LRFD Steel Design

AASHTO LRFD Bridge Design Specifications

Slide Shows
LRFD Steel Design

AASHTO LRFD Bridge Design Specification

Slide Shows

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AASHTO LRFD
Review of Loads and Analysis

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References

- “Four LRFD Design Examples of Steel Highway Bridges,” Vol. II, Chapter 1A Highway Structures Design Handbook, Published by American Iron and Steel Institute in cooperation with HDR Engineering, Inc. Available at http://www.aisc.org/
References

- AASHTO Web Site: [http://bridges.transportation.org/](http://bridges.transportation.org/)

References

- AISC / National Steel Bridge Alliance Web Site: [http://www.steelbridges.org/](http://www.steelbridges.org/)
- “Steel Bridge Design Handbook”
References


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References

Philosophies of Design

LRFD: Load & Resistance Factor Design

- For Safety:
  \[ \sum \gamma Q \leq \phi R \]

  - \( \gamma \) - Load Factor
  - \( R \) - Component Resistance
  - \( \phi \) - Resistance Factor

The LRFD philosophy provides a more uniform, systematic, and rational approach to the selection of load factors and resistance factors than LFD.

Philosophies of Design - LRFD Fundamentals

Reliability Index:

ASD / LFD Bridge Designs

LRFD Bridge Designs (Expected)
Chapter 1 – Introduction

§1.3.2: Limit States

- **Service:**
  - Deals with restrictions on stress, deformation, and crack width under regular service conditions.
  - Intended to ensure that the bridge performs acceptably during its design life.

- **Strength:**
  - Intended to ensure that strength and stability are provided to resist statistically significant load combinations that a bridge will experience during its design life.
  - Extensive distress and structural damage may occur at strength limit state conditions, but overall structural integrity is expected to be maintained.

- **Extreme Event:**
  - Intended to ensure structural survival of a bridge during an earthquake, vehicle collision, ice flow, or foundation scour.

- **Fatigue:**
  - Deals with restrictions on stress range under regular service conditions reflecting the number of expected cycles.
Chapter 1 – Introduction

§1.3.2: Limit States

\[ Q = \sum \eta_i \gamma_i Q_i \]  
(1.3.2.1-1)

\( \gamma_i \) - Load Factor
\( Q_i \) - Load Effect
\( \eta_i \) - Load Modifier

When the maximum value of \( \gamma_i \) is appropriate

\[ \eta_i = \eta_D \eta_R \eta_I \geq 0.95 \]  
(1.3.2.1-2)

When the minimum value of \( \gamma_i \) is appropriate

\[ \eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.00 \]  
(1.3.2.1-3)

Pg 1.3
AASHTO-LRFD 2007

Chapter 1 – Introduction

§1.3.2: Limit States - Load Modifiers

Applicable only to the Strength Limit State

- \( \eta_D \) – Ductility Factor:
  - \( \eta_D = 1.05 \) for nonductile members
  - \( \eta_D = 1.00 \) for conventional designs and details complying with specifications
  - \( \eta_D = 0.95 \) for components for which additional ductility measures have been taken

- \( \eta_R \) – Redundancy Factor:
  - \( \eta_R = 1.05 \) for nonredundant members
  - \( \eta_R = 1.00 \) for conventional levels of redundancy
  - \( \eta_R = 0.95 \) for exceptional levels of redundancy

- \( \eta_I \) – Operational Importance:
  - \( \eta_I = 1.05 \) for important bridges
  - \( \eta_I = 1.00 \) for typical bridges
  - \( \eta_I = 0.95 \) for relatively less important bridges

These modifiers are applied at the element level, not the entire structure.

Pgs. 1.5-7; Chen & Duan
AASHTO-LRFD 2007
§ 3.4 - Load Factors and Combinations

§1.3.2: ODOT Recommended Load Modifiers

For the Strength Limit States:

- $\eta_D$ - Ductility Factor:
  - Use a ductility load modifier of $\eta_D = 1.00$ for all strength limit states

- $\eta_R$ - Redundancy Factor:
  - Use $\eta_R = 1.05$ for "non-redundant" members
  - Use $\eta_R = 1.00$ for "redundant" members
  - Bridges with 3 or fewer girders should be considered "non-redundant."
  - Bridges with 4 girders with a spacing of 12’ or more should be considered “non-redundant.”
  - Bridges with 4 girders with a spacing of less than 12’ should be considered “redundant.”
  - Bridge with 5 or more girders should be considered “redundant.”

For information on other substructure types, refer to NCHRP Report 458 Redundancy in Highway Bridge Substructures.

$\eta_R$ does NOT apply to foundations. Foundation redundancy is included in the resistance factor.
§ 1.3.2: ODOT Recommended Load Modifiers

For the Strength Limit States:

- **\( \eta_I \) - Operational Importance:**
  - In General, use \( \eta_I = 1.00 \) unless one of the following applies
  - Use \( \eta_I = 1.05 \) if any of the following apply
    - Design ADT \( \geq 60,000 \)
    - Detour length \( \geq 50 \) miles
    - Any span length \( \geq 500' \)
  - Use \( \eta_I = 0.95 \) if both of the following apply
    - Design ADT \( \leq 400 \)
    - Detour length \( \leq 10 \) miles

Detour length applies to the shortest, emergency detour route.
§ 3.4 - Loads and Load Factors

§ 3.4.1: Load Factors and Load Combinations

Permanent Loads
- DD - Downdrag
- DC - Structural Components and Attachments
- DW - Wearing Surfaces and Utilities
- EH - Horizontal Earth Pressure
- EL - Locked-In Force Effects Including Pretension
- ES - Earth Surcharge Load
- EV - Vertical Pressure of Earth Fill

Transient Loads
- BR - Veh. Braking Force
- CE - Veh. Centrifugal Force
- CR - Creep
- CT - Veh. Collision Force
- CV - Vessel Collision Force
- EQ - Earthquake
- FR - Friction
- IC - Ice Load
- LL - Veh. Live Load
- IM - Dynamic Load Allowance
- LS - Live Load Surcharge
- PL - Pedestrian Live Load
- SE - Settlement
- SH - Shrinkage
- TG - Temperature Gradient
- TU - Uniform Temperature
- WA - Water Load
- WL - Wind on Live Load
- WS - Wind Load on Structure
§ 3.4 - Loads and Load Factors

§ 3.4.1: Load Factors and Load Combinations

| Load Combination | DC | DD | DW | EH | EV | ES | LL | IM | CE | BR | PL | LS | WA | WS | WL | FR | TG | CS | SH | TG | SE | EQ | IC | CT | CV |
|------------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| STRENGTH I       | $\gamma_p$ | 1.75 | 1.00 | -- | -- | 1.00 | 0.50/1.20 | $\gamma_{SE}$ | $\gamma_{TG}$ | $\gamma_{SE}$ | -- | -- | -- | -- |
| STRENGTH II      | $\gamma_p$ | 1.35 | 1.00 | -- | -- | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | -- | -- | -- | -- |
| STRENGTH III     | $\gamma_p$ | 1.00 | 1.40 | -- | -- | 1.00 | 0.50/1.20 | $\gamma_{SE}$ | $\gamma_{TG}$ | $\gamma_{SE}$ | -- | -- | -- | -- |
| STRENGTH IV      | $\gamma_p$ | 1.00 | -- | -- | 1.00 | 0.50/1.20 | -- | -- | -- | -- | -- | -- | -- | -- |
| STRENGTH V       | $\gamma_p$ | 1.35 | 1.00 | 0.40 | 1.0 | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | -- | -- | -- | -- | -- |

Table 3.4.1-1 Load Combinations and Load Factors (cont.)

| Load Combination | DC | DD | DW | EH | EV | ES | LL | IM | CE | BR | PL | LS | WA | WS | WL | FR | TG | CS | SH | TG | SE | EQ | IC | CT | CV |
|------------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| EXTREME EVENT I  | $\gamma_p$ | $\gamma_{EQ}$ | 1.00 | -- | -- | 1.00 | -- | -- | -- | -- | -- | -- | -- | -- | -- | 1.00 | -- | -- | -- |
| EXTREME EVENT II | $\gamma_p$ | 0.50 | 1.00 | -- | -- | 1.00 | -- | -- | -- | -- | -- | -- | -- | -- | -- | 1.00 | 1.00 | 1.00 |
| FATIGUE – LL, IM, & CE ONLY | -- | 0.75 | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |

-- 10 --
§ 3.4 - Loads and Load Factors

§3.4.1: Load Factors and Load Combinations

Table 3.4.1-1 Load Combinations and Load Factors (cont.)

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>DC</th>
<th>DD</th>
<th>DW</th>
<th>LL</th>
<th>IM</th>
<th>CE</th>
<th>BR</th>
<th>PL</th>
<th>LS</th>
<th>WA</th>
<th>WS</th>
<th>WL</th>
<th>TU</th>
<th>CR</th>
<th>SH</th>
<th>TG</th>
<th>SE</th>
<th>EQ</th>
<th>IC</th>
<th>CT</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>SERVICE I</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.30</td>
<td>1.0</td>
<td>1.00</td>
<td>1.00/1.20</td>
<td>7TE</td>
<td>7TE</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
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<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SERVICE II</td>
<td>1.00</td>
<td>1.30</td>
<td>1.00</td>
<td>--</td>
<td>--</td>
<td>1.00</td>
<td>1.00/1.20</td>
<td>--</td>
<td>--</td>
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<td>--</td>
<td>--</td>
<td>--</td>
<td></td>
<td></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>7TE</td>
<td>7TE</td>
<td>--</td>
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<td>--</td>
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<td>--</td>
<td>--</td>
<td>--</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SERVICE IV</td>
<td>1.00</td>
<td>--</td>
<td>1.00</td>
<td>0.70</td>
<td>--</td>
<td>1.00</td>
<td>1.00/1.20</td>
<td>--</td>
<td>--</td>
<td>--</td>
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<td>--</td>
<td>--</td>
<td>--</td>
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<td>--</td>
<td>--</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Use One of These at a Time:

- SETG
- TU
- CR
- SHFR
- WLWSWA
- LL
- IM
- CE
- BR
- PL
- LS
- DC
- DD
- DW
- EH
- EV
- ES
- EL

---

§ 3.4 - Loads and Load Factors

§3.4.1: Load Factors and Load Combinations

- **Strength I:** Basic load combination relating to the normal vehicular use of the bridge without wind.

- **Strength II:** Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both, without wind.

- **Strength III:** Load combination relating to the bridge exposed to wind in excess of 55 mph.

- **Strength IV:** Load combination relating to very high dead load to live load force effect ratios. (Note: In commentary it indicates that this will govern where the DL/LL >7, spans over 600', and during construction checks.)

- **Strength V:** Load combination relating to normal vehicular use with a wind of 55 mph.
§ 3.4 - Loads and Load Factors

§ 3.4.1: Load Factors and Load Combinations

- **Extreme Event I**: Load combination including earthquakes.

- **Extreme Event II**: Load combination relating to ice load, collision by vessels and vehicles, and certain hydraulic events with a reduced live load.

- **Fatigue**: Fatigue and fracture load combination relating to repetitive gravitational vehicular live load and dynamic responses under a single design truck.

- **Service I**: Load combination relating to normal operational use of the bridge with a 55 mph wind and all loads at nominal values. Compression in precast concrete components.

- **Service II**: Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular load.

- **Service III**: Load combination relating only to tension in prestressed concrete superstructures with the objective of crack control.

- **Service IV**: Load combination relating only to tension in prestressed concrete columns with the objective of crack control.
§ 3.4 - Loads and Load Factors

§3.4.1: Load Factors and Load Combinations

Table 3.4.1-2 Load Factors for Permanent Loads, $\gamma_p$

<table>
<thead>
<tr>
<th>Type of Load, Foundation Type, and Method Used to Calculate Downdrag</th>
<th>Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum</td>
</tr>
<tr>
<td>$DC$: Component and Attachments</td>
<td>1.25 0.90</td>
</tr>
<tr>
<td>$DC$: Strength IV only</td>
<td>1.50 0.90</td>
</tr>
<tr>
<td>$DD$: Downdrag</td>
<td>1.4 0.25</td>
</tr>
<tr>
<td>$Piles, \alpha$ Tomlinson Method</td>
<td>1.05 0.30</td>
</tr>
<tr>
<td>$Piles, \lambda$ Method</td>
<td>1.25 0.35</td>
</tr>
<tr>
<td>$DW$: Wearing Surfaces and Utilities</td>
<td>1.50 0.65</td>
</tr>
<tr>
<td>$EH$: Horizontal Earth Pressure</td>
<td>1.50 0.90</td>
</tr>
<tr>
<td>$\bullet$ Active</td>
<td>1.35 0.90</td>
</tr>
<tr>
<td>$\bullet$ At-Rest</td>
<td></td>
</tr>
<tr>
<td>$EL$: Locked in Erections Stresses</td>
<td>1.00 1.00</td>
</tr>
</tbody>
</table>

Common load combinations for Steel Design

- **Strength I:** $1.25DC + 1.50DW + 1.75(LL+IM)$
- **Service II:** $1.00DC + 1.00DW + 1.30(LL+IM)$
- **Fatigue:** $0.75(LL+IM)$
§ 3.5 – Permanent Loads

§3.5.1 Dead Loads: DC and DW

- DC is the dead load of the structure and components present at construction. These have a lower load factor because they are known with more certainty.

- DW are future dead loads, such as future wearing surfaces. These have a higher load factor because they are known with less certainty.

§ 3.6 - Live Loads

§3.6.1.1: Lane Definitions

- # Design Lanes = INT(w/12.0 ft)
  - w is the clear roadway width between barriers.

- Bridges 20 to 24 ft wide shall be designed for two traffic lanes, each ½ the roadway width.

- Examples:
  - A 20 ft. wide bridge would be required to be designed as a two lane bridge with 10 ft. lanes.
  - A 38 ft. wide bridge has 3 design lanes, each 12 ft. wide.
  - A 16 ft. wide bridge has one design lane of 12 ft.
§ 3.6 - Live Loads

§3.6.1.3.1: Application of Design Vehicular Loads

- The governing force effect shall be taken as the larger of the following:
  - The effect of the design tandem combined with the design lane load
  - The effect of one design truck (HL-93) combined with the effect of the design lane load
  - For negative moment between inflection points, 90% of the effect of two design trucks (HL-93 with 14 ft. axle spacing) spaced at a minimum of 50 ft. combined with 90% of the design lane load.

§ 3.6 - Live Loads

§3.6.1.2.2: Design Truck

8 kip
32 kip
14' - 0" to 30' - 0"
6' - 0"

14' - 0"
§ 3.6 - Live Loads

§3.6.1.2.3: Design Tandem

0.640kip/ft is applied SIMULTANEOUSLY with the design truck or design tandem over a width of 10 ft. within the design lane.

NOTE: the impact factor, IM, is NOT applied to the lane load. It is only applied to the truck or tandem load.

This is a big change from the Standard Specifications…
§ 3.6 - Live Loads

AASHTO Standard Spec vs LRFD Spec:

Old Std Spec Loading:
- HS20 Truck, or
- Alternate Military, or
- Lane Load

New LRFD Loading:
- HL-93 Truck and Lane Load, or
- Tandem and Lane Load, or
- 90% of 2 Trucks and Lane Load

---

§ 3.6 - Live Loads

Live Loads for Maximum Positive Moment in Span 1

- The impact factor is applied only to the truck, not the lane load
- Although a truck in the third span would contribute to maximum response, by specification only one truck is used.
§ 3.6 - Live Loads

Live Loads for Shear at Middle of Span 1

- Impact is applied only to the truck.
- In this case, the front axle is ignored as it does not contribute to the maximum response.

Live Loads for Maximum Moment Over Pier 1

- Impact is applied to the trucks only.
- The distance between rear axles is fixed at 14 ft.
- The distance between trucks is a minimum of 50 ft.
- This applies for negative moment between points of contraflexure and reactions at interior piers.
§ 3.6 - Live Loads

§3.6.1.3: Application of Design Vehicular Live Loads

- In cases where the transverse position of the load must be considered:
  - The design lanes are positioned to produce the extreme force effect.
  - The design lane load is considered to be 10 ft. wide. The load is positioned to maximize the extreme force effect.
  - The truck/tandem is positioned such that the center of any wheel load is not closer than:
    - 1.0 ft. from the face of the curb/railing for design of the deck overhang.
    - 2.0 ft. from the edge of the design lane for design of all other components.

---

§ 3.6 - Live Loads

Both the Design Lanes and 10’ Loaded Width in each lane shall be positioned to produce extreme force effects.

- Center of truck wheels must be at least 2’ from the edge of a design lane
- The lane load may be at the edge of a design lane.
§ 3.6 - Live Loads

### Multiple Presence Factor

<table>
<thead>
<tr>
<th># of Loaded Lanes</th>
<th>MP Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.20</td>
</tr>
<tr>
<td>2</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>0.85</td>
</tr>
<tr>
<td>&gt;3</td>
<td>0.65</td>
</tr>
</tbody>
</table>

These factors are based on an assumed ADTT of 5,000 trucks:
- If the ADTT is less than 100, 90% of the specified force may be used
- If the ADTT is less than 1,000, 95% of the specified force may be used

*Multiple Presence Factors are NOT used with the Distribution Factors*

§ 3.6 - Live Loads

### §3.6.2: Dynamic Load Allowance

**Impact Factors, IM**
- Deck Joints 75% **ODOT EXCEPTION**
  - 125% of static design truck or 100% of static design tandem
- Fatigue 15%
- All other cases 33%

The Dynamic Load Allowance is applied only to the truck load (including fatigue trucks), not to lane loads or pedestrian loads.
§6.6 - Fatigue and Fracture Considerations

§3.6.1.4.1: Fatigue Truck

The fatigue truck is applied alone – lane load is NOT used. The dynamic allowance for fatigue is $IM = 15\%$. The load factor for fatigue loads is 0.75 for $LL$, $IM$ and $CE$ ONLY.

No multiple presence factors are used in the Fatigue Loading, the distribution factors are based on one lane loaded, and load modifiers ($\eta$) are taken as 1.00.

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AASHTO-LRFD  
Chapter 4: Structural Analysis and Evaluation

James A Swanson

§4.4 – Acceptable Methods of Structural Analysis

- **Simplified Analysis**
  - Distribution Factor

- **Refined Analysis**
  - Finite Element Modeling

---

§4.6.2 - Approximate Methods of Analysis – Dist Factors

§ 4.6.2.2 Lateral Load Distribution Beam and Slab Bridges

- Design live load bending moment or shear force is the product of a lane load on a beam model and the appropriate distribution factor.

\[ M_{U,LL} = (DF)(M_{Beam,Line}) \]

- The following Distribution Factors are applicable to Reinforced Concrete Decks on Steel Girders, CIP Concrete Girders, and Precast Concrete I or Bulb-Tee sections.

- Also applies to Precast Concrete Tee and Double Tee Sections when sufficient connectivity is present.
§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2 Lateral Load Distribution Beam and Slab Bridges

The simplified distribution factors may be used if:

- Width of the slab is constant
- Number of beams, $N_b \geq 4$
- Beams are parallel and of similar stiffness
- Roadway overhang $d_e \leq 3\text{ ft}^*$
- Central angle $< 40^\circ$
- Cross section conforms to AASHTO Table 4.6.2.1-1

*ODOT Exception: The roadway overhang $d_e \leq 3\text{ ft.}$ does not apply to interior DFs for sections (a) and (k).

---

§4.6.2 - Approximate Methods of Analysis – Distribution Factors

This is part of Table 4.6.2.1-1 showing common bridge types.

The letter below the diagram correlates to a set of distribution factors.

Slab-on-Steel-Girder bridges qualify as type (a) cross sections.
§4.6.2 - Approximate Methods of Analysis – Distribution Factors

This is a part of Table 4.6.2.2b-1 showing distribution factors for moment. A similar table exists for shear distribution factors.

The table gives the DF formulae and the limits on the specific terms. If a bridge does NOT meet these requirements or the requirements on the previous slide, refined analysis must be used.

---

Concrete Deck, Filled Grid, Partially Filled Grid, Unfilled Grid, Filled Grid Composite, Reinforced Concrete Slab on Steel or Concrete Beams, T-Beams, and Double T-Sections

- $n$, $e$, $k$ and also $i, j$ if sufficiently connected to act as a unit
- One Design Lane Loaded:
  \[
  0.06 + \frac{S}{14} \left( \frac{S}{L} \right)^{0.5} \left( \frac{K_1}{12.0L^2} \right)^{0.61}
  \]
  Two or More Design Lanes Loaded:
  \[
  0.075 + \frac{S}{9.5} \left( \frac{S}{L} \right)^{0.62} \left( \frac{K_2}{12.0L^2} \right)^{1.31}
  \]

Use lesser of the values obtained from the equation above with $N_i = 3$ or the lever rule $N_i = 3$.
§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.2 Moment Distribution - Interior Girders

- **Interior Girders:**
  - **One Lane Loaded:**
    \[
    DF_{M,Int} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.1} \frac{K_g}{12L^1.1} 
    \]
  - **Two or More Lanes Loaded:**
    \[
    DF_{M,Int} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \frac{K_g}{12L^1.1} 
    \]

This term may be taken as 1.00 for prelim design.

Parameter Definitions & Limits of Applicability:

- **S** - Beam or girder spacing (ft.) \(3.5 \leq S \leq 16.0\)
- **L** - Span length of beam or girder (ft.) \(20 \leq L \leq 240\)
- **Kg** - Longitudinal stiffness parameter (in\(^4\)) \(10k \leq Kg \leq 7M\)
- **t_s** - Thickness of concrete slab (in) \(4.5 \leq t_s \leq 12.0\)
- **d_e** - Distance from exterior beam to interior edge of curb (ft.) (Positive if the beam is “inside” of the curb.) \(-1.0 \leq d_e \leq 5.5\)
Parameter Definitions & Limits of Applicability:

\[ K_g = n \left( I + Ae_g^2 \right) \]  \hspace{1cm} (4.6.2.2.1-1)

- \( n \) - Modular ratio, \( E_{Beam} / E_{Deck} \)  
- \( I \) - Moment of inertia of beam (in\(^4\))
- \( A \) - Area of beam (in\(^2\))
- \( e_g \) - Distance between CG steel and CG deck (in)

ODOT Exception: For interior beam DF, include monolithic wearing surface and haunch in \( e_g \) and \( K_g \) when this increases the DF.

**§ 4.6.2.2d Moment Distribution - Exterior Beams**

**Exterior Girders:**

- One Lane Loaded: 
  
  Lever Rule

- Two or More Lanes Loaded:
  
  \[ DF_{ext} = e \cdot DF_{int} \]

\[ e = 0.77 + \frac{d}{9.1} \]
§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2d Moment Distribution - Exterior Beams

- **Lever Rule:**
  - Assume a hinge develops over each interior girder and solve for the reaction in the exterior girder as a fraction of the truck load.

This example is for one lane loaded. Multiple Presence Factors apply. 1.2 is the MPF

\[
\sum M_y \rightarrow 1.2Pe - RS = 0
\]

\[
R = \frac{1.2Pe}{S} \quad \therefore DF = \frac{1.2e}{S}
\]

In the diagram, \( P \) is the axle load.

---

§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2e Moment Distribution - Skewed Bridges

- **Correction for Skewed Bridges:**
  - The bending moment may be reduced in bridges with a skew of \( 30^\circ \leq \theta \leq 60^\circ \)

\[
DF'_M = \left(1 - C_i (\tan \theta)^{0.5}\right)DF_M
\]

\[
C_i = 0.25 \left(\frac{K_g}{12Li}\right)^{0.25} S^{0.5}
\]

- When the skew angle is greater than 60°, take \( \theta = 60^\circ \)
§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.3a Shear Distribution - Interior Beams

- **Interior Girders:**
  - One Lane Loaded:
    \[ DF_{int} = 0.36 + \frac{S}{25.0} \]
  - Two or More Lanes Loaded:
    \[ DF_{int} = 0.2 + \frac{S}{12} - \left( \frac{S}{35} \right)^2 \]

Pg 4.41 - Table 4.6.2.3a-1  AASHTO-LRFD 2007
ODOT Short Course Created July 2007 Review of Loads: Slide #55

§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.3b Shear Distribution - Exterior Beams

- **Exterior Girders:**
  - One Lane Loaded:
    Lever Rule
  - Two or More Lanes Loaded:
    \[ DF_{ext} = e \cdot DF_{int} \]
    \[ e = 0.60 + \frac{d}{10} \]

Pg 4.43 - Table 4.6.2.3b-1  AASHTO-LRFD 2007
ODOT Short Course Created July 2007 Review of Loads: Slide #56
§ 4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.3c Shear Distribution - Skewed Bridges

- Correction for Skewed Bridges:
  - The shear forces in beams of skewed bridges shall be adjusted with a skew of $0^\circ \leq \theta \leq 60^\circ$

$$DF'_s = \left(1.0 + 0.20 \left(\frac{12L_s}{K_v}T_{\theta/2}\right)^{1/3}\right)DF_s$$

Pg 4.44 - Table 4.6.2.2.3c-1

§ 4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.2d Exterior Beams

- Minimum Exterior DF: (Rigid Body Rotation of Bridge Section)

$$DF_{Ext,ab} = \frac{N_L}{N_b} \frac{X_{Ext}}{\sum e} \left(\sum X^2\right)$$  \hspace{1cm} (C4.6.2.2d-1)

- $N_L$ - Number of loaded lanes under consideration
- $N_b$ - Number of beams or girders
- $e$ - Eccentricity of design truck or load from CG of pattern of girders (ft.)
- $x$ - Distance from CG of pattern of girders to each girder (ft.)
- $X_{Ext}$ - Distance from CG of pattern of girders to exterior girder (ft.)
§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.2d Exterior Beams

- Minimum Exterior DF: (Rigid Body Rotation of Bridge Section)

\[ DF_{\text{ext,2d}} = \frac{N_L}{N_b} + \frac{X_{E_x}}{\sum X^2} \]  

(N4.6.2.2d-1)

- \( N_L \): Number of loaded lanes under consideration
- \( N_b \): Number of beams or girders
- \( e \): Eccentricity of design truck or load from CG of pattern of girders (ft.)
- \( x \): Distance from CG of pattern of girders to each girder (ft.)
- \( X_{E_x} \): Distance from CG of pattern of girders to exterior girder (ft.)

§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.1 Dead Load Distribution

“Where bridges meet the conditions specified herein, permanent loads of and on the deck may be distributed uniformly among the beams and/or stringers. For this type of bridge, the conditions are:”

- Width of deck is constant
- Unless otherwise specified, the number of beams is not less than four
- Beams are parallel and have approximately the same stiffness
- Unless otherwise specified, the roadway part of the overhang, \( d_e \), does not exceed 3.0 ft
- Curvature in plan is less then the limit specified in Article 4.6.1.2
- Cross-section is consistent with one of the cross-sections shown Table 4.6.2.2.1-1
§6.1 - Scope

This chapter covers the design of steel components, splices and connections for straight or horizontally curved beam and girder structures, frames, trusses and arches, cable-stayed and suspension systems, and metal deck systems, as applicable.

Although horizontally curved girder structures are now included in the AASHTO-LRFD Specification, they will not be specifically addressed in this course.

§6.4 - Materials

- 6.4.1 Structural Steels
- 6.4.2 Pins, Roller, and Rockers
- 6.4.3 Bolts, Nuts, and Washers
- 6.4.4 Stud Shear Connectors
- 6.4.5 Weld Metal
- 6.4.6 Cast Metal
- 6.4.7 Stainless Steel
- 6.4.8 Cables
§6.4 - Materials

§6.4.1: Structural Steels

Table 6.4.1-1 Minimum Mechanical Properties of Structural Steel

<table>
<thead>
<tr>
<th>Equivalent ASTM Designation</th>
<th>M270</th>
<th>M270</th>
<th>M270</th>
<th>M270</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of Plate (in)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Up to 4.0 incl.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Not Applicable</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Tensile Strength, $F_u$ (ksi)</td>
<td>58</td>
<td>65</td>
<td>65</td>
<td>70</td>
</tr>
<tr>
<td>Minimum Yield Strength, $F_y$ (ksi)</td>
<td>36</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Equivalent ASTM Designation</th>
<th>M270</th>
<th>M270</th>
<th>M270</th>
<th>M270</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of Plate (in)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Up to 4.0 incl.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Up to 4.0 incl.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Tensile Strength, $F_u$ (ksi)</td>
<td>70</td>
<td>85</td>
<td>110</td>
<td>100</td>
</tr>
<tr>
<td>Minimum Yield Strength, $F_y$ (ksi)</td>
<td>50</td>
<td>70</td>
<td>100</td>
<td>90</td>
</tr>
</tbody>
</table>

Pgs 6.20-22  AASHTO-LRFD 2007

ODOT Short Course  Created July 2007  Materials and Limit States: Slide #5

§6.4 - Materials

BDM §302.4.1.1: Material Requirements

Types of steel to be selected in the design of bridges is as follows:

- ASTM A709 grade 50W shall be specified for an un-coated weathering steel bridge.
- ASTM A709 grade 50 shall be specified for a coated steel bridge.
- ASTM A709 grade 36 is not recommended and is being discontinued by the steel mills.
- High Performance Steel (HPS), A709 grade 70W, un-coated weathering steel is most economical when used in the flanges of hybrid girders. Consult the Office of Structural Engineering for recommendations prior to specifying its use. A plan note is provided in the appendix.

BDM Pg 3-19  AASHTO-LRFD 2007

ODOT Short Course  Created July 2007  Materials and Limit States: Slide #6
§6.4 - Materials

§6.4.3: Bolts, Nuts, and Washers

- Bolts shall conform to one of the following:
  - ASTM A307 $F_u = 60$ksi
  - AASHTO M164 (ASTM A325) $F_u = 120$ksi / $105$ksi
  - AASHTO M253 (ASTM A490) $F_u = 150$ksi \(\leftarrow\) Prohibited by ODOT

- Nuts shall conform to:
  - AASHTO M291 (ASTM A563) for use with M164 and M253 bolts

- Washers shall conform to:
  - AASHTO M293 (ASTM F436)

§6.4.4: Stud Shear Connectors

- Stud connectors shall conform to one of the following:
  - AASHTO M169 (ASTM A108) $F_u = 50$ksi or $60$ksi

AISC Now Lists $F_u = 65$ksi for ASTM A108 Shear Studs
§6.4 - Materials

§6.4.5: Weld Metal

- Refers to AWS D1.5 - Bridge Welding Code

§6.5 - Limit States

- 6.5.1 General
- 6.5.2 Service Limit State
- 6.5.3 Fatigue and Fracture Limit State
- 6.5.4 Strength Limit State
- 6.5.5 Extreme Event Limit State

§6.5.1: General

- Structural behavior of steel components shall be investigated for each stage that may be critical during Construction, Handling, Transportation, and Erection as well as during the Service life of the structure.

