AASHTO T-14
2016 Agenda Items
Proposed Revisions to LRFD BDS Section 6

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T-14 Agenda Item No. 12
Section 6, Articles 6.2, 6.6.1.2.4 and 6.6.1.3.2

• Description of Proposed Revisions:
  • Item #1:
    ➢ Add new definitions in Article 6.2 for a ‘Lateral Connection Plate’ and a ‘Transverse Connection Plate’.
  • Items #2 through #4:
    ➢ Add two new tables in Article 6.6.1.2.4 providing recommended details to avoid conditions susceptible to constraint-induced fracture in regions subject to net tensile stress under Strength I.
      • Table 6.6.1.2.4-1 -> intersections of longitudinal stiffeners & vertical stiffeners welded to the web
      • Table 6.6.1.2.4-2 -> intersections of lateral connection plates & vertical stiffeners welded to the web
  • Items #5 and #6:
    ➢ Revise provisions of Article 6.6.1.3.2 to be consistent with the details shown in the new Table 6.6.1.2.4-2.
Note 1: If a gap is specified between the weld toes, the recommended minimum distance between the weld toes is 0.75 in., but shall not be less than 0.5 in. Larger gaps are also acceptable.
T-14 Agenda Item No. 13
Section 6, Articles 6.2, 6.7.2 and 6.17

Description of Proposed Revisions:

- NCHRP Project 12-79: “Guidelines for Analysis Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges”
- NCHRP Project 20-07, Task 355: “Guidelines for Reliable Fit-Up of Steel I-Girder Bridges”

- Item #1:
  - Introduces 9 new definitions in Article 6.2.
- Item #2:
  - States more explicitly issues to be considered in analysis for phased construction or staged deck placement when establishing girder cambers (Article 6.7.2).
- Items #3 and #4:
  - Revise language to indicate that for certain specified bridge types, the contract documents should state the fit condition for which the cross-frames are to be detailed (Articles 6.7.2 & C6.7.2).
T-14 Agenda Item No. 13
Section 6, Articles 6.2, 6.7.2 and 6.17

• Description of Proposed Revisions:

• The contract documents should state the fit condition for which the cross-frames or diaphragms are to be detailed for the following I-girder bridges:

  ➢ Straight bridges where one or more support lines are skewed more than 20 degrees from normal;
  ➢ Horizontally curved bridges where one or more support lines are skewed more than 20 degrees from normal and with an $L/R$ in all spans less than or equal to 0.03; and
  ➢ Horizontally curved bridges with or without skewed supports and with a maximum $L/R$ greater than 0.03.

where:
$L =$ span length bearing to bearing along the centerline of the bridge
$R =$ radius of the centerline of the bridge cross-section
Fit Condition – deflected girder geometry associated with a targeted dead load condition for which the cross-frames are detailed to connect to the girders.
<table>
<thead>
<tr>
<th>Loading Condition Fit</th>
<th>Construction Stage Fit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>No-Load Fit (NLF)</td>
<td>Fully-Cambered Fit</td>
<td>The cross-frames are detailed to fit to the girders in their fabricated, plumb, fully-cambered position under zero dead load.</td>
</tr>
<tr>
<td>Steel Dead Load Fit (SDLF)</td>
<td>Erected Fit</td>
<td>The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under bridge steel dead load at the completion of the erection.</td>
</tr>
<tr>
<td>Total Dead Load Fit (TDLF)</td>
<td>Final Fit</td>
<td>The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge total dead load.</td>
</tr>
</tbody>
</table>
### T-14 Agenda Item No. 13
Section 6, Articles 6.2, 6.7.2 and 6.17

**Recommended Fit Conditions for Straight I-Girder Bridges**
(including Curved I-Girder Bridges with $L/R$ in all spans $\leq 0.03$)

