AASHTO T-14
2019 Agenda Items
Proposed Revisions to LRFD BDS Section 6

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T-14 Agenda Item No. 27
Section 6, Articles 6.3 and 6.8.2.2

Description of Proposed Revisions:

• Item #1:
  ➢ Revise the Notation list in Article 6.3.

• Item #2:
  ➢ Revise the 3rd paragraph of Article 6.8.2.2 as follows:

6.8.2.2—Reduction Factor, $U$

The shear lag reduction factor, $U$, may be calculated as specified in Table 6.8.2.2-1. For open cross-section members composed of more than one element, the calculated value of $U$ should not be taken to be less than the ratio of the gross area of the connected element or elements to the member gross area.
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Section 6, Articles 6.3 and 6.8.2.2

• Item #3:

➢ In Article C6.8.2.2, revise the 1st paragraph and add a new 2nd paragraph as follows:

C6.8.2.2

The provisions of Article 6.8.2.2 are adapted from the 2005 2016 AISC Specification Section D3-3, Effective Net Area for design of tension members. The 2005 AISC provisions are adapted such that they are consistent with updated draft 2010 AISC provisions. These updated provisions specify that, for members composed of more than one element open cross-section members, such as W, M, S, C or HP shapes, tees, and single and double angles, the calculated value of $U$ should not be taken to be less than the ratio of the gross area of the connected element or elements to the member gross area. The preceding provision does not apply to closed sections, such as HSS, nor to plates.

The effect of the moment due to the eccentricities in the connection in angle members and light structural tee members loaded eccentrically in axial tension may be ignored in the design of the member and the connections (AISC, 2016); the effect of the connection eccentricity is addressed through the use of the shear lag reduction factor, $U$. 

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### Section 6, Articles 6.3 and 6.8.2.2

**Table 6.8.2.2-1—Shear Lag Factors for Connections to Tension Members**

<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 cross-sectional elements by fasteners or welds (except as in Cases 4, 4, 5, and 6).</td>
<td>$U = 1.0$</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 shapes, Case 7 may be used. For angles, Case 8 may be used.)</td>
<td>$U = 1 - \frac{x}{L}$</td>
<td>—</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-sectional elements.</td>
<td>$U = 1.0$ and $A_{A_n} = \text{area of the directly connected elements}$</td>
<td>—</td>
</tr>
<tr>
<td>4</td>
<td>Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $x/L$ shall not be less than 4 times the weld size.</td>
<td>$L \geq 2w \ldots U = 1.0$ [L \geq 1.5w \ldots U = 0.87] [1.5w \geq L \geq w \ldots U = 0.75] $U = \frac{3L^2}{3L^2 + w^2} \left(1 - \frac{x}{L}\right)$</td>
<td>—</td>
</tr>
<tr>
<td>5</td>
<td>Round HSS with a single concentric gusset plate through slots in the HSS.</td>
<td>$L \geq 1.3D \ldots U = 1.0$ [D \leq L &lt; 1.3D \ldots U = 1 - \frac{x}{L}] $U = \frac{3L^2}{3L^2 + w^2} \left(1 - \frac{x}{L}\right)$</td>
<td>—</td>
</tr>
</tbody>
</table>

Where:
- $x$ is the distance from the centroid of the cross-sectional element to the line of action of the shear force.
- $L$ is the length of the cross-sectional element.
- $w$ is the weld size.
- $D$ is the diameter of the round HSS.
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<table>
<thead>
<tr>
<th>7</th>
<th>W, M, S, or HP Sshapes or Ttees cut from these shapes (If U is calculated per Case 2, the larger value is permitted to be used.) with flange connected with 3 or more fasteners per line in direction of loading</th>
<th>$b_f \geq \frac{2}{3}d \ldots U = 0.90$</th>
<th>—</th>
<th>$b_f &lt; \frac{2}{3}d \ldots U = 0.85$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>with web connected with 4 or more fasteners per line in direction of loading</td>
<td>$U = 0.70$</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Single and double angles (If U is calculated per Case 2, the larger value is permitted to be used.) with 4 or more fasteners per line in direction of loading</td>
<td>$U = 0.80$</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td></td>
<td>with 2 or 3 fasteners per line in direction of loading (with fewer than 3 fasteners per line in direction of loading, use Case 2)</td>
<td>$U = 0.60$</td>
<td>—</td>
<td></td>
</tr>
</tbody>
</table>