- Structural components shall be proportioned to satisfy requirements at Service, Strength, Extreme Event, and Fatigue and Fracture Limit States.
§6.5 - Limit States

§6.5.2: Service Limit State

- Covers Elastic Deformations

- For flexural members (§6.10 and §6.11), provides limits to prevent permanent deformations due to localized yielding.

§6.5.3: Fatigue and Fracture Limit State

- Components and details shall be investigated for Fatigue as specified in §6.6 for the combinations and loads specified in §3.4.1 and §3.6.1.4, respectively.

- Flexural members shall be investigated as specified in §6.10 and §6.11. Special fatigue requirements for thin webs and shear connectors.

- Bolts subject to tensile fatigue shall be investigated as specified in §6.13.2.10.3.

- Fracture toughness requirements shall be in conformance with §6.6.2.
§6.5 - Limit States

§6.5.4: Strength Limit State

- Strength and Stability shall be considered using the applicable load combinations in Table 3.4.1-1
- The Design Resistance, \( R_{dr} \), shall be taken as \( \varphi R_n \).
- Resistance Factors
  - Gross-Section Yielding, \( \varphi_y = 0.95 \)
  - Net-Section Fracture, \( \varphi_u = 0.80 \)
  - Axial Compression, \( \varphi_c = 0.90 \)
  - Flexure, \( \varphi_f = 1.00 \)
  - Shear, \( \varphi_s = 1.00 \)
  - A325 & A490 Bolt Tension, Shear, and Bearing, \( \varphi_t = \varphi_s = \varphi_{bb} = 0.80 \)

§6.5 - Limit States

§6.5.5: Extreme Event Limit State

- All applicable extreme event load combinations in Table 3.4.1-1 shall be investigated.
- All resistance factors for the extreme event limit state, except for bolts, shall be taken as 1.00
- Bolted joints not protected by capacity design or structural fuses may be assumed to behave as bearing-type connections at the extreme event limit states.
§6.7 - General Dimension and Detail Requirements

- 6.7.1 Effective Length of Spans
- 6.7.2 Dead Load Camber
- 6.7.3 Minimum Thickness of Steel
- 6.7.4 Diaphragms and Cross Frames
- 6.7.5 Lateral Bracing
- 6.7.6 Pins

Span lengths shall be taken as the distance between centers of bearings or other points of support.

Effective span lengths may be different for effective width and DF calcs.
§6.7 - General Dimension and Detail Requirements

§6.7.2: Dead Load Camber

Steel structures should be cambered during fabrication to compensate for dead load deflection and vertical alignment.

- Deflection due to steel weight and concrete weight shall be reported separately.
- Deflections due to future wearing surfaces or other loads not applied at the time of construction shall be reported separately.
- Vertical camber shall be specified to account for the computed dead load deflection.
- If staged construction is specified, the sequence of load application should be recognized in determining the camber and stresses.

§6.7 - General Dimension and Detail Requirements

§6.7.3: Minimum Thickness of Steel

- Structural steel, including bracing, cross-frames, and all types of gusset plates, except for webs of rolled shapes, closed ribs in orthotropic decks, fillers, and in railings, shall be not less than $\frac{5}{16}$" in thickness.
- The web thickness of rolled beams or channels and of closed ribs in orthotropic decks shall not be less than $\frac{1}{4}$" in thickness.
§6.7 - General Dimension and Detail Requirements

§6.7.4: Diaphragms and Cross-Frames

- Diaphragms or cross frames may be placed at the ends of the structure, across interior supports, and intermittently along the span to:
  - transfer lateral wind loads from the bottom flange of a girder to the deck and from the deck to the bearings,
  - provide stability to the bottom flange for all loads when it is in compression,
  - provide stability to the top flange in compression prior to curing of the deck,
  - aid in distributing lateral flange bending effects, and
  - aid in transverse distribution of vertical loads applied to the structure.

---

§6.7 - General Dimension and Detail Requirements

§6.7.4: Diaphragms and Cross-Frames

- Diaphragms or cross-frames for rolled beams and plate girders should be as deep as practicable
  - As a minimum, they should be at least:
  - 1/2 of the beam depth for rolled beams
  - 3/4 of the girder depth for plate girders
§6.7 - General Dimension and Detail Requirements

BDM §302.4.2.3: Intermediate Cross-Frames

- Skewed crossframes at intermediate support points should be avoided.

- Crossframes shall be oriented perpendicular to the main steel members regardless of the structure’s skew angle.

- Cross frames shall be perpendicular to stringers and be in line across the total width of the structure.

- Cross frame spacings between points of dead load contraflexure in the positive moment regions shall not exceed 25 ft.

- Cross frame spacings between points of dead load contraflexure in the negative moment regions shall not exceed 15 ft.
§6.6 - Fatigue and Fracture Considerations

- 6.6.1 Fatigue
- 6.6.2 Fracture
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

Each fatigue detail shall satisfy,

\[ \gamma (\Delta f) \leq (\Delta F)^n \]  

(6.6.1.2.1)

where,

\[ \gamma \] - load factor specified in Table 3.4.1-1 for fatigue (\( \gamma_{\text{fatigue}} = 0.75 \))

\[ (\Delta f) \] - live load stress range due to the passage of the fatigue load specified in §3.6.1.4

\( \eta \) and \( \phi \) are taken as 1.00 for the fatigue limit state

The live-load stress due to the passage of the fatigue load is approximately one-half that of the heaviest truck expected in 75 years.

§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- The force effect considered for the fatigue design of a steel bridge detail shall be the live load stress range.

- For flexural members with shear connectors provided throughout their entire length, and with concrete deck reinforcement satisfying the provisions of Article 6.10.1.7 (Minimum Negative Flexure Deck Reinforcement), live load stresses and stress ranges for fatigue design may be computed using the short-term composite section assuming the concrete deck to be effective for both positive and negative flexure.

- Residual stresses shall not be considered in investigating fatigue.
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- These provisions shall be applied only to details subjected to a net applied tensile stress.

- In regions where the unfactored permanent loads produce compression, fatigue shall be considered only if the compressive stress is less than twice the maximum tensile live load stress resulting from the fatigue load combination.

i.e., where: \( f_{\text{comp. DL}} \leq 2f_{\text{fat load, tension}} \)

---

§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- This is based on the typical S-N diagram:
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

\[
(\Delta F)_a = \left(\frac{A}{N}\right) \left(\frac{(\Delta F)_{TH}}{2}\right)
\]

- \(A\) - Fatigue Detail Category Constant - Table 6.6.1.2.5-1
- \(N = (365)(75)n(ADTT)_{SL}\) (75 Year Design Life) - Table 6.6.1.2.5-2
- \(n\) - # of stress ranges per truck passage - Table 6.6.1.2.5-2
- \((ADTT)_{SL}\) - Single-Lane ADTT from §3.6.1.4
- \((\Delta F)_{TH}\) - Constant amplitude fatigue threshold - Table 6.6.1.2.5-3

**ODOT is planning to simply design for infinite life on Interstate Structures**

---

§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

**Tables 6.6.1.2.5-1&3 Fatigue Constant and Threshold Stress Range**

<table>
<thead>
<tr>
<th>Detail Category</th>
<th>(A \times 10^3) (ksi³)</th>
<th>((\Delta F)_{TH}) (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>250</td>
<td>24.0</td>
</tr>
<tr>
<td>B</td>
<td>120</td>
<td>16.0</td>
</tr>
<tr>
<td>B'</td>
<td>61.0</td>
<td>12.0</td>
</tr>
<tr>
<td>C</td>
<td>44.0</td>
<td>10.0</td>
</tr>
<tr>
<td>C'</td>
<td>44.0</td>
<td>12.0</td>
</tr>
<tr>
<td>D</td>
<td>22.0</td>
<td>7.0</td>
</tr>
<tr>
<td>E</td>
<td>11.0</td>
<td>4.5</td>
</tr>
<tr>
<td>E'</td>
<td>3.9</td>
<td>2.6</td>
</tr>
<tr>
<td>M164 Bolts</td>
<td>17.1</td>
<td>31.0</td>
</tr>
<tr>
<td>M253 Bolts</td>
<td>31.5</td>
<td>38.0</td>
</tr>
</tbody>
</table>

**More about fatigue categories in a minute...**
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

Table C6.6.1.2.5-1 75-Year (ADTT)\textsubscript{SL} Equivalent to Infinite Life

<table>
<thead>
<tr>
<th>Detail Category</th>
<th>75-Year (ADTT)\textsubscript{SL} Equivalent to Infinite Life (Trucks / Day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>535</td>
</tr>
<tr>
<td>B</td>
<td>865</td>
</tr>
<tr>
<td>B'</td>
<td>1035</td>
</tr>
<tr>
<td>C</td>
<td>1290</td>
</tr>
<tr>
<td>C'</td>
<td>745</td>
</tr>
<tr>
<td>D</td>
<td>1675</td>
</tr>
<tr>
<td>E</td>
<td>3545</td>
</tr>
<tr>
<td>E'</td>
<td>6525</td>
</tr>
</tbody>
</table>

This Table shows the values of (ADTT)\textsubscript{SL} above which the Infinite Life check governs (Assuming one cycle per truck passage).

Fatigue details located within L/10 of a support are considered to be “near” the support.
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- In the absence of better information,

\[(ADTT)_{SL} = p \cdot ADTT\]  \hspace{1cm} \text{(3.6.1.4.2-1)}

where,

- \(p\) - The fraction of truck traffic in a single lane

**Table 3.6.1.4.2-1 Single Lane Truck Fraction**

<table>
<thead>
<tr>
<th># Lanes Available to Trucks</th>
<th>(p)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.00</td>
</tr>
<tr>
<td>2</td>
<td>0.85</td>
</tr>
<tr>
<td>3 or more</td>
<td>0.80</td>
</tr>
</tbody>
</table>

*Must consider the number of lanes available to trucks in each direction!*

§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- In the absence of better information,

\[ADTT = (TF) \cdot ADT\]

where,

- \(TF\) - The fraction trucks in the average daily traffic

**Table C3.6.1.4.2-1 ADT Truck Fraction**

<table>
<thead>
<tr>
<th>Class of Highway</th>
<th>(TF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural Interstate</td>
<td>0.20</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>

*ODOT is suggesting that the \(ADTT\) be taken as 4 x 20-year-avg \(ADT\)*
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2.3: Fatigue Detail Categories

<table>
<thead>
<tr>
<th>GENERAL CONSIDERATION</th>
<th>SITUATION</th>
<th>DETAIL CATEGORY</th>
<th>ILLUSTRATIVE EXAMPLE SEE [PAGES 6.35-6.37, 6.41]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate Members</td>
<td>Base metal:</td>
<td>A, 2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• With rolled or channel surfaces; flange- and edge- welded members; 6.0 ksi or low flow</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Of equivalent welding metal, all grades, designed and detailed in accordance with AWS A7.060</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• At section of cyster heads and gus sets</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Boltup Members</td>
<td>Base metal and weld metal in components without attachments, connected by:</td>
<td>B, B'</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Continuous full penetration groove welds with backing bars removed, or</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Continuous full penetration groove welds with backing bars parallel to the direction of applied stress</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Continuous full penetration groove welds with backing bars in place, or</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Continuous partial penetration groove welds parallel to the direction of applied stress</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Base metal at ends of partial length cover plate:</td>
<td>B, B'</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• With bolted slip critical end connections</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3, 4, 5, 7</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

-- 51 --
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2.3: Fatigue Detail Categories

| Longitudinally | Diagonal | Transversely | Externally |ave Metal | weld metal or full-penetration groove-welded splice type.
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Groove-Welded</td>
<td>3 Curves</td>
<td>Groove-Welded</td>
<td>3 Curves</td>
<td>B, C, F</td>
<td>8, 10, 11, 12, 13, 15, 16</td>
</tr>
<tr>
<td>Connections w/ Weld</td>
<td></td>
<td>Connections w/ Weld</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Curves Established by NPT and All Required Grooving to the Dimensions of Plate</td>
<td></td>
<td>Curves Established by NPT and All Required Grooving to the Dimensions of Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fatigue Detail Categories</td>
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<td>Fatigue Detail Categories</td>
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<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base metal or weld metal or full-penetration groove-welded splice type.</td>
<td></td>
<td>Base metal or weld metal or full-penetration groove-welded splice type.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With transitions in width or thickness with welds ground flush</td>
<td></td>
<td>With transitions in width or thickness with welds ground flush</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With or without transitions having slope no greater than 1:2.5 when weld reinforcement is not removed</td>
<td></td>
<td>With or without transitions having slope no greater than 1:2.5 when weld reinforcement is not removed</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>When the detail length in the direction of applied stress is:</td>
<td></td>
<td>When the detail length in the direction of applied stress is:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>less than 2.0 m.</td>
<td>less than 2.0 m.</td>
<td>less than 2.0 m.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>between 2.0 m. and 12 times the detail thickness, but less than 4.0 m.</td>
<td>between 2.0 m. and 12 times the detail thickness, but less than 4.0 m.</td>
<td>between 2.0 m. and 12 times the detail thickness, but less than 4.0 m.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>greater than either 12 times the detail thickness or 4.0 m.</td>
<td>greater than either 12 times the detail thickness or 4.0 m.</td>
<td>greater than either 12 times the detail thickness or 4.0 m.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With a transition radius with the end welds ground smooth, regardless of detail length:</td>
<td></td>
<td>With a transition radius with the end welds ground smooth, regardless of detail length:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>transition radius ≥ 240 mm.</td>
<td>transition radius ≥ 240 mm.</td>
<td>transition radius ≥ 240 mm.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60 mm. ≤ transition radius &lt; 240 mm.</td>
<td>60 mm. ≤ transition radius &lt; 240 mm.</td>
<td>60 mm. ≤ transition radius &lt; 240 mm.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ODOT Short Course Created July 2007</td>
<td>ODOT Short Course Created July 2007</td>
<td>ODOT Short Course Created July 2007</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fatigue and Fracture: Slide #15</td>
<td>Fatigue and Fracture: Slide #16</td>
<td>Fatigue and Fracture: Slide #16</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### §6.6 - Fatigue and Fracture Considerations

#### §6.6.1.2.3: Fatigue Detail Categories

<table>
<thead>
<tr>
<th>Transverse Welded Connection with Welds Normal to the Direction of Stress</th>
<th>Base metal at detail attached by fillet welds with a transition radius:</th>
<th>16</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>• With equal plate thickness and weld reinforcement removed:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>o transition radius ≥ 20 in.</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>o 20 in. &lt; transition radius ≥ 6.0 in.</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>o transition radius &lt; 6.0 in.</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>• With equal plate thickness and weld reinforcement removed:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>o transition radius ≥ 20 in.</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>o 20 in. &lt; transition radius ≥ 6.0 in.</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>o transition radius &lt; 6.0 in.</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>• With unequal plate thickness and weld reinforcement removed:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>o transition radius ≥ 20 in.</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>o transition radius &lt; 20 in.</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>• For any transition radius with equal plate thickness and weld reinforcement not removed:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>o transition radius ≥ 20 in.</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>o transition radius &lt; 20 in.</td>
<td>E</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fillet Welded Connections with Welds Normal to the Direction of Stress</th>
<th>Base metal:</th>
<th>14</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>• At details other than transverse stiflenees to-flange or transverse stfflenees to web connections:</td>
<td>Lower of C or Eq. 6.6.1.2.3.3</td>
</tr>
<tr>
<td></td>
<td>• At the toe of transverse stfflenees to-flange and transverse stfflenees to web welds:</td>
<td>C'</td>
</tr>
<tr>
<td></td>
<td>Base metal at end of weld</td>
<td>9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fillet Welded Connections with Welds Normal and/or Parallel to the Direction of Stress</th>
<th>Base metal at end of weld</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>• At details other than transverse stiflness to-flange or transverse stfflness to web connections:</td>
<td>Lower of C or Eq. 6.6.1.2.3.3</td>
</tr>
<tr>
<td></td>
<td>• At the toe of transverse stfflness to-flange and transverse stfflness to web welds:</td>
<td>C'</td>
</tr>
<tr>
<td></td>
<td>Base metal at end of weld</td>
<td>9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Longitudinally Loaded Fill Welded Attachment</th>
<th>Base metal at details attached by fillet welds:</th>
<th>16</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>• Where the detail length in the direction of applied stress is:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>o less than 2.0 in. or end detail connection:</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>o between 2.0 in. and 12 times the detail thickness, for less than 4.0 in.</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>o greater than either 12 times the detail thickness or 4.0 in.</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>o detail thickness ≤ 1.0 in.</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>• With a transition radius with the end welds ground smooth, regardless of detail length:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>o transition radius ≥ 20 in.</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>o transition radius &lt; 20 in.</td>
<td>E</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Transverse Loaded Fill Welded Attachment</th>
<th>Base metal at details attached by fillet welds:</th>
<th>16</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>• With a transition radius with end welds not ground smooth:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>o transition radius ≥ 20 in.</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>o transition radius &lt; 20 in.</td>
<td>E</td>
</tr>
</tbody>
</table>
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2.3: Fatigue Detail Categories

<table>
<thead>
<tr>
<th>Longitudinally Loaded Fillet-Welded Attachments</th>
<th>Base metal at detail attached by fillet welds:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Where the weld length is in the direction of applied stress:</td>
</tr>
<tr>
<td></td>
<td>less than 2.0 in., or stub-type shear connections:</td>
</tr>
<tr>
<td></td>
<td>less than 2.0 in., or stub-type shear connections:</td>
</tr>
<tr>
<td></td>
<td>between 2.0 in. and 12 times the detail thickness, but less than 4.0 in.:</td>
</tr>
<tr>
<td></td>
<td>greater than either 12 times the detail thickness of 4.0 in.:</td>
</tr>
<tr>
<td></td>
<td>3 times the detail thickness of 1.0 in.</td>
</tr>
<tr>
<td></td>
<td>3 times the detail thickness of 1.0 in.</td>
</tr>
<tr>
<td></td>
<td>With a transition radius with end welds ground smooth, regardless of detail length:</td>
</tr>
<tr>
<td></td>
<td>transition radius &gt; 2.0 in.:</td>
</tr>
<tr>
<td></td>
<td>transition radius ≤ 2.0 in.:</td>
</tr>
<tr>
<td></td>
<td>With a transition radius with end welds ground smooth:</td>
</tr>
<tr>
<td></td>
<td>transition radius &gt; 2.0 in.:</td>
</tr>
<tr>
<td></td>
<td>transition radius ≤ 2.0 in.:</td>
</tr>
<tr>
<td></td>
<td>With a transition radius with end welds not ground smooth:</td>
</tr>
<tr>
<td></td>
<td>transition radius &gt; 2.0 in.:</td>
</tr>
<tr>
<td></td>
<td>transition radius ≤ 2.0 in.:</td>
</tr>
<tr>
<td></td>
<td>16</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Transversely Loaded Fillet-Welded Attachments with Welds Parallel to the Direction of Primary Stress</th>
<th>Base metal at detail attached by fillet welds:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>With a transition radius with end welds ground smooth:</td>
</tr>
<tr>
<td></td>
<td>transition radius &gt; 2.0 in.:</td>
</tr>
<tr>
<td></td>
<td>transition radius ≤ 2.0 in.:</td>
</tr>
<tr>
<td></td>
<td>With a transition radius with end welds not ground smooth:</td>
</tr>
<tr>
<td></td>
<td>transition radius &gt; 2.0 in.:</td>
</tr>
<tr>
<td></td>
<td>transition radius ≤ 2.0 in.:</td>
</tr>
<tr>
<td></td>
<td>16</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mechanically Fastened Connections</th>
<th>Base metal:</th>
</tr>
</thead>
<tbody>
<tr>
<td>At gross section of high-strength bolted connections, except maidly loaded joints in which use of plate gusset is intended to connect transverse</td>
<td></td>
</tr>
<tr>
<td>At net section of high-strength bolted transverse connections</td>
<td></td>
</tr>
<tr>
<td>At net section of gussetted connections</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Yoke or Pin Plates</th>
<th>Base metal at the shank of oxy-arc, or through the gross section of pin plates with:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rolled or smoothly ground surfaces.</td>
</tr>
<tr>
<td></td>
<td>Flange cut-off pin.</td>
</tr>
<tr>
<td></td>
<td>E</td>
</tr>
</tbody>
</table>

Pgs 6.35-6.37, 6.41  AASHTO-LRFD 2007  ODOT Short Course  Created July 2007  Fatigue and Fracture: Slide #20
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2.4: Fatigue Detail Categories

- Transversely loaded partial-pen groove welds shall not be used except in some metal deck details.

- Gusset plates attached to girder flanges with only transverse fillet welds shall not be used.

§6.6 - Fatigue and Fracture Considerations

§6.6.2: Fracture

- The appropriate temperature zone shall be determined from Table 6.6.2-1

- Fracture toughness requirements shall be in conformance with Table 6.6.2-2

Table 6.6.2-1 Temperature Zone Designations

<table>
<thead>
<tr>
<th>Min Service Temperature</th>
<th>Temperature Zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>0°F and above</td>
<td>1</td>
</tr>
<tr>
<td>-1°F to -30°F</td>
<td>2</td>
</tr>
<tr>
<td>-31°F to -60°F</td>
<td>3</td>
</tr>
</tbody>
</table>

← ODOT Designs
§6.6 - Fatigue and Fracture Considerations

§6.6.2: Fracture

- Except as specified herein, all primary longitudinal superstructure components and connections sustaining tensile force effects due to Strength Load Combination I, and transverse floorbeams subject to such effects, shall require mandatory Charpy V-notch fracture toughness.

- Other primary components and connections sustaining tensile force effects due to the Strength Load Combination I may require mandatory Charpy V-notch fracture toughness at the discretion of the Owner.

- All components and connections requiring Charpy V-notch fracture toughness shall be so designated on the contract plans.

- Unless otherwise indicated on the contract plans, Charpy V-notch fracture toughness requirements shall not be considered mandatory for the following items:
  - Splice plates and filler plates in bolted splices
  - Intermediate transverse web stiffeners not serving as connection plates
  - Bearings, sole plates, and masonry plates
  - Expansion dams
  - Drainage material
§6.6 - Fatigue and Fracture Considerations

§6.6.2: Fracture Critical Members

- Fracture Critical Member (FCM) - Component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function.

- Unless a rigorous analysis with assumed hypothetical cracked components confirms the strength and stability of the hypothetically damaged structure, the location of all FCMs shall be clearly delineated on the contract plans.

*FCMs are subject to more stringent toughness requirements than non-FCMs*

---

BDM §302.4.3.2: Fracture Critical Members

- The designer should make all efforts to not develop a structure design that requires fracture critical members. As specified in Section 301.2, structures with fracture critical details require a concurrent detail design review to be performed by the Office of Structural Engineering.

- If a girder is non-redundant, include the entire girder in the pay quantity for Item 513 - Structural Steel Members, Level 6. The designer shall designate the tension and compression zones in the fracture critical members.

  This basically means that you have to have a “top-of-the-line fabricator…”
### §6.6 - Fatigue and Fracture Considerations

#### §6.6.2: Fracture

**Table 6.6.2-2 Fracture Toughness Requirements**

<table>
<thead>
<tr>
<th>Grade</th>
<th>Thickness</th>
<th>Min Test Energy</th>
<th>Temperature Zone 1</th>
<th>Temperature Zone 2</th>
<th>Temperature Zone 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(in)</td>
<td>(ft-lbs) @ °F</td>
<td>(ft-lbs @ °F)</td>
<td>(ft-lbs @ °F)</td>
<td>(ft-lbs @ °F)</td>
</tr>
<tr>
<td>36</td>
<td>≤ 4</td>
<td>20</td>
<td>25 @ 70</td>
<td>25 @ 40</td>
<td>25 @ 10</td>
</tr>
<tr>
<td>50/50S/50W</td>
<td>≤ 2</td>
<td>20</td>
<td>25 @ 70</td>
<td>25 @ 40</td>
<td>25 @ 10</td>
</tr>
<tr>
<td>2&lt; t ≤ 4</td>
<td>24</td>
<td>30 @ 70</td>
<td>30 @ 40</td>
<td>30 @ 10</td>
<td>30 @ 10</td>
</tr>
<tr>
<td>HPS 50W</td>
<td>t ≤ 4</td>
<td>28</td>
<td>35 @ -10</td>
<td>35 @ -10</td>
<td>35 @ -10</td>
</tr>
<tr>
<td>HPS 70W</td>
<td>t ≤ 4</td>
<td>28</td>
<td>35 @ 30</td>
<td>35 @ 0</td>
<td>35 @ -30</td>
</tr>
<tr>
<td>100/100W</td>
<td>t ≤ 2.5</td>
<td>28</td>
<td>35 @ 30</td>
<td>35 @ 0</td>
<td>35 @ -30</td>
</tr>
<tr>
<td>2.5 &lt; t ≤ 4</td>
<td>36</td>
<td>45 @ 30</td>
<td>45 @ 0</td>
<td>Not Permitted</td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>≤ 4</td>
<td>20</td>
<td>25 @ 70</td>
<td>25 @ 40</td>
<td>25 @ 10</td>
</tr>
<tr>
<td>50/50S/50W</td>
<td>≤ 4</td>
<td>20</td>
<td>25 @ 70</td>
<td>25 @ 40</td>
<td>25 @ 10</td>
</tr>
<tr>
<td>HPS 50W</td>
<td>t ≤ 4</td>
<td>24</td>
<td>30 @ 10</td>
<td>30 @ 10</td>
<td>30 @ 10</td>
</tr>
<tr>
<td>HPS 70W</td>
<td>t ≤ 4</td>
<td>28</td>
<td>35 @ -10</td>
<td>35 @ -10</td>
<td>35 @ -10</td>
</tr>
<tr>
<td>100/100W</td>
<td>t ≤ 4</td>
<td>28</td>
<td>35 @ 30</td>
<td>35 @ 0</td>
<td>35 @ -30</td>
</tr>
</tbody>
</table>

---

### §6.6 - Fatigue and Fracture Considerations

#### §6.6.2: Fracture

**Table 6.6.2-2 Fracture Toughness Requirements**

<table>
<thead>
<tr>
<th>Grade</th>
<th>Thickness</th>
<th>Min Test Energy</th>
<th>Temperature Zone 1</th>
<th>Temperature Zone 2</th>
<th>Temperature Zone 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(in)</td>
<td>(ft-lbs) @ °F</td>
<td>(ft-lbs @ °F)</td>
<td>(ft-lbs @ °F)</td>
<td>(ft-lbs @ °F)</td>
</tr>
<tr>
<td>36</td>
<td>≤ 4</td>
<td>20</td>
<td>25 @ 70</td>
<td>25 @ 40</td>
<td>25 @ 10</td>
</tr>
<tr>
<td>50/50S/50W</td>
<td>≤ 2</td>
<td>20</td>
<td>25 @ 70</td>
<td>25 @ 40</td>
<td>25 @ 10</td>
</tr>
<tr>
<td>2&lt; t ≤ 4</td>
<td>24</td>
<td>30 @ 70</td>
<td>30 @ 40</td>
<td>30 @ 10</td>
<td>30 @ 10</td>
</tr>
<tr>
<td>HPS 50W</td>
<td>t ≤ 4</td>
<td>28</td>
<td>35 @ -10</td>
<td>35 @ -10</td>
<td>35 @ -10</td>
</tr>
<tr>
<td>HPS 70W</td>
<td>t ≤ 4</td>
<td>28</td>
<td>35 @ 30</td>
<td>35 @ 0</td>
<td>35 @ -30</td>
</tr>
<tr>
<td>100/100W</td>
<td>t ≤ 2.5</td>
<td>28</td>
<td>35 @ 30</td>
<td>35 @ 0</td>
<td>35 @ -30</td>
</tr>
<tr>
<td>2.5 &lt; t ≤ 4</td>
<td>36</td>
<td>45 @ 30</td>
<td>45 @ 0</td>
<td>Not Permitted</td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>≤ 4</td>
<td>20</td>
<td>25 @ 70</td>
<td>25 @ 40</td>
<td>25 @ 10</td>
</tr>
<tr>
<td>50/50S/50W</td>
<td>≤ 4</td>
<td>20</td>
<td>25 @ 70</td>
<td>25 @ 40</td>
<td>25 @ 10</td>
</tr>
<tr>
<td>HPS 50W</td>
<td>t ≤ 4</td>
<td>24</td>
<td>30 @ 10</td>
<td>30 @ 10</td>
<td>30 @ 10</td>
</tr>
<tr>
<td>HPS 70W</td>
<td>t ≤ 4</td>
<td>28</td>
<td>35 @ -10</td>
<td>35 @ -10</td>
<td>35 @ -10</td>
</tr>
<tr>
<td>100/100W</td>
<td>t ≤ 4</td>
<td>28</td>
<td>35 @ 30</td>
<td>35 @ 0</td>
<td>35 @ -30</td>
</tr>
</tbody>
</table>

---
AASHTO-LRFD
Chapter 6: Tension Members

James A Swanson

§6.8 - Tension Members

- 6.8.1 General
- 6.8.2 Tensile Resistance
- 6.8.3 Net Area
- 6.8.4 Limiting Slenderness Ratio
- 6.8.5 Built-Up Members
- 6.8.6 Eyebars
- 6.8.7 Pin-Connected Members

§6.8.1: General
Members and splices subjected to axial tension shall be investigated for:

- Gross Section yielding
- Net Section Fracture
§6.8 - Tension Members

§6.8.2: Tensile Resistance

- **Gross Section Yielding:**
  \[ P_r = \phi_y P_{my} = \phi_y F_y A_g \]  
  \[ \phi_y = 0.95 \]

  \( F_y \) - Specified minimum yield strength.
  \( A_g \) - Gross Cross-sectional area of the member.

Yielding of the member in the gross section is considered a limit state because it could lead to excessive elongation of the member that could compromise the stability or safety of the structure.

- **Net Section Fracture:**
  \[ P_n = \phi_u P_{mu} = \phi_u F_u A_n U \]  
  \[ \phi_u = 0.80 \]

  \( F_u \) - Specified minimum tensile strength.
  \( A_n \) - Net area of the member.
  \( U \) - Shear lag reduction coefficient.

