tr>
<tr>
<td>500 to 1000</td>
<td>20</td>
</tr>
<tr>
<td>Over 1000</td>
<td>25</td>
</tr>
</tbody>
</table>

### 2.9 (B) Lateral Bracing

Max. stress of the bottom lateral wind bracing

\[
f_b = f_d \times (DF)_{b\ell}
\]

\[
f_d = \text{Max. Stress in Crossframes from grid analysis}
\]

\[
(DF)_{b\ell} = 2.5 \frac{-SD + (0.16L - 10)}{(SG)^{4/3}} \times \left( \frac{L}{R} \right) + \left( \frac{L}{160} + 0.652 \right)
\]
Intermediate cross frames should be spaced as nearly uniform as practical to ensure that the flange strength equations are appropriate. Equation (C9-1) may be used as a guide for preliminary framing.

\[ \ell = \sqrt[36]{\frac{5}{36}} r_\sigma R b_f \]  
Eq. (C9-1)

Where:

- \( \ell \) = cross frame spacing (ft)
- \( r_\sigma \) = desired bending stress ratio, \( \frac{f_\ell}{f_b} \)
- \( R \) = girder radius (ft)
- \( b_f \) = flange width (in)
\[ f_\ell \leq 0.5F_y \]  
Eq. (5-1)

where:
\[ F_y = \text{specified minimum yield stress (ksi)} \]

When \( f_b \) is greater than or equal to the smaller of \( 0.33F_y \) or 17 ksi, then:
\[ \left| \frac{f_\ell}{f_b} \right| \leq 0.5 \]  
Eq. (5-2)

The unbraced arc length, \( l \), in feet between brace points at cross frames or diaphragms shall satisfy the following:
\[ l \leq 25b \quad \text{and} \quad \leq R/10 \]  
Eq. (5-3)

where:
\[ b = \text{minimum flange width in the panel (in)} \]
\[ R = \text{minimum girder radius within the panel (ft)} \]
2.12 NONCOMPOSITE GIRDER DESIGN

(B) Maximum Normal Flange Stress

- Compact Sections

  - Compression Flange

Requirement:

\[
\frac{b}{t} \leq \frac{3200}{\sqrt{F_y}}
\]

Allowables:

\[
F_{cr1} = F_{bs} \rho_B \rho_w
\]

where

\[
F_{bs} = F_y (1 - 3 \lambda^2)
\]

where

\[
\lambda = \frac{1}{\pi} \left( \frac{12 \ell}{b} \right) \sqrt{\frac{F_y}{E}}
\]

** Revised in 2003

** 2003 New Guide Specs use the same equation, except \( l/b \) in all the equations to be consistent with the units used
2.12 NONCOMPOSITE GIRDER DESIGN (cont.)

\[
\bar{\rho}_b = \frac{1}{1 + \frac{12\ell}{b_f} \left(1 + \frac{2\ell}{6b_f}\right) \left(\frac{\ell}{R} - 0.01\right)^2}
\]

** Revised in 2003

\[
\bar{\rho}_w = 0.95 + 18 \left[0.1 - \frac{\ell}{R}\right]^2 + \frac{f_w}{f_b} \left[0.3 - 1.2 \frac{\ell}{R b_f}\right] \frac{F_{bs}}{\rho_b F_y}
\]

** Revised in 2003

\[
\bar{\rho}_B \bar{\rho}_w \leq 1.0
\]

Tension Flange

\[
f_b < F_{bu} = F_y
\]

** Superseded
Compact Compression (Art. 5.2.1)

\[ F_{cr} = \text{smaller} \begin{cases} F_{cr1} = F_{bs} \overline{\rho}_b \overline{\rho}_w \\ F_{cr2} = F_y - \frac{|f_\ell|}{3} \end{cases} \]  

(Eq. 5-4)  

(Eq. 5-5)

- \( F_{cr} \) for Partially Braced Tension Flange is the same as \( F_{cr} \) for compact compression.
- \( F_{cr} \) for Continuously Braced Tension Flange is \( F_y \).
(B) Maximum Normal Flange Stress (cont.)