<table>
<thead>
<tr>
<th>Square Bridges and Skewed Bridges up to 20 deg Skew</th>
<th>Recommended</th>
<th>Acceptable</th>
<th>Avoid</th>
</tr>
</thead>
<tbody>
<tr>
<td>Any span length</td>
<td>Any</td>
<td></td>
<td>None</td>
</tr>
<tr>
<td>Skewed Bridges with Skew $&gt; 20$ deg and $I_s \leq 0.30$ +/-</td>
<td>TDLF or SDLF</td>
<td>NLF</td>
<td></td>
</tr>
<tr>
<td>Skewed Bridges with Skew $&gt; 20$ deg and $I_s &gt; 0.30$ +/-</td>
<td>SDLF</td>
<td>TDLF</td>
<td>NLF</td>
</tr>
</tbody>
</table>

Span lengths up to 200’ +/-

Span lengths greater than 200’ +/-

![Formula Image](Image)

$$I_s = \frac{w_g \tan \theta}{L_s}$$
T-14 Agenda Item No. 13
Section 6, Articles 6.2, 6.7.2 and 6.17

Recommended Fit Conditions for Horizontally Curved I-Girder Bridges

\((L/R)_{MAX} > 0.03\)

<table>
<thead>
<tr>
<th>Radial or Skewed Supports</th>
<th>Recommended</th>
<th>Acceptable</th>
<th>Avoid</th>
</tr>
</thead>
<tbody>
<tr>
<td>((L/R))_{MAX} ≥ 0.2</td>
<td>NLF</td>
<td>SDLF</td>
<td>TDLF</td>
</tr>
<tr>
<td>All other cases</td>
<td>SDLF</td>
<td>NLF</td>
<td>TDLF</td>
</tr>
</tbody>
</table>

- Total Dead Load Fit should not be specified for curved I-girder bridges with or without skew and with a maximum \(L/R\) greater than 0.03.
- Detail for a Steel Dead Load Fit, unless than maximum \(L/R\) is greater than or equal to 0.2.
- When \((L/R)_{MAX} ≥ 0.2\), detail for No-Load Fit, unless the additive locked-in force effects from Steel Dead Load Fit detailing are considered.
In straight skewed I-girder bridges, dead load force effects are partially *offset* by the corresponding locked-in force effects.

For straight skewed I-girder bridges detailed for a Total Dead Load Fit, a net reduced load factor may be applied to the unfactored total dead load cross-frame forces and flange lateral bending stresses (Article C6.7.2):

\[
(\gamma_p)_{\text{red}} = (\gamma_p - 0.4)
\]

**Anticipated Effect on Bridges:**

- Stronger understanding of the implications of dead load camber and cross-frame detailing methods for skewed and curved bridge amongst the various stakeholders.
- Safer and more consistent practices to reduce the possibility of construction delays and claims.
- Use of a Total Dead Load Fit discouraged for curved I-girder bridges with a maximum \(L/R\) greater than 0.03.
T-14 Agenda Item No. 16
Section 6, Articles 6.7.4.1, 6.7.4.2 & 6.17

• Description of Proposed Revisions:
  
  • Item #1:
    ➢ Additional bullet item in Article 6.7.4.1 to emphasize importance of cross-frames in controlling torsional stresses & rotations due to eccentric deck overhang loads.
  
  • Item #2:
    ➢ Language added to Article C6.7.4.2 to discuss beneficial framing arrangements in skewed and curved I-girder bridges to alleviate deleterious transverse stiffness effects.
    ➢ Revision to recommended offset of first intermediate cross-frame placed normal to the girders adjacent to a skewed support.