Rupture of the member at the net section is considered a limit state because the member would no longer be able to carry load.
§6.8 - Tension Members

§6.8.2: Tensile Resistance

Gross Section

Net Section

---

§6.8 - Tension Members

§6.8.2: Tensile Resistance

Gross Section

Net Section
§6.8 - Tension Members

§6.8.2: Tensile Resistance

- Net Section Fracture: Elastic Stress Concentrations

Yielding is not checked on the net section because it will be localized and will not lead to excessive elongation of the member.
§6.8 - Tension Members

§6.8.3: Net Area

- **Effective Hole Diameter**
  - For Standard Holes,
    \[
    d_{\text{eff}} = d_{\text{bolt}} - \frac{1}{64} \cdot \frac{d_{\text{bolt}}}{16}
    \]
  - Std holes are \(\frac{1}{8}\)" larger than the bolt
  - \(\frac{1}{64}\)" damage during fabrication

- **ODOT CMS Spec 513.19**
  - Holes in primary members cannot be punched full-size

- **Staggered Fasteners**
  - For Each Diagonal Segment, add
    \[
    \frac{s^2}{4g}
    \]

§6.8.2.2: Shear Lag Reduction

- \(U = 1.00\) when the tension load is transmitted directly to each of the cross sectional elements within the cross section.
- \(U = 0.90\) for rolled I-shapes and tees cut from I-shapes where the flange width is not less than 2/3 the depth when no fewer than 3 fasteners are used in the direction of stress.
- \(U = 0.85\) for all other members having no fewer than 3 fasteners in the direction of stress.
- \(U = 0.75\) for all members having only 2 fasteners in the direction of stress.

_When a tension load is transmitted by fillet welds to some but not all elements of a cross section, the weld strength shall control._
§6.8 - Tension Members

§6.8.2.2: Shear Lag Reduction - Commentary

- The provisions of Article 6.8.2.2 are adapted from the commentary to the 1999 AISC LRFD Specification, Article B3, Effective Net Area for Tension Members.

- Similar simple provisions appear in previous issues of the AISC LRFD Specification prior to 1993, but were replaced in the 1993 edition by a more precise equation for shear-lag effects, Equation B3-3.

- The 1999 AISC LRFD Commentary suggests that the complication and preciseness of Equation B3-3 is not warranted for design.

The AISC provisions are now found in Article D3.3 of the 2005 13th Ed.

Table D3.1

<table>
<thead>
<tr>
<th>Case</th>
<th>Description of Element</th>
<th>Shear Lag Factor, U</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except cases 3, 4, 5 and 6.)</td>
<td>$U = 1.00$</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds. (alternatively, for W, M, S, and HP case 7 may be used.)</td>
<td>$U = 1 - \frac{L}{W}$</td>
<td>[Diagram]</td>
</tr>
<tr>
<td>3</td>
<td>All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sections elements.</td>
<td>$U = 1.00$ and $A_s = area$ of the directly connected elements</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Plates where the tension load is transmitted by longitudinal welds only.</td>
<td>$L \geq 2.0W \ldots U = 1.00$</td>
<td>$2.0W &gt; L \geq 1.5W \ldots U = 0.87$ and $1.5W &gt; L \geq 1.0W \ldots U = 0.75$</td>
</tr>
</tbody>
</table>

‡ Most General Case

‡ For Welded Plates
§6.8 - Tension Members

§6.8.2.2: Shear Lag Reduction - AISC Provisions

(a) T

(b) T

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AISC Pg 16.1-251

AISC Pg 16.1-251
AASHTO-LRFD 2007

Tension Members: Slide #13

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§6.8 - Tension Members

§6.8.2: Tensile Resistance

Net Section

Less than 100% Effective

Effective Net Section

For Main Members Subject to Stress Reversals

\[
\frac{L}{r_{\text{min}}} \leq 140
\]

For Main Members Not Subject to Stress Reversals

\[
\frac{L}{r_{\text{min}}} \leq 200
\]

For Bracing Members

\[
\frac{L}{r_{\text{min}}} \leq 240
\]
§6.9 - Compression Members

- 6.9.1 General
- 6.9.2 Compressive Resistance
- 6.9.3 Limiting Slenderness Ratio
- 6.9.4 Noncomposite Members
- 6.9.5 Composite Members

§6.9.1: General

The provisions of this Article shall apply to prismatic noncomposite and composite steel members with at least one plane of symmetry and subjected to either axial compression or combined axial compression and flexure about an axis of symmetry.

“Torsional buckling or flexural-torsional buckling of singly symmetric and unsymmetric compression members and doubly-symmetric compression members with very thin walls should be investigated.”

(Covered Later)
§6.9 - Compression Members

Theoretical Basis of Compression Provisions

Axial Capacity, $P_n$

$P_y = F_y A_i$

Crushing Load

Elastic Buckling

$P_e = \frac{\pi^2 E A}{(KL/r)^2}$

Slenderness, $\lambda$

---

$\lambda = 2.25$

---

§6.9 - Compression Members

Theoretical Basis of Compression Provisions

Axial Capacity, $P_n$

Crushing Load

Inelastic Buckling (Residual Stresses)

Elastic Buckling

Slenderness, $\lambda$

---

$\lambda = 2.25$
§6.9 - Compression Members

Theoretical Basis of Compression Provisions

Axial Capacity, $P_n$

Residual Stresses and Initial Out-of-Straightness

Slenderness, $\lambda$

§6.9.2: Compressive Resistance

$P_p = \phi_c P_n$  \hspace{1cm} (6.9.2.1-1)

$\phi_c = 0.90$
§6.9 - Compression Members

§6.9.3: Limiting Slenderness Ratio

Compression Members shall satisfy the following slenderness limits:

- For Main Members

\[ \frac{KL}{r} \leq 120 \]

- For Bracing Members

\[ \frac{KL}{r} \leq 140 \]

§6.9.4.1: Noncomposite Compressive Strength

- If \( \lambda \leq 2.25 \), (Inelastic Flexural Buckling)

\[ P_n = 0.66 F_y A_s \]  \hspace{1cm} (6.9.4.1-1)

- If \( \lambda > 2.25 \), (Elastic Flexural Buckling)

\[ P_e = \frac{0.88 F_y A_s}{\lambda} \]  \hspace{1cm} (6.9.4.1-2)

where,

\[ \lambda = \left( \frac{KL}{r \pi} \right)^\frac{1}{2} \frac{F_y}{E} \]  \hspace{1cm} (6.9.4.1-3)

Refer to AISC for torsional and flexural-torsional buckling.
§6.9 - Compression Members

§6.9.4.2: Local Buckling Limits

- For Most Cases, the Plate Slendernesses Shall Satisfy,
  \[
  \frac{b}{t} \leq k \sqrt{\frac{F_y}{E}} \quad (6.9.4.2-1)
  \]

- Since yielding is an upper bound on the flexural-buckling strength, this check, which is based on the critical stress of plates, is used to ensure that the section will fail by flexural buckling prior to the components buckling locally.

- \( F_y \) in the equations used to check for local buckling may be replaced by the maximum computed compressive stress due to the factored loads and concurrent bending moments.

Table 6.9.4.2-1 Plate Buckling Coefficients and Widths for Axial Compression

<table>
<thead>
<tr>
<th>Plates Supported Along One Edge</th>
<th>( k )</th>
<th>( b )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flanges and Projecting Legs or Plates</td>
<td>0.56</td>
<td>• Half-flange width of rolled I-sections</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Full-flange width of channels</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Distance between free edge and first line of bolts or welds in plate</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Full width of an outstanding leg for pairs of angles in continuous contact</td>
</tr>
<tr>
<td>Stems of Rolled Tees</td>
<td>0.75</td>
<td>• Full depth of tee</td>
</tr>
<tr>
<td>Other Projecting Elements</td>
<td>0.45</td>
<td>• Full width of outstanding leg for single angle strut or double angle strut with spacers</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Full projecting width for others</td>
</tr>
<tr>
<td>Plates Supported Along Two Edges</td>
<td>( k )</td>
<td>( b )</td>
</tr>
<tr>
<td>Box Flanges and Cover Plates</td>
<td>1.40</td>
<td>• Clear distance between webs minus inside corner radius on each side for box flanges</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Distance between lines of webs or bolts for flange cover plates</td>
</tr>
<tr>
<td>Webs and Other Plate Elements</td>
<td>1.49</td>
<td>• Clear distance between flanges minus fillet radius for webs of rolled beams</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Clear distance between edge supports for all others</td>
</tr>
<tr>
<td>Perfkoated Cover Plates</td>
<td>1.36</td>
<td>• Clear distance between edge supports</td>
</tr>
</tbody>
</table>
§6.9 - Compression Members

§6.9.4.2: Local Buckling Limits

- For Built-Up I-Sections, the Following Shall be Satisfied,

\[ \frac{b}{t} = \frac{b_f}{2t_f} \leq 0.64 \sqrt[4]{\frac{k_c E}{F_y}} \]  \hspace{1cm} (6.9.4.2-2)

where,

\[ 0.35 \leq k_c \leq 0.76 \]  \hspace{1cm} (6.9.4.2-3)

and

\[ k_c = \frac{4}{D}\sqrt{\frac{F_y}{t_w}} \]  \hspace{1cm} (6.9.4.2-4)

- The parameter \( k_c \) provides a measure of the amount of local-buckling restraint that the web provides to the flange and accounts for interaction between FLB and WLB.

Although AASHTO states that \( b/t \) limits “shall be satisfied,” they still refer to AISC for strength determination of slender members.
§6.9 - Compression Members

§6.9.4.3: Built-Up Compression Members

- If the buckling mode of a built-up column involves deformations that cause shear in the connectors between individual sections, the original slenderness ratio \( \frac{KL}{r_o} \), shall be replaced by a modified value, \( \frac{KL}{r_m} \).

\[
\left( \frac{KL}{r} \right)_m = \left( \frac{KL}{r} \right)_o + 0.82 \frac{a^2}{(1 + \alpha^2)} \left( \frac{a}{r_{ib}} \right)^2
\]

- Modified slenderness ratio of the built-up member

- Original slenderness ratio of the built-up member

\[\alpha = \frac{h}{2r_{ib}}\]

\[r_{ib} = \text{radius of gyration of an individual component relative to its axis parallel to the member axis of buckling}\]

\[h = \text{distance between centroids of individual components measured perpendicular to the member axis of buckling}\]

\[a = \text{distance between connectors, determined by}\]

\[
\frac{a}{r} \leq \left( \frac{3}{4} \right) \left( \frac{KL}{r} \right)_{min}
\]

\[r_i = \text{minimum radius of gyration of an individual component}\]
§6.9 - Compression Members

§4.6.2.5: Effective Length Factor

- For triangulated trusses, trusses, and frames, the effective length factor in the braced plane may be taken as:
  - For bolted or welded end connections at both ends
    \[ K = 0.750 \]
  - For pinned connections at both ends
    \[ K = 0.875 \]
  - For single angles, regardless of end connection
    \[ K = 1.00 \]

- Otherwise, use SSRC Charts and Tables

---

### TABLE C-C2.2

Approximate Values of Effective Length Factor, \( K \)

<table>
<thead>
<tr>
<th>Buckled shape of column is shown by dashed line.</th>
<th>(a)</th>
<th>(b)</th>
<th>(c)</th>
<th>(d)</th>
<th>(e)</th>
<th>(f)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Theoretical ( K ) value</td>
<td>0.5</td>
<td>0.7</td>
<td>1.0</td>
<td>1.6</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Recommended design value when ideal conditions are approximated</td>
<td>0.65</td>
<td>0.80</td>
<td>1.2</td>
<td>1.6</td>
<td>2.10</td>
<td>2.0</td>
</tr>
<tr>
<td>End condition code</td>
<td>Rotation fixed and translation fixed</td>
<td>Rotation fixed and translation fixed</td>
<td>Rotation fixed and translation fixed</td>
<td>Rotation fixed and translation fixed</td>
<td>Rotation fixed and translation fixed</td>
<td></td>
</tr>
</tbody>
</table>
§6.9 - Compression Members

\[ G = \frac{\Sigma (EI/L)c}{\Sigma \lambda (EI/L)c} \]  
(C4.6.2.5-3)

§4.6.2.5: Effective Length Factor

The “Girder” term in the above equation is modified by the parameter \( \lambda \) to reflect the degree of fixity of the connection at the far end of the girder.

<table>
<thead>
<tr>
<th></th>
<th>Sidesway Inhibited</th>
<th>Sidesway Uninhibited</th>
</tr>
</thead>
<tbody>
<tr>
<td>Far End Fixed</td>
<td>( \lambda = 2 )</td>
<td>( \lambda = \frac{2}{3} )</td>
</tr>
<tr>
<td>Far End Pinned</td>
<td>( \lambda = \frac{3}{2} )</td>
<td>( \lambda = \frac{3}{2} )</td>
</tr>
</tbody>
</table>

Additional details are available in the AISC Specification
§6.9 - Compression Members

§4.6.2.5: Effective Length Factor

- For column ends supported by but not rigidly connected to a footing or foundation, $G$ is theoretically equal to infinity, but unless actually designed as a true frictionless pin, may be taken equal to 10 for practical design.

- If the column end is rigidly attached to a properly designed footing, $G$ may be taken equal to 1.0. Smaller values may be taken if justified by analysis.

- In computing effective length factors for members with monolithic connections, it is important to properly evaluate the degree of fixity in the foundation using engineering judgment. In absence of a more refined analysis, the following values can be used:
  - Footing anchored on rock: $G = 1.5$
  - Footing not anchored on rock: $G = 3.0$
  - Footing on soil: $G = 5.0$
  - Footing on multiple rows of end bearing piles: $G = 1.0$

§6.9 - Compression Members

§6.9.4.1: Torsional and Flexural-Torsional Buckling

- Singly symmetric and unsymmetric compression members, such as angles or tees, and doubly-symmetric compression members, such as cruciform members or built-up members with very thin walls, may be governed by the modes of flexural-torsional buckling or torsional buckling rather than the conventional (flexural) buckling mode reflected on Slide #8.

- The design of these members for these less conventional buckling modes is covered in AISC (2005).
§6.9 - Compression Members

§6.9.4.1: Torsional and Flexural-Torsional Buckling

Flexural Buckling

Flexural or Torsional Buckling

Flexural-torsional buckling about the axis of symmetry. Flexural buckling about the other axis.
§6.9 - Compression Members

§6.9.4.1: Torsional and Flexural-Torsional Buckling

Torsional and Flexural-Torsional Buckling Provisions can be written as,

- If $\lambda \leq 2.25$, (Inelastic Buckling)
  \[ P_n = 0.66\lambda F_f A_y \]  (6.9.4.1-1)

- If $\lambda > 2.25$, (Elastic Buckling)
  \[ P_n = \frac{0.88F_f A_y}{\lambda} \]  (6.9.4.1-2)

where,

\[ \lambda = \frac{F_y}{F_f} \]

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§6.9 - Compression Members

§6.9.4.1: Torsional and Flexural-Torsional Buckling

- For Torsional Buckling of Doubly Symmetric Sections

  \[ F_f = \left[ \frac{\pi^2 EC}{(K_f L)^2 + GJ} \right] \frac{1}{I_x + I_y} \]  (AISC E4-4)
§6.9 - Compression Members

§6.9.4.1: Torsional and Flexural-Torsional Buckling

For Flexural-Torsional Buckling of Singly-Symmetric Sections where the y axis is the axis of symmetry,

\[ F_y = \left( \frac{F_{ey} + F_{ez} + F_{ez}}{2H} \right) \left[ 1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2} \right] \]  
(AISC E4-5)

\[ H = 1 - \frac{x^2 + y^2}{F_{ey}^2} \]  
(AISC E4-8)

\[ F_{ez} = x^2 + y^2 + \frac{I_x}{A_y} \]  
(AISC E4-7)

\[ F_{cy} = \frac{\pi^2 E}{(KL/r)^2} \]  
(AISC E4-10)

\[ F_{ez} = \left( \frac{\pi^2 E}{(KL/r)^2} + GJ \right) \frac{1}{A_y F_{ey}} \]  
(AISC E4-11)

AISC Pg 16.1-34

For Tees and Double Angles, the provisions are simplified since the \( C_w \) term in AISC Eqn E4-11 can be taken as zero.

\[ F_{cy} = \left[ \frac{\pi^2 E}{(KL/r)^2} + GJ \right] \frac{1}{I_x + I_y} \]  
\[ F_{cez} = \frac{GJ}{A_y F_{ey}} \]  
(AISC E4-3)

AISC Eqn E4-5 can then be rewritten as

\[ F_{cey} = \left( \frac{F_{cey} + F_{cez}}{2H} \right) \left[ 1 - \frac{4F_{cey}F_{cez}H}{(F_{cey} + F_{cez})^2} \right] \]  
(AISC E4-2)
§6.9 - Compression Members

§6.9.4.1: Torsional and Flexural-Torsional Buckling

- AISC Eqn E4-5 can then be rewritten as

\[
F_{cty} = \left( \frac{F_{cry} + F_{crz}}{2H} \right) \left[ 1 - \sqrt{\frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}} \right]
\]

(AISC E4-2)

where \( F_{cry} \) is taken as the critical stress for flexural buckling about the \( Y \) axis (i.e. \( P_n / A \), from either Eqn 6.9.4.1-1 or Eqn 6.9.4.1-2), and

\[
F_{cry} = \frac{P_{cry}}{A}
\]

\[
P_{n,y} = F_{cty}A
\]
In General:

- "Beams" are members that are composed of elements (flanges, webs, etc) that are stocky enough that moment capacity can reach or approach the yielding moment, \( M_y \), or possibly the plastic moment, \( M_p \).
  - "Beams" can be rolled sections or built-up sections

- "Plate Girders" are members that are composed of elements that are slender enough that buckling of one or more of the elements occurs before the yield moment, \( M_y \), can be reached
  - "Plate Girders" are almost always built-up sections
Flexural Behavior - Theory

**Beams vs Plate Girders**

**In General:**
- The most commonly accepted delineation is the web slenderness:

\[
\frac{h}{t_w} \leq 5.70 \sqrt{\frac{E}{F_y}} \quad \text{when the section is classified as a “Beam”}
\]

\[
\frac{h}{t_w} > 5.70 \sqrt{\frac{E}{F_y}} \quad \text{when the section is classified as a “Plate Girder”}
\]

---

**Theoretical Flexural Failure Modes: “Beams”**

**Primary Failure Modes:**
- **Yielding - Development of Plastic Hinge:** \( M_n = M_p \)
- **Local Buckling:** \( M_w = M_{cr} \)
  - Flange Local Buckling
  - Web Local Buckling
- **Lateral-Torsional Buckling:** \( M_n = M_{cr} \)

**Other:**
- **Shear (Shear Yielding, Shear Buckling)**
Flexural Behavior - Theory

Theoretical Flexural Failure Modes: “Plate Girders”

Primary Failure Modes:
- Yielding - “Reaching First Yield”
  - Occasionally, Plate Girder Capacity can Exceed $M_y$

- Compression Flange Local Buckling: $M_n = M_{cr}$

- Compression Flange Lateral-Torsional Buckling: $M_n = M_{cr}$

Secondary Failure Modes:
- Vertical Flange Buckling
- Web Bend Buckling

Other:
- Shear (Shear Yielding, Shear Buckling)
- Tension Field Action

Flexural Behavior - Theory

Yield Moment and Plastic Moment

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Created July 2007

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**Flexural Behavior - Theory**

**Yield Moment and Plastic Moment**

\[ M_y = F_c a = F_t a \]
\[ F_c = F_t = \left( \frac{1}{2} \right) \left( \frac{b}{2} \right) (b) = \frac{b^2}{4} F_y \]
\[ a = h - (2) \left( \frac{1}{2} \right) = \frac{2}{3} h \]
\[ M_y = F_a = \left( \frac{b^2}{4} F_y \right) \left( \frac{2}{3} h \right) = \frac{b^2 h}{6} F_y = S_y F_y \]

\[ M_p = F_c a = F_t a \]
\[ F_c = F_t = \left( \frac{1}{2} \right) \left( \frac{b}{2} \right) (b) = \frac{b^2}{4} F_y \]
\[ a = h - (2) \left( \frac{1}{2} \right) = \frac{2}{3} h \]
\[ M_p = F_a = \left( \frac{b^2}{4} F_y \right) \left( \frac{2}{3} h \right) = \frac{b^2 h}{6} F_y = Z_y F_y \]

**Shape Factor:**

The Shape factor is defined as the ratio of the plastic moment, \( M_p \), to the yield moment, \( M_y \).

For the Rectangular Cross Section:

\[ SF = \frac{M_p}{M_y} = \frac{\left( \frac{bh^2}{4} \right) F_y}{\frac{bh^2}{6} F_y} = 1.5 \]

The SF is a measure of a section’s efficiency as a bending member.

The SF will always be \( \geq 1.00 \), with 1.00 being most efficient.
Flexural Behavior - Theory

Yield Moment and Plastic Moment

\[ M_p = \sum F_i a_i \]

\[ F_k = F_e = b_f t_f F_y = F_f \]

\[ a_1 = a_4 = \frac{b}{2} + \frac{t_f}{2} = \frac{1}{2}(h + t_f) = a_f \]

\[ F_w = F_w = \left(\frac{h}{2}\right) t_w F_y = F_w \]

\[ a_2 = a_3 = \left(\frac{1}{2}\right) \left(\frac{h}{2}\right) = \frac{1}{4} = a_w \]

\[ M_p = \left(2[F_f a_f + F_w a_w]\right) \]

The Shape factor for most I-shaped cross sections ranges from 1.10 to 1.20, which means that they are more efficient in bending than rectangular sections.

Flexural Behavior - Theory

Yield Moment and Plastic Moment

- Up to this point, we have used doubly symmetric sections. In that case, the Elastic Neutral Axis and Plastic Neutral Axis are at the mid-height of the section.

- When the section is only singly symmetric (or nonsymmetric) the ENA and PNA will be at different locations.
If the section is homogenous, find the PNA by setting the area above the PNA to the area below the PNA.

Otherwise, set the force above the PNA to the force below the PNA.
Flexural Behavior - Theory

Plastic Moment: Composite Sections

- Plastic Neutral Axis in Top Flange

Plastic Moment: Composite Sections

- Plastic Neutral Axis in Web
### Flexural Behavior - Theory

**Plastic Moment: Appendix D6 - Positive Moment**

#### Table D6.1-1: Calculation of PNA and $M_P$ for Section in Positive Flexure

<table>
<thead>
<tr>
<th>CASE</th>
<th>PNA CONDITION</th>
<th>$\Gamma$ AND $M_P$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>In Web $P + P_r \geq P_r + P_o + P_{a}$</td>
<td>$\Gamma = \left( \frac{D}{2} \right) \left[ \frac{B - P_r - P_o - P_{a}}{P_r} \right]$ + $1$; $M_P = \frac{P}{2D} \left[ \Gamma \left( D - \Gamma \right) \right] + \left[ P_{d_1} + P_{d_2} + P_{d_{a_1}} + P_{d_{a_2}} + P_{d} \right]$</td>
</tr>
<tr>
<td>H</td>
<td>In Top Flange $P + P_r \geq P_r + P_o + P_{a}$</td>
<td>$\Gamma = \left( \frac{t}{2} \right) \left[ \frac{P + P_r - P_o - P_{a}}{P_r} \right]$ + $1$; $M_P = \frac{P}{2t} \left[ \Gamma \left( t - \Gamma \right) \right] + \left[ P_{d_1} + P_{d_2} + P_{d_{a_1}} + P_{d_{a_2}} + P_{d} \right]$</td>
</tr>
<tr>
<td>III</td>
<td>Concrete Deck, Below $P_o$ $P + P_r + P_o \geq \left( \frac{C}{t} \right) P + P_o + P_{a}$</td>
<td>$\Gamma = \left( \frac{t}{2} \right) \left[ \frac{P + P_r + P - P_o}{P} \right]$; $M_P = \frac{P}{2t} \left[ \Gamma \left( t - \Gamma \right) \right] + \left[ P_{d_1} + P_{d_2} + P_{d_{a_1}} + P_{d_{a_2}} + P_{d} \right]$</td>
</tr>
<tr>
<td>IV</td>
<td>Concrete Deck, at $P_o$ $P + P_r + P_o \geq \left( \frac{C}{t} \right) P + P_o + P_{a}$</td>
<td>$\Gamma = \left( \frac{t}{2} \right) \left[ \frac{P + P_r + P - P_o}{P} \right]$; $M_P = \frac{P}{2t} \left[ \Gamma \left( t - \Gamma \right) \right] + \left[ P_{d_1} + P_{d_2} + P_{d_{a_1}} + P_{d_{a_2}} + P_{d} \right]$</td>
</tr>
</tbody>
</table>

---

### Work Flexure Examples #2 & #3

**Plastic Moment: Appendix D6 - Positive Moment**

#### Table D6.1-1: Calculation of PNA and $M_P$ for Section in Positive Flexure

<table>
<thead>
<tr>
<th>CASE</th>
<th>PNA CONDITION</th>
<th>$\Gamma$ AND $M_P$</th>
</tr>
</thead>
<tbody>
<tr>
<td>V</td>
<td>Concrete Deck, Above $P_o$, Below $P_a$ $P + P_r + P_o \geq \left( \frac{C}{t} \right) P + P_o$</td>
<td>$\Gamma = \left( \frac{t}{2} \right) \left[ \frac{P + P_r + P - P_o}{P} \right]$; $M_P = \frac{P}{2t} \left[ \Gamma \left( t - \Gamma \right) \right] + \left[ P_{d_1} + P_{d_2} + P_{d_{a_1}} + P_{d_{a_2}} + P_{d} \right]$</td>
</tr>
<tr>
<td>VI</td>
<td>Concrete Deck, at $P_o$ $P + P_r + P_o \geq \left( \frac{C}{t} \right) P + P_o$</td>
<td>$\Gamma = \left( \frac{t}{2} \right) \left[ \frac{P + P_r + P - P_o}{P} \right]$; $M_P = \frac{P}{2t} \left[ \Gamma \left( t - \Gamma \right) \right] + \left[ P_{d_1} + P_{d_2} + P_{d_{a_1}} + P_{d_{a_2}} + P_{d} \right]$</td>
</tr>
<tr>
<td>VII</td>
<td>Concrete Deck, Above $P_a$ $P + P_r + P_a + P_o \geq \left( \frac{C}{t} \right) P + P_r$</td>
<td>$\Gamma = \left( \frac{t}{2} \right) \left[ \frac{P + P_r + P - P_o}{P} \right]$; $M_P = \frac{P}{2t} \left[ \Gamma \left( t - \Gamma \right) \right] + \left[ P_{d_1} + P_{d_2} + P_{d_{a_1}} + P_{d_{a_2}} + P_{d} \right]$</td>
</tr>
</tbody>
</table>
Flexural Behavior - Theory

Plastic Moment: Composite Sections

- The Plastic Moment of Composite Sections under Negative Moment can be computed based on the steel section alone or can be computed accounting for the rebar, assuming that shear connectors are placed throughout the negative moment region and that the rebar has been properly developed.
Flexural Behavior - Theory

Plastic Moment: Composite Sections

**Table D6.1-2: Calculation of PNA and \( M_p \) for Section in Negative Flexure**

<table>
<thead>
<tr>
<th>CASE</th>
<th>PNA CONDITION</th>
<th>( T ) AND ( M_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>In Web ( P_1 + P_2 \geq P_n + P_c )</td>
<td>( T = \frac{D}{3} \left[ \frac{P_1 - P_2 - P_n + P_c}{P_c} + 1 \right] )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( M_p = \frac{P_c}{220} \left[ T + (D - T) \right] \left[ P_d d_1 + P_d d_2 + P_d d_3 + P_d d_4 \right] )</td>
</tr>
<tr>
<td>II</td>
<td>In Top Flange ( P_1 + P_2 \geq P_n + P_c )</td>
<td>( T = \frac{1}{3} \left[ \frac{P_1 - P_2 - P_n + P_c}{P_c} + 1 \right] )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( M_p = \frac{P_c}{220} \left[ T + (D - T) \right] \left[ P_d d_1 + P_d d_2 + P_d d_3 + P_d d_4 \right] )</td>
</tr>
</tbody>
</table>
Flexural Behavior - Theory

Yield Moment and Plastic Moment

\[ M_y = S_y F_y \]

\[ M_p = Z_y F_y \]
Flexural Behavior - Theory

Section Classification for “Beams”

Flanges (Unstiffened):

\[ \lambda = \frac{b_f}{2t_f} \]

Compact if \( \lambda \leq \lambda_p \)

\[ \lambda_p = 0.38 \frac{E}{F_y} \]

Non-Compact if \( \lambda_p < \lambda \leq \lambda_r \)

Slender if \( \lambda < \lambda_s \)

\[ \lambda_s = 0.83 \frac{E}{F_y} \]
Flexural Behavior - Theory

Section Classification for “Beams”

Webs (Stiffened): \[ \lambda = \frac{h}{t_w} \]

Unstiffened

- Compact if \( \lambda \leq \lambda_p \)
  \[ \lambda_p = 3.76 \frac{E}{F_y} \]

- Non-Compact if \( \lambda_p < \lambda \leq \lambda_r \)

Stiffened

- Slender if \( \lambda_r < \lambda \)
  \[ \lambda_r = 5.70 \frac{E}{F_y} \]

Based on this, all rolled sections in AISC have compact webs for \( F_y \leq 50 \text{ksi} \)

Section Classification for “Plate Girders”

Flanges (Unstiffened): \[ \lambda = \frac{b_f}{2t_f} \]

Unstiffened

- Compact if \( \lambda \leq \lambda_p \)
  \[ \lambda_p = 0.38 \frac{E}{F_y} \]

- Non-Compact if \( \lambda_p < \lambda \leq \lambda_s \)

Stiffened

- Slender if \( \lambda_s < \lambda \)
  \[ \lambda_s = 0.95 \frac{Ek_c}{F_y} \]

\[ k_c = \frac{4}{\sqrt{h/t_w}} \quad 0.35 \leq k_c \leq 0.76 \]

\( k_c \) is a measure of how much restraint the web provides to the flanges
Flexural Behavior - Theory

Section Classification for “Plate Girders”

Webs (Stiffened): \[ \lambda = \frac{h}{t_w} \]

Slender if \( \lambda_s < \lambda \)

\[ \lambda_s = 5.70 \sqrt{\frac{E}{F_y}} \]

A section is a “Plate Girder” only when it has a slender web.

Flexural Behavior - Theory

Solution Space for Local Buckling

- Plastic Moment
- Inelastic FLB/WLB
- Elastic FLB/WLB

\( M_p \)

\( M_n \)

Compact
Non-Compact
Slender

Flange or Web Slenderness, \( \lambda \)
Flexural Behavior - Theory

Strain Demand at the Plastic Moment

Loosely Speaking:
- When $\lambda \leq \lambda_r$ - the element can reach $F_y$ before buckling locally
- When $\lambda \leq \lambda_p$ - the element can sustain “significant inelastic strain” before buckling locally

How much inelastic strain must a section sustain to reach its plastic moment?