Non-Compact Sections

— Compression Flange

Requirement:

\[
\frac{3200}{\sqrt{F_y}} < \frac{b}{t} \leq \frac{4400}{\sqrt{F_y}}
\]

** Superseded

Allowables:

\[
f_b \leq F_{by} = F_{bs} \rho_w \rho_s
\]

\[
(f_b + f_w) \leq F_y
\]

where

\[
F_{bs} = F_y (1 - 3 \lambda^2)
\]

(Same definition for \(\rho_b\) and \(\rho_w\))

— Tension Flange

\[
(f_b + f_w) \leq F_{by} = F_y
\]

** Superseded
2003 New Guide Specifications Section 5

Non-Compact Compression (Art. 5.2.2)

\[
\frac{b_f}{t_f} \leq 1.02 \sqrt{\frac{E}{(f_b + f_\ell)}} \leq 23
\]  
Eq. (5-7)

\[
F_{cr} = \text{smaller}\left\{ \begin{array}{l}
F_{cr1} = F_{bs} \rho_b \rho_w \\
F_{cr2} = F_y - |f_\ell|
\end{array} \right.
\]  
Eq. (5-8)

Eq. (5-9)
** 2003 New Guide Specifications Section 5

\[
\rho_b = \frac{1}{1 + \frac{\ell}{R} \frac{12\ell}{75b_f}}
\]

\[
\rho_{w1} = \frac{1}{1 - \frac{f_{\ell}}{f_b} \left(1 - \frac{12\ell}{75b_f}\right)} + \frac{12\ell}{b_f} \cdot 0.95 + \frac{30 + 8,000 \left(0.1 - \frac{\ell}{R}\right)^2}{1 + 0.6 \left(\frac{f_{\ell}}{f_b}\right)}
\]

When \(\frac{f_{\ell}}{f_b} \geq 0\) \(\rho_w = \min(\rho_{w1}, \rho_{w2})\); \(\frac{f_{\ell}}{f_b} < 0\), \(\rho_w = \rho_{w1}\)
** 2003 New Guide Specifications Section 5

- Partially Braced Tension Flanges (Art. 5.3)

\[ F_{cr} = \text{smaller} \begin{cases} F_{cr1} = F_{bs} \bar{\rho}_b \bar{\rho}_w \\ F_{cr2} = F_y - \frac{|f_\ell|}{3} \end{cases} \quad \text{Eq. (5-10)}

- Tension Flanges and Continuously Braced Flanges (Art. 5.4)

\[ F_{cr} = F_y \quad \text{Eq. (5-11)} \]
Unstiffened Web (Art. 6.2)

for \( R \leq 700 \) feet

\[ \frac{D}{t_w} \leq 100 \]  
Eq. (6-1)

for \( R > 700 \) feet

\[ \frac{D}{t_w} \leq 100 + 0.038(R - 700) \leq 150 \]  
Eq. (6-2)

Where:

- \( D \) = distance along the web between flanges (in)
- \( t_w \) = web thickness (in)
- \( R \) = minimum girder radius in the panel (ft)
Web Bending Stresses (Art. 6.2.1, 6.3.1, & 6.4.1)

\[ F_{cr} = \frac{0.9Ek}{D^2} \leq F_y \]

where:

- \( k \) = bend-duckling coefficient
- \( k = 7.2 \frac{D}{D_c}^2 \) for unstiffened webs
- \( k = 9.0 \frac{D}{D_c}^2 \) for transversely stiffened webs
- \( k = 5.17 \left( \frac{D}{d_s} \right)^2 \) for \( \frac{d_s}{D_c} \geq 0.4 \) for longitudinally stiffened webs
- \( k = 11.64 \left( \frac{D}{D_c - d_s} \right)^2 \) for \( \frac{d_s}{D_c} < 0.4 \) for longitudinally stiffened webs

- \( D_c \) = depth of web in compression (in)
- \( d_s \) = distance between long. stiffener and comp. flange (in)

** 2003 New Guide Specifications Section 6
2.12 NONCOMPOSITE GIRDER DESIGN

(cont.)

(C) Web Design

- Webs Without Stiffeners

\[ V_u = C \, V_p \]

where

\[ V_p = 0.58 \, F_y \, D \, t \]

for

\[ \frac{D}{t_w} < \frac{6,000\sqrt{K}}{\sqrt{F_y}} \]

\[ C = 1.0 \]

for

\[ \frac{6,000\sqrt{K}}{\sqrt{F_y}} \leq \frac{D}{t_w} \leq \frac{7,500\sqrt{K}}{\sqrt{F_y}} \]

\[ C = \frac{6,000\sqrt{K}}{\left(\frac{D}{t_w}\right) \sqrt{F_y}} \]
2.12 NONCOMPOSITE GIRDER DESIGN (cont.)

\[
\frac{D}{t_w} > \frac{7500\sqrt{K}}{\sqrt{F_y}}
\]

for \( K = 5 \)

\[
C = \frac{4.5 \times 10^7 K}{\left(\frac{D}{t_w}\right)^2 F_y}
\]

where buckling coefficient \( K = 5 \)

** 2003 New Guide Specs use the same equation, except \( E \) is included in all the equations to be consistent with LRFD format.
Transversely Stiffened Webs

D/t_w < 150

For R ≤ 700 feet

\[ d_o = D \]  

Eq. (6-7a)

For R > 700 feet

\[ d_o = [1.0 + 0.00154(R - 700)]D \leq 3D \]

Eq. (6-7b)

where:

R = minimum girder radius in the panel (ft)
2.12 NONCOMPOSITE GIRDER DESIGN
(cont.)