<diagram>
T-14 Agenda Item No. 16
Section 6, Articles 6.7.4.1, 6.7.4.2 & 6.17

• Description of Proposed Revisions:

- Framing of a normal intermediate cross-frame into or near a bearing location along a skewed support line is strongly discouraged unless the cross-frame diagonals are omitted.
- At skewed interior piers, place cross-frames along the skewed bearing line and locate intermediate cross-frames greater than or equal to the recommended minimum offset from the bearing lines.
- For curved I-girder bridges, provide contiguous intermediate cross-frame lines within the span in combination with the recommended offset at skewed bearing lines.
T-14 Agenda Item No. 16
Section 6, Articles 6.7.4.1, 6.7.4.2 & 6.17

• Anticipated Effect on Bridges:
  • Stronger understanding of the implications of various framing arrangements for skewed and curved I-girder bridges amongst the various stakeholders.
  • Greater avoidance of deleterious transverse stiffness effects in sharply skewed bridge spans.
  • Achieve design economies by reducing the number of cross-frames and avoiding the large transverse stiffness load paths.
T-14 Agenda Item No. 18  
Section 6, Article 6.10.3.4.2

- Description of Proposed Revisions:

- Items #1 through #3:
  - Eigenvalue buckling & large displacement analyses were recently conducted at the University of Texas.
  - Revise Eq. 6.10.3.4.2-1 to include a system moment-gradient modifier, $C_{bs}$, as follows:

\[
M_{gs} = C_{bs} \frac{\pi^2 w_g E}{L^2} \sqrt{I_{eff} I_x}
\]

-\[C_{bs} = \begin{cases} 1.1 & \text{for simply-supported units} \\ 2.0 & \text{for continuous-span units} \end{cases}\]
T-14 Agenda Item No. 18
Section 6, Article 6.10.3.4.2

• Description of Proposed Revisions:
  • Items #1 through #3:
    ➢ Compare the buckling capacity, \( M_{gs} \), with the sum of the largest total factored girder moments within the span (rather than the sum of the largest total factored positive girder moments).
    ➢ Raised the limit on the sum of the largest moments within the span from 50% to 70% of \( M_{gs} \).
    ➢ Narrow curved I-girder units should be analyzed using a global second-order load-deflection analysis. Alternatively, add lateral bracing adjacent to supports of the span, or brace unit to other units or with external bracing (Article C6.10.3.4.2).

• Anticipated Effect on Bridges:
  • Larger elastic global lateral-torsional buckling resistances should result for narrow straight 2- and 3-girder systems in their noncomposite condition during the deck placement.
T-14 Agenda Item No. 21
Section 6, Articles 6.13.2.7, 6.13.2.12 & 6.17

• Description of Proposed Revisions:

• Items #1 through #5:
  ➢ The built-in joint length reduction factor in lap splice tension connections increased from 0.80 to 0.90.
  ➢ Limiting connection length for the above decreased from 50.0 in. to 38.0 in.
  ➢ Bolt shear-to-tensile strength ratio increased from 0.60 to 0.625.
  ➢ Result: increase in the nominal shear resistance of high-strength bolts at the strength limit state (Article 6.13.2.7):

\[
R_n = 0.48A_bF_{ub}N_s \quad R_n = 0.56A_bF_{ub}N_s \quad (X)
\]

\[
R_n = 0.38A_bF_{ub}N_s \quad R_n = 0.45A_bF_{ub}N_s \quad (N)
\]

➢ For lap splice tension connections greater than 38.0 in. in length, the reduction factor is 0.75. Multiply \( R_n \) by 0.83 (or 0.75/0.90).
T-14 Agenda Item No. 21
Section 6, Articles 6.13.2.7, 6.13.2.12 & 6.17

• Description of Proposed Revisions:

• Items #6 and #7:
  ➢ Shear resistance equation for anchor rods implicitly assumes threads included in shear plane since thread length not limited.
  ➢ ASTM A 307 Grade C eliminated.
  ➢ Joint length factor of 0.90 is not applied to anchor rods.
  ➢ Result (Article 6.13.2.12):

\[
R_n = 0.48A_b F_{ub} N_s \quad R_n = 0.50A_b F_{ub} N_s \quad (N)
\]

• Anticipated Effect on Bridges:

• In general, fewer bolts will be required in high-strength bolted connections to satisfy the shear resistance provisions at the strength limit state.
T-14 Agenda Item No. 22
Section 6, Articles 6.13.2.8 & 6.17