Define:

$$R = \frac{\theta_f - \theta_p}{\theta_p}$$

Moment Capacity, $M_c$

Rotation, $\theta$

Rotation Capacity, $R$
Strain Demand at the Plastic Moment

- Using the Arc-Length Formula,

\[ L - \delta = \theta \left( r - \frac{d}{2} \right) \rightarrow r = \frac{L - \delta + d}{\theta} \]

\[ L + \delta = \theta \left( r + \frac{d}{2} \right) \rightarrow r = \frac{L + \delta - d}{\theta} \]

\[ \frac{L - \delta}{\theta} + \frac{d}{2} = \frac{L + \delta - d}{\theta} \]

\[ \theta = \frac{2\delta}{d} \]
Flexural Behavior - Theory

Strain Demand at the Plastic Moment

Then,

\[ R = \frac{\theta_f - \theta_p}{\theta_p} = \left( \frac{2\delta}{d} \right)_f - \left( \frac{2\delta}{d} \right)_p = \frac{\delta_f - \delta_p}{\delta_p} \]

Since \( \varepsilon = \frac{\delta}{L} \rightarrow \delta = \varepsilon L \)

\[ R = \frac{\varepsilon_f L - \varepsilon_p L}{\varepsilon_p L} = \frac{\varepsilon_f - \varepsilon_p}{\varepsilon_p} \rightarrow \frac{\varepsilon_f}{\varepsilon_p} = (R + 1) \]

Strain Demand at the Plastic Moment

Take \( \frac{\varepsilon_f}{\varepsilon_p} = \frac{M_p}{M_y} \theta_y = SF \theta_y \rightarrow \varepsilon_p = SF \varepsilon_y \)

\[ \frac{\varepsilon_f}{\varepsilon_p} = \frac{\varepsilon_f}{SF \varepsilon_y} \rightarrow \frac{\varepsilon_f}{\varepsilon_y} = SF (R + 1) \]

Take \( SF = 1.2 \) for I-shaped Sections and \( R = 3 \)

\[ \frac{\varepsilon_f}{\varepsilon_y} = 4.8 \]

Most texts estimate the strain demand at the plastic moment at roughly 7 to 9 times the yield strain. Comparatively speaking, the strain at the onset of strain hardening is roughly 15 to 20 times the yield strain.
Flexural Behavior - Theory

Strain Demand at the Plastic Moment

- For Buckling of a Plate Under a Uniform Compression:

\[ F_{cr} = \frac{\pi^2 kE}{12(1 - \nu^2)(b/t)^2} \]

- Solving for \( b/t \), taking \( \nu = 0.30 \):

\[ \frac{b}{t} = \sqrt{\frac{\pi^2 kE}{12(1 - \nu^2)F_{cr}}} = 0.951 \sqrt{\frac{kE}{F_{cr}}} \]

Strain Demand at the Plastic Moment

- In order to achieve significant plasticity, substitute \( F_y \) for \( F_{cr} \) and take 0.46 of the limiting value of \( b/t \):

\[ \frac{b}{t} \leq (0.46)(0.951) \sqrt{\frac{kE}{F_y}} = 0.437 \sqrt{\frac{kE}{F_y}} \]
Flexural Behavior - Theory

### Strain Demand at the Plastic Moment

- In order to achieve significant plasticity, substitute $F_y$ for $F_{cr}$ and take 0.46 of the limiting value of $\frac{b}{t}$.

\[
\frac{b}{t} \leq (0.46) \left( \frac{b}{t} \right)
\]

\[
\frac{F_y}{E} \leq \frac{F_y}{E} \leq \frac{F_y}{E} \leq \frac{F_y}{E} \leq \frac{F_y}{E}
\]

- For the compression flange of a beam or girder,

- Judgment...take $k$ as $\frac{1}{3}$ of the way between pinned and fixed

\[
k = 0.425 + (0.33)(1.277 - 0.425) = 0.709
\]

\[
\lambda_p = \frac{F_y}{E} \leq \frac{F_y}{E} \leq \frac{F_y}{E} \leq \frac{F_y}{E} \leq \frac{F_y}{E}
\]

- Compare this with the limit used in AASHTO

\[
\lambda_p = 0.38 \sqrt{\frac{E}{F_y}} \quad (6.10.8.2.2-4)
\]
Flexural Behavior - Theory

Lateral-Torsional Buckling – “Beams”

\[ M_{cr} = \frac{\pi}{L} \sqrt{\frac{EI}{GJ}} \sqrt{1 + W^2} \]

\[ W = \frac{\pi}{L} \sqrt{\frac{EC}{GJ}} \]
Lateral buckling of plate girders is more often characterized by lateral buckling of the compression flange than by LTB of the section.

Because the web of a plate girder is so thin, the entire section may not twist like that of a rolled section.

The radius of gyration of a hypothetical tee made up of the compression flange and a portion of the web, \( r_t \), is needed.
Flexural Behavior - Theory

Compression Flange Lateral Buckling – “Plate Girders”

- This hypothetical tee section is composed of the compression flange and 1/3 of the depth of the web that is in compression.

\[ M_{cr} = S_{ec} F_{cr} \]
\[ F_{cr} = \frac{R_b \pi^2 E}{(L_b/r_t)^2} \]

- \( S_{ec} \) - Elastic Section Modulus for the compression flange
- \( R_b \) - Load Shedding Factor (more on this later)
- \( L_b \) - Unbraced length of the beam

Flexural Behavior - Theory

Solution Space for Lateral-Torsional Buckling
Flexural Behavior - Theory

Lateral-Torsional Buckling

- For Beams:

\[ L_p = 1.76r_y \frac{E}{F_y} \quad L_r = \text{Complex...} \]

- For Plate Girders:

\[ L_p = 1.0r_y \frac{E}{F_{jc}} \quad L_r = \pi r_y \frac{E}{F_y} \]

The equation proposed for the critical buckling strength were derived for the case of a uniform bending moment over the length of the beam.

This is the most critical case but is very often overly conservative when the moment diagram is not uniform.

A factor, the moment gradient modifier, \( C_m \), based on the shape of the moment diagram, is used to increase the moment capacity when the moment is not uniform.
Flexural Behavior - Theory

Lateral-Torsional Buckling

- Many forms of the equation for $C_b$ can be found. The first of two prevailing formulations is shown here,

$$C_b = 1.75 + 1.05 \left( \frac{M_1}{M_2} \right) + 0.3 \left( \frac{M_1}{M_2} \right)^2 \leq 2.3$$

where $M_1$ and $M_2$ are the moments at the ends of an unbraced length. The ratio of $M_1/M_2$ is negative when they cause single curvature and is positive when they cause double curvature.

- This formulation is widely accepted but has the limitation that it assumes a linearly varying moment between end points. I.e., it doesn’t account for the case of a load on the beam between end points.

The second of two prevailing formulations for $C_b$ is shown here,

$$C_b = \frac{12.5M_{\text{max}}}{2.5M_{\text{max}} + 3M_A + 4M_B + 3M_C} \leq 3.0$$

where $M_{\text{max}}$ is the maximum moment in an unbraced length and $M_A$, $M_B$, and $M_C$ are the moments at the quarter point, mid point, and three-quarter point of the unbraced length. Absolute values of the moments are used.

- A form of the first formulation for $C_b$ is adopted by AASHTO-LRFD but the application is considerably complicated by the fact that the moments (or stresses) are taken from moment envelopes instead of from moment diagrams. More on this later…
Flexural Behavior - Theory

Solution Space for Lateral-Torsional Buckling

Without \( C_b \)

With \( C_b \)

Flexural Behavior - Theory

More on Rotation Capacity

\[ \Delta_{\text{max}} \]

Rotation capacity = \( R \Delta_{\text{pl}} \)

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AASHTO-LRFD 2007
Flexural Behavior - Theory

More on Rotation Capacity

1. Plastic moment strength $M_p$ is achieved along with large deformation. Deformation ability, called rotation capacity as shown is essentially the ability to undergo large flange strain without instability.

2. Inelastic behavior where plastic moment strength $M_p$ is achieved but little rotation capacity is exhibited, because of inadequate stiffness of the flange and/or web to resist local buckling, or inadequate lateral support to resist lateral-torsional buckling, while the flange is inelastic.

3. Inelastic behavior where the moment strength $M_r$, the moment above which residual stresses cause inelastic behavior to begin, is reached or exceeded; however, local buckling of the flange or web, or lateral-torsional bucking prevent achieving the plastic moment strength $M_p$.

4. Elastic behavior where moment strength $M_{cr}$ is controlled by elastic buckling; any or all of local flange buckling, local web buckling, or lateral-torsional buckling.

Vertical Flange Buckling - “Plate Girders”
Flexural Behavior - Theory

Vertical Flange Buckling - “Plate Girders”

The vertical component of the flange force, $F_{vert}$, can be written as

$$F_{vert} = \sigma_f A_f \, d\theta$$

Substituting,

$$F_{vert} = (\sigma_f A_f) \left( \frac{2\varepsilon_f \, dx}{h} \right)$$

If we divide $F_{vert}$ by the area $A = t_w \, d_x$ then we get the stress $f_c$ shown above

$$f_c = (\sigma_f A_f) \left( \frac{2\varepsilon_f \, dx}{h \cdot t_w \cdot dx} \right) = \left( \frac{A_f}{A_w} \right) (2\sigma_f \varepsilon_f)$$
Flexural Behavior - Theory

**Vertical Flange Buckling - “Plate Girders”**

where:
- \( A_f \) - the area of the compression flange
- \( A_w \) - the area of the web

Recall that the critical buckling stress for a plate is given by

\[
F_{cr} = \frac{k \cdot \pi^2 \cdot E}{12(1-\nu^2)(b/t)^2}
\]

Let \( f_c = F_{cr} \), \( k = 1.00 \) (pinned top & bottom; other edges free), \( bli = hlt_w \)

\[
\left(\frac{A_f}{A_w}\right)(2\sigma_f e_f) = \frac{\pi^2 \cdot E}{12(1-\nu^2)(h/t_w)^2}
\]

Solving for \( h/t_w \), with \( \nu = 0.30 \),

\[
\frac{h}{t_w} = 0.672 \sqrt{\left(\frac{A_w}{A_f}\right)\left(\frac{E}{\sigma_f e_f}\right)}
\]

Now suppose that the flange is at its yield stress, which leads to \( \sigma_f = F_y \) and the strain in the flange is equal to \( (F_y + F_r) / E \)

\[
\frac{h}{t_w} = 0.672E \sqrt{\left(\frac{A_w}{A_f}\right)\left(\frac{1}{F_y(F_y + F_r)}\right)}
\]
Flexural Behavior - Theory

Vertical Flange Buckling - “Plate Girders”

Estimate the residual stress as $F_r = 0.3 \cdot F_y$ and recognize that the ratio of $A_w$ to $A_f$ likely to have a lower bound of 0.5

\[
\frac{h}{t_w} = \frac{0.475E}{F_y(1.3F_y)} = 0.4166 \frac{E}{F_y}
\]

Vertical Flange Buckling is not explicitly considered by AASHTO. Later, however, we’ll see that $D / t_w$ is limited to a maximum of 150. Substitute 150 and solve for $F_y$.

\[
F_y = \frac{(0.4166)(29,000^{ksi})}{(150)} = 80.5^{ksi}
\]

VFB is precluded so long as the yield stress is not greater than $80^{ksi}$. The commentary on Pg 6-85 says that you’re safe up to $85^{ksi}$.

Web Bend Buckling - “Plate Girders”

Consider a Plate under Flexure

\[
F_{cr} = \frac{k \cdot \pi^2 \cdot E}{12(1 - v^2)(b/t)^2}
\]

- For a web panel of the girder defined by vertical stiffeners with the aspect ratio of $a / h$,
  - $k = 39.6$ if full fixity is assumed at the flanges
  - $k = 23.9$ if the web is assumed to be pinned at the flanges.
- Take 80% of the difference towards the higher value:
  - $k = 23.9 + (0.8)(39.6 - 23.9) = 36.5$
Flexural Behavior - Theory

Web Bend Buckling - “Plate Girders”

\[ F_{cr} = \frac{36.5 \cdot \pi^2 \cdot E}{12(1 - 0.3^2)(h/t_w)^2} = \frac{33.0 \cdot E}{(h/t_w)^2} \]

- Solve for \( h/t_w \)

\[ \frac{h}{t_w} = 5.74 \sqrt{\frac{E}{F_{cr}}} \]

- If you set \( F_{cr} \geq F_y \), then,

\[ \frac{h}{t_w} \leq 5.74 \sqrt{\frac{E}{F_y}} \]

which is roughly the limit for a slender web, of

\[ \frac{h}{t_w} = 5.70 \sqrt{\frac{E}{F_y}} \]

Flexural Behavior - Theory

Web Bend Buckling - “Plate Girders”

- Going back to \( F_{cr} \) as a function of \( k \), using \( D \) instead of \( h \):

\[ F_{cr} = \frac{k \cdot \pi^2 \cdot E}{12(1 - 0.3^2)(D/t_w)^2} = \frac{0.9038 \cdot k \cdot E}{(D/t_w)^2} \]

- \( k \) can be defined as,

\[ k = \frac{9}{(D_c/D)^2} \]

- For the case of the doubly symmetric shape, \( D = 2D_c \), \( k = 36.0 \), and

\[ F_{cr} = \frac{32.5E}{(D/t_w)^2} \]

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Since plate-girder webs usually have high $h/t_w$ ratios, buckling may occur as a result of the bending about the strong axis of the girder. Generally speaking, webs with $h/t_w > \lambda_r$ are susceptible to buckling.

Since the web carries only a small portion of the bending moment on the section, however, this buckling does not generally represent the end of the usefulness of the girder.

Consider the figure on the following slide, which illustrates the relationship between nominal moment, $M_{n}$, and $h/t_w$ when lateral-torsional Buckling and flange-local buckling are precluded...

When the post-buckling strength of the girder is considered, the strength is increased from line BC to line BD in chart.

The amount of increase of a function mostly of the ratio of $A_s$ to $A_f$. 

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ODOT Short Course Created July 2007 Flexure: Slide #65
Flexural Behavior - Theory

Load Shedding Factor - “Plate Girders”

To derive a reduction factor for moment capacity to account for the post buckling strength of the web, the portion of the web that has buckled is disregarded for moment capacity, as is shown here for the case of $h/\tw = 320$ (quite slender).

It can be shown that for this case, the ratio $\frac{M_n}{M_f}$ can be adequately approximately linearly as,

$$\frac{M_n}{M_f} = 1.0 - 0.09 \frac{A_w}{A_f}$$

This is for only one value of $h/\tw$, though... What happens when $h/\tw$ varies?

---

Flexural Behavior - Theory

Load Shedding Factor - “Plate Girders”

- The strength represented by line BD in the chart two slides back can be approximated as linear.

- At point D, $h/\tw = 320$, the limit above which VFB may govern.

- At Point B, $\frac{h}{\tw} = 5.70 \frac{E}{\sqrt{F_y}} \approx 162$ (for $F_y = 36$ksi)

- The slope of line BD, then is...

$$\text{Slope of BD} = \frac{\text{Slope per } A_r/A_f}{320 - 162} = \frac{0.09}{158} = 0.00057, \text{ say } 0.0005$$
Flexural Behavior - Theory

Load Shedding Factor - “Plate Girders”

- Then,

\[
\frac{M}{M_y} = 1.0 - 0.0005 \left( \frac{A_w}{A_f} \right) \left( \frac{h}{t_w} - 5.70 \sqrt[5]{\frac{E}{F_{yw}}} \right)
\]

- The coefficient of 0.0005 was originally developed by Basler and is valid for the ratio of \( \frac{A_w}{A_f} \) up to 3.0. An updated version of the above equation is valid for the ratio of \( \frac{A_w}{A_f} \) up to 10.0.

\[
\frac{M}{M_y} = 1.0 - \left( \frac{a_{wc}}{1200 + 300a_{wc}} \right) \left( \frac{h}{t_w} - 5.70 \sqrt[5]{\frac{E}{F_{yw}}} \right)
\]

- When \( M_y \) is replaced by the critical moment, \( M_{cr} = S_{wc} F_{cr} \), which may be less than \( M_y \),

\[
M_y = S_{wc} F_{cr} \left[ 1.0 - \left( \frac{a_{wc}}{1200 + 300a_{wc}} \left( \frac{h}{t_w} - 5.70 \sqrt[5]{\frac{E}{F_{yw}}} \right) \right) \right]
\]

\[
M_y = S_{wc} F_{cr} R_b
\]

- Thus the load shedding factor (or plate girder factor) can be written as,

\[
R_b = 1 - \left( \frac{a_{wc}}{1200 + 300a_{wc}} \right) \left( \frac{2D}{t_w} - 5.70 \sqrt[5]{\frac{E}{F_{yw}}} \right) \leq 1.0 \quad (6.10.1.10.2-3)
\]

- In this form, \( R_b \) is limited to a value not greater than 1.00, \( h \) is replaced with \( 2D \), for the case where the N.A. is not at mid-height, and \( F_{yw} \) is replaced with \( F_{yc} \).
Hybrid Girder Factor - “Plate Girders”

- It is often economical to proportion a built-up girder with a web that has a lower strength than the flange(s). In this case the girder is referred to as a hybrid girder.

- In general the strength of a girder is defined by yielding of the flanges and not yielding of the web.

One approach to determine the moment capacity of a hybrid girder is to use moment equilibrium of the stress distribution present at first yield or at the plastic moment.

…but this is rather tedious.
A second approach is to derive a reduction factor that can be applied to a moment capacity that is determined assuming that the girder is made up of material for $F_y = F_y^f$

$$M_v = S_m F_y R_v$$

where,

$$R_v = \frac{12 + a_v (3m - m^3)}{12 + 2a_w}$$

$$a_v = \frac{A_w}{A_f} \quad m = \frac{F_u}{F_y}$$

Compare this form of the hybrid factor to the form in AASHTO...

---

$m$ is replaced by $\rho$ and $A_{w_e}$ is replaced by $\beta$, wherein the web area is taken as $2D_n t_w$ instead of $h t_w$

$$R_v = \frac{12 + \beta (3\rho - \rho^3)}{12 + 2\beta}$$

$$\beta = \frac{2D_n t_w}{A_{w_e}}$$

$$\rho = \frac{F_u}{f_y} \leq 1.0$$

$D_n$ - Larger of the distances from the E.N.A. to the inside face of either flange.

$f_y$ - Yield stress of the flange corresponding the $D_n$.

$A_{w_e}$ - Area of the flange corresponding to $D_n$. 

---
Flexural Behavior - Theory

Depth of the Web in Compression

- Most of the discussion of Flexural Theory focusing on the web behavior has included the height of the web, \( h \) or depth of the web \( D \).

- These discussions have focused primarily on the stability of the web with regard to Web Local Buckling, Bend Buckling, Vertical Flange Buckling, Load Shedding, etc.

- Really, we're interested more in the depth of the web that is in compression than we are in the overall depth of the web.
Flexural Behavior - Theory

Depth of the Web in Compression

- When proportioning composite plate girders, it is common to use a top flange that is significantly smaller than the bottom flange. When this section is subjected to moments before the deck has cured, the ENA can be quite a bit lower than mid-height of the web creating a situation that is more critical than was assumed for doubly symmetric sections.

![Diagram showing depth of compression web]

- To account for this possibly unconservative situation, AASHTO uses the depth of the web in compression, either \( D_c \) or \( D_{cp} \), instead of \( h \) or \( D \) in most equations addressing web stability.

- For a non-composite doubly symmetric section, half of the web is in compression...
  \[
  \frac{h}{t_w} = \frac{2D_c}{t_w}
  \]

- Since the ENA and PNA are coincident for a non-composite doubly symmetric section...
  \[
  \frac{h}{t_w} = \frac{2D_{cp}}{t_w}
  \]
Flexural Behavior - Theory

Depth of the Web in Compression in the Elastic Range, $D_c$

- For composite sections in positive flexure, the depth of the web in compression in the elastic range, $D_c$, shall be the depth over which the algebraic sum of the stresses in the steel, long-term composite and short-term composite sections from the dead and live loads, plus impact, is compressive.

- At sections in positive flexure, $D_c$ of the composite section will increase with increasing span length because of the increasing dead-to-live load ratio. Therefore, in general it is important to recognize the effect of the deadload stress on the location of the neutral axis of the composite section in regions of positive flexure.

\[
D_c = \left( \frac{-f_c}{f_y} \right) \left( d - t_c \right) \geq 0
\]
Flexural Behavior - Theory

Depth of the Web in Compression in the Elastic Range, $D_c$

- $D_c$ - depth of the steel section (in.)
- $f_c$ - sum of the compression-flange stresses caused by the different loads, i.e., $DC1$, the permanent load acting on the noncomposite section; $DC2$, the permanent load acting on the long-term composite section; $DW$, the wearing surface load; and $LL+IM$; acting on their respective sections (ksi).
- $f_c$ shall be taken as negative when the stress is in compression.
- $f_t$ - the sum of the various tension-flange stresses caused by the different loads (ksi).

Flange lateral bending shall be disregarded in these calculations.

---

Flexural Behavior - Theory

Depth of the Web in Compression in the Elastic Range, $D_c$

- For composite sections in negative flexure, $D_c$ shall be computed for the section consisting of the steel girder plus the longitudinal reinforcement with the exception of the following:
  - For composite sections in negative flexure at the service limit state where the concrete deck is considered effective in tension for computing flexural stresses on the composite section due to Load Combination Service II, $D_c$ shall be computed from Eq. 1.

- For composite sections in negative flexure, the concrete deck is typically not considered to be effective in tension. Therefore, the distance between the neutral axis locations for the steel and composite sections is small and the location of the neutral axis for the composite section is largely unaffected by the dead-load stress. The exception is for Service Checks where the deck is considered effective in tension.
Flexural Behavior - Theory

Depth of the Web in Compression in the Plastic Range, $D_{cp}$

- For composite sections in positive flexure, the depth of the web in compression at the plastic moment, $D_{cp}$, shall be taken as follows for cases from Table D6.1-1 where the plastic neutral axis is in the web:

$$D_{cp} = D \left( \frac{F_{yr}A_t - F_{ws}A_c - 0.85f'cA_c - F_{yr}A_{ws}}{F_{yr}A_c} + 1 \right)$$  \hspace{1cm} (D6.3.2-1)

- For all other composite sections in positive flexure, $D_{cp}$ shall be taken equal to zero.

- For composite sections in negative flexure, $D_{cp}$ shall be taken as follows for cases from Table D6.1-2 where the plastic neutral axis is in the web:

$$D_{cp} = \frac{D}{2A_yF_{yr}} \left[ F_{yr}A_t + F_{ws}A_c + F_{yr}A_{ws} - F_{yr}A_c \right]$$  \hspace{1cm} (D6.3.2-2)

- For all other composite sections in negative flexure, $D_{cp}$ shall be taken equal to $D$. 
**Flexural Behavior - Theory**

**Depth of the Web in Compression in the Plastic Range, \( D_{cp} \)**

- **For noncomposite sections** where:

  \[
  F_y A_y \geq \left| F_y A_k - F_y A_t \right|
  \]  
  \( \text{(D6.3.2-3)} \)

  \( D_{cp} \) shall be taken as:

  \[
  D_{cp} = \frac{D}{2 A_y F_y} \left[ F_y A_k + F_y A_w - F_y A_t \right]
  \]  
  \( \text{(D6.3.2-4)} \)

- **For all other noncomposite sections**, \( D_{cp} \) shall be taken equal to \( D \).

---

**Yield Moment - Non-composite Sections**

- The yield moment, \( M_y \), of a noncomposite section shall be taken as the smaller of the moment required to cause nominal first yielding in the compression flange, \( M_{yc} \), and the moment required to cause nominal first yielding in the tension flange, \( M_{yt} \), at the strength limit state.

- Flange lateral bending in all types of sections and web yielding in hybrid sections shall be disregarded in this calculation.
Flexural Behavior - Theory

Yield Moment - Composite Sections

- The yield moment of a composite section in positive flexure shall be taken as the sum of the moments applied separately to the steel and the short-term and long-term composite sections to cause nominal first yielding in either steel flange at the strength limit state. Flange lateral bending in all types of sections and web yielding in hybrid sections shall be disregarded in this calculation.
- For a composite section in positive flexure may be determined as follows:
  - Calculate the moment \( M_{D1} \) caused by the factored permanent load applied before the concrete deck has hardened or is made composite. Apply this moment to the steel section.
  - Calculate the moment \( M_{D2} \) caused by the remainder of the factored permanent load. Apply this moment to the long-term composite section.
  - Calculate the additional moment \( M_{AD} \) that must be applied to the short-term composite section to cause nominal yielding in either steel flange.
  - The yield moment is the sum of the total permanent load moment and the additional moment.

Symbolically, the procedure is:

- Solve for \( M_{AD} \) from the equation:

\[
F_s = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \quad (D6.2.2-1)
\]

- Then calculate:

\[
M_y = M_{D1} + M_{D2} + M_{AD} \quad (D6.2.2-2)
\]
§6.10 - I-Section Flexural Members

- 6.10.1 General
- 6.10.2 Cross-Section Proportion Limits
- 6.10.3 Constructability
- 6.10.4 Service Limit State
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- 6.10.9 Shear Strength
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§6.10.1 - I-Sections: General

- 6.10.1 General
  - 6.10.1.1 Composite Sections
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Modular Ratio,
\[ n = \frac{E_s}{E_c} \]

\[ 2.4 \leq f'_c \leq 2.9 \quad n = 10 \]
\[ 2.9 \leq f'_c \leq 3.6 \quad n = 9 \]
\[ 3.6 \leq f'_c \leq 4.6 \quad n = 8 \]
\[ 4.6 \leq f'_c \leq 6.0 \quad n = 7 \]
\[ 6.0 \leq f'_c \quad n = 6 \]

- Short-Term Modular Ratio \( \to n \)
- Long-Term Modular Ratio \( \to 3n \)
§6.10.1 - I-Sections: General

§6.10.1.1: Composite Sections - Effective Flange Width (§4.6.2.6)

- \( b_{\text{int}} \), for interior beams, may be taken as the least of:
  - One-quarter the effective span length
  - 12.0 times the slab thickness plus the greater of the web thickness or one-half the width of the top flange
  - The average spacing of adjacent beams

- \( b_{\text{ext}} \), for exterior beams, may be taken as one half \( b_{\text{int}} \), plus the least of:
  - One-eighth the effective span length
  - 6.0 times the slab thickness plus the greater of one-half the web thickness or one-quarter the width of the top flange
  - The width of the overhang
§6.10.1 - I-Sections: General

§6.10.1.1: Composite Sections - Effective Flange Width (§4.6.2.6)

- Interior Beams,
  \[ b_{s,int} = \min \left( \frac{1}{12} \frac{L_{eff}}{t_s} + \max \left( t_s, \frac{h_i}{2} \right) \right) \]

- Exterior Beams:
  \[ b_{s,ext} = \frac{b_{s,int}}{2} + \min \left( \frac{1}{12} \frac{L_{eff}}{t_s} + \max \left( t_s, \frac{h_i}{2} \right) \right) \]

- For effective flange width calculations, the effective span length is the:
  - actual span length for simply supported spans, and
  - distance between permanent load inflection points for continuous spans

ODOT: Don’t include the sacrificial wearing surface in eff width calcs.

§6.10.1 - I-Sections: General

§6.10.1.1: Composite Sections - Stress Calculations

For Positive Moment Regions:
- Stresses computed based on the transformed area of the deck
  - Use properties of steel alone (DC₁), short-term composite (LL), or long-term composite (DC₂) as appropriate.

For Negative Moment Regions:
- Use the cracked section properties consisting of the steel section and longitudinal slab reinforcement within \( b_p \) except:
  - Fatigue loads and Service II loads act on the uncracked composite section so long as adequate shear connectors are in place and the deck has minimum reinforcement.

For Concrete Stresses:
- Use the short-term composite section properties

ODOT: Don’t include the sacrificial wearing surface or the haunch in when computing composite section properties.
§6.10.1 - I-Sections: General

§6.10.1.3: Hybrid Sections
- The yield strength of the web should not be less than 70% of the yield strength of the higher strength flange or 36 ksi.

§6.10.1 - I-Sections: General

§6.10.1.5: Stiffness
- For loads applied to the noncomposite section,
  - use the stiffness properties of the steel alone.

- For permanent loads applied to the composite section,
  - use the stiffness properties of the long-term composite section assuming the concrete deck to be effective over the entire span length

- For transient loads applied to the composite section,
  - use the stiffness properties of the short-term composite section assuming the concrete deck to be effective over the entire span length
§6.10.1 - I-Sections: General

§6.10.1.6: Flange Stresses and Moments

- $f_{bu}$: The compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of lateral bending.

- $M_o$: The largest major-axis bending moment throughout the unbraced length causing compression in the flange under consideration.

- $f_l$: The largest stress due to lateral bending throughout the unbraced length in the flange under consideration. Shall not exceed $0.6 F_{yf}$
  - Eccentric Concrete Deck Overhangs §6.10.3.4 → Constructability
  - Wind loads (WS) §4.6.2.7
  - Effects of staggered cross frames and/or skewed supports.

---

§6.10.1 - I-Sections: General

Eccentric Concrete Deck Overhangs

---

---
The weight of the screed goes into the brackets, too.

Lateral Flange Stresses due to Wind ($W_S$)

- Wind pressure acting on a girder is assumed to be evenly distributed to the flanges of the girder.

- When the top flange is braced by a slab, etc., the wind acting on the upper half of the girder is disregarded (i.e. goes directly into the slab).

- The Wind pressure acting on the lower half of the girder is carried by the weak-axis bending in the bottom flange to the cross frames where it is transmitted into the deck.
§6.10.1 - I-Sections: General

§6.10.1.6: Flange Stresses and Moments

Review: §3.8.1 Determination of Horizontal Wind Load on Structure (W/S)

1. Determine Design Velocity, \( V_{DZ} \)
   \[ V_{DZ} = 2.5 V_{n} \left( \frac{V_{o}}{V_{b}} \right) \ln \left( \frac{Z}{Z_{o}} \right) \]  
   (3.8.1.1-1)

2. Determine Design Pressure, \( P_{D} \)
   \[ P_{D} = \frac{V_{DZ}}{V_{b}} \]  
   (3.8.1.2.1-1)

- \( Z \) - Elevation of the bridge above the ground or water level (ft)
- \( Z_{o} \) - Friction length of the upstream fetch (ft) (Table 3.8.1.1-1)
- \( V_{o} \) - Friction velocity (mph) (Table 3.8.1.1-1)
- \( V_{b} \) - Base velocity, taken as 100\text{mph} in the absence of better information
- \( V_{30} \) - velocity at \( Z = 30' \), taken as \( V_{o} \) in the absence of better information
- \( P_{b} \) - base wind pressure, (Table 3.8.1.2.1-1) taken as 50\text{psf} for beams

The total wind loading shall not be taken less than 30\text{lb/ft} on beam spans.