- Transversely Stiffened Girders
  
  \[ V_u = C \ V_p \]

  (Calculations for \( C \) and \( V_p \) are the same as for webs without stiffeners except

  \[ K = 5 + \frac{5}{\left( \frac{d_o}{D} \right)^2} \]

- Requirements for Transverse Stiffeners
  
  \[ I \leq d_o \ t^3 \ J \]

  where

  \[ J = \left[ 2.5 \left( \frac{D}{d_o} \right)^2 - 2 \right] X \]
Transverse Web Stiffeners

Width-to-Thickness Ratio Requirement

\[
\frac{b_s}{t_s} \leq 0.48 \sqrt{\frac{E}{F_y}}
\]

Eq. (6-13)

Where:

\( t_s \) = stiffener thickness (in)
\( b_s \) = stiffener width (in)
\( F_y \) = specified minimum yield stress of stiffener (ksi)
**2003 New Guide Specifications Section 6**

**Moment of Inertia Requirement**

\[ I_{ts} = d_o t_w^3 J \]  
Eq. (6-14)

where:

\[ J = \left[ \left( \frac{1.58}{d/D} \right)^2 - 2 \right] X \geq 0.5 \]  
Eq. (6-15)

\[ X = \begin{cases} 1.0 & \text{for } a \leq 0.78 \\ 1 + \left( \frac{a - 0.78}{1.775} \right) Z^4 & \text{otherwise} \end{cases} \]  
Eq. (6-16)

\[ a = \text{aspect ratio} \]

\[ d_o = \frac{d}{D} \]

\[ Z = \frac{0.079 d_o^2}{R t_w} \leq 10 \]  
Eq. (6-18)

\[ d_o = \text{actual distance between transverse stiffeners (in)} \]

\[ R = \text{minimum girder radius in the panel (ft)} \]
2.12 NONCOMPOSITE GIRDER DESIGN

(cont.)

Longitudinally Stiffened Girders

Not required if

\[
\frac{D}{t} \leq \frac{36,500}{\sqrt{F_y}} \left[ 1 - 8.6\left(\frac{d_o}{R}\right) + 34\left(\frac{d_o}{R}\right)^2 \right]
\]

One stiffener required (D/5 from the compression flange) if

\[
\frac{D}{t} \leq \frac{73,000}{\sqrt{F_y}} \left[ 1 - 2.9\sqrt{d_o / R} + 2.2\left(\frac{d_o}{R}\right) \right]
\]

Two stiffeners required (D/5 from both flanges) if

\[
\frac{D}{t} \leq \frac{73,500}{\sqrt{F_y}}
\]

Requirements for Longitudinal Stiffeners
Longitudinal Web Stiffeners

Moment of inertia Requirement

\[ I_{ls} \geq D t_w^3 \left( 2.4a^2 - 0.13 \right) \beta \]  

Eq. (6-19)

where:

\[ \beta = \frac{Z}{6} + 1 \]  
when the longitudinal stiffener is on the side of the web away from the center of curvature

\[ \beta = \frac{Z}{12} + 1 \]  
when the longitudinal stiffener is on the side of the web toward the center of curvature
When single transverse stiffeners are used, they preferably shall be attached to both flanges. When pairs of transverse stiffeners are used, they shall be fitted tightly to both flanges.

Transverse stiffeners used as connection plates shall be attached to the flanges by welding or bolting with adequate strength to transfer horizontal force in the cross members to the flanges.

Two longitudinal stiffeners may be used on web section where stress reversal occurs.

Longitudinal and transverse stiffeners preferably shall be attached on opposite sides of the web.