where:
- $L$ = length of connection (in.)
- $w$ = plate width (in.)
- $\bar{x}$ = connection eccentricity (in.)
- $B$ = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection (in.)
- $D$ = outside diameter of round HSS (in.)
- $H$ = overall height of rectangular HSS member, measured in the plane of the connection (in.)
- $d$ = full nominal depth of the section; for tees, depth of the section from which the tee was cut (in.)
- $b_f$ = flange width (in.)
Description of Proposed Revisions:

• Item #1:
  ➢ Revise Article 6.10.1.4 on Variable Web Depth Members as follows:

6.10.1.4—Variable Web Depth Members

The effect of bottom flange inclination shall be considered in determining the bottom flange stress caused by bending about the major-axis of the cross-section, and any potential modifications to the vertical web shear. **In cases where permitted by static equilibrium permits the vertical web shear to be reduced in variable web depth members, only the web dead-load shear may be reduced by the vertical component of the bottom flange force.** At points where the bottom flange becomes horizontal, the transfer of the vertical component of the flange force back into the web shall be considered. **full- or partial-depth transverse stiffening of the web shall be provided, unless the provisions of Article D6.5.2 are satisfied for the factored vertical component of the inclined flange force using a length of bearing** \(N\) **equal to zero.**
Variable Web Depth Members
Parabolic Haunch

Horizontal component of force in flange:
\[ P_h = M \frac{A_f}{S_x} \]

Normal stress in inclined flange:
\[ f_n = \frac{P_h}{A_f \cos \theta} \]

Vertical component of force in flange:
\[ P_v = P_h \tan \theta \]
T-14 Agenda Item No. 28
Section 6, Article 6.10.1.4

Item #2:
 ➢ Revise the 3rd paragraph of Article C6.10.1.4 as follows:

C6.10.1.4

This component of the flange force affects the vertical web shear. In regions of positive flexure with tapered or parabolic haunches sloping downward toward the supports, the vertical web shear is increased by $P_v$. For fish belly haunches, $P_v = 0$ near the interior supports because the slope of the bottom flange is small in that area. For all other cases, the vertical web shear is reduced by $P_v$. In cases where the vertical web shear is reduced, the Specifications permit the Engineer to reduce the web dead-load shear accordingly in these cases. Calculation of the reduced live-load shear is problematic because numerous sets of concurrent moments and shears must be evaluated in order to determine the critical or smallest shear reduction, and thus is not likely worth the effort. Also, variable depth webs are used most often on longer-span girders where dead load is more predominant. The total modified vertical web shear may be used in the design of the sloping flange-to-web welds.
T-14 Agenda Item No. 28
Section 6, Article 6.10.1.4

• Item #2:
  ➢ Revise the 4th paragraph of Article C6.10.1.4 as follows:

C6.10.1.4

In fish belly haunches, where the slope of the bottom flange is smaller at positions closer to the interior support, the convex bottom flange in compression produces a uniformly distributed radial tensile stress on the web. In parabolic haunches, where the downward slope of the bottom flange is larger at positions closer to the interior support, the change in the bottom-flange inclination in combination with compressive stress in the bottom flange induces a compressive distributed transverse force on the web (Blodgett, 1982) the concave bottom flange in compression produces a uniformly distributed radial compressive stress on the web. The magnitude of the radial stress in each case is dependent on the radius of curvature of the flange. Blodgett (1982) provides a rational approach for computing and evaluating the effect of the combined stresses on the web, which typically are not of significant concern unless the radius of curvature of the flange is unusually sharp. If the girder web is unstiffened or transversely-stiffened with a stiffener spacing \(d_o\) greater than approximately 1.5\(D\) within this type of haunch a parabolic haunch adjacent to an interior support, the Engineer should consider checking the stability of the web under this the effect of the radial compressive force.
T-14 Agenda Item No. 28
Section 6, Article 6.10.1.4