• Description of Proposed Revisions:

• Items #1 through #6:
  - Values of surface condition factor, $K_s$, in Table 6.13.2.8-3 used in calculating nominal slip resistance of a high-strength bolt in a slip-critical connection revised as follows:

<table>
<thead>
<tr>
<th>Surface Conditions</th>
<th>$K_s$ Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A</td>
<td>0.33 0.30</td>
</tr>
<tr>
<td>Class B</td>
<td>0.50</td>
</tr>
<tr>
<td>Class C</td>
<td>0.33 0.30</td>
</tr>
<tr>
<td>Class D</td>
<td>0.45</td>
</tr>
</tbody>
</table>

- Requirement to roughen hot-dip galvanized surfaces after galvanizing by hand-wire brushing is removed.
- Unsealed thermal-sprayed pure zinc or 85/15 zinc aluminum coatings ($t_{coating} \leq 16$ mils) are classified as Class B surfaces.
- Sealed thermal-spray coatings not included – must be qualified by test.
Anticipated Effect on Bridges:

- The introduction of a Class D surface condition is not anticipated to have a significant effect on the overall number of bolts and permits more coating options.
- The reduction in $K_s$ for Class A and Class C surface conditions from 0.33 to 0.30 may potentially lead to a small increase in the number of bolts for these surface conditions since the smaller coefficient may lead to slip controlling the number of bolts (vs. strength).
T-14 Agenda Item No. 24
Section 6, Various Articles (2)

• Description of Proposed Revisions:

  • Items #1 through #4:
    ➢ New Article 6.4.3.1 on “High-Strength Fasteners” – introduces new ASTM F3125 standard for high-strength bolts.
    ➢ Streamline articles on nuts and washers – refer to F3125 standard.
    ➢ New article on “Direct Tension Indicators” (DTIs)
  • Items #5 and #6:
    ➢ New Article 6.4.3.2 on “Low-Strength Steel Bolts” (ASTM A307 bolts)
  • Items #7 and #8:
    ➢ New Article 6.4.3.3 on “Fasteners for Structural Anchorage”
    ➢ ASTM F1554 only; nuts ASTM A563 or A194 Grade 2H
    ➢ Term “anchor bolts” changed to “anchor rods” throughout Section 6.
T-14 Agenda Item No. 24
Section 6, Various Articles (2)

• Description of Proposed Revisions:

  • Item #9:
    ➢ Revise resistance factor definitions for high-strength bolts in Article 6.5.4.2 to reflect new ASTM F3125 standard.
  • Items #11 and #12:
    ➢ Revise Tables 6.6.1.2.5-1 (Detail Category Constant, A) and 6.6.1.2.5-3 (Constant Amplitude Fatigue Threshold) to reflect new F3125 standard.
  • Items #13 and #14:
    ➢ Remove list of conditions/exceptions where hardened washers are required for connections using F3125 bolts – defer to similar list in Article 11.5.6.4.3 of LRFD Bridge Construction Specs.
  • Item #16:
    ➢ Revise minimum required bolt tension values in Table 6.13.2.8-1 to reflect new strength level of 120 ksi for Grade A325 or F1852 high-strength bolts over 1 inch in diameter.
ASTM

Combined Structural Bolt Specification

F3125-15a Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions

Slides courtesy of:

Chad Larson
President - LeJeune Bolt Company
Producer Vice Chair - ASTM F16 Fastener Committee
Chair - ASTM Subcommittee F16.02 on Steel Bolts, Nuts, Rivets and Washers
What is F3125?

New Standard F3125

A325M
F1852
F2280
A490M

A325
New Standard F3125

A490
Structural Bolt Grades

120 ksi Min Tensile – “Group A”
- A325
- A325M
- F1852

150 ksi Min Tensile – “Group B”
- A490
- A490M
- F2280
Structural Bolt Styles

- Heavy Hex Head
  - A325 Group A
  - A325M Group A
  - A490 Group B
  - A490M Group B

- Twist-Off
  - F1852 Group A
  - F2280 Group B
What Now?