Bearing stiffeners shall be placed in pairs, bolted or welded to the girder web or diaphragm.
2.13      GENERAL

2.14      EFFECTIVE FLANGE WIDTH

2.15      NONCOMPOSITE DEAD LOAD STRESSES

2.16      COMPOSITE SECTION STRESSES

2.17      SHEAR CONNECTORS
2.13 **GENERAL**

- Girders shall be proportioned for ordinary bending and for nonuniform torsion (also called warping or lateral flange bending)

- Usual moment of inertia method shall be used for bending

- Any rational method shall be used for nonuniform torsion

- For fatigue, perform stress analysis including bending and nonuniform torsion
2.14 EFFECTIVE FLANGE WIDTH

- AASHTO Standard Spec. (Art. 10.38.3;**2003 Art. 4.5.2)
  not to exceed:
  - ¼ of the span length of the girder
  - the distance center to center of girders
  - 12 x the least thickness of the slab

- AASHTO LRFD (Art. 4.6.2.6)
  For interior beams, the least of:
  - ¼ of the effective span length
  - 12 x the average thickness of the slab + greater of
    (web thickness, ½ the width of the top flange of the girder)
  - the average spacing of adjacent beams
2.15 **NONCOMPOSITE DEAD LOAD STRESSES**

- \((f_b)_{DL1} \leq F_b\) determined from Art. 2.12 (B)
- \((f_b + f_w)_{DL1} \leq F_y\) determined from Art. 2.12 (B)

\[ \frac{b}{t} \leq \frac{4400}{\sqrt{1.3f_{DL1}}} \]

where \(f_{DL1} = (f_b + f_w)_{DL1}\)

**Revised in 2003, same equations for noncomposite and composite**
2.16  COMPOSITE SECTION STRESSES

- Compact within Concrete  \( f_b \leq F_y \)

- Noncompact, compression flange
  - braced by cross-frames or diaphragms
    \[
    f_b \leq F_{by} \\
    f_b + f_w \leq F_y
    \]
  - braced by concrete
    \[
    f_b + (f_w)_{DL} \leq F_y
    \]

** Revised in 2003, same equations for noncomposite and composite**
2.17 SHEAR CONNECTORS

Same as current AASHTO for straight girders, except as modified by Article 1.19

(A) Fatigue
(B) Ultimate Strength

** Covered in 2003 Section 7 – Shear Connectors
- Strength

\[ N = \frac{P}{\phi_{sc} S_u} \]  
Eq. (7-1)

where:
\[ \phi_{sc} = 0.85 \]
\[ S_u = \text{strength of one shear connector according to AASHTO Article 10.38.5.1.2 (kip)} \]
\[ P = \text{force in slab at point of maximum positive live load moment given by Equation (7-2) (kip)} \]

\[ P = \sqrt{P_p^2 + F_p^2} \]  
Eq. (7-2)
where:

\[ P_{lp} = \text{longitudinal force in the slab at point of maximum positive live load moment computed as the smaller of } P_{1p} \text{ or } P_{2p} \text{ from Equations (7-3) and (7-4) (kip)} \]

\[ F_{rp} = \text{radial force in the slab at point of maximum positive live load moment computed from Equation (7-5) (kip)} \]

\[ P_{1p} = A_s F_y \quad \text{Eq. (7-3)} \]

where:

\[ A_s = \text{area of steel girder (in}^2) \]

\[ P_{2p} = 0.85 f_c' b_d t_d \quad \text{Eq. (7-4)} \]

where:

\[ f_c' = \text{specified 28-day compressive stress of concrete (ksi)} \]

\[ b_d = \text{effective concrete with as specified in Article 4.5.2 (in)} \]

\[ t_d = \text{average concrete thickness (in)} \]
Eq. (7-5)

\[
\overline{F_p} = \overline{P_p} \frac{L_p}{R}
\]

where:

\( L_p \) = arc length between an end of the girder and an adjacent point of maximum positive live load moment (ft)

\( R \) = minimum girder radius over the length, \( L_p \) (ft)

Between \( M_{LL}^+ \) and \( M_{LL}^- \):

Eq. (7-6)

\[
P = \sqrt{P_T^2 + F_T^2}
\]

where:

\( \overline{P_T} \) = longitudinal force in the concrete slab at point of greatest negative live load moment computed from Equation (7-7) (kip)

\( \overline{F_T} \) = radial force in the concrete slab at point of greatest negative live load moment computed from Equation (7-10) (kip)
\[
\bar{P}_T = \bar{P}_p + \bar{P}_n
\]  
Eq. (7-7)

where:

\[
\bar{P}_n = \text{the smaller of } P_{1n} \text{ or } P_{2n} \text{ (kip)}
\]

\[
P_{1n} = A_s F_y
\]  
Eq. (7-8)

\[
P_{2n} = 0.45 f_c' b_d t_d
\]  
Eq. (7-9)

\[
\bar{F}_T = \frac{\bar{P}_n L_n}{R}
\]  
Eq. (7-10)

where:

\[
L_n = \text{arc length between a point of maximum positive live load moment and an adjacent point of greatest negative live load moment (ft)}
\]

\[
R = \text{minimum girder radius over the length, } L_n \text{ (ft)}
\]
** 2003 New Guide Specifications Section 7