• Item #1:
  ➢ Revise Article 6.10.1.4 on Variable Web Depth Members as follows:

6.10.1.4—Variable Web Depth Members

The effect of bottom flange inclination shall be considered in determining the bottom flange stress caused by bending about the major-axis of the cross-section, and any potential modifications to the vertical web shear. In cases where permitted by static equilibrium permits the vertical web shear to be reduced in variable web depth members, only the web dead-load shear may be reduced by the vertical component of the bottom flange force. At points where the bottom flange becomes horizontal, the transfer of the vertical component of the flange force back into the web shall be considered. Full- or partial-depth transverse stiffening of the web shall be provided, unless the provisions of Article D6.5.2 are satisfied for the factored vertical component of the inclined flange force using a length of bearing $N$ equal to zero.
D6.5.2—Web Local Yielding

Webs subject to compressive or tensile concentrated loads shall satisfy:

\[ R_u \leq \phi_b R_n \]  

(D6.5.2-1)

in which:

- \( R_n \) = nominal resistance to the concentrated loading (kip)

\begin{itemize}
  \item For interior-pier reactions and for concentrated loads applied at a distance from the end of the member that is greater than \( d \):
    \[ R_n = (5k + N) F_{yw} t_w \]  
    (D6.5.2-2)
  \item Otherwise:
    \[ R_n = (2.5k + N) F_{yw} t_w \]  
    (D6.5.2-3)
\end{itemize}

where:

- \( \phi_b \) = resistance factor for bearing specified in Article 6.5.4.2
- \( d \) = depth of the steel section (in.)
- \( k \) = distance from the outer face of the flange resisting the concentrated load or bearing reaction to the web toe of the fillet (in.)
- \( N \) = length of bearing (in.). \( N \) shall be greater than or equal to \( k \) at end bearing locations.
- \( R_u \) = factored concentrated load or bearing reaction (kip)
Item #2:

Revise the 5th paragraph of Article C6.10.1.4 as follows:

C6.10.1.4

At points where an inclined bottom flange becomes horizontal, the vertical component of the inclined flange force is transferred back into the web as a concentrated load. This concentrated load causes additional stress in the web, and therefore, full- or partial-depth stiffening of the web must be provided at these points, except as discussed below. Full-depth stiffeners should be positively attached to both flanges and partial-depth stiffeners should be positively attached to the bottom flange, and web-to-bottom flange welds, and will often require additional local stiffening. At these locations, the web is sufficient without additional stiffening if the requirements of Article D6.5.2 are satisfied for the factored vertical component of the inclined flange force using a length of bearing N equal to zero. At locations where the concentrated load is compressive and N is equal to zero, the provisions of Article D6.5.2 generally govern relative to those of Article D6.5.3; therefore, satisfaction of the requirements of Article D6.5.2 using a length of bearing N equal to zero ensures that the web is adequate without additional stiffening for locations subjected to compressive or tensile concentrated transverse loads.
T-14 Agenda Item No. 28
Section 6, Article 6.10.1.4
T-14 Agenda Item No. 29
Section 6, Articles 6.3 and 6.12.2.2.4

Description of Proposed Revisions:

• Item #1:
  ➢ Revise the Notation list in Article 6.3.

• Items #2 & #3:
  ➢ Revisions are made to Articles 6.12.2.2.4 and C6.12.2.2.4 for determining the flexural resistance of tees and double angles loaded in the plane of symmetry in order to bring the provisions up-to-date with the latest provisions in AISC (2016).
  
  o Prior editions of the AISC Specification did not distinguish between tees and double angles and as a result, there were instances when double angles would appear to have less strength than two single angles. This concern is now addressed by providing separate provisions for tees and double angles.
  o In those cases where double angles should have the same strength as two single angles, the revised provisions make use of the equations for single angles, as applicable, given in Section F10 of AISC (2016).
T-14 Agenda Item No. 29
Section 6, Articles 6.3 and 6.12.2.2.4

Items #2 & #3:

- In addition, a new linear transition equation from $M_p$ to $M_y$ is introduced for the limit state of lateral-torsional buckling when the stem of the member is in tension; that is, when the flange is subject to compression. Previous specifications transitioned abruptly from the full plastic moment to the elastic buckling range.