• What does this mean for existing inventory, projects underway, and future structures which will end up with bolts made to both the new specification and the previous specifications? In short, nothing.

• There will be a difference in how the fasteners are called out, and in some cases minor differences in technical requirements, but no changes resulting in the need for connection design or code changes. Users must simply understand that the grades they are familiar with now reside under a parent document.
T-14 Agenda Item No. 24
Section 6, Various Articles (2)

• Anticipated Effect on Bridges:
  • Allows the use of a single specification for high-strength bolts.
  • Allows for greater consistency in the specification, provision, and testing of all types of high-strength bolts.
  • Single higher strength level for Grade A325 and F1852 bolts over 1 inch in diameter may make larger diameter bolts more attractive for design.
T-14 Agenda Item No. 25
Section 6, Various Articles (3)

- Description of Proposed Revisions:
  - Introduces the unified effective width approach for the calculation of the nominal compressive resistance of members with slender element cross-sections (Articles 6.9.4.1 & 6.9.4.2).
    - Accounts for effect of potential local buckling of slender elements, supported along one or two longitudinal edges, on the overall column-buckling resistance of the member.
    - Replaces the current Q-factor approach to handle compression members with slender elements – originally adopted in the 1969 AISC and AISI Specifications.
    - Revises Table 6.9.4.2.1-1 to replace “plate-buckling coefficients”, $k$, with corresponding width-to-thickness ratio limits, $\lambda_r$. 
Description of Proposed Revisions:

- Removes reference to the terms “unstiffened elements” and “stiffened elements” in the specification and commentary.
- Nominal compressive resistance, $P_n$, obtained by multiplying $F_{cr}$ based on the gross cross-sectional area by an effective area, $A_{eff}$.
- $A_{eff}$ generally computed as the summation of effective areas of the cross-section based on reduced effective widths, $b_e$, for each slender element in the cross-section (Article 6.9.4.2.2a).
- For circular tubes and round HSS, $A_{eff}$ is computed directly from equations (Article 6.9.4.2.2b).

Anticipated Effect on Bridges:

- More streamlined approach that applies to slender plate elements supported along one or two longitudinal edges.
- Adoption will assist with the future implementation of new LRFD Design Specifications for noncomposite steel box sections.
T-14 Agenda Item No. 26
Section 6, Various Articles (4)

• Description of Proposed Revisions:
  • Items #1 and #4:
    ➢ De-emphasis of language permitting bearing stiffeners to be attached to flange receiving its load with full penetration groove welds.
    ➢ Bearing stiffener should be finished-to-bear against flange receiving its load (Article 6.10.11.2.1).
    ➢ Bearing stiffeners serving as connection plates are to be attached to both flanges.
  • Items #2 and #3:
    ➢ Minimum distance requirements between end of web-to-transverse stiffener weld and longitudinal stiffener-to-web weld is removed (Article 6.10.11.1.1).
    ➢ Language regarding avoidance of intersecting welds is removed (Article C6.10.11.1.1).
T-14 Agenda Item No. 26
Section 6, Various Articles (4)

- Items #6 through #8:
  - Revise language in Article 6.10.11.3.1 to be consistent with the recommended details shown in Table 6.6.1.2.4-1.
  - Where more than one pair of bearing stiffeners is used, longitudinal stiffeners are not to be continued in-between the stiffener pair.