---

§6.10.1 - I-Sections: General

§4.6.2.7.1: Lateral Wind Load (W/S) Distribution

The wind force tributary to the bottom flange may be determined as,

\[ W = \eta \gamma P_{D} \frac{d}{2} \]  
(C4.6.2.7.1-1)

The weak-axis moment in the bottom flange between brace points, where the brace carries the wind load to the deck, may be determined as,

\[ M_{w} = \frac{W L_{b}^{2}}{10} \]  
(C4.6.2.7.1-2)

The weak-axis moment in the bottom flange, where the overall wind load is carried by weak-axis bending of the girder system alone (no deck), may be determined as,

\[ M_{w} = \frac{W L_{b}^{2}}{10} + \frac{W L^{2}}{8N_{s}} \]  
(C4.6.2.7.1-3)
\[ f_l = \frac{M_{w\ell}}{S_{lf}} = \frac{6M_{w\ell}}{t_l b_f} \]

The horizontal wind force applied at the bottom flange to each brace point, \( P_{w\ell} \), may be calculated as:

\[ P_{w\ell} = W L_b \]  

§6.10.1.7: Minimum Negative Concrete Reinforcement

- When the longitudinal tension stress in the concrete deck due to Load Combination Service II exceeds the factored modulus of rupture for the concrete, 1% reinforcement shall be provided in the deck.

- The reinforcement used shall be grade 60 bars not larger than #6.

- The reinforcement should be placed in two layers uniformly distributed across the deck width with 2/3 of the bars placed in the top layer.

- The individual bars shall be placed at intervals not exceeding 12”.

- Where shear connectors are omitted from the negative-moment region, bars shall be tied into the positive moment region (see §6.10.10.3).
§6.10.1 - I-Sections: General

§6.10.1.8: Net Section Fracture

- For flexural members at the Strength Limit State or for constructability, the following shall be satisfied at all sections with holes in the tension flange:

\[
f_i \leq 0.84 \left( \frac{A_t}{A_y} \right) F_u \leq F_{yt}
\]  

(6.10.1.8-1)

- \(f_i\) - Stress on the gross area of the tension flange due to factored loads.
- \(A_t\) - Net area of the tension flange (§6.8.3).
- \(A_y\) - Gross area of the tension flange.

§6.10.1.9: Web Bend-Buckling Resistance

- Elastic Bend-Buckling of Web

\[
F_{crw} = \frac{0.9E_k}{\left( \frac{D}{t_w} \right)^2}
\]  

(6.10.1.9.1-1)

\[
k = \frac{9}{\left( \frac{D_w}{D} \right)^2}
\]  

(6.10.1.9.1-2)

- \(F_{crw}\) - Bend-buckling resistance, not to exceed the smaller of \(R_y F_{yw}\) or \(F_{yw} / 0.7\).
- \(k\) - Bend-buckling coefficient.
- \(D_w\) - Depth of the web in compression (Elastic Section).

*Post buckling strength of the web is not considered under service loads.*
§6.10.1 - I-Sections: General

§6.10.1.10: Flange Strength Reduction

- Hybrid Girder Factor

\[
R_s = \frac{12 + \beta (3 \rho - \rho^2)}{12 + 2 \beta} \tag{6.10.1.10.1-1}
\]

\[
\beta = \frac{2 D_n f_w}{A_{fl}} \tag{6.10.1.10.1-2}
\]

\[
\rho = \frac{F_{yw}}{f_w} \leq 1.0
\]

- Load Shedding Factor (Post-Buckling Strength of Web)

\[
R_s = 1 - \left( \frac{a_{wc}}{1200 + 300 a_{wc}} \right) \left[ \frac{2 D_n f_w}{I_w} - 5.70 \sqrt{\frac{E}{F_{yw}}} \right] \leq 1.0 \tag{6.10.1.10.2-3}
\]

\[
a_{wc} = \frac{2 D_n f_w}{b_{wc} t_{yc}} \tag{6.10.1.10.2-5}
\]
§6.10 - I-Sections: Cross-Section Proportion Limits

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- 6.10.2 Cross-Section Proportion Limits
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Without Longitudinal Stiffeners:

\[
\frac{D}{t_w} \leq 150 \quad (6.10.2.1.1-1)
\]

With Longitudinal Stiffeners:

\[
\frac{D}{t_w} \leq 300 \quad (6.10.2.1.2-1)
\]

- **D**: Depth of the web.
- **\( t_w \)**: Thickness of the web.

Practical upper limit on web slenderness. VFB not explicitly considered. ODOT Prohibits the use of longitudinal stiffeners.
§6.10.2.2: Flange Proportion Limits

\[ \frac{b_f}{2t_f} \leq 12.0 \]
Practical upper limit to preclude flange distortions during welding. (6.10.2.2-1)

\[ b_f \geq \frac{D}{6} \]
Precludes the use of very narrow flanges – limited experimental Data. (6.10.2.2-2)

\[ t_f \geq 1.1t_{wc} \]
Ensures that the flanges can restrain the web during shear buckling. (6.10.2.2-3)

\[ 0.1 \leq \frac{I_{wc}}{I_{w}} \leq 10 \]
Ensures that the section is proportioned and behaves like an I-shape as opposed to a “tee” shape. (6.10.2.2-4)

- \( b_f \): Flange width.
- \( t_f \): Flange thickness.
- \( D \): Depth of the web.
- \( I_{wc} \): Moment of inertia of the compression flange about the vertical axis of the member.
- \( I_{w} \): Moment of inertia of the tension flange about the vertical axis of the member.

BDM §302.4.3.3: Width and Thickness Requirements

- In addition to design limitations of width to thickness, flanges shall be wide enough that the girder will have the necessary lateral strength for handling and erection. An empirical rule is that the minimum width of top flange should be:
  \[ b_f \geq (D/6 + 2.5) \geq 12" \]

- Whenever possible, use constant flange widths throughout the length of the girder.

- The minimum thickness for any girder flange shall be \( \frac{7}{8} " \).

- Generally, flange thicknesses should conform to the following:
  - For material \( \frac{7}{8} " \) to 3" thick, specify thickness in \( \frac{1}{8} " \) increments.
  - For material greater than 3" thick, specify thickness in \( \frac{1}{4} " \) increments.

- The minimum web thickness shall be \( \frac{3}{8} " \)
In determining the points where changes in flange thickness occur, the designer should weigh the cost of butt-welded splices against extra plate thickness. In many cases it may be advantageous to continue the thicker plate beyond the theoretical step-down point to avoid the cost of the butt-welded splice.

The amount of steel that must be saved to justify providing a welded splice should be as follows:

- For A709-36 steel:
  - \[300 \text{ lb} + 25 \text{ lb} \times \text{cross sectional area (in}^2\text{)} \text{ of the lighter flange plate}\]

- For A709-50 & 50W steel, the cutoff is 85% of the value for A709-36.

Consider a 16" x 2" flange transitioning to a 16" by \(\frac{7}{8}\)" flange, A709-50

\[
\left(0.85\right)\left[300\text{lb} + \left(25\frac{\text{lb}}{\text{in}^2}\right)\left(\frac{7}{8}\right)\left(16\right)\right] = 552.5\text{lbs}
\]

The savings in weight is:

\[
\frac{\left(16\right)\left(1\frac{7}{8}\right)}{\left(12\frac{7}{8}\right)^2} \left(490\frac{\text{lb}}{\text{in}}\right) = 61.25\frac{\text{lb}}{\text{ft}} \quad \frac{552.5\text{lbs}}{61.25\frac{\text{lb}}{\text{ft}}} = 9.02\text{'}
\]

So it makes sense to change thickness only if the smaller flange can be used for more than 9'.
§6.10 - I-Sections: Constructability

- 6.10.1 General
- 6.10.2 Cross-Section Proportion Limits

- 6.10.3 Constructability
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  - 6.10.3.2 Flexure
  - 6.10.3.3 Shear
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- 6.10.6 Strength Limit State

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§6.10 - I-Sections: Service Limit State

- 6.10.1 General
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- 6.10.3 Constructability

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  - 6.10.4.2 Permanent Deformations

- 6.10.5 Fatigue and Fracture
- 6.10.6 Strength Limit State
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§6.10 - I-Sections: Service Limit State

§6.10.4.1: Elastic Deformations

- Refers to optional live load deflection criteria in §2.5.2.6
  - “In the absence of other criteria:"
    - Typical:
      \[
      \Delta_{\text{Vehicle Load}} \leq \frac{L}{800} \\
      \Delta_{\text{Vehicle + Pedestrian Load}} \leq \frac{L}{1000}
      \]
    - On Cantilevers:
      \[
      \Delta_{\text{Vehicle Load}} \leq \frac{L}{300} \\
      \Delta_{\text{Vehicle + Pedestrian Load}} \leq \frac{L}{375}
      \]

- Use the \((\text{LL+IM})\) portion of the Service I combination - multiple presence factors apply.
  - Section 6.10.4.1 Refers to Section 2.5.2.6
  - Section 2.5.2.6 includes a reference to Section 3.6.1.3.2

- Section 3.6.1.3.2 reads, “If the owner invokes the optional live load deflection criteria, the deflection should be taken as the larger of:"
  - “That resulting from the design truck alone, or
  - That resulting from 25% of the design truck taken together with the design lane load,”
§6.10 - I-Sections: Service Limit State

§6.10.4.1: Elastic Deformations

- When investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded, and all supporting components should be assumed to deflect equally;
  - For a straight multibeam bridge, this is equivalent to saying that the distribution factor for deflection is equal to the number of lanes divided by the number of beams.

- When investigating maximum relative displacements, the number and position of loaded lanes should be selected to provide the worst differential effect;

---

§6.10 - I-Sections: Service Limit State

§6.10.4.1: Elastic Deformations

- For composite design, the stiffness of the design cross-section used for the determination of deflection should include the entire width of the roadway and the structurally continuous portions of the railings, sidewalks, and median barriers;

*ODOT prohibits the use of “the stiffness contribution of railings, sidewalks and median barriers in the design of the composite section.”*
§6.10 - I-Sections: Service Limit State

§6.10.4.1: Elastic Deformations

- Optional span-to-depth ratios are also provided

Table 2.5.2.6.3-1: Traditional Minimum Depths for Constant Depth Structures

<table>
<thead>
<tr>
<th>Material</th>
<th>Type</th>
<th>Minimum Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Overall Depth of Composite I-Beam</td>
<td>0.040L</td>
</tr>
<tr>
<td>Steel</td>
<td>Steel Depth of Composite I-Beam</td>
<td>0.033L</td>
</tr>
<tr>
<td></td>
<td>Trusses</td>
<td>0.100L</td>
</tr>
</tbody>
</table>

**ODOT states that “designers shall apply the span-to-depth ratios shown.”**

§6.10.4.2: Permanent Deformations

**Composite Sections:**

Top Flange: \[ f_f \leq 0.95R_s F_{sf} \] (6.10.4.2.2-1)

Bottom Flange: \[ f_f + \frac{f_t}{2} \leq 0.95R_s F_{sf} \] (6.10.4.2.2-2)

**Noncomposite Sections:**

Both Flanges: \[ f_f + \frac{f_t}{2} \leq 0.80R_s F_{sf} \] (6.10.4.2.2-3)

\( f_f \): Flange stress due to Service II Combination without consideration to lateral bending.

\( f_t \): Flange stress due to Service II Combination due to lateral bending.
§6.10.4.2: Permanent Deformations

For compact composite sections in positive flexure utilized in shored construction, the longitudinal compressive stress in the concrete deck due to the Service II loads, determined using the short-term composite section, shall not exceed $0.6f'_{c}$.

All sections except composite sections in positive flexure:

Web: $f_c \leq F_{crw}$  \hspace{1cm} (6.10.4.2.2-4)

- $f_c$ - Compression flange stress due to Service II Combination without consideration to lateral bending.
- $F_{crw}$ - Nominal bend-buckling strength of the web (§6.10.1.9).

Post buckling strength of the web is not considered under service loads.
§6.10 - I-Sections: Fatigue and Fracture

- 6.10.1 General
- 6.10.2 Cross-Section Proportion Limits
- 6.10.3 Constructability
- 6.10.4 Service Limit State

- 6.10.5 Fatigue and Fracture
  - 6.10.5.1 Fatigue
  - 6.10.5.2 Fracture
  - 6.10.5.3 Special Fatigue Requirements for Webs

- 6.10.6 Strength Limit State
- 6.10.7 Flexural Resistance: Composite Sections in Positive Flexure
- 6.10.8 Flexural Resistance: Composite Sections in Negative Flexure and Noncomposite Sections
- 6.10.9 Shear Strength
- 6.10.10 Shear Connectors
- 6.10.11 Stiffeners
- 6.10.12 Cover Plates

Check elastic flexing of the web under shear loads... We'll cover this later.
§6.10 - I-Sections: Fatigue and Fracture

C6.4.3 Flowchart for LRFD Article 6.10.5

Figure C6.4.3-1 Flowchart for LRFD Article 6.10.5—Fatigue and Fracture Limit State.

§6.10 - I-Sections: Strength Limit State (1 of 5)

- 6.10.1 General
- 6.10.2 Cross-Section Proportion Limits
- 6.10.3 Constructability
- 6.10.4 Service Limit State
- 6.10.5 Fatigue and Fracture
- 6.10.6 Strength Limit State
  - 6.10.6.1 General
  - 6.10.6.2 Flexure
  - 6.10.6.3 Shear
  - 6.10.6.4 Shear Connectors
- 6.10.7 Flexural Resistance: Composite Sections in Positive Flexure
- 6.10.8 Flexural Resistance: Composite Sections in Negative Flexure and Noncomposite Sections
- 6.10.9 Shear Strength
- 6.10.10 Shear Connectors
- 6.10.11 Stiffeners
- 6.10.12 Cover Plates
§6.10 - I-Sections: Strength Limit State

§6.10.6.2: Strength Limit State: Flexure

- **Work With Stresses When…**
  - The beam is behaving elastically (i.e. local buckling, lateral buckling)

- **Work With Moments When…**
  - The beam is behaving plastically (i.e. plastic moment, composite sections, moment redistribution)

---

§6.10 - I-Sections: Strength Limit State

§6.10.6.2: Strength Limit State: Flexure

- **Section Classification**

  - **Compact if:**
    \[
    \frac{2D_c}{t_w} \leq 3.76 \sqrt{\frac{E}{F_{yc}}} \tag{6.10.6.2.2-1}
    \]

  - **Nonslender:**
    \[
    \frac{2D_n}{t_w} \leq 5.70 \sqrt{\frac{E}{F_{yc}}} \tag{6.10.6.2.3-1}
    \]
    and
    \[
    \frac{I_{wc}}{I_{wc}} \geq 0.3 \tag{6.10.6.2.3-2}
    \]

  - **Noncompact if:**
    \[
    \frac{2D_n}{t_w} \leq 5.70 \sqrt{\frac{E}{F_{yc}}} \tag{6.10.6.2.3-1}
    \]
    and
    \[
    \frac{I_{wc}}{I_{wc}} \geq 0.3 \tag{6.10.6.2.3-2}
    \]

  - **Otherwise Slender**

*Symbols:*
- \(D_c\) - Depth of Web in Compression (@ Plastic Moment).
- \(D_n\) - Depth of Web in Compression (Elastic).
- \(F_{yc}\) - Yield Stress of Compression Flange.
§6.10 - I-Sections: Strength Limit State

§6.10.6.2: Strength Limit State: Flexure

- **Composite Sections in Positive Flexure**
  - Compact Sections → §6.10.7.1 Moments Upper Bound: $M_p$
  - Noncompact Sections → §6.10.7.2 Stresses Upper Bound: $M_y$

- **Composite Sections in Negative Flexure and Noncomposite Sections**
  - Nonslender Sections → §App A Moments Upper Bound: $M_p$
  - Slender Sections → §6.10.8 Stresses Upper Bound: $M_y$

*Optional*

Sections with $F_y > 70$ksi are limited to their yield moment, $M_y$. 

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§6.10 - I-Sections: Strength Limit State

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§6.10 - I-Sections: Composite Sections in Pos Flexure

- 6.10.1 General
- 6.10.2 Cross-Section Proportion Limits
- 6.10.3 Constructability
- 6.10.4 Service Limit State
- 6.10.5 Fatigue and Fracture
- 6.10.6 Strength Limit State

- 6.10.7 Flexural Resistance: Composite Sections in Positive Flexure
  - 6.10.7.1 Compact Sections
  - 6.10.7.2 Noncompact Sections
  - 6.10.7.3 Ductility Requirement

- 6.10.8 Flexural Resistance: Composite Sections in Negative Flexure and Noncomposite Sections
- 6.10.9 Shear Strength
- 6.10.10 Shear Connectors
- 6.10.11 Stiffeners
- 6.10.12 Cover Plates

---

§6.10.7.1: Compact Sections

- for Compact Sections
  At the Strength Limit State, the section must satisfy:

\[
M_a + \frac{1}{3} f_{fl} S_{xt} \leq \phi M_a
\]

\[
(6.10.7.1.1-1)
\]

- \( M_a \): Major-axis bending moment due to factored loads (§6.10.1.6).
- \( f_{fl} \): Flange lateral bending stress (§6.10.1.6).
- \( S_{xt} \): Major-axis elastic section modulus to the tension flange. \( M_a / F_{yt} \)
§6.10 - I-Sections: Composite Sections in Pos Flexure

§6.10.7.1: Compact Sections

- **for Compact Sections**

  If \( D_p \leq 0.1D_t \), then:
  \[
  M_n = M_p \tag{6.10.7.1.2-1}
  \]

  Otherwise:
  \[
  M_n = M_p \left( 1.07 - 0.7 \frac{D_p}{D_t} \right) \tag{6.10.7.1.2-2}
  \]

  For continuous spans:
  \[
  M_n \leq 1.3R_hM_y \tag{6.10.7.1.2-3}
  \]

\( D_p \) - Distance from the top of the concrete deck to the PNA of the composite section.
\( D_t \) - Total depth of the composite section.
\( M_p \) - Plastic moment of the composite section (App. D6.1).
\( M_y \) - Yield moment (App. D6.2).
\( R_h \) - Hybrid girder factor.

§6.10.7.2: Noncompact Sections

- **for Noncompact Sections at the Strength Limit State**, the compression flange must satisfy:

  \[
  f_{bcu} \leq \phi_f F_{nc} \tag{6.10.7.2.1-1}
  \]

  where:

  \[
  F_{nc} = R_{cc}R_{fc}F_{yr} \tag{6.10.7.2.2-1}
  \]

  \( f_{bcu} \) - Flange stress calculated without consideration of lateral bending (§6.10.1.6).
§6.10 - I-Sections: Composite Sections in Pos Flexure

§6.10.7.2: Noncompact Sections

- for Noncompact Sections at the Strength Limit State, the tension flange must satisfy:

\[ f_{tu} + \frac{1}{3} f_l \leq \phi f_w \]  

\[ (6.10.7.2.1-2) \]

where:

\[ F_w = R_s F_c \]  

\[ (6.10.7.2.2-2) \]

- \( f_{tu} \): Flange stress calculated without consideration of lateral bending (§6.10.1.6).
- \( f_l \): Flange lateral bending stress (§6.10.1.6).

§6.10.7.3: Ductility Requirement

- Compact and Noncompact sections shall satisfy:

\[ D_p \leq 0.42 D_f \]  

\[ (6.10.7.3-1) \]

This limit is required to avoid premature crushing of the concrete slab.

**ODOT Exception:** The haunch should not be included in \( D_p \) and \( D_f \).
§6.10 - I-Sections: Composite Sections in Pos Flexure

6.10.1 General
6.10.2 Cross-Section Proportion Limits
6.10.3 Constructability
6.10.4 Service Limit State
6.10.5 Fatigue and Fracture
6.10.6 Strength Limit State
6.10.7 Flexural Resistance: Composite Sects in Pos Flexure

6.10.8 Flexural Resistance: Composite Sections in Negative Flexure and Noncomposite Sections
   - 6.10.8.1 General
   - 6.10.8.2 Compression Flange Flexural Resistance
   - 6.10.8.3 Tension Flange Flexural Resistance

6.10.9 Shear Strength
6.10.10 Shear Connectors
6.10.11 Stiffeners
6.10.12 Cover Plates
§6.10 - I-Sections: Comp Sections in Neg Flexure / Noncomp Sections (2 of 16)

§6.10.8.1: General

At the Strength Limit State:

- Sections with Discretely Braced Compression Flanges

\[ f_{bw} + \frac{1}{3} f_t \leq \phi_f F_{bw} \]  

(6.10.8.1.1-1)

- Sections with Discretely Braced Tension Flanges

\[ f_{bw} + \frac{1}{3} f_t \leq \phi_f F_{bw} \]  

(6.10.8.1.2-1)

- Sections with Continuously Braced Flange in Tension or Compression

\[ f_{bw} \leq \phi_f R_y F_{yy} \]  

(6.10.8.1.3-1)

\( F_{nc} \) - Nominal Flexural Resistance for the Compression Flange from §6.10.8.2
\( F_{nt} \) - Nominal Flexural Resistance for the Tension Flange from §6.10.8.3

§6.10 - I-Sections: Comp Sections in Neg Flexure / Noncomp Sections (3 of 16)

§6.10.8.2: Compression-Flange Flexural Resistance

- Compression Flange Local Buckling

If \( \lambda_t \leq \lambda_{ypt} \), then:

\[ F_{nc} = R_y R_x F_{yc} \]  

(6.10.8.2.2-1)

Otherwise:

\[ F_{nc} = \left[ 1 - \frac{F_{yr}}{R_x F_{yc}} \left( \frac{\lambda_f - \lambda_{yf}}{\lambda_y - \lambda_{yf}} \right) \right] R_y R_x F_{yc} \]  

(6.10.8.2.2-2)

where:

\[ F_{yr} = \min \left( 0.7 F_{yc}, F_{yw} \right) \geq 0.5 F_{yc} \]  

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§6.10 - I-Sections: Comp Sections in Neg Flexure / Noncomp Sections (4 of 16)

§6.10.8.2: Compression-Flange Flexural Resistance

$\lambda_f = \frac{b_c}{2t_c}$  \hspace{2cm} (6.10.8.2.2-3)

$\lambda_{yr} = 0.38 \sqrt{\frac{E}{F_{yc}}}$ \hspace{2cm} (6.10.8.2.2-4)

$\lambda_{cr} = 0.56 \sqrt{\frac{E}{F_{yc}}}$ \hspace{2cm} (6.10.8.2.2-5)

- $b_c$: Width of Compression Flange.
- $t_c$: Thickness of Compression Flange.
- $F_{yc}$: Yield Stress of Compression Flange.
- $F_{yr}$: Yield Stress of Compression Flange – Residual Stress

Elastic Flange Local Buckling is not explicitly considered since it is precluded for $F_{yc} \leq 90 \text{ksi}$ by Eqn 6.10.2.2-1 in the General Limitations.

§6.10 - I-Sections: Comp Sections in Neg Flexure / Noncomp Sections (5 of 16)

§6.10.8.2: Compression-Flange Flexural Resistance

- **Compression Flange Lateral-Torsional Buckling**

  If $L_b \leq L_p$, then:

  $$F_{nc} = R_p R_{cr} F_{yc}$$ \hspace{2cm} (6.10.8.2.3-1)

  If $L_p < L_b \leq L_r$, then:

  $$F_{nc} = C_b \left[ 1 - \left( 1 - \frac{F_{yc}}{R_r F_{cr}} \right) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] R_p R_{cr} F_{yc} \leq R_p R_{cr} F_{yc}$$ \hspace{2cm} (6.10.8.2.3-2)

  Otherwise:

  $$F_{nc} = F_{cr} \leq R_p R_{cr} F_{yc}$$ \hspace{2cm} (6.10.8.2.3-3)
§6.10.8.2: Compression-Flange Flexural Resistance

\[ L_p = 1.0 r_p \sqrt{\frac{E}{F_{c'}}} \]  
\[ L_y = \pi r_y \sqrt{\frac{E}{F_{y'}}} \]  
\[ F_{cr} = \frac{C_r R_y \pi^2 E}{(L_p / r_p)^2} \]  
\[ r_i = \sqrt{\frac{b_{iy}}{12\left(1 + \frac{L_p}{b_w t_w} \frac{D_i}{b_{iy} t_{iy}}\right)}} \]

§6.10.8.2: Moment Gradient Modifier

- In General:
  \[ C_s = 1.75 - 1.05 \left(\frac{f_{1i}}{f_{2i}}\right) + 0.3 \left(\frac{f_{1i}}{f_{2i}}\right)^2 \leq 2.3 \]  

- For Unbraced Cantilevers and Members where \( f_{mid} / f_2 > 1 \) or \( f_2 = 0 \)
  \[ C_s = 1.00 \]
§6.10 - I-Sections: Comp Sections in Neg Flexure / Noncomp Sections (8 of 16)

§6.10.8.2: Moment Gradient Modifier

\( f_2 \) - except as noted below, largest compressive stress without consideration of lateral bending at either end of the unbraced length of the flange under consideration, calculated from the critical moment envelope value.

\( f_2 \) shall be due to the factored loads and shall be taken as positive. If the stress is zero or tensile in the flange under consideration at both ends of the unbraced length, \( f_2 \) shall be taken as zero.

\( f_0 \) - stress without consideration of lateral bending at the brace point opposite to the one corresponding to \( f_2 \), 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.

\( f_0 \) shall be due to the factored loads and shall be taken as positive in compression and negative in tension.

§6.10.8.2: Moment Gradient Modifier

\( f_{mid} \) - 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.

\( f_{mid} \) shall be due to the factored loads and shall be taken as positive in compression and negative in tension.
§6.10 - I-Sections: Comp Sections in Neg Flexure / Noncomp Sections

§6.10.8.2: Moment Gradient Modifier

- \( f_1 \) - stress without consideration of lateral bending at the brace point opposite to the one corresponding to \( f_2 \), calculated as the intercept of the most critical assumed linear stress variation passing through \( f_2 \) and either \( f_{\text{mid}} \) or \( f_0 \), whichever produces the smaller value of \( C_b \).

\( f_1 \) may be determined as follows:

- When the variation in the moment along the entire length between the brace points is concave in shape:
  \[
  f_1 = f_0 \quad (6.10.8.2.3-10)
  \]

- Otherwise:
  \[
  f_1 = 2f_{\text{mid}} - f_2 \geq f_0 \quad (6.10.8.2.3-11)
  \]
§6.10 - I-Sections: Comp Sections in Neg Flexure / Noncomp Sections

§6.10.8.2: Moment Gradient Modifier

- Strict application of the $C_g$ provisions would require the consideration of the concurrent moments along the unbraced length.

- However, since concurrent moments are normally not tracked in the analysis, it is convenient and always conservative to use the worst-case moment values to compute the above stresses.

- The worst-case moment for calculation of $f_s$ is the critical envelope value, or the moment causing the largest value of $f_s$ in the flange under consideration.

- The worst case moments used to compute $f_{o}$ and $f_{mid}$ are the values obtained from the moment envelopes that produce the largest compressive stress, or the smallest tensile stress if the point is never in compression, within the flange under consideration at each of these locations.
§6.10.8.2: Moment Gradient Modifier

- For unbraced lengths containing a transition to a smaller section at a distance greater than 20% of $L_b$ from the brace point with the smaller moment, the lateral-torsional buckling resistance should be taken as the smallest resistance, $F_{nc}$, within the unbraced length under consideration.

- This resistance is to be compared to the largest value of the compressive stress due to the factored loads, $f_{nu}$, throughout the unbraced length calculated using the actual properties of the section.

- The moment gradient modifier, $C_m$, should be taken equal to 1.0 in this case and $L_b$ should not be modified by an effective length factor. A suggested procedure to provide a more refined estimate of the lateral-torsional buckling resistance for this case is presented in Grubb and Schmidt (2004).

$§6.10.8.3: Tension-Flange Flexural Resistance$

- Tension Flange Yielding

$$F_{nu} = R_n F_{yf} \quad (6.10.8.3-1)$$
§6.10 - I-Sections: Comp Sections in Neg Flexure / Noncomp Sections (16 of 16)

Figure 6.44 - Flowchart for Article 6.10.1 - Composite Sections in Negative Flexure and Noncomposite Sections.

Chapter 6 Appendices

- **App A6**  Pgs. 212-223  Post Elastic Moment Capacity
- **App B6**  Pgs. 224-234  Inelastic Moment Redistribution
- **App C6**  Pgs. 235-239  Step-by-step Instructions
  
- **App D6**  Pgs. 250-256  Fundamentals of Flexure
§A6 - I-Sections: Post Elastic Strength

- **A6.1 General**
- **A6.2 Web Plastification Factors**
- **A6.3 Flexural Resistance Based on the Compression Flange**
- **A6.4 Flexural Resistance Based on the Tension Flange**

**Composite Sections in Positive Flexure**
- Compact Sections \(\rightarrow \) §6.10.7.1 Moments Upper Bound: \(M_p\)
- Noncompact Sections \(\rightarrow \) §6.10.7.2 Stresses Upper Bound: \(M_y\)

**Composite Sections in Negative Flexure and Noncomposite Sections**
- Nonslender Sections \(\rightarrow \) §App A Moments Upper Bound: \(M_p\)
- Slender Sections \(\rightarrow \) §6.10.8 Stresses Upper Bound: \(M_y\)

*Optional*

*Potential Gains in Strength Decrease with Increasing Web Slenderness.*

---

§A6 - I-Sections: Post Elastic Strength

**§A6.1: General**

- Appendix A6 can be applied to sections where:
  - \(F_y \leq 70\text{kpsi}\) for the web and flanges
  - The web is not slender, i.e.

\[
\frac{2D}{t_w} < 5.7 \sqrt{\frac{E}{F_{yc}}}
\]  

\text{(A6.1-1)}

and

- The flanges satisfy the following:

\[
\frac{I_{xc}}{I_{yt}} \geq 0.3
\]  

\text{(A6.1-2)}

---

---
§A6 - I-Sections: Post Elastic Strength

§A6.1: General

At the Strength Limit State:

- Sections with Discretely Braced Compression Flanges

\[ M_x + \frac{1}{3} f_S S_{xc} \leq \phi M_{nc} \]  

(A6.1.1-1)

- Sections with Discretely Braced Tension Flanges

\[ M_x + \frac{1}{3} f_S S_{xt} \leq \phi M_{nt} \]  

(A6.1.2-1)

- Sections with Continuously Braced Compression Flanges

\[ M_x \leq \phi R_{pc} M_{yc} \]  

(A6.1.3-1)

- Sections with Continuously Braced Tension Flanges

\[ M_x \leq \phi R_{pt} M_{yt} \]  

(A6.1.4-1)

\( M_{nc} \) - Nominal Moment Capacity for the Compression Flange from §A6.3.
\( M_{nt} \) - Nominal Moment Capacity for the Tension Flange from §A6.4.
\( R_{pc} \) - Web Plastification Factor for the Compression Flange.
\( R_{pt} \) - Web Plastification Factor for the Tension Flange.
\( M_{yc} \) - Yield Moment for the Compression Flange.
\( M_{yt} \) - Yield Moment for the Tension Flange.
§A6 - I-Sections: Post Elastic Strength

§A6.2: Web Plastification Factors

- **Web Classification**

  Compact if:
  \[
  \frac{2D_w}{t_w} \leq \lambda_{yw(D_p)}
  \]  
  \(A6.2.1-1\)

  Noncompact if:
  \[
  \frac{2D_w}{t_w} \leq \lambda_{yw}
  \]  
  \(A6.2.2-1\)

  \(D_p\) - Depth of Web in Compression (@ Plastic Moment).
  \(D_w\) - Depth of Web in Compression (Elastic).
  \(F_y\) - Yield Stress of Compression Flange.