For lateral torsional buckling tee stems and double angle web legs subject to tension, the nominal flexural resistance based on lateral-torsional buckling shall be taken as:

- If $L_b \leq L_p$, then lateral-torsional buckling shall not apply.
- If $L_p < L_b \leq L_r$, then:

$$M_n = M_p - \left( M_p - M_y \right) \left( \frac{L_b - L_p}{L_r - L_p} \right)$$

(6.12.2.2.4c-1)

- If $L_b > L_r$, then:

$$M_n = M_{cr}$$

(6.12.2.2.4c-2)
T-14 Agenda Item No. 30
Section 6, Articles 6.3, 6.13.2.9 and 6.13.6.1.3

Description of Proposed Revisions:

• Item #1:
  ➢ Revise the Notation list in Article 6.3 to catch up with all the previous revisions made to the provisions for the design of bolted splices for flexural members.

• Item #2:
  ➢ Add the following paragraph to the end of Article 6.13.2.9:

    If the nominal bearing resistance of a bolt hole exceeds the nominal shear resistance of the bolt determined as specified in Article 6.13.2.7, the nominal bearing resistance of the bolt hole shall be limited to the nominal shear resistance of the bolt.
T-14 Agenda Item No. 30
Section 6, Articles 6.3, 6.13.2.9 and 6.13.6.1.3

• Item #3:
  ➢ Revise the 4th paragraph of Article C6.13.2.9 as follows:

  In these Specifications, the nominal bearing resistance of an interior hole is based on the clear distance between the hole and the adjacent hole in the direction of the bearing force. The nominal bearing resistance of an end hole is based on the clear distance between the hole and the end of the member. The nominal bearing resistance of the connected member may be taken as the sum of the smaller of the nominal shear resistance of the individual bolts and the nominal bearing resistances of the individual bolt holes parallel to the line of the force. The clear distance is used to simplify the computations for oversize and slotted holes.

• Item #4:
  ➢ Add the following paragraph underneath Fig. C6.13.6.1.3b-2:

  The moment resistance provided by the flanges can potentially be increased by staggering the flange bolts.
T-14 Agenda Item No. 30
Section 6, Articles 6.3, 6.13.2.9 and 6.13.6.1.3

• Item #5:
  ➢ Revise the 1st sentence of the last paragraph of Article C6.13.6.1.3b as follows:

  For flanges with one web in straight girders and for in horizontally curved girders, the effects of flange lateral bending need not be considered in the design of the bolted flange splices since the combined areas of the flange splice plates will typically equal or exceed the area of the smaller flange to which they are attached.

• Items #6 & #7:
  ➢ Miscellaneous revisions are made to the changes to Articles 6.13.6.1.3c and C6.13.6.1.3c (Web Splices) that were balloted on and approved in 2017 Agenda Item 16. The revisions are necessary for consistency with changes that were previously made to these articles in an Errata to the 8th Edition BDS.
T-14 Agenda Item No. 32
Section 6, Various Articles (2)

Description of Proposed Revisions:

- Item #1:
  - Add new definitions for an Internally Redundant Member (IRM) and a Load Path Redundant Member (LPRM) to Article 6.2:

  - Internally Redundant Member (IRM)—A steel primary member in tension, or with a tension element, that is not qualified as an LPRM but has redundancy in the cross-section such that fracture of one element will not propagate through the entire member and is discoverable by the applicable inspection procedures.

  - Load Path Redundant Member (LPRM)—A steel primary member in tension, or with a tension element, that has redundancy based on the number of main supporting members between points of support, such that fracture of one cross-section of one member will not cause a portion of or the entire bridge to collapse.
Revise the definitions for a Fracture-Critical Member (FCM) and a System Redundant Member (SRM) in Article 6.2 as follows:

• **Fracture-Critical Member (FCM)**—As defined in the Code of Federal Regulations (CFR), a steel primary member or portion thereof subject to in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse.