- Anticipated Effect on Bridges (Agenda Items 12 & 26):
  - Improved details at intersections of longitudinal stiffeners & lateral connection plates with vertical stiffeners should help avoid conditions susceptible to constraint-induced fracture.
  - Continuous longitudinal stiffeners will be less susceptible to load-induced fatigue concerns & will perform as intended to control web bend-buckling.
  - Discouraging full-penetration groove welds to attach bearing stiffeners to the flange will provide economy & less welding deformation.
T-14 Agenda Item No. 27
Section 6, Various Articles (5)

• Description of Proposed Revisions:
  • Item #1:
    ➢ Adds explanation of ‘matching weld metal’ to Article C6.13.3.1.
  • Item #2:
    ➢ Clarifies definition of the weld metal classification strength, $F_{exx}$, given in Article C6.13.3.2.1.
  • Item #3:
    ➢ Removes language in Article C6.13.3.2.3a leftover from the 4th Edition Specification.
  • Items #4 through #6:
    ➢ Removes Article 6.13.3.2.4a on fillet-welded connections loaded in tension and compression.
    ➢ Adds language indicating the shear resistance of a fillet-welded connection is to be taken as the smaller of the factored shear rupture resistance of the connected material adjacent to the welded leg, and the factored shear resistance of the weld metal.
T-14 Agenda Item No. 27
Section 6, Various Articles (5)

• Description of Proposed Revisions:
  
  • Items #7 through #10:
    - Renames Article 6.13.3.7 to “Fillet Welds for Sealing”.
    - Addresses sealing fillet welds around ends of transverse or bearing stiffeners, connection plates, and lap-splice connections.
    - Return portions of the sealing welds exempted from the minimum weld size requirements and the resistance calculation for the connection.
    - Commentary added to recommend use of such welds in galvanized structures, with vent holes added where required.
    - Further indicates that undercutting of the corner of a stiffener when the fillet weld is wrapped around the end does not affect the fatigue performance of the weld.

• Anticipated Effect on Bridges:
  
  • Improved sealing against corrosion at ends of stiffeners.
  • Less chance of inadvertently limiting a fabricator to manual stick welding when defining the classification strength of the weld metal.
T-14 Agenda Item No. 28
Section 6, Various Articles (6)

• Description of Proposed Revisions:
  • Items #1 through #6 & Item #23:
    - Remove applicability of the 75 percent and average rules in Article 6.13.1 to the design of bolted and welded splices for flexural members.
    - Rules are applicable to connections and splices for primary members subject to axial tension or compression only.
    - Clarify application of rules to primary members subjected to force effects acting in multiple directions due to combined loading.
T-14 Agenda Item No. 28
Section 6, Various Articles (6)

• Description of Proposed Revisions:

  • Items #7 through #11:
    ➢ Revise general article on design of bolted splices for flexural members to reflect new procedures.
    ➢ Renumbering of articles
    ➢ Removal of check for slip of bolts during erection of steel

  • Items #12 through #22:
    ➢ Implement new simplified bolted splice design procedure within Article 6.13.6.1.
Splice Design Procedure

1. Design Flange Connection to Develop the Smallest Design Yield Resistance of the Flanges

Design Yield Resistance: \( P_{fy} = F_{yf}A_e \)

\[
A_e = \left( \frac{\phi_u F_u}{\phi_y F_{yf}} \right) A_n \leq A_g
\]

2. Design Web Connection to Develop the Smallest Factored Shear Resistance of the Web

\( V_r = \phi_v V_n \)

Two Rows of bolts minimum on each side of splice.

“Simplified Design of Bolted Splice Connections for Steel Girders” – Frank, Ocel, and Grubb
Positive Moment Capacity Check
Bottom Flange in Tension

Moment Capacity = \( P_{fy} \) for the Bottom Flange x Moment Arm to Mid-Depth of Deck
= \( (F_{yf} \times A_e) \times A \)

\[ A = D + \frac{t_{ft}}{2} + t_{haunch} + \frac{t_s}{2} \]

\[ P_{fy} = F_{yf} A_e \]
Negative Moment Capacity Check

Ignore Tensile Strength of Reinforcement

\[ P_{fy} (\text{top}) = F_{yf} A_e \]
\[ A = D + \frac{t_{ft}}{2} + \frac{t_{fc}}{2} \]
\[ P_{fy} (\text{bot.}) = F_{yf} A_e \]