  **Similar to §6.10.6.2 Except for More Stringent Limits on \(\lambda_p\)**

- **Web Classification**

  \[
  \lambda_{yw(D_p)} = \left( \frac{E}{F_y} \right)^{\frac{1}{2}} \leq \lambda_{yw}\left( \frac{D_p}{D_c} \right)
  \]  
  \(A6.2.1-2\)

  \[
  \lambda_{yw} = 5.7 \left( \frac{E}{F_y} \right)^{\frac{1}{2}}
  \]  
  \(A6.2.1-3\)

  \(D_p\) - Depth of Web in Compression (@ Plastic Moment).
  \(D_c\) - Depth of Web in Compression (Elastic).
  \(F_y\) - Yield Stress of Compression Flange.
§A6 - I-Sections: Post Elastic Strength

§A6.2: Web Plastification Factors

- For Compact Webs

\[ R_{pc} = \frac{M_p}{M_{pc}} \]  
(A6.2.1-4)

\[ R_{pt} = \frac{M_p}{M_{pt}} \]  
(A6.2.1-5)

- For Noncompact Webs

\[ R_{pc} = \left[ 1 - \left( 1 - \frac{R_{M_{pc}}}{M_p} \right) \left( \frac{\lambda_{yw} - \lambda_{pw}(D_t)}{\lambda_{yw}} \right) \right] \frac{M_p}{M_{pc}} \leq \frac{M_p}{M_{pc}} \]  
(A6.2.2-4)

\[ R_{pt} = \left[ 1 - \left( 1 - \frac{R_{M_{pt}}}{M_p} \right) \left( \frac{\lambda_{yt} - \lambda_{pt}(D_t)}{\lambda_{yt}} \right) \right] \frac{M_p}{M_{pt}} \leq \frac{M_p}{M_{pt}} \]  
(A6.2.2-5)

\[ \lambda_w = \frac{2D}{I_w} \quad \lambda_{pw}(D_t) = \lambda_{pw}(D_{ct}) \left( \frac{D_t}{D_{ct}} \right) \leq \lambda_{yw} \]  
(A6.2.2-6)
§A6 - I-Sections: Post Elastic Strength

§A6.3.1: Compression Flange Capacity

- The Flexural Resistance Based on the Compression Flange Shall be Taken as the Smaller of:
  - Compression Flange Local Buckling Strength (§A6.3.2)
  - Lateral-Torsional Buckling Strength (§A6.3.3)

\[
M_{nc} = R_{nc} M_{pc}
\]  
\[M_{nc} = \left[1 - \left(1 - \frac{F_{nc} S_{nc}}{R_{pc} M_{pc} \lambda f_{nc}^2} \right)\lambda f_{nc}^2\right] R_{pc} M_{pc}
\]

\[
\lambda f = \frac{b_{fc}}{2t_{fc}}
\]
§A6 - I-Sections: Post Elastic Strength

§A6.3.2: Compression Flange Local Buckling Strength

- Flange Classification:

  Compact if: \[ \frac{\lambda_f}{\lambda_p} \leq 0.38 \frac{E}{F_{yc}} \] (A6.3.2-4)

  Noncompact if: \[ \frac{\lambda_f}{\lambda_p} \leq 0.95 \frac{E}{F_{yc}} \] (A6.3.2-5)

\[ k_c = \frac{4D}{\sqrt{I_w}} \], \( 0.35 \leq k_c \leq 0.76 \) (A6.3.2-6)

For Rolled Shapes, Take \( k_c = 0.76 \)

§A6 - I-Sections: Post Elastic Strength

§A6.3.3: Lateral-Torsional Buckling Strength

- When \( L_b \leq L_p \) (Plastic Moment):

\[ M_{wc} = R_{m_c} M_{yc} \] (A6.3.3-1)

- When \( L_p < L_b \leq L_t \) (Inelastic LTB):

\[ M_{wc} = C_p \left[ 1 - \left( 1 - \frac{F_{yc} S_{wc}}{R_{m_c} M_{yc}} \left( \frac{L_p}{L_t} - 1 \right) \right) \right] R_{m_c} M_{yc} \leq R_{m_c} M_{yc} \] (A6.3.3-2)

- When \( L_t < L_b \) (Elastic LTB):

\[ M_{wc} = F_{yc} s_{wc} \leq R_{m_c} M_{yc} \] (A6.3.3-3)
§A6 - I-Sections: Post Elastic Strength

§A6.3.3: Lateral-Torsional Buckling Strength

where:

\[ L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} \]  
(A6.3.3-4)

\[ L_y = 1.95 r_x \frac{E}{F_{yc}} \sqrt{\frac{J}{S_y h}} \left[ 1 + \frac{1 + 6.76 F_{yc} S_y h}{E J} \right] \]  
(A6.3.3-5)

\[ F_{yr} = \frac{C_e \pi^2 E}{(L_y / r_y)^2} \left[ 1 + 0.078 \frac{J}{S_y h} \left( \frac{L_y}{r_y} \right)^2 \right] \]  
(A6.3.3-8)

\[ F_{yr} = \min \left( 0.7 F_{yc}, R_y F_{Fy} \frac{S_y}{S_{yc}} F_{yc} \right) \geq 0.5 F_{yc} \]  
(Pg 6-222)

Inconsistent Definition of \( F_{yr} \)!!! (Relative to §6.10.8, Pg 6-109)


§A6 - I-Sections: Post Elastic Strength

§A6.3.3: Lateral-Torsional Buckling Strength

where:

\[ J = \frac{D t_w^3}{3} + b_x t_x^3 \left( 1 - 0.63 \frac{t_x}{b_x} \right) + b_y t_y^3 \left( 1 - 0.63 \frac{t_y}{b_y} \right) \]  
(A6.3.3-9)

\[ r_t = \frac{b_y}{\sqrt{12 \left[ 1 + \frac{1}{3} \frac{D t_w}{b_x t_w} \right]}} \]  
(A6.3.3-10)

\( h = \) Depth Between Centerlines of Flanges (in)
§A6 - I-Sections: Post Elastic Strength

§A6.3.3: Lateral-Torsional Buckling Strength

- Moment Gradient Modifier
  - In General:
    \[
    C_s = 1.75 - 1.05 \left( \frac{M_L}{M_2} \right) + 0.3 \left( \frac{M_L}{M_2} \right)^2 \leq 2.3 \tag{A6.3.3-7}
    \]
  - For Unbraced Cantilevers and Members where \( M_{mid} / M_2 > 1 \) or \( M_2 = 0 \)
    \[
    C_s = 1.00 \tag{A6.3.3-6}
    \]
§A6 - I-Sections: Post Elastic Strength

§A6.4: Tension Flange Yielding

- The Nominal Flexural Resistance Based on Tension Flange Yielding Shall be Taken as

\[ M_{nt} = R_{pl} M_{fs} \]  

(A6.4-1)
§6.10 - I-Sections: Constructability

- 6.10.1 General
- 6.10.2 Cross-Section Proportion Limits

- 6.10.3 Constructability
  - 6.10.3.1 General
  - 6.10.3.2 Flexure
  - 6.10.3.3 Shear
  - 6.10.3.4 Deck Placement
  - 6.10.3.5 Dead Load Deflection

- 6.10.4 Service Limit State
- 6.10.5 Fatigue and Fracture
- 6.10.6 Strength Limit State
- 6.10.7 Flexural Resistance: Composite Sections in Positive Flexure
- 6.10.8 Flexural Resistance: Composite Sections in Negative Flexure and Noncomposite Sections
- 6.10.9 Shear Strength
- 6.10.10 Shear Connectors
- 6.10.11 Stiffeners
- 6.10.12 Cover Plates

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§6.10.3.1: General

The provisions of Article 2.5.3 shall apply.

- Constructibility issues should include, but not be limited to, consideration of deflection, strength of steel and concrete, and stability during critical stages of construction.

- Bridges should be designed in a manner such that fabrication and erection can be performed without undue difficulty or distress and that locked-in construction force effects are within tolerable limits.

- When the designer has assumed a particular sequence of construction in order to induce certain stresses under dead load, that sequence shall be defined in the contract documents.

- Where there are, or are likely to be, constraints imposed on the method of construction by environmental considerations or for other reasons, attention shall be drawn to those constraints in the contract documents.
§6.10 - I-Sections: Constructability

§6.10.3.1: General

- The provisions of Article 2.5.3 shall apply.
  - Where the bridge is of unusual complexity, such that it would be unreasonable to expect an experienced contractor to predict and estimate a suitable method of construction while bidding the project, at least one feasible construction method shall be indicated in the contract documents.
  - If the design requires some strengthening and/or temporary bracing or support during erection by the selected method, indication of the need thereof shall be indicated in the contract documents.
  - Details that require welding in restricted areas or placement of concrete through congested reinforcing should be avoided.
  - Climatic and hydraulic conditions that may affect the construction of the bridge shall be considered.

- For investigating the constructibility of flexural members, all loads shall be factored as specified in Article 3.4.2 (Load Factors for Construction Loads).
  - All appropriate strength load combinations in Table 3.4.1-1, modified as specified herein, shall be investigated.
  - When investigating Strength Load Combinations I, III, and V during construction, load factors for the weight of the structure and appurtenances, DC and DW, shall not be taken to be less than 1.25.
  - Unless otherwise specified by the Owner, the load factor for construction loads and for any associated dynamic effects shall not be less than 1.5 in Strength Load Combination I. The load factor for wind in Strength Load Combination III shall not be less than 1.25.

- For the calculation of deflections, load factors shall be taken as 1.0.
§6.10 - I-Sections: Constructability

§6.10.3.1: General

- In addition to providing adequate strength, nominal yielding or reliance on post-buckling resistance shall not be permitted for main load-carrying members during critical stages of construction, except for yielding of the web in hybrid sections. This shall be accomplished by satisfying the requirements of Articles 6.10.3.2 (Flexural Strength) and 6.10.3.3 (Shear Strength) at each critical construction stage.

- Potential uplift at bearings shall be investigated at each critical construction stage.

- Webs without bearing stiffeners at locations subjected to concentrated loads not transmitted through a deck or deck system shall satisfy the provisions of Article D6.5.

- If there are holes in the tension flange at the section under consideration, the tension flange shall also satisfy the requirement specified in Article 6.10.1.8.

- Load-resisting bolted connections either in or to flexural members shall be proportioned to prevent slip under the factored loads at each critical construction stage. The provisions of Article 6.13.2.8 shall apply for investigation of connection slip.
§6.10 - I-Sections: Constructability

§6.10.3.2: Flexure

Discretely Braced Compression Flanges

- For critical stages of construction, each of the following requirements shall be satisfied. For sections with slender webs, Eq. 1 shall not be checked when $f_t$ is equal to zero. For sections with compact or noncompact webs, Eq. 3 shall not be checked.

Check for Flange Yielding: $f_{bu} + f_t \leq \phi f_y R_y F_{yf}$  \hspace{1cm} (6.10.3.2.1-1)

Check for LTB and FLB: $f_{bu} + \frac{1}{3} f_t \leq \phi f_{crw}$  \hspace{1cm} (6.10.3.2.1-2)

Check for Web Bend-Buckling: $f_{bw} \leq \phi f_{crw}$  \hspace{1cm} (6.10.3.2.1-3)

Appendix A can be used to check for LTB and/or FLB for Constructability.

Pg 6.102-103

§6.10 - I-Sections: Constructability

§6.10.3.2: Flexure

- Discretely Braced Tension Flanges

Check for Flange Yielding: $f_{bu} + f_t \leq \phi f_y R_y F_{yf}$  \hspace{1cm} (6.10.3.2.2-1)

- Continuously Braced Tension or Compression Flanges

Check for Flange Yielding: $f_{bw} \leq \phi f_y R_y F_{yf}$  \hspace{1cm} (6.10.3.2.3-1)
§6.10.3.3: Shear

Interior panels of webs with transverse stiffeners shall satisfy the following requirement during critical stages of construction:

\[ V_u \leq \phi V_{cr} \]  

(6.10.3.3-1)

\( V_u \) - shear in the web at the section under consideration due to the factored permanent loads and factored construction loads applied to the non-composite section

The nominal shear resistance for this check is limited to the shear yielding or shear-buckling resistance. The use of tension-field action is not permitted under these loads during construction.

(Use of tension-field action is permitted after the deck has hardened or is made composite, if tension-field action is permitted in Section 6.10.9)

§6.10.3.4: Deck Placement

Sections in positive flexure that are composite in the final condition, but are non-composite during construction, shall be investigated for flexure during the various stages of the deck placement using the geometric properties, bracing lengths and stresses used in calculating the nominal flexural resistance shall be for the steel section only.
§6.10 - I-Sections: Constructability

§6.10.3.4: Deck Placement

- Changes in load, stiffness and bracing during the various stages of the deck placement shall be considered.
  - The entire concrete deck may not be placed in one stage; thus, parts of the girders may become composite in sequential stages.
  - If certain deck placement sequences are followed, the temporary moments induced in the girders during the deck placement can be considerably higher than the final non-composite dead load moments after the sequential placement is complete.

\[ b_f \geq \frac{L}{85} \]

where \( L \) is the length of the shipping piece (in).
§6.10 - I-Sections: Constructability

§6.10.3.4: Deck Placement

- The effects of forces from deck overhang brackets acting on the fascia girders shall be considered
  - The applied torsional moments bend the exterior girder top flanges outward. The resulting flange lateral bending stresses tend to be largest at the brace points at one or both ends of the unbraced length. The lateral bending stress in the top flange is tensile at the brace points on the side of the flange opposite from the brackets. These lateral bending stresses should be considered in the design of the flanges.
§6.10 - I-Sections: Constructability

§6.10.3.4: Deck Placement

- The effects of forces from deck overhang brackets acting on the fascia girders shall be considered
  - The horizontal components of the reactions on the cantilever-forming brackets are often transmitted directly onto the exterior girder web. The girder web may exhibit significant plate bending deformations due to these loads. The effect of these deformations on the vertical deflections at the outside edge of the deck should be considered. The effect of the reactions from the brackets on the cross-frame forces should also be considered.

  - Excessive deformation of the web or top flange may lead to excessive deflection of the bracket supports causing the deck finish to be problematic.

- Where practical, forming brackets should be carried to the intersection of the bottom flange and the web.

- Alternatively, the brackets may bear on the girder webs if means are provided to ensure that the web is not damaged and that the associated deformations permit proper placement of the concrete deck.

- The provisions of Article 6.10.3.2 allow for the consideration of the flange lateral bending stresses in the design of the flanges.
§6.10 - I-Sections: Constructability

§6.10.3.4: Deck Placement

- In the absence of a more refined analysis, either of the following equations may be used to estimate the maximum flange lateral bending moments due to the eccentric loadings depending on how the lateral load is assumed applied to the top flange:

\[ M_i = \frac{F_i L_b^2}{12} \quad (C6.10.3.4-1) \]

\[ M_i = \frac{P_i L_b}{8} \quad (C6.10.3.4-2) \]

- The magnitude and application of the overhang loads assumed in the design should be shown in the contract documents.

§6.10 - I-Sections: Constructability

§6.10.3.5: Dead Load Deflections

From §6.7.2: Dead Load Camber:

- Steel structures should be cambered during fabrication to compensate for dead load deflection and vertical alignment. Deflection due to steel weight and concrete weight shall be reported separately.

- Deflections due to future wearing surfaces or other loads not applied at the time of construction shall be reported separately.

- Vertical camber shall be specified to account for the computed dead load deflection.

- If staged construction is specified, the sequence of load application should be recognized in determining the camber and stresses.
§6.10 - I-Sections: Constructability

Figure: A4.41 - Flowchart for LRFD Table AL4.3

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§6.10 - I-Sections: Shear Strength

- 6.10.1 General
- 6.10.2 Cross-Section Proportion Limits
- 6.10.3 Constructability
- 6.10.4 Service Limit State
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- 6.10.7 Flexural Resistance: Composite Sects in Pos Flexure
- 6.10.8 Flexural Resistance: Composite Sections in Negative Flexure and Noncomposite Sections

- 6.10.9 Shear Strength
  - 6.10.9.1 General
  - 6.10.9.2 Nominal Resistance of Unstiffened Webs
  - 6.10.9.3 Nominal Resistance of Stiffened Webs

- 6.10.10 Shear Connectors
- 6.10.11 Stiffeners
- 6.10.12 Cover Plates
§6.10 - I-Sections: Shear Strength

Theoretical Basis: Shear Strength

\[ V_s = V_{wp} = CV_p \]  

(6.10.9.2-1)

- \( C \) - Ratio of shear buckling strength to shear yielding strength
- \( V_p \) - Plastic shear force, \( 0.58 F_{yw} D t_w \)

Theoretical Basis: Shear Buckling
§6.10 - I-Sections: Shear Strength

**Theoretical Basis: Shear Buckling**

**In General,**

\[ \tau_{cr} = \frac{\pi^2 k_r E}{12(1-v^2)(D/t_o)^2} \]

Define \( C \) as the ratio of buckling strength to yielding strength

\[ C = \frac{V_{cr}}{V_p} = \frac{\tau_{cr}}{0.58 F_{yw}} \]

\[ = \frac{\pi^2 k_r E}{12(1-v^2)(D/t_o)^2(0.58 F_{yw})} = \frac{1.57 k_r E}{(D/t_o)^2 F_{yw}} \]

---

§6.10 - I-Sections: Shear Strength

**Theoretical Basis: Shear Buckling**

**Theoretical Solution:**

when \( d_o \geq D \):

\[ k_r = 4.0 + \frac{5.34}{(d_o / D)^2} \]

when \( d_o \leq D \):

\[ k_r = \frac{4.0}{(d_o / D)^2} + 5.34 \]

**Practical Solution:**

Split the difference...

\[ k_r = 5 + \frac{5}{(d_o / D)^2} \]

---
§6.10 - I-Sections: Shear Strength

Theoretical Basis: Shear Buckling

Theoretical Basis: Solution Space
§6.10 - I-Sections: Shear Strength

§6.10.9.1: Shear Strength: General

\[ \phi_v = 1.00 \]

- At the Strength Limit State

\[ V_s \leq \phi_v V_n \]

(6.10.9.1-1)

- Stiffened Interior Web Panels of I-shaped Girders:
  - without longitudinal stiffeners, the transverse stiffener spacing shall not exceed 3D
  - with longitudinal stiffeners, the transverse stiffener spacing shall not exceed 1.5D

- Stiffened End Web Panels of I-shaped Girders:
  - the transverse stiffener spacing shall not exceed 1.5D

*ODOT Prohibits the Use of Longitudinal Stiffeners.*

§6.10 - I-Sections: Shear Strength

§6.10.9: Nominal Shear Resistance

\[ V_n = V_{cr} = CV_p \]

(6.10.9.2-1)

\[ C \] - Ratio of shear buckling strength to shear yielding strength

\[ V_p \] - Plastic shear force, \( 0.58 F_{yw} D t_w \)
§6.10 - I-Sections: Shear Strength

§6.10.9: Nominal Shear Resistance

For Unstiffened Webs:

$k = 5$

For Stiffened Webs:

$k = 5 + \frac{5}{d_D}$

(6.10.9.3.2-7)
§6.10 - I-Sections: Shear Strength

Theoretical Basis: Tension Field Action
§6.10 - I-Sections: Shear Strength

Theoretical Basis: Tension Field Action

\begin{align*}
\left[ \frac{E_{k}}{F_{c}} \right] \cdot 1.12 & \\
\left[ \frac{E_{k}}{F_{c}} \right] \cdot 1.40 & \\
\end{align*}

\begin{itemize}
\item TFA Strength
\item Buckling Strength
\end{itemize}
§6.10 - I-Sections: Shear Strength

§6.10.9.3: Nominal Resistance Including Tension Field Action

- for Interior Panels

\[
V_s = V_p \left( C \frac{0.87(1 - C)}{1 + \left( \frac{d_o}{D} \right)^2} \right) \tag{6.10.9.3.2-1}
\]

\[
V_s = V_p \left( C \frac{0.87(1 - C)}{1 + \left( \frac{d_{o}}{D} \right)^2} \right) + \left( \frac{d_{o}}{D} \right) \tag{6.10.9.3.2-2}
\]

else

\[
V_s = V_p \left( C \frac{0.87(1 - C)}{1 + \left( \frac{d_{o}}{D} \right)^2} \right) \tag{6.10.9.3.2-8}
\]

\(d_o\) - Transverse stiffener spacing

Tension Field Action is now permitted for Hybrid Girders.

- for End Panels

\[
V_s = V_{cr} = CV_p \tag{6.10.9.3.3-1}
\]

\(C\) - Ratio of shear buckling strength to shear yielding strength.

\(V_p\) - Plastic shear force, \(0.58 F_{yw} D t_w\).

Tension Field Action is not permitted in end panels.
§6.10 - I-Sections: Shear Strength

§6.10.5.3: Special Fatigue Requirements for Webs

- Interior web panels with transverse stiffeners shall satisfy:

\[ V_s \leq V_p = CV_p \]  

(6.10.5.3-1)

- \( V_s \) - Shear in the web panel due to unfactored permanent loads plus twice the factored fatigue load.
- \( C \) - Ratio of shear buckling strength to shear yielding strength.
- \( V_p \) - Plastic shear force, \( 0.58 F_y \frac{D}{f_{tw}} \).
§6.10 - I-Sections: Shear Connectors

6.10.1 General
6.10.2 Cross-Section Proportion Limits
6.10.3 Constructability
6.10.4 Service Limit State
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6.10.9 Shear Strength

6.10.10 Shear Connectors
   6.10.10.1 General
   6.10.10.2 Fatigue Resistance
   6.10.10.3 Special Requirements for Inflection Points
   6.10.10.4 Strength Limit State

6.10.11 Stiffeners
6.10.12 Cover Plates

§6.10.10: General

In composite sections, stud or channel shear connectors shall be provided at the interface between the concrete deck and the steel section to resist interface shear. Channels are prohibited by ODOT.

Simple span composite bridges shall be provided with shear connectors throughout the length of the span.

Straight continuous composite bridges should normally be provided with shear connectors throughout the length of the bridge.
   - In negative flexure regions, shear connectors shall be provided if longitudinal reinforcement is considered as part of the composite section.
   - Otherwise, shear connectors need not be provided in negative flexure regions but additional connectors shall be place near the inflection points, as specified in §6.10.10.3.
   - When shear connectors are omitted in the negative flexure regions, longitudinal reinforcement shall be extended into the positive flexure region as is specified in §6.10.1.7

**ODOT Exception:** “…shall have shear connectors for the full length of…”
§6.10 - I-Sections: Shear Connectors

§6.10.10.1: General (cont)

- The ratio of the height to diameter of a stud shear connector shall not be less than 4.0
- The depth of clear cover over the tops of the shear connectors should not be less than 2.0"
- The shear connectors should penetrate at least 2.0” into the concrete deck.

BDM §304.4.1.15: ODOT Prefers the use of 7/8” diameter shear studs.
§6.10 - I-Sections: Shear Connectors

§6.10.10.1: General (cont)

- The longitudinal pitch of the shear connectors shall satisfy:

\[
p \leq \frac{nZ_r}{V_{sr}} \quad (6.10.10.1.2-1)
\]

- \( n \) - Number of shear connectors in a cross-section.
- \( Z_r \) - Shear fatigue resistance of an individual shear connector (§6.10.10.2).
- \( V_{sr} \) - Horizontal fatigue shear range per unit length.

\[
V_{sr} = \sqrt{(V_{fat})^2 + (F_{fat})^2} \quad (6.10.10.1.2-2)
\]

\[
V_{fat} = \frac{V_{f}Q}{I} \quad (6.10.10.1.2-3)
\]

\[
F_{fat} = \max \left\{ \frac{A_{eff} \sigma_{f,t}}{wR}, \frac{F_{s,rc}}{w} \right\} \quad (6.10.10.1.2-4)
\]

- \( V_{fat} \) - Longitudinal fatigue shear range per unit length.
- \( F_{fat} \) - Radial fatigue shear range per unit length.
- \( V_{f} \) - Vertical shear force range due to the fatigue load combination.
§6.10 - I-Sections: Shear Connectors

§6.10.10.1: General (cont)

$A_{bot}$ - Area of the bottom flange.

$\sigma_{fbl}$ - Range of longitudinal fatigue stress in bottom flange.

$l$ - Distance between brace points.

$w$ - Effective length of deck taken as 48” except at end supports where $w$ may be taken as 24”.

$R$ - Minimum girder radius within the panel.

$F_{FC}$ - Net range of cross-frame or diaphragm force at the top flange, taken as zero except for discontinuous cross-frame or diaphragm lines in bridges with skew angles greater than 20°.

**Bottom Line:** $F_{fat} = 0$ for most typical straight bridges...

---

§6.10 - I-Sections: Shear Connectors

§6.10.10.2: Fatigue Resistance

- The fatigue resistance of an individual shear stud shall be taken as:

$$Z_r = \alpha d^2 \geq \frac{5.5d^2}{2} \quad (6.10.10.2-1)$$

$$\alpha = 34.5 - 4.28 \log(N) \quad (6.10.10.2-2)$$

$d$ - Shear stud diameter.

$N$ - Number of fatigue cycles (§6.6.1.2.5).

*Must also check the effect of the connector on the fatigue resistance of the flange.*
§6.10 - I-Sections: Shear Connectors

§6.10.10.3: Special Requirement for Inflection Points

- For members that are noncomposite for negative flexure in the final condition, additional shear connectors shall be provided in the region of points of permanent load contraflexure.

The number of additional shear connectors shall be taken as:

\[ n_{sc} = \frac{A_y f_{sr}}{Z_r} \]  

(6.10.10.3-1)

- \( A_y \) - Total area of reinforcement over the interior support within the effective width.
- \( f_{sr} \) - Stress range in the longitudinal reinforcement under the Fatigue combination.

The additional shear connectors shall be placed within a distance equal to 1/3 the effective width on each side of the inflection point.

§6.10 - I-Sections: Shear Connectors

§6.10.10.4: Strength Limit State

- The factored shear resistance of a single shear connector shall be taken as:

\[ Q_{sc} = \phi_{sc} Q_{cr} \]  

(6.10.10.4.1-1)

\[ \phi_{sc} = 0.85 \]
§6.10 - I-Sections: Shear Connectors

§6.10.10.4: Strength Limit State (cont)

- The nominal strength of one stud connector shall be taken as:
  \[ Q_s = 0.5 A_{sc} \sqrt{f'c E_c} \leq A_{sc} F_{sc} \]  
  (6.10.10.4.3-1)

- The nominal strength of one channel connector shall be taken as:
  \[ Q_c = 0.3 (t_f + 0.5 t_w) L_c \sqrt{f'c E_c} \]  
  (6.10.10.4.3-2)

\( A_{sc} \) - Cross-sectional area of the stud shear connector.
\( f'c \) - Specified minimum strength of the concrete in the deck.
\( E_c \) - Modulus of elasticity of the concrete in the deck (\( E_c = 33,000 w_c 1.5 \sqrt{f'c} \) ksi).
\( F_{sc} \) - Specified minimum strength of the stud shear connector.
\( t_f, t_w, L_c \) - Flange thickness, web thickness, and length of a channel shear connector.

For curved bridges, \( P \) must include radial forces as well.
§6.10 - I-Sections: Shear Connectors

§6.10.10.4: Strength Limit State (cont)

For straight continuous spans that are composite for negative flexure, the shear force, \( P \), between the point of maximum positive moment \((LL + IM)\) and an adjacent end of the member shall be determined based on shown on the previous slide.

For straight continuous spans that are composite for negative flexure, the shear force, \( P \), between the point of maximum positive moment \((LL + IM)\) and the centerline of an adjacent interior support shall be taken as \( P_i \):

\[
P = P_i = P_p + P_n
\]  
\[
P_n = \min \begin{cases} 
P_{ts} = F_{yw} b_t n_t + F_{yw} b \beta t \beta + F_{yw} b \beta t \beta \\
P_{ps} = 0.45 f_y b_y t_y
\end{cases}
\]

For curved bridges, \( P \) must include radial forces as well.
§6.10 - I-Sections: Shear Connectors

§6.10.10.4: Strength Limit State (cont)

\[ n^+ = \frac{P}{Q} = \frac{P_1}{Q} = \frac{P_1 + P_2}{Q} \]

\[ n^- = \frac{P}{Q} = \frac{P_2}{Q} = \frac{P_1 + P_2}{Q} \]
§6.10 - I-Sections: Stiffeners

- 6.10.1 General
- 6.10.2 Cross-Section Proportion Limits
- 6.10.3 Constructability
- 6.10.4 Service Limit State
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- 6.10.6 Strength Limit State
- 6.10.7 Flexural Resistance: Pos Flexure
- 6.10.8 Flexural Resistance: Neg Flexure
- 6.10.9 Shear Strength
- 6.10.10 Shear Connectors

- 6.10.11 Stiffeners
  - 6.10.11.1 Transverse Stiffeners
  - 6.10.11.2 Bearing Stiffeners
  - 6.10.11.3 Longitudinal Stiffeners

- 6.10.12 Cover Plates
§6.10 - I-Sections: Stiffeners

§6.10.11.1: Transverse Stiffeners

- Transverse stiffeners shall consist of plates or angles welded or bolted to either one or both sides of the web.

- Stiffeners in straight girders not used as connection plates shall be tight fit at the compression flange, but need not be in bearing with the tension flange.

- Stiffeners used as connecting plates for diaphragms or cross-frames shall be attached to both flanges.

---

The distance between the end of the web-to-stiffener weld and the near edge of the adjacent web-to-flange or longitudinal stiffener-to-web weld shall not be less than $4t_w$ or more than the lesser of $6t_w$ and 4.0 in.