## Fatigue

\[
V_{sr} = \sqrt{(V_{\text{fat}})^2 + (F_{\text{fat}})^2}
\]

Eq. (7-11)

Where:

- \(V_{\text{fat}}\) = longitudinal fatigue shear range/unit length (k/in)
- \(F_{\text{fat}}\) = radial fatigue shear range/unit length (k/in)

\[
F_{\text{fat}} = \frac{A_{\text{bot}}\sigma_{\text{flg}}\ell}{wR}
\]

Eq. (7-12)

Where:

- \(A_{\text{bot}}\) = area of bottom flange (in²)
- \(\sigma_{\text{flg}}\) = range of fatigue stress in the bottom flange (ksi)
- \(\ell\) = distance between brace points (ft)
- \(R\) = minimum girder radius within the panel (ft)
- \(w\) = effective length of deck (in)
$F_{\text{fat}} = \frac{F_{CR}}{W}$

Eq. (7-13)

where:

$F_{CR} =$ net range of cross frame force at the top flange (kip)

$p = \frac{nZ_r}{V_{sr}}$

Eq. (7-14)

where:

$n =$ number of shear connectors in a cross section

$Z_r =$ shear fatigue strength of an individual shear connector determined as specified in AASHTO LRFD Article 6.10.7.4.2 (kip)

$V_{sr} =$ range of horizontal shear for fatigue from Equation (7-11) (kip)
2.18 GENERAL

2.19 ALLOWABLE STRESSES

2.20 PLATE THICKNESS REQUIREMENTS

** not covered in 2003 new Guide Specifications
2.18 GENERAL

- Pertains to hybrid I-girders having a vertical axis of symmetry through the middle-plane of the web plate.
2.19 ALLOWABLE STRESSES

(A) Bending-Noncomposite Girders

- Allowable flange stress

Equal to allowable stress from Article 2.12 multiplied by reduction factor

\[ R = 1 - \frac{B\Psi(1 - \alpha')^2(3 - \Psi + \Psi\alpha')}{6 + B\Psi(3 - \Psi)} \]

where \( F_y \) = yield strength of compression flange

\[ B = \frac{\text{web area}}{\text{tension flange area}} \]

\[ \Psi = \frac{\text{C of tension flange}}{\text{depth of section}} \]
\[
\alpha' = \alpha \left(1 + \left| \frac{f_w}{f_b} \right|_t \right)
\]

\[
\alpha = \frac{\text{web yield strength}}{\text{tension flange yield strength}}
\]

\[
\left| \frac{f_w}{f_b} \right|_t = \text{absolute value of (lateral flange bending stress / bending stress) for tension flange}
\]

If \( \left| \frac{f_w}{f_b} \right|_t \geq \frac{1 - \alpha}{\alpha} \), then \( R = 1 \)
(B) Bending – Composite Girders

- Allowable flange stress – positive moment region
  Same as for noncomposite girder

- Allowable flange stress – negative moment region
  Same as for noncomposite girder, except $\alpha$ is defined as follows:

\[
\alpha' = \alpha \\
\text{when } \left| \frac{f_w}{f_b} \right|_c \leq \frac{2\Psi - 1}{1 - \Psi}, \quad \alpha' = \alpha
\]

\[
\text{when } \frac{2\Psi - 1}{1 - \Psi} < \left| \frac{f_w}{f_b} \right|_c < \frac{\Psi}{\alpha(1 - \Psi)} - 1
\]
\[ \alpha' = \alpha \left[ 1 + \left| \frac{f_w}{f_b} \right| \frac{1 - \Psi}{\Psi} \right] \]

where \( \left| \frac{f_w}{f_b} \right| \) is the same as \( \left| \frac{f_w}{f_b} \right| \) except that it is for the compression flange

if \( \left| \frac{f_w}{f_b} \right| \geq \frac{\Psi}{\alpha(1 - \Psi)} - 1 \), then \( R = 1 \)
2.19 ALLOWABLE STRESSES (Cont.)

- **Shear** (same as homogeneous section in Art. 2.12(C))
  \[ V_u = C \, V_p \]

- **Fatigue**

  Same as current AASHTO for straight hybrid girders
2.20 PLATE THICKNESS REQUIREMENTS

- Web plate thickness requirements
  
  Same as for non-hybrid curved girders, except that $f_b$ in the formulas is replaced by $f_b / R$

- Flange plate thickness requirements
  
  Same as for non-hybrid curved girders
THE END

Questions ??