Moment Capacity = Smallest Value of \( P_{fy} \) x Distance Between Flange Centroids
\[ = (F_{yf} x A_e)_{\text{smallest}} x A \]

“Simplified Design of Bolted Splice Connections for Steel Girders” – Frank, Ocel, and Grubb
Web Resultant

\[ H_w = \frac{(\text{Additional required web moment})}{A_w} \]

\[ R = \sqrt{(V_r)^2 + (H_w)^2} = \sqrt{(\phi_v V_n)^2 + (H_w)^2} \]

Number Bolts Required = \( R / \text{Bolt Capacity} \)

Minimum of Two Rows each side of splice
Additional Web Force
If Moment From Flanges is Not Sufficient

- Additional Required Web Moment = Factored Design Moment \( - (P_{fy} \times A) \) (flange moment)

- Web Contribution: \( H_w \times A_w \) (web force at mid depth)

\[
A_w = \frac{D}{2} + t_{haunch} + \frac{t_s}{2}
\]

Positive Moment

\[
P_{fy} = F_{yf}A_e
\]

Negative Moment

\[
A_w = \frac{D}{2} + \frac{t_f}{2}
\]

Largest flange force = \( P_{fy} = F_{yf}A_e \)
## Design Comparisons

<table>
<thead>
<tr>
<th>Girder Depth In.</th>
<th>Required Number of Bolts One Side</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number of Top Flange Bolts</td>
<td>Number of Web Bolts</td>
</tr>
<tr>
<td>72</td>
<td>12</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>22</td>
</tr>
<tr>
<td>111</td>
<td>24</td>
<td>102</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>80 Tub Girder</td>
<td>16</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>28</td>
</tr>
</tbody>
</table>
Model Description

- Shell element models in Abaqus
- Adapted fastener models from NCHRP 12-84
- Five loading scenarios
  - Pure positive moment
  - Pure negative moment *
  - High shear (as little moment as possible) *
  - Proportion design positive moment/shear
  - Proportional design negative moment/shear *

* = deck not present
Results – High Shear

![Graph showing the comparison between current and proposed methods for high shear analysis. The graph displays the relationship between left support reaction (kip) and left support displacement (inches). The proposed method is represented by a red line, while the current method is shown in black. Step 26 is indicated on the graph.]

“Simplified Design of Bolted Splice Connections for Steel Girders” – Frank, Ocel, and Grubb
Results – High Shear
Von Mises Stresses @ Step 26

CURRENT

PROPOSED

“Simplified Design of Bolted Splice Connections for Steel Girders” – Frank, Ocel, and Grubb
Results – High Shear

Bolt Shear Forces @ Step 26

CURRENT

PROPOSED

"Simplified Design of Bolted Splice Connections for Steel Girders" – Frank, Ocel, and Grubb
Results – PropNegMomShear

![Graph showing the comparison between the Current Method and the Proposed Method for Left Support Reaction and Left Support Displacement. The graph includes a horizontal line representing the Vu value.](image-url)

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Von Mises Stresses @ $V_u$

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Bolt Shear Forces @ $V_u$

“Simplified Design of Bolted Splice Connections for Steel Girders” – Frank, Ocel, and Grubb
T-14 Agenda Item No. 28
Section 6, Various Articles (6)

• Anticipated Effect on Bridges:
  • Application of the new proposed design provisions for bolted field splices will typically result in a few more bolts in the flange splices and significantly fewer bolts in the web splices than under the current design provisions.
  • The overall simplification in the design procedure should result in easier interpretation of the provisions, faster and more efficient design of field splices, and more consistent and cost-effective designs.
  • Clarifications to the application of the 75 percent and average rules to the design of connections and splices in primary members at the strength limit state subject to combined force effects should also be beneficial to designers.
T-14/T-5 Agenda Item No. 29
Section 6, Various Articles; Section 4, Articles 4.6.1.2.4b, 4.6.1.2.4c and 4.9

• Description of Proposed Revisions:
  
  • Item #1:
    ➢ Revised definitions in Article 6.2:

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. A steel primary member or portion thereof subject to tension whose failure would probably cause a portion of or the entire bridge to collapse.