*The gap is limited to $6t_w$ to avoid vertical bucking of the unsupported web*
§6.10 - I-Sections: Stiffeners

**ODOT BDM §302.4.3.4: Intermediate Stiffeners**

- Intermediate web stiffeners shall be a minimum 3/8 inch thickness.

- Stiffeners that extend beyond the edge of flange shall be clipped at a 45° angle.

- All intermediate stiffeners should be the same size.

- Where intermediate stiffeners are to be used for the purpose of stiffening the web, it is preferable to use single stiffeners on alternate sides of the web of interior girders and only the inside of the web for fascia girders.
  - These stiffeners shall be welded to the web and the compression flange.
  - The tension flange of these stiffeners shall be a tight fit.

---

§6.10 - I-Sections: Stiffeners

**ODOT BDM §302.4.3.4: Intermediate Stiffeners**

- Stiffeners shall be provided for the attachment of cross frames and shall be welded to the web and both flanges to help eliminate cracking of the web due to out of plane bending. The designer shall investigate that the fatigue criteria is met in these areas.

- Stitch welding or single sided welding is not acceptable.

- Stiffener plates shall have corners in contact with both web and flange clipped. The clip dimensions shall be 1 inch horizontally and 2½ inches vertically.

- Intermediate stiffeners shall only be used on rolled beams when required for cross frames.

*Violation of the $\delta_n$ requirement of this article due to the requirement for clipping stiffeners and stiffener weld terminations is acceptable.*
§6.10 - I-Sections: Stiffeners

§6.10.11.1: Transverse Stiffeners

- The width, \( b_p \), of each projecting stiffener element shall satisfy:

\[
 b_p \geq 2.0 + \frac{D}{30} \tag{6.10.11.1-1}
\]

and

\[
 16t_p \geq b_p \geq \frac{b_f}{4} \tag{6.10.11.1-2}
\]

where:
- \( D \) - Depth of the web
- \( b_f \) - Full width of the widest compression flange.
- \( t_p \) - thickness of the projecting stiffener element

When neither tension field action nor post buckling strength are used in adjacent web panels, the moment of inertia of the transverse stiffener shall satisfy the smaller of:

\[
 I_i \geq b t_p J \tag{6.10.11.1.3-1}
\]

and

\[
 I_i \geq \frac{D^3 \rho_{ij}^{1.3}}{40} \left( \frac{F_{u*}}{E} \right)^{1.5} \tag{6.10.11.1.3-2}
\]

where:
- \( I_i \) - Moment of inertia of the stiffener about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for pairs of stiffeners.
§6.10 - I-Sections: Stiffeners

§6.10.11.1: Transverse Stiffeners

and where:
- \( b \) - The smaller of \( d_o \) and \( D \).
- \( d_o \) - The smaller of the adjacent web panel widths.
- \( J \) - Stiffener bending rigidity parameter.

\[
J = 2.5 \left( \frac{D}{d_o} \right)^2 - 2.0 \geq 0.5 \quad (6.10.11.1.3-3)
\]

- \( t_p \) - thickness of the projecting stiffener element.
- \( \rho_t \) - The larger of \( \frac{F_{yw}}{F_{crs}} \) and 1.0, where:

\[
F_{crs} = \frac{0.31E}{(b/t_p)^2} \leq F_{yw} \quad (6.10.11.1.3-4)
\]

- \( F_{yw} \) - Specified minimum yield strength of the stiffener.

---

§6.10 - I-Sections: Stiffeners

§6.10.11.1: Transverse Stiffeners

- For transverse stiffeners adjacent to web panels in which the shear force is larger than the shear buckling resistance and thus the web post buckling or tension field resistance is required in one or both panels, the moment of inertia of the transverse stiffeners shall satisfy Eq. 2.

\[
i \geq \frac{D^4}{40} \left( \frac{F_{yw}}{E} \right)^{1.3} \quad (6.10.11.1.3-2)
\]

---
§6.10 - I-Sections: Stiffeners

§D6.5: Concentrated Loads Applied without Bearing Stiffeners

- At bearing locations and at other locations subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, webs without bearing stiffeners shall be investigated for the limit states of web local yielding and web crippling according to the provisions of Articles D6.5.2 and D6.5.3.

§6.10 - I-Sections: Stiffeners

§D6.5.2: Web Local Yielding

- For interior-pier reactions and for concentrated loads applied at a distance from the end of the member that is greater than \(d\):

\[
R_u = (5k + N)F_{yw} \quad \text{(D6.5.2-2)}
\]

where:

- \(k\) - distance from the outer face of the flange resisting the concentrated load or bearing reaction to the web toe of the fillet
- \(N\) - length of bearing

(Recall...\(\phi_b = 1.00\))
§6.10 - I-Sections: Stiffeners

§D6.5.2: Web Local Yielding

- For interior-pier reactions and for concentrated loads applied at a distance from the end of the member that is greater than \( d \):

\[
(5k + N) = \frac{(5k + N)}{2.5n_w w R_w t_w} = \frac{(5k + N)}{2.5n_w w R_w t_w}
\]  

(Recall \( \phi_b = 1.00 \))
§6.10 - I-Sections: Stiffeners

§D6.5.2: Web Local Yielding

- For end reactions and for concentrated loads applied at a distance from the end of the member that is less than or equal to \( d \):

\[
N = \frac{(2.5k + N)}{d}
\]

\[
R_w = 0.80 \left[ 1 + 3 \left( \frac{N}{d} \right)^{1.5} \right] \frac{E F_{w, f}}{t_w}
\]

(Recall \( \phi_w = 0.80 \))

§6.10 - I-Sections: Stiffeners

§D6.5.2: Web Crippling

- For interior-pier reactions and for concentrated loads applied at a distance from the end of the member that is greater than \( \frac{d}{2} \):

\[
1.5 \frac{20.80}{204}
\]
For end reactions and for concentrated loads applied at a distance from the end of the member that is less than or equal to \(d/2\):

\[
\begin{align*}
\text{if } \frac{N}{d} & \leq 0.2 & R_w &= 0.40t_w \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \left( \frac{EF}{t_w} \right) \\
\text{if } \frac{N}{d} & > 0.2 & R_w &= 0.40t_w \left[ 1 + \left( \frac{4N}{d} - 0.2 \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \left( \frac{EF}{t_w} \right)
\end{align*}
\]

(Recall… \(\phi_w = 0.80\))

Bearing stiffeners shall be placed on the webs of built-up sections at all bearing locations.

At bearing locations on rolled shapes and at other locations on built-up sections or rolled shapes subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, either bearing stiffeners shall be provided or the web shall satisfy the provisions of Article D6.5 (Web Yielding / Web Crippling)
§6.10 - I-Sections: Stiffeners

§6.10.11.2: Bearing Stiffeners - Commentary

- Webs of built-up sections and rolled shapes without bearing stiffeners at the indicated locations must be investigated for the limit states of web local yielding and web crippling according to the procedures specified in Article D6.5. The section should either be modified to comply with these requirements or else bearing stiffeners designed according to these Specifications should be placed on the web at the location under consideration.

- In particular, inadequate provisions to resist temporary concentrated loads during construction that are not transmitted through a deck or deck system can result in failures. The Engineer should be especially cognizant of this issue when girders are incrementally launched over supports.

Bearing stiffeners shall consist of one or more plates or angles welded or bolted to both sides of the web. The connections to the web shall be designed to transmit the full bearing force due to the factored loads.
§6.10 - I-Sections: Stiffeners

§6.10.11.2: Bearing Stiffeners

- The stiffeners shall extend the full depth of the web and as closely as practical to the outer edges of the flanges.

- Each stiffener shall be either milled to bear against the flange through which it receives its load or attached to that flange by a full penetration groove weld.

\[
b_t \leq 0.48 t_p \sqrt{\frac{F_{ys}}{F_{ys}}} \tag{6.10.11.2.2-1}\]

where:
- \( F_{ys} \) - Specified minimum yield stress of the stiffener
- \( t_p \) - Thickness of the projecting stiffener element

This provision is in place to prevent local buckling of the stiffener.
§6.10 - I-Sections: Stiffeners

Local Buckling of a Bearing Stiffener

\[ R_{st} = \phi_b (R_s)_{u} = \phi_b 1.4 A_{pm} F_{ys} \]

(6.10.11.2.3-1,2)

where:
- \( A_{pm} \) - Area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange.
- \( F_{ys} \) - Specified minimum yield stress of the stiffener

(Recall...\( \phi_b = 1.00 \))
§6.10 - I-Sections: Stiffeners

§6.10.11.2: Bearing Stiffeners

- The bearing area is contact area between the end of the stiffener and the flange.
- The area lost due to the fillet clip must be subtracted.

The factored axial resistance, $P_{n}$, shall be determined as specified in Article 6.9.2.1 (Compression Members) using the specified minimum yield strength of the stiffener plates $F_{ys}$.

The radius of gyration shall be computed about the mid-thickness of the web and the effective length shall be taken as $0.75D$, where $D$ is the web depth.
§6.10 - I-Sections: Stiffeners

§6.10.11.2: Bearing Stiffeners

- For stiffeners bolted to the web, the effective column section shall consist of the stiffener elements only.

- For stiffeners consisting of two plates welded to the web, the effective column section shall consist of the two stiffener elements, plus a centrally located strip of web extending not more than 9tw on each side of the stiffeners.

- If more than one pair of stiffeners is used, the effective column section shall consist of all stiffener elements, plus a centrally located strip of web extending not more than 9tw on each side of the outer projecting elements of the group.

Effective Column Sections

A Single Pair of Stiffeners

Multiple Pairs of Stiffeners
§6.10 - I-Sections: Stiffeners

§6.10.11.2: Bearing Stiffeners

- The strip of the web shall not be included in the effective section at interior supports of continuous-span hybrid members for which the specified minimum yield strength of the web is less than 70 percent of the specified minimum yield strength of the higher strength flange.

i.e. when:

\[ F_{yw} \leq 0.7F_{ys} \]

- If the specified minimum yield strength of the web is less than that of the stiffener plates, the strip of the web included in the effective section shall be reduced by the ratio \( F_{yw}/F_{ys} \).

§6.10 - I-Sections: Stiffeners

§6.10.11.3: Longitudinal Stiffeners

§302.4.3.1 of the ODOT BDM Prohibits the Use of Longitudinal Stiffeners

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§6.13 - Connections and Splices

- 6.13.1 General
- 6.13.2 Bolted Connections
- 6.13.3 Welded Connections
- 6.13.4 Block Shear Rupture
- 6.13.5 Connection Elements
- 6.13.6 Splices
- 6.13.7 Rigid Frame Connections
§6.13 - Connections and Splices

§6.13.1: General

- Connections and splices for primary members shall be designed at the Strength Limit State for the larger of:
  - The average of the actual force and resisting force
    \[ M_{v,\text{max}} = \frac{M_{v,\text{act}} + \phi M_{v,\text{res}}}{2} \]
  - 75% of the factored resistance of the member.
    \[ M_{v,\text{max}} = 0.75\phi M_{v,\text{res}} \]
  - Where a section changes at a splice, \( \phi M_e \) is to be based on the smaller section.

§6.13.2: Bolted Connections

- Slip-Critical Connections shall be proportioned to prevent slip under the Service II Load Combination with \( R_r = R_n \) (i.e. no resistance factor)
- Bearing Connections are designed at the Strength Limit State

In general, bearing connections permitted in axial compression or bracing members. Otherwise slip-critical connections are required.
§6.13 - Connections and Splices

§6.4.3: Bolts, Nuts, and Washers

- Bolts shall conform to one of the following:
  - ASTM A307 $F_{u} = 60\text{ksi}$
  - AASHTO M164 (ASTM A325) $F_{u} = 120\text{ksi} / 105\text{ksi}$
  - AASHTO M253 (ASTM A490) $F_{u} = 150\text{ksi}$

- Nuts shall conform to:
  - AASHTO M291 (ASTM A563) for use with M164 and M253 bolts

- Washers shall conform to:
  - AASHTO M293 (ASTM F436)

§6.13 - Connections and Splices

§6.13.2.3: Bolts, Nuts, and Washers

Washers for high-strength bolted connections shall be required where:

- Where the outer face of the bolted parts has a slope greater than 1:20, with respect to a plane normal to the bolt axis

- Where tightening is to be performed by the calibrated wrench method, in which case the washer shall be used under the element turned in tightening;

- Where AASHTO M253 (ASTM A490) bolts are to be installed in material having a specified minimum yield strength less than 50 ksi, irrespective of the tightening method;

- Where AASHTO M253 (ASTM A490) bolts over 1.0 in. in diameter are to be installed in an oversize or short-slotted hole in an outer-ply, in which case a minimum thickness of 0.3125 in. shall be used under both the head and the nut. Multiple hardened washers shall not be used.

- Where needed for oversize or slotted holes
§6.13 - Connections and Splices

§6.13.2.3: Bolts, Nuts, and Washers

- Hardened washers shall be installed over oversize and short-slotted holes in an outer ply.

- Structural plate washers or a continuous bar with standard holes, not less than 0.3125 in. in thickness, shall be required to completely cover long-slotted holes. Hardened washers for use with high-strength bolts shall be placed over the outer surface of the plate washer or bar.

- Load indicator devices shall not be installed over oversize or slotted holes in an outer ply, unless a hardened washer or a structural plate washer is also provided.

§6.13.2.4: Holes

Unless specified otherwise, standard holes shall be used in bolted cnxns

- **Oversize holes** may be used in any or all plies of slip-critical connections. They shall not be used in bearing-type connections.

- **Short-slotted holes** may be used in any or all plies of slip-critical or bearing-type connections. The slots may be used without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections.

- **Long-slotted holes** may be used in only one ply of either a slip-critical or bearing-type connection. Long-slotted holes may be used without regard to direction of loading in slip-critical connections but shall be normal to the direction of load in bearing-type connections.
§6.13 - Connections and Splices

§6.13.2.6: Spacing of Bolts

- Minimum spacing between centers of bolts in standard holes shall not be less than 3 times the diameter of the bolt (3d).

- For oversize or slotted holes, maintain at least 2d of clear spacing between edges of adjacent holes.

For oversize or slotted holes, maintain at least 2d of clear spacing between edges of adjacent holes.

§6.13.2.6: Edge Distance

- The minimum edge distance shall be as specified in Table 1.

Table 6.13.2.6.6-1 Minimum Edge Distances

<table>
<thead>
<tr>
<th>Bolt Diameter</th>
<th>Sheared Edges</th>
<th>Rolled or Gas-Cut Edges</th>
<th>ODOT Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/8&quot;</td>
<td>1-1/8&quot;</td>
<td>7/8&quot;</td>
<td>---</td>
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<tr>
<td>3/4&quot;</td>
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<td>1&quot;</td>
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<td>1-1/8&quot;</td>
<td>...</td>
</tr>
<tr>
<td>1&quot;</td>
<td>1-3/4&quot;</td>
<td>1-1/4&quot;</td>
<td>2&quot;</td>
</tr>
<tr>
<td>1-1/8&quot;</td>
<td>2&quot;</td>
<td>1-1/2&quot;</td>
<td>2-1/4&quot;</td>
</tr>
<tr>
<td>1-1/4&quot;</td>
<td>2-1/4&quot;</td>
<td>1-5/8&quot;</td>
<td>2-1/2&quot;</td>
</tr>
<tr>
<td>1-3/8&quot;</td>
<td>2-3/8&quot;</td>
<td>1-3/4&quot;</td>
<td>2-5/8&quot;</td>
</tr>
</tbody>
</table>

- The maximum edge distance shall not be more than eight times the thickness of the thinnest outside plate or 5.0 in.

**ODOT prefers the use of 1” or 1-1/8” diameter bolts for splices.**
§6.13 - Connections and Splices

§6.13.2.6: End Distance

- The end distance for all types of holes measured from the center of the bolt shall not be less than the edge distances specified in Table 1.

  When oversize or slotted holes are used, the minimum clear end distance shall not be less than the bolt diameter.

- The maximum end distance shall not be more than eight times the thickness of the thinnest outside plate or 5.0 in.

§6.13.2.7: Bolted Connections - Shear Resistance

The nominal shear resistance of a bolt,

- Where threads are excluded from the shear plane

  \[ R_n = 0.48 A_b F_{ub} N_s \quad (6.13.2.7-1) \]

- Where threads are included in the shear plane

  \[ R_n = 0.38 A_b F_{ub} N_s \quad (6.13.2.7-2) \]

where:

- \( A_b \) - Area of Bolt Corresponding to the Nominal Diameter.
- \( F_{ub} \) - Specified Minimum Tensile Strength of the Bolt.
- \( N_s \) - Number of Shear Planes per Bolt.

(Recall…\( \phi_s = 0.80 \))
§6.13 - Connections and Splices

§6.13.2.7: Bolted Connections - Shear Resistance

- Individual bolts in shear with the shear plane in the shank of the bolt demonstrated a strength approximately corresponding to 60% of tensile strength of the material.

For threads excluded:

\[ R_u = 0.6 A_F \]

- When several bolts are used in the same shear connection, the bolts are somewhat less effective than when tested individually. The overall strength is approximately 80% of the strength of the individual bolts.

\[ R_{n,\text{group}} = (0.8) \sum (0.6 A_F) = \sum (0.48 A_F) \]

When the bolts are sheared on a plane that passes through the threads, the strength was found to be 83.3% of that when they were sheared on a plane passing through the shank. Taking 80%, then...

For threads included:

\[ R_{n,\text{group}} = 0.80 \sum (0.48 A_F) = \sum (0.38 A_F) \]

- When the length of a connection exceeds 50" in length, the strengths calculated by equations 1 and 2 should be further reduced by 20% (i.e. multiply by a second reduction factor of 0.80).
§6.13 - Connections and Splices

§6.13.2.7: Bolted Connections - Shear Resistance

When A307 Bolts Are Used:

- The resistance factor for A307 Bolts in Shear is $\phi_s = 0.65$ instead of $\phi_s = 0.80$ that is used for A325 and A490.

- Their strength in shear shall be based on the threads included condition, regardless of the actual configuration.

- Because of bending that is common in A307 bolts, their strength in shear shall be reduced by 1.0% per $\frac{1}{16}$" when their length exceeds 5 diameters.

§6.13 - Connections and Splices

§6.13.2.8: Bolted Connections - Slip Resistance

- Bearing-type connections shall be permitted only for joints subjected to axial compression or joints on bracing members and shall satisfy the factored resistance, $R_r$, at the Strength Limit State.

- Slip-critical connections shall be proportioned to prevent slip under Load Combination Service II and to provide bearing, shear, and tensile resistance at the applicable Strength Limit State load combinations.

- Joints subject to stress reversal, heavy impact loads, severe vibration or located where stress and strain due to joint slippage would be detrimental to the serviceability of the structure shall be designated as slip-critical. They include....

*In general, bearing connections permitted in axial compression or bracing members. Otherwise slip-critical connections are required.*
§6.13 - Connections and Splices

### §6.13.2.8: Bolted Connections - Slip Resistance

- Joints subject to fatigue loading;
- Joints in shear with bolts installed in oversized holes;
- Joints in shear with bolts installed in short- and long-slotted holes where the force on the joint is in a direction other than perpendicular to the axis of the slot, except where the Engineer intends otherwise and so indicates in the contract documents;
- Joints with significant load reversal;
- Joints in which welds and bolts share in transmitting load at a common faying surface
- Joints in axial tension or combined axial tension and shear;
- Joints in axial compression only, with standard or slotted holes in only one ply of the connection with the direction of the load perpendicular to the direction of the slot, except for splices in compression members (6.13.6.1.3)
- Joints in which, in the judgment of the Engineer, any slip would be critical to the performance of the joint or the structure and which are so designated in the contract documents.

The nominal slip resistance of a bolt shall be taken as

\[ R_n = K_h K_s N_s P_t \]  \hspace{1cm} (6.13.2.8-1)

- **\( K_h \)** - Hole Size Factor
  - Standard Holes: 1.00
  - Oversize or Short Slots: 0.85
  - Long Slots Perpendicular: 0.70
  - Long Slots Parallel: 0.60
- **\( K_s \)** - Surface Condition Factor
  - Class A or C: 0.33
  - Class B: 0.50
- **\( N_s \)** - Number of Slip Planes per Bolt
- **\( P_t \)** - Minimum Required Bolt Tension

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§6.13 - Connections and Splices

### §6.13.2.8: Bolted Connections - Slip Resistance

The nominal slip resistance of a bolt shall be taken as

\[ R_n = K_h K_s N_s P_t \]  \hspace{1cm} (6.13.2.8-1)

- **\( K_h \)** - Hole Size Factor
  - Standard Holes: 1.00
  - Oversize or Short Slots: 0.85
  - Long Slots Perpendicular: 0.70
  - Long Slots Parallel: 0.60
- **\( K_s \)** - Surface Condition Factor
  - Class A or C: 0.33
  - Class B: 0.50
- **\( N_s \)** - Number of Slip Planes per Bolt
- **\( P_t \)** - Minimum Required Bolt Tension

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§6.13 - Connections and Splices

§6.13.2.8: Bolted Connections - Pretension

- High-strength bolts subjected to axial tension shall be tensioned to the force specified in Table 6.13.2.8-1.
- The pretension is not included in the bolt load, $T_u$.

<table>
<thead>
<tr>
<th>Bolt Diameter</th>
<th>Required Tension, $P_t$ (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M164 (A325)</td>
</tr>
<tr>
<td>5/8&quot;</td>
<td>19</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>28</td>
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<td>7/8&quot;</td>
<td>39</td>
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<tr>
<td>1&quot;</td>
<td>51</td>
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<tr>
<td>1-1/8&quot;</td>
<td>56</td>
</tr>
<tr>
<td>1-1/4&quot;</td>
<td>71</td>
</tr>
<tr>
<td>1-3/8&quot;</td>
<td>85</td>
</tr>
<tr>
<td>1-1/2&quot;</td>
<td>103</td>
</tr>
</tbody>
</table>

&enspace;\[ R_s = K_s K_{ns} P_t \left(1 - \frac{T_u}{P_t} \right) \quad (6.13.2.11-3) \]

where:
- $T_u$ - Tensile force due to factored loads under combination Service II.
- $P_t$ - Minimum required pretension in Table 6.13.2.8-1.
§6.13 - Connections and Splices

§6.13.2.8: Bolted Connections - Slip Resistance

- **Class A Surface**: unpainted clean mill scale, and blast-cleaned surfaces with Class A coatings,

- **Class B Surface**: unpainted blast-cleaned surfaces and blast-cleaned surfaces with Class B coatings

- **Class C Surface**: hot-dip galvanized surfaces roughened by wire brushing after galvanizing

The contract documents shall specify that in uncoated joints, paint, including any inadvertent overspray, be excluded from areas closer than one bolt diameter but not less than 1.0 in. from the edge of any hole and all areas within the bolt pattern.

§6.13 - Connections and Splices

§6.13.2.8: Bolted Connections - Slip Resistance

- The contract documents shall specify that joints having painted faying surfaces be blast-cleaned and coated with a paint that has been qualified by test as a Class A or Class B coating.

- Subject to the approval of the Engineer, coatings providing a surface condition factor less than 0.33 may be used, provided that the mean surface condition factor is established by test.

- The contract documents shall specify that coated joints not be assembled before the coatings have cured for the minimum time used in the qualifying test.
§6.13 - Connections and Splices

§6.13.2.8: Bolted Connections - Slip Resistance

- The contract documents shall specify faying surfaces to be galvanized shall be hot-dip galvanized in accordance with the AASHTO M111 (ASTM A123). The surfaces shall subsequently be roughened by means of hand wire brushing. Power-wire brushing shall not be permitted.

- When using galvanized steel in the connections, beware of:
  - Creep with regard to the slip resistance, and
  - Loss of pretension in the bolts

§6.13.2.9: Bolted Connections - Bearing Resistance

The nominal bearing resistance of a bolt shall be taken as

With a clear bolt-to-bolt distance of $2.0d$ and a clear end distance of $2.0d$:

$$ R_n = 2.4 \, d \, t \, F_u $$  \hspace{1cm} (6.13.2.9-1)

Otherwise

$$ R_n = 1.2 \, L_c \, t \, F_u $$  \hspace{1cm} (6.13.2.9-2)

$L_c$ - Clear distance between holes or clear end distance (ksi).
$F_u$ - Tensile strength of the connected material (ksi).
$t$ - Thickness of base material (in)

Use “2.0” and “1.0” for long slots arranged perpendicular.

The nominal bearing resistance of the connected member may be taken as the sum of the resistances of the individual holes.
§6.13 - Connections and Splices

§6.13.2.9: Bolted Connections - Bearing Resistance

When the clear distance is large enough (greater than $2d$), the bearing strength is based on deformations around the bolt holes of approximately 1/4”.

\[ R_n = (2)(L_c)(0.6F_u) = 1.2L_cF_u \]
§6.13 - Connections and Splices

§6.13.2.9: Bolted Connections - Bearing Resistance

\[ T_n = 0.76 A_b F_{ub} \]

- \( A_b \) - Area of Bolt Corresponding to the Nominal Diameter.
- \( F_{ub} \) - Specified Minimum Tensile Strength of the Bolt.

(Recall…\( \phi = 0.80 \))
§6.13 - Connections and Splices

§6.13.2.10: Bolted Connections - Tensile Resistance

\[ A_{\text{nom}} = \left( \frac{\pi}{4} \right) d_{\text{nom}}^2 \]

\[ A_{\text{nom}} = \left( \frac{\pi}{4} \right) \left( d_{\text{nom}} - \frac{1.3}{n} \right)^2 \]

\[ A_{\text{eff}} = \left( \frac{\pi}{4} \right) \left( d_{\text{nom}} - \frac{0.9743}{n} \right)^2 \]

\[ n \text{ is the number of threads per inch (not the thread pitch).} \]

Test results show that the tensile strength is best predicted by \( A_{\text{eff}} \).

\[ \frac{T_u}{A_{\text{nom}}} = 0.76 A_{\text{eff}} \]

\[ T_u = A_{\text{eff}} F' \approx 0.76 A_{\text{nom}} F' \]

\[ \text{Average: 0.765} \]
§6.13 - Connections and Splices

§6.13.2.10: Bolted Connections – Prying Action

- The applied tensile force shall be taken as the force due to the external factored loadings, plus any tension resulting from prying action produced by deformation of the connected parts, as specified in Article 6.13.2.10.4.

\[ \sum F_y \rightarrow 2P + 2Q = 2T \]

Solving for the bolt force, \( T \):

\[ T = P + Q \]

The force in the bolt is the sum of the applied tension, \( P \), and the internal prying force, \( Q \).

\[ Q_a = \left( \frac{3b}{8a} - \frac{t'}{20} \right) P_a \]

\( Q_a \) - Total tension per bolt (including prying action) due to factored loadings (kip).
\( P_a \) - Direct tension per bolt due to factored loadings (kip).
\( a \) - Distance from the center of bolt to the edge of plate (in).
\( b \) - Distance from center of bolt to the toe of fillet of connected part (in).
\( t \) - Thickness of thinnest part connected (in).
§6.13 - Connections and Splices

§6.13.2.10: Bolted Connections – Prying Action

\[
\begin{align*}
\gamma (\Delta F) &\leq (\Delta F)_n \quad (6.6.1.2.2-1) \\
(\Delta F)_n = \left( \frac{A}{N} \right)^{\frac{1}{2}} &\geq \frac{(\Delta F)_{TH}}{2} \quad (6.6.1.2.5-1)
\end{align*}
\]

- Distance from the center of bolt to the edge of plate (in).
- Distance from center of bolt to the toe of fillet of connected part (in).

\textit{This model is based on work by Douty and McGuire and provides conservative results. There are better models available, but...}

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§6.13 - Connections and Splices

§6.13.2.10: Bolted Connections – Fatigue Resistance

- Where high-strength bolts in axial tension are subject to fatigue, the stress range, $\Delta F$, in the bolt, due to the fatigue design live load, plus the dynamic load allowance, plus the prying force, shall satisfy,

\[
\gamma (\Delta F) \leq (\Delta F)_n
\]

- For M164 Bolts (A325) in tension: $A = 17.1 \times 10^8$ ksi$^3$, $(\Delta F)_{TH} = 31.0$ ksi
- For M253 Bolts (A490) in tension: $A = 31.5 \times 10^8$ ksi$^3$, $(\Delta F)_{TH} = 38.0$ ksi
§6.13 - Connections and Splices

§6.13.2.10: Bolted Connections – Fatigue Resistance

- The nominal diameter of the bolt shall be used in calculating the bolt stress range. In no case shall the calculated prying force exceed 60 percent of the externally applied load.

- Low carbon ASTM A307 bolts shall not be used in connections subjected to fatigue.

Commentary: “Properly tightened A325 and A490 bolts are not adversely affected by repeated application of the recommended service load tensile stress, provided that the fitting material is sufficiently stiff that the prying force is a relatively small part of the applied tension.”

§6.13.2.11: Bolted Connections – Combined Tension and Shear

The nominal tensile resistance of a bolt subjected to tension and shear shall be taken as

- If \( \frac{P}{R_n} \leq 0.33 \),

\[
T_n = 0.76 A_b F_{ub} \quad (6.13.2.11-1)
\]

- Otherwise

\[
T_n = 0.76 A_b F_{ub} \sqrt{1 + \left( \frac{P_n}{\phi R_n} \right)^2} \quad (6.13.2.11-2)
\]

\( P_n \) - Shear force on the bolt due to factored loads.
\( R_n \) - Nominal shear resistance of the bolt.
§6.13 - Connections and Splices

§6.13.2.11: Bolted Connections – Combined Tension and Shear

Test have shown that the strength of bearing fasteners subject to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse (Kulak et al., 1987). The relationship can be expressed as

\[
\left( \frac{T_u}{\phi T_u} \right)^2 + \left( \frac{P_u}{\phi P_u} \right)^2 = 1
\]

§6.13 - Connections and Splices

§6.13.4: Block Shear Resistance

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§6.13 - Connections and Splices

§6.13.4: Block Shear Resistance

Tension

Shear

Shear
§6.13 - Connections and Splices

§6.13.4: Block Shear Resistance

The factored resistance corresponding to Block Shear Rupture is:

- If $A_{nv} \geq 0.58 A_{nv'}$
  \[ R_r = \phi_{bs} \left(0.58 F_y A_{vg} + F_u A_{tn} \right) \]  \hspace{1cm} (6.13.4-1)

- Otherwise
  \[ R_r = \phi_{bs} \left(0.58 F_u A_{nv} + F_y A_{tg} \right) \]  \hspace{1cm} (6.13.4-2)

- $A_{vg}$ - Gross area in Shear
- $A_{nv}$ - Net area in Shear
- $A_{tg}$ - Gross area in Tension
- $A_{tn}$ - Net area in Tension
- $F_y$ - Specified minimum yield strength of the connected material.
- $F_u$ - Specified minimum tensile strength of the connected material.
- $\phi_{bs} = 0.80$
§6.13 - Connections and Splices

§6.13.3: Welded Connections

- Base metal, weld metal, and welding design details shall conform to the requirements of the AASHTO/AWS D1.5M/D1.5 Bridge Welding Code. Welding symbols shall conform to those specified in AWS Publication A2.4.

- Matching weld metal shall be used in groove and fillet welds, except that the Engineer may specify electrode classifications with strengths less than the base metal when detailing fillet welds, in which case the welding procedure and weld metal shall be selected to ensure sound welds.

Commentary: “Use of undermatched weld metal is highly encouraged for fillet welds connecting steels with specified minimum yield strength greater than 50 ksi. Research has shown that undermatched welds are much less sensitive to delayed hydrogen cracking and are more likely to produce sound welds on a consistent basis.”

The factored resistance of a welded connection is governed by the resistance of the base metal or the tensile strength of the deposited weld metal. The nominal resistance of fillet welds is determined from the effective throat area, whereas the nominal strength of the connected parts is governed by their respective thickness.

Commentary: “Shear yielding is not critical in welds because the material strain hardens without large overall deformations occurring. Therefore, the factored shear resistance is based on the shear strength of the weld metal multiplied by a suitable resistance factor to ensure that the connected part will develop its full strength without premature failure of the weldment.”

Three types of welds are considered:

- Complete-Penetration Groove Welds
- Partial-Penetration Groove Welds
- Fillet Welds
§6.13 - Connections and Splices

§6.13.3: Complete-Penetration Groove Welds

- **Tension and Compression:**
  The factored resistance of complete penetration groove-welded connections subjected to tension or compression normal to the effective area or parallel to the axis of the weld shall be taken as the factored resistance of the base metal.

- **Shear:**
  The factored resistance of complete penetration groove-welded connections subjected to shear on the effective area shall be taken as the lesser of 60% of the factored resistance of the base metal in tension, and,

  \[ 0.60 \phi_{e1} F_{exx} \]  
  \[ \phi_{e1} = 0.85 \]  
  (for Shear on effective area in Full Pen Welds)
§6.13 - Connections and Splices

§6.13.3: Partial-Penetration Groove Welds

Beware of Transversely-Loaded Partial-Pen Groove Welds!!!

Section 6.6.1.2.4: “Transversely loaded partial penetration groove welds shall not be used, except as permitted in Article 9.8.3.7.2, (which covers detailing requirements for orthotropic steel decks).”

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§6.13 - Connections and Splices

§6.13.3: Partial-Penetration Groove Welds

- Tension and Compression:
  The factored resistance of partial penetration groove-welded connections subjected to tension or compression parallel to the axis of the weld or compression normal to the effective area shall be taken as the factored resistance of the base metal.
§6.13 - Connections and Splices

§6.13.3: Partial-Penetration Groove Welds

- **Tension and Compression:**
  The factored resistance for partial penetration groove-welded connections subjected to tension normal to the effective area shall be taken as the lesser the factored resistance of the base metal, or,

  \[ 0.60 \phi_{e1} F_{exx} \quad (6.13.3.2.3a-1) \]

  \[ \phi_{e1} = 0.80 \quad \text{(for Tension normal to the effective area of Partial Pen Welds)} \]

- **Shear:**
  The factored resistance of partial penetration groove-welded connections subjected to shear parallel to the axis of the weld shall be taken as the lesser of either the factored nominal resistance of the connected material specified in Article 6.13.5 or the factored resistance of the weld metal taken as:

  \[ 0.60 \phi_{e2} F_{exx} \quad (6.13.3.2.3b-1) \]

  \[ \phi_{e2} = 0.80 \quad \text{(for Shear on effective area in Partial Pen Welds)} \]
§6.13 - Connections and Splices

§6.13.3: Fillet Welds

- **Tension and Compression:**
  
  The factored resistance for fillet-welded connections subjected to tension or compression parallel to the axis of the weld shall be taken as the factored resistance of the base metal.

  Commentary: “Flange-to-web fillet-welded connections may be designed without regard to the tensile or compressive stress in those elements parallel to the axis of the welds.”

  *In other words, you design these welds only for the shear transferred between the flange and web regardless of the net tension or compression actually in the flange or web.*

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§6.13 - Connections and Splices

§6.13.3: Fillet Welds

- **Shear:**

  The resistance of fillet welds in shear which are made with matched or undermatched weld metal and which have typical weld profiles shall be taken as the product of the effective area specified in Article 6.13.3.3 and the factored resistance of the weld metal taken as:

  \[
  0.60 \phi_{e2} F_{exx} \quad (6.13.3.2b-1)
  \]

  \( \phi_{e2} = 0.80 \) (for Shear in the Throat of Weld Metal in Fillet Welds)
§6.13 - Connections and Splices

§6.13.3: Fillet Welds

Commentary: “It is seldom that weld failure will ever occur at the weld leg in the base metal. The applicable effective area for the base metal is the weld leg, which is 30% greater than the weld throat. If overstrength weld metal is used or the weld throat has excessive convexity, the capacity can be governed by the weld leg and the shear fracture resistance of the base metal 0.6 $F_u$.”

§6.13 - Connections and Splices

§6.13.3: Fillet Welds

Shear:

Commentary: “The factored resistance of fillet welds subjected to shear along the length of the weld is dependent upon … the direction of the applied load, which may be parallel or transverse to the weld. In both cases, the weld fails in shear, but the plane of rupture is not the same.”
§6.13 - Connections and Splices

§6.13.3: Fillet Welds

Shear:
Commentary: “If fillet welds are subjected to eccentric loads that produce a combination of shear and bending, they must be proportioned on the basis of a direct vector addition of the shear forces on the weld (i.e. elastic vector method).”
§6.13 - Connections and Splices

§6.13.3: Welded Connections - Effective Area

- The effective area shall be the effective weld length multiplied by the effective throat. The effective throat shall be the shortest distance from the joint root to the weld face.

- Additional requirements can be found in the AASHTO/AWS D1.5M/D1.5 Bridge Welding Code, Article 2.3.

AWS §2.3.1: Effective Weld Area - Groove Welds

- Full Pen Welds: The effective weld size of a complete joint penetration groove weld shall be the thickness of the thinner part joined. No increase is permitted for weld reinforcement.

- Partial Pen Welds: The effective weld size of a partial joint penetration groove weld is either (1) the depth of bevel less 1/8" or (2) the depth of bevel without reduction, depending on the angle of the groove and the welding process used.

The ODOT CMS permits the use of SMAW, SAW and FCAW processes.
§6.13 - Connections and Splices

AWS §2.3.1: Effective Weld Area - Fillet Welds

- The effective throat shall be the shortest distance from the joint root to the weld face of the diagrammatic weld.

\[
t_e = 0.707 \times w
\]

§6.13.3: Welded Connections - Size of Fillet Welds

- The maximum size of fillet weld that may be used along edges of connected parts shall be taken as:
  - For material less than 1/4" thick: the thickness of the material, and
  - For material 1/4" or more in thickness: 1/16" less than the thickness of the material, unless the weld is designated on the contract documents to be built out to obtain full throat thickness.

- The minimum size of fillet weld should be taken as below. The weld size need not exceed the thickness of the thinner part joined. Smaller fillet welds may be approved by the Engineer based upon applied stress and the use of appropriate preheat.

\[
\begin{align*}
\text{For } T \leq 3/4" & \quad \text{For } T > 3/4" \\
W & \geq 1/4" & W & \geq 5/16" \\
\end{align*}
\]

T is the thickness of the thicker part
§6.13 - Connections and Splices

§6.13.3: Welded Connections - Length of Fillet Welds

- The minimum effective length of a fillet weld shall be four times its size and in no case less than 1.5 in.

§6.13 - Connections and Splices

§6.13.3: Welded Connections - Fillet Weld End Returns

- Fillet welds that resist a tensile force not parallel to the axis of the weld or proportioned to withstand repeated stress shall not terminate at corners of parts or members. Where such returns can be made in the same plane, they shall be returned continuously, full size, around the corner, for a length equal to twice the weld size. End returns shall be indicated in the contract documents.

- Fillet welds deposited on the opposite sides of a common plane of contact between two parts shall be interrupted at a corner common to both welds.
§6.13 - Connections and Splices

§6.13.6: Bolted Splices

Tension Members:
- Splices for tension members shall satisfy the requirements specified for net section fracture, gross yielding, and block-shear rupture
- Splices for tension members shall be designed using slip-critical connections as specified in Article 6.13.2.1.1.

Compression Members:
- Splices for compression members detailed with milled ends in full contact bearing at the splices and for which the contract documents specify inspection during fabrication and erection, may be proportioned for not less than 50% of the lower factored resistance of the sections spliced.
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Flexural Members

- In continuous spans, splices should be made at or near points of dead load contraflexure.

- Web and flange splices in areas of stress reversal shall be investigated for both positive and negative flexure.

- In both web and flange splices, there shall not be less than two rows of bolts on each side of the joint.

- Oversize or slotted holes shall not be used in either the member or the splice plates at bolted splices.

- Bolted splices for flexural members shall be designed using slip-critical connections as specified in Article 6.13.2.1.1. The connections shall also be proportioned to prevent slip during the erection of the steel and during the casting of the concrete deck.

\[ f_i \leq 0.84 \left( \frac{A_u}{A_g} \right) F_u \leq F_{ml} \]  

(6.10.1.8-1)

- The flexural stresses due to the factored loads at the strength limit state and for checking slip of the bolted connections at the point of splice shall be determined using the gross section properties.
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Web Splices

Web Splices are to be designed for:
- The direct shear transferred across the splice,
- The moment created by the eccentric shear, and
- The portion of the beam moment that is carried by the web.

The eccentricity of the shear force shall be taken as the distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration.

As a minimum, at the strength limit state, the design shear, $V_{uw}$, shall be taken as follows:

if $V_u < 0.5\phi_v V_n$, then:

$$V_{uw} = 1.5V_u$$  \hspace{1cm} (6.13.6.1.4b-1)

Otherwise:

$$V_{uw} = \frac{V_u + \phi_v V_n}{2}$$  \hspace{1cm} (6.13.6.1.4b-2)
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Web Splices

- Webs shall be spliced symmetrically by plates on each side.

- The splice plates shall extend as near as practical for the full depth between flanges.

- At the strength limit state, the flexural stress in the web splice plates shall not exceed the specified minimum yield strength of the splice plates times the resistance factor for flexure.

- Shear yielding and block shear of the plates shall be checked.

- Bolted connections from web splices shall be designed as slip critical connections for the maximum resultant bolt design force. As a minimum, for checking slip of the web splice bolts, the design shear shall be taken as the shear at the point of splice under Load Combination Service II.

---

§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Web Splices

- For bolt groups subjected to eccentric shear, the use of the elastic vector method is preferred over the ultimate strength method because “it provides a more uniform level of safety.”

- To effectively utilize the elastic vector method to compute the maximum resultant bolt force, all actions should be applied at the mid-depth of the web and the polar moment of inertia of the bolt group should be computed about the centroid of the connection.

- Shifting the polar moment of inertia of the bolt group to the neutral axis of the composite section (which is typically above the mid-depth of the web) may cause the bolt forces to be underestimated.

- To simplify the computations and avoid possible errors, it is recommended that all calculated actions in the web be applied at the mid-depth of the web for design of the splice.
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Web Splices

The following equations are suggested to determine a design moment, $M_{uw}$, and a design horizontal force, $H_{uw}$, to be applied at the mid-depth of the web for designing the splice plates and their connections at the strength limit.

\[
M_{uw} = \frac{t_w D^2}{12} |R_h F_{cf} - R_{cf} f_{ncf}| \quad \text{(C6.13.6.1.4b-1)}
\]

\[
H_{uw} = \frac{t_w D}{2} |R_h F_{cf} + R_{cf} f_{ncf}| \quad \text{(C6.13.6.1.4b-2)}
\]

where:
- $t_w$, $D$ – thickness and depth of the web
- $R_h$ – Hybrid Girder factor
- $F_{cf}$ – Design stress in the controlling flange, (+) Ten (-) Comp
- $R_{cf}$ – the absolute value of the ratio of $F_{cf}$ to $f_{cf}$
- $f_{cf}$ – Flexural stress due to the factored loads in the controlling flange
- $f_{ncf}$ – Flexural stress due to the factored loads in the noncontrolling flange
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Web Splices

- Modified versions of these equations can also be used to determine slip loads

\[ M_{sw} = \frac{t_w D^2}{12} |f_s - f_{os}| \]  
\[ H_{sw} = \frac{t_w D}{2} |f_s + f_{os}| \]

where:
- \( t_w \) - thickness and depth of the web.
- \( f_s \) - Max stress due to Service II at mid thickness of flange under consideration.
- \( f_{os} \) - Max stress due to Service II as mid thickness of the other flange at the point of the splice concurrent with \( f_s \).

§6.13.6: Bolted Splices - Flange Splices

- At the strength limit state, the splice plates on the controlling flange shall provide a minimum resistance taken as the design stress, \( F_{cf} \), times the smaller effective flange area, \( A_{cf} \), on either side of the splice.

\[ F_{cf} = \left( \frac{1}{2} \right) \left( \frac{f_{cf}}{R_y} \right) + \alpha \phi \sigma F_{cf} \geq 0.75 \alpha \phi \sigma F_{cf} \]

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§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Flange Splices

- At the strength limit state, the splice plates on the noncontrolling flange shall provide a minimum resistance taken as the design stress, $F_{nf}$, times the smaller effective flange area, $A_{e}$, on either side of the splice.

$$F_{nf} = R_{sf} \left( \frac{f_{nf}}{R_{th}} \right) \geq 0.75 \alpha \phi f_{yf}$$  \hspace{1cm} (6.13.6.1.4c-3)

$A_{e}$ is the effective area of the flange.

- For compression flanges, $A_{e}$ shall be taken as the gross area of the flange.
- For tension flanges, $A_{e}$ shall be taken as:

$$A_{e} = \left( \frac{\phi f_{u}}{\phi f_{yf}} \right) A_{n} \leq A_{g}$$ \hspace{1cm} (6.13.6.1.4c-2)

By designing for the effective area, $A_{e}$, net fracture of the tension flange is theoretically precluded.
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Flange Splices

- The controlling flange is defined as either the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its mid-thickness due to the factored loads for the loading condition under investigation to its factored flexural resistance.

\[
\frac{f}{\phi F_n} = \left( \frac{f}{\phi M_n / S_x} \right)
\]

- The other flange is termed the noncontrolling flange.
- In areas of stress reversal, the splice must be checked independently for both positive and negative flexure.
- For composite sections in positive flexure, the controlling flange is typically the bottom flange.
- For sections in negative flexure, either flange may qualify as the controlling flange.

\[f_{cf}\] and \[f_{ncf}\] are taken at the mid-thickness of their respective flanges and are taken at concurrent locations in the splice.
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Flange Splices

- $F_u$ - nominal flexural resistance of the flange
- $F_{yf}$ - specified minimum yield strength of the flange
- $F_{yw}$ - specified minimum yield strength of the web
- $F_{yt}$ - specified minimum yield strength of the tension flange
- $F_{tu}$ - specified minimum tensile strength of the tension flange
- $\phi_f$ - resistance factor for flexure (1.00)
- $\phi_u$ - resistance factor for fracture of tension members (0.80)
- $\phi_y$ - resistance factor for yielding of tension members (0.95)

The factor $\alpha$ is generally taken as 1.0, except that a lower value equal to the ratio of $F_u$ to $F_{yf}$ may be used for flanges where $F_u$ is less than $F_{yf}$. Potential cases include bottom flanges of I-sections in compression, or bottom box flanges in compression or tension at the point of splice. In these cases, the calculated $F_u$ of the flange at the splice may be significantly below $F_{yf}$ making it overly conservative to use $F_{yf}$ to determine the flange design force for designing the splice.

For I-section flanges in compression, the reduction in $F_u$ below $F_{yf}$ is typically not as large as for box flanges. Thus, for simplicity, a conservative value of $\alpha$ equal to 1.0 may be used for this case even though the specification would permit the use of a lower value.
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Flange Splices

- Flange splice plates subjected to tension are checked for net section fracture, gross yielding, and block shear rupture at the strength limit state (though block shear rupture will typically not govern).

- Flange plates subjected to compression are to be checked for gross section yielding at the strength limit state (i.e. the unbraced length of the plate is taken as zero) with the resistance factor taken as 0.90 from §6.5 as for compression members.

- For a flange splice with inner and outer splice plates, the flange design force at the strength limit state may be assumed to be divided equally between the inner and outer plates when the areas of the inner and outer plates do not differ by more than 10%.

(See commentary for guidance when difference in area exceeds 10%)

§6.13.6: Bolted Splices - Flange Splices

- Bolted connections for flange splices shall be designed as slip-critical connections for the flange design force.

- As a minimum, for checking slip of the flange splice bolts, the design force for the flange under consideration shall be taken as the Service II design stress, $F_s$, times the smaller gross flange area on either side of the splice, where,

$$F_s = \frac{f_s}{R_h}$$

(6.13.6.1.4c-5)

- $f_s$ - maximum flexural stress due to Load Combination Service II at the mid-thickness of the flange under consideration for the smaller section at the point of splice

- $R_h$ - hybrid girder factor. For hybrid sections in which $f_s$ in the flange with the larger stress does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Flange Splices

- Where applicable, lateral bending effects in discretely braced flanges of I-sections (and in discretely braced top flanges of tub sections) shall be considered in the design of the bolted flange splices.

- The traditional elastic vector method may also be used in these cases to account for the effects of flange lateral bending on the design of the splice bolts.

- Splice plates subject to flange lateral bending should also be designed at the strength limit state for the combined effects of the calculated design shear and design moment acting on the bolt group.

- The shear on the flange bolt group is assumed caused by the flange force, which is calculated without consideration of the flange lateral bending.

- At the strength limit state, the design moment is taken as the lateral bending moment due to the factored loads multiplied by the factor, $R_{Y'}$.

- Lateral flange bending can be ignored in the design of top flange splices once the flange is continuously braced.
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Fillers

When bolts carrying loads pass through fillers 1/4” or more in thickness in axially loaded connections, including girder flange splices, either:

- The fillers shall be extended beyond the gusset or splice material, and the filler extension shall be secured by enough additional bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler, or…

\[ R = \frac{(1+\gamma)}{(1+2\gamma)} \quad (6.13.6.1.5-1) \]

\[ \gamma = \frac{A_f}{A_p} \]

- \( A_f \) - sum of the area of the fillers on the top and bottom of the connected plate
- \( A_p \) - smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate
§6.13 - Connections and Splices

§6.13.6: Bolted Splices - Fillers

- For slip-critical connections, the factored slip resistance of a bolt at the Service II load combination shall not be adjusted for the effect of the fillers.

- Fillers 1/4" or more in thickness shall consist of not more than two plates, unless approved by the Engineer.

- For bolted web splices with thickness differences of 1/16" or less, no filler plates are required.

- The specified minimum yield strength of fillers 1/4" or greater in thickness should not be less than the larger of 70% of the specified minimum yield strength of the connected plate and 36 ksi.

§6.13 - Connections and Splices

§6.13.6: Elastic Vector Method

The Elastic Vector Method Produces Conservative Results

- Assume Zero Friction on the Faying Surface
- Replace Eccentric Load with a Concentric Load and Torque
- Distribute Forces in Proportion to the Distance from the Center of Gravity of the Bolt Group
§6.13 - Connections and Splices

§6.13.6: Elastic Vector Method

Consider the bracket shown in the following figure.

\[ T = e P \]

§6.13 - Connections and Splices

§6.13.6: Elastic Vector Method

Equivalent Actions Distributed to the Bolts

\[ P \]
§6.13 - Connections and Splices

§6.13.6: Elastic Vector Method

\[ V_{total} = V_{direct} + V_{torsion} \]  
(Shear forces must be added vectoraly)

\[
P_{Vtotal} = P_{direct} + P_{torsion} \]

(Shear forces must be added vectoraly)

In the context of shear stress:

\[
\tau_{total} = \tau_{direct} + \tau_{torsion} = \frac{P}{nA} + \frac{Td}{J} \]

where:

- \( n \) - Number of bolts in the group
- \( A \) - Area of one of the bolts in the group
- \( J \) - Polar moment of inertia of the bolt group

\[
J = I_P = I_X + I_Y
\]

\[
I_X = \sum_{i=1}^{n} \left[ I_x + Ad_i^2 \right]
\]

\[
I_Y = \sum_{i=1}^{n} \left[ I_y + Ad_i^2 \right]
\]

\[
I_z = I_y = \left( \frac{\pi}{64} \right) d_b^4
\]

Since \( I_x \) and \( I_y \) are generally small compared to \( Ad^2 \), they can be neglected, which greatly simplifies calculations while introducing only a small error.
§6.13 - Connections and Splices

§6.13.6: Elastic Vector Method

\[ J = I_p = I_x + I_y = \sum_{i=1}^{n} \left[ \frac{d_{i,x}^2}{d_i} + A d_{i,y}^2 \right] + \sum_{i=1}^{n} \left[ \frac{d_{i,y}^2}{d_i} + A d_{i,x}^2 \right] \]

Since \( I_x \) and \( I_y \) are generally small compared to \( Ad_i^2 \), they can be neglected, which greatly simplifies calculations while introducing only a small error.

\[ J = \sum_{i=1}^{n} \left[ A d_{i,x}^2 + A d_{i,y}^2 \right] = A \sum_{i=1}^{n} \left[ d_{i,x}^2 + d_{i,y}^2 \right] = A \Sigma d_i^2 \]

\[ \tau_{\text{total}} = \frac{P}{nA} + \frac{T d_i}{A \Sigma d_i^2} \quad \left( \tau_{\text{total}} \right) (A) = \frac{V_{\text{total}}}{n} + \frac{T d_i}{\Sigma d_i^2} \]

§6.13 - Connections and Splices

§6.13.6: Elastic Vector Method

The horizontal and vertical components of shear due to torsion can be found as,

\[ V_{i,x} = \left( \frac{d_{i,x}}{d_i} \right) V_i = \left( \frac{d_{i,x}}{d_i} \right) \left( \frac{T d_i}{\sum (d_i^2)} \right) = \left( \frac{T d_{i,x}}{\sum (d_i^2)} \right) \]

\[ V_{i,y} = \left( \frac{d_{i,y}}{d_i} \right) V_i = \left( \frac{d_{i,y}}{d_i} \right) \left( \frac{T d_i}{\sum (d_i^2)} \right) = \left( \frac{T d_{i,y}}{\sum (d_i^2)} \right) \]

These can then be added to the direct shear in each direction to find the Maximum shear force in the bolt.

\[ V_{\text{total}} = \sqrt{V_{i,x} + \frac{P}{n}}^2 + \left( V_{i,y} + \frac{P}{n} \right)^2 \]
Steel Bridges:
Cost Effective Design

James A Swanson

References

- National Steel Bridge Alliance Web Site
  - http://www.steelbridge.org/

- Preferred Practices for Steel Bridge Design, Fabrication, and Erection
  - Texas Dept. of Transportation

- Design for Constructability
  - Tom Wandzilak – High Steel Structures
Span Configuration

Simple-Span Girders

- Often results in heavier girder sections
- Usually easier / faster to erect than continuous girders
- Erection savings may offset material costs
- Drawback: Extra expansion joints = Maintenance headaches…
- Simple for DL / Continuous for LL gaining popularity

Two-Span Girders

- Not often the most economical choice b/c of high negative moments
- 3 or 4 span girder are usually preferable
- Clear Zones can sometimes drive the decision making process
**Span Configuration**

*Three- and Four-Span Girders*

- Generally the preferred solution
- Arrangements over four spans are typically discouraged
- Interior spans are generally 20 to 30% longer than exterior spans
- Shorter exterior spans may result in uplift at abutments (bad)

**Span Arrangements:**
- End Spans $\cong 0.8$ Interior Spans is economical
- End Spans $\cong$ Interior Spans is OK, too.
- With integral abutments acting as counter weights…
  …End Spans $\cong 0.6$ Interior Spans can work

---

**Girder Spacing**

*High Steel Suggests….*

- Increased girder spacing leads to lower costs:
  - Fewer girders to fabricate
  - Fewer girders to ship
  - Fewer girders to erect
- Added weight per girder is offset by fewer girders
- Use to $S = 10 - 11$ ft. with $L \leq 140$ ft.
- Use to $S = 11 - 12$ ft. with $L > 140$ ft.
- Increased deck thickness may lengthen service life
- Increased DL in the deck may reduce vibrations

---
Girder Spacing

**TxDOT Suggests....**

- Increased girder spacing leads to lower costs, but...
- Wider spacing creates difficulties with erection and deck design
  - Max spacing of 8' to 9' needed for removable formwork
- Max girder spacing should be based on span length of an 8” deck
- Precast deck panels are preferred → $S_{\text{max}} = 8' - 6”$
- Use a minimum of 4 girder lines in any structure

---

**NSBA Suggests....**

- SIP forms permit the use of larger girder spacings, which can be more efficient
- Girder spacings in the range of 11’ to 14’ generally provide the most economical solution
  - Reduces web material, which is not 100% utilized
  - Somewhat smaller spacing is economical for rolled sections
- Important to balance deck overhang so that the exterior girder moments are similar to interior girder moments.
  - An overhang = 30% of interior spacing is a good rule of thumb.
  - Remember the $d_{\text{eq}}$ limitation of 3’ for $DF$ eqns
Span Arrangement

Longer Spans or Shorter Spans???

- Try to balance superstructure and substructure costs...
  ...Goal = lower total cost

- Longer spans = fewer substructure units = lower substructure costs...
  ...but also = higher superstructure costs

- Shorter spans = lower superstructure costs...
  ...but also = additional substructure units = high substr costs
Span Arrangement

**Fewer Girders = …**
- …Less Welding
- …Fewer Cross Frames
  - Reduced crane time
  - Reduced labor costs
- …Fewer lifts during erection
  - Reduced crane time
  - Reduced labor costs
- …Heavier lifts, though
  - Larger crane may be required

**Longer Spans or Shorter Spans???**
- Piers in water = increased substructure costs
  - Cofferdams
  - Dewatering
  - Barge-mounted equipment
- Poor soil conditions = increased substructure costs
- Construction near or over railroads = increased costs

*Must consider site-access costs when developing a preliminary plan…*
Span Arrangement

Consider Minimized Life-Cycle Costs

- Future Redecking
  
  Many owners are now requiring a plan for future deck replacement using staged construction maintaining traffic on half of the structure.

  This may require the an extra girder but the added costs may be easily offest by reduced costs of redecking in the future.

Rolled Beam vs. Plate Girders

Rolled Beams

- Reduced fabrication cost may lead to economy
- Availability may be an issue
- Allow a plate-girder alternate in the contract
Rolled Beam vs. Plate Girders

Plate Girders

- **Easier to inventory**
  - It is easier for fabricators for stock plates than shapes

- **Allow more customization by designers**
  - Flange Thicknesses / Web Thicknesses
  - Hybrid girder options
  - $F_y > 50$ksi

- **Be Careful!!**  Least Weight ≠ Least Cost

- **Savings in steel can easily be overshadowed by increased labor**

Steel Selection

Plate Girders...

- **Use of HPS-70W is encouraged**
  - Life-cycle maintenance costs are lower
  - Generally most economical in hybrid configurations…
  - 70W in bottom flanges and top flanges in Negative Moment Regions

- **Avoid the use of Grade 36 steel for primary members**
  - No cost difference with Grade 50…

- **Use of Grade 100 or 100W steel is strongly discouraged at this point**

- **Rolled sections are not available in HPS-70W or HPS-100W**

- **Angles (for cross frames) are generally not available in Grade 50**
Handling and Shipping Considerations

General Handling Considerations

- Length ≤ 125 ft.
- Weight ≤ 35 tons
- Height ≤ 9 ft. tall

Highway Shipping Considerations (Varies by State)

- Length ≤ 175 ft.
- Weight ≤ 80 tons
- Height ≤ 9.5 ft. Upright or 13.5 ft. Horizontal
Flange Details

Flange Width

- Preferable to have a constant flange width along length of the girder
  - It is cheaper to vary flange thickness than flange width
  - If a width must change, do it at a field splice

- Flange width should be specified in increments of 2 to 3"
  - Think about how pieces will be cut from plates (42" or 48" wide)
  - Given time, fabricators can order plates in custom widths

- Flange width should not be less than 12"
  - Think about supporting decking forms or precast deck panels
  - Think about positioning of shear studs in composite bridges

- Preferable to use same width for top and bottom flanges

Flange Details

Flange Width (continued)

- Girder stability during erection = \( f(\text{flange width}) \)

- In general:
  - \( \frac{L}{b_i} \leq 60 \) are stable during erection
  - \( 60 < \frac{L}{b_i} \leq 80 \) are questionable during erection
  - \( \frac{L}{b_i} > 80 \) require temporary bracing / support during erection
Flange Details

Flange Thickness

- **Good Practice:** Min Flange Thickness = 3/4”
- Use maximum thicknesses of 3”
  - Weld time increases disproportionately with thicker plates
  - Grade 50 and HPS-70 Q&T available up to 4”
  - HPS-70W TMCP available to up 2”

- Flange thickness should be specified in:
  - 1/8” increments from 3/4” to 1”
  - 1/4” increments from 1” to 3”
  - 1/2” increments from 3” to 4”

Flange Details

Flange Thickness (continued)

- Consider a flange splice only if it will save more than 800 to 1,000 lbs
- Use minimum segment lengths of 10 ft.
- Four to six flange sizes are reasonable for continuous girders
- Two to three flange sizes are reasonable for simple girders
- Use common flange sizes (~8) on jobs with multiple structures
- Be aware of “Slabbing and Stripping” practices
Flange Details

Flange Thickness (continued)

- Flange splices should result in a change in area of at least 25%
- The thinner flange should not be less than 1/2 the thickness of the thicker flange
Web Details

- Specify web depths in increments of 2 or 3".
- Try to maintain a ratio of $L / D_w$ in the range of 25:1 or 30:1.
- Specify a minimum web thickness of 1/2".
- Longitudinal stiffeners complicate fabrication. Increase web thickness to preclude their need.
- Consider the added cost of labor in considering transverse stiffeners vs. added web thickness.
- **Web thickness should be specified in:**
  - 1/16" increments from 1/2" to 3/4"
  - 1/8" increments from 3/4" to 1"
  - 1/4" increments above 1"

Flange-to-Web Welds

- 5/16" AWS Minimum usually works.
- Anything larger than 3/8" will require multiple passes, which will substantially increase cost.
Field Splices

- Located at DL inflection points
- Offer the option to eliminate field splices

Expansion Joints