Primary Member—A member designed to carry the internal forces determined from an analysis. A steel member or component that transmits gravity loads through a necessary as-designed load path. These members are therefore subjected to more stringent fabrication and testing requirements; considered synonymous with the term “main member”.

Secondary Member—A member in which stress is not normally evaluated in the analysis. A steel member or component that does not transmit gravity loads through a necessary as-designed load path.

Component—A constituent part of a structure or member.
T-14/T-5 Agenda Item No. 29
Section 6, Various Articles; Section 4, Articles 4.6.1.2.4b, 4.6.1.2.4c and 4.9

• Description of Proposed Revisions:
  • Items #2 through #5:
    ➢ Introduce a new Article 6.6.2.1 entitled ‘Member or Component Designations & Charpy V-Notch Testing Requirements’.
      • New Table 6.6.2.1-1 is provided designating various members or components as primary or secondary.
      • Primary members (or portions thereof) subject to a net tensile stress under Strength I are to be designated on contract plans.
      • Charpy V-notch testing is required for primary members subject to a net tensile stress (or portions thereof located in designated tension zones) under Strength I, except for diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in horizontally curved bridges.
T-14/T-5 Agenda Item No. 29
Section 6, Various Articles; Section 4, Articles 4.6.1.2.4b, 4.6.1.2.4c and 4.9

• Description of Proposed Revisions:
  • Items #2 through #5 and #12:
    ➢ Introduce a new Article 6.6.2.2 entitled ‘Fracture-Critical Members (FCMs)’.
      • Contains all existing requirements related to FCMs.
      • Primary members (or portions thereof) that are FCMs are to be designated on contract plans.
      • Members (or portions thereof) not subject to a net tensile stress under Strength I are not to be designated as FCMs.
      • Introduces the concept of a ‘System Redundant Member (SRM)’

_System Redundant Member (SRM)—_A steel primary member or portion thereof subject to tension for which the load-path redundancy is not known by engineering judgment, but which is demonstrated to have load-path redundancy through a refined analysis. SRMs must be identified and designated as such by the Engineer on the contract plans, and designated in the contract documents to be fabricated according to Clause 12 of the *AASHTO/AWS D1.5M/D1.5 Bridge Welding Code*. An SRM need not be subject to the hands-on in-service inspection protocol for a FCM as described in 23 CFR 650.
Description of Proposed Revisions:

Items #6 through #9:
- Revisions to various articles in Section 6 for consistency with the proposed designations of primary and secondary members.
  - Emphasize that force effects in primary cross-frame or diaphragm members in horizontally curved bridges, exceeding one or more of the specified conditions for neglect of curvature, are to be quantified by analysis.

Items #10 and #11:
- Clarify application of limiting slenderness ratios for tension and compression members (Articles 6.8.4 & 6.9.3).

Items #13 through #17:
- Clarify force effects that do not need to be explicitly considered in curved bridges when effects of curvature can be neglected in the analysis (Articles 4.6.1.2.4b & 4.6.1.2.4c).
T-14/T-5 Agenda Item No. 29
Section 6, Various Articles; Section 4, Articles
4.6.1.2.4b, 4.6.1.2.4c and 4.9

• Anticipated Effect on Bridges:

  • Enhanced communication between the Engineer and the Fabricator in identifying primary & secondary members, and primary members and portions thereof subject to tension.
  • Greater clarity is provided related to the identification of FCMs.
  • Introduction of the concept of System Redundant Members (SRMs) into the specifications.
  • Greater economy should result from reducing the need for more costly fabrication & testing requirements for certain members that add little or no value.
  • Greater clarity is provided on the specific force effects that do not need to be explicitly considered in curved bridges when the effects of curvature can be neglected in the analysis.
?? QUESTIONS ??