• **System Redundant Member (SRM)**—A steel primary member or portion thereof subject to in tension, or with a tension element, that is not qualified as an LPRM but has redundancy in the bridge system, such that fracture of one cross-section of the member will not cause a portion of or the entire bridge to collapse, for which the redundancy is not known by engineering judgment, but which is demonstrated to have 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.
Items #2 and #4:

- Revise the next-to-the-last paragraph of Article 6.6.1.2.3 as follows:

  For components and details on members in tension, or with a tension element, that are classified as Fracture-Critical Members, the Fatigue load combination specified in Table 3.4.1-1 should be used in combination with the nominal fatigue resistance for infinite life specified in Article 6.6.1.2.5.

- Add the following paragraph after the 7th paragraph of Article C6.6.1.2.3:

  The use of fatigue details classified as Detail Category C or better is encouraged on longitudinal members in tension, or with a tension element, that are classified as Fracture-Critical Members. This does not apply to certain transverse and/or secondary members in these structures.
Item #3:

- Change the term “flame-cut” to the more general term “thermal-cut” in the descriptions for Conditions 1.1, 1.2, and 8.7 in Table 6.6.1.2.3-1.

- Revise the sketches in Condition 4.3 in Table 6.6.1.2.3-1 as follows:
Item #5:

- Revise the last paragraph of Article C6.6.1.2.3 as follows:

  Considering the increased Fatigue I live load factor, $\gamma_{LL}$, for orthotropic deck details specified in Article 3.4.4 and the cycles per truck passage, $\{n\}$, in for orthotropic decks plate connections subjected to wheel load cycling, e.g., rib-to-deck welds, specified in Table 6.6.1.2.5-2, the 75-year $ADTT_{SL}$ equivalent to infinite life (trucks per day) calculated from Eq. C6.6.1.2.3-1 is results in 870 740 trucks per day for deck plate fatigue Category C orthotropic deck-plate connection details and 4,350 3,700 trucks per day for all other fatigue Category C orthotropic deck details, based on Category C. Thus, finite life design may produce more economical designs for the detail under consideration on lower-volume roadways with 75-year $ADTT_{SL}$ values equal to or less than these values.
T-14 Agenda Item No. 32
Section 6, Various Articles (2)

- Item #6:
  
  Revise the 1st two paragraphs of Article 6.6.2.2 to read as follows:

The Engineer shall have the responsibility for identifying and designating on the contract plans—classification of primary members or portions thereof in tension, or with a tension element, as one of the following: Load Path Redundant Member (LPRM), System Redundant Member (SRM), Internally Redundant Member (IRM), or Fracture-Critical Member—are fracture-critical members (FCMs). The contract plans shall clearly delineate all members classified as a SRM, IRM, or FCM. The contract documents shall require that all members meeting the definition of a FCM or SRM be fabricated according to the provisions of Clause 12 specified in the AASHTO/AWS D1.5M/D1.5 Bridge Welding Code. Members or portions thereof that are not subject to a net tensile stress under Strength Load Combination I shall not require classification be designated as FCMs.

Any classification of a SRM or IRM shall be supported by FHWA-approved calculation, analysis, or other criteria supported by experimental verification. A primary member—or portion thereof subject to tension, for which the redundancy is not known by engineering judgment but which is demonstrated to have redundancy in the presence of a simulated fracture in that member through the use of a refined analysis, shall be designated as a System Redundant Member (SRM) in the contract documents. The contract documents shall further indicate that SRMs are to be fabricated according to the provisions of Clause 12 specified in the AASHTO/AWS D1.5M/D1.5 Bridge Welding Code. The criteria, assumptions, and other pertinent information related to the refined analysis used to demonstrate the redundancy classify the member shall be retained and included in the inspection records or permanent bridge file.
T-14 Agenda Item No. 32
Section 6, Various Articles (2)

Revise Article C6.6.2.2 as follows:

As defined in the Code of Federal Regulations (CFR), a Fracture-Critical Member (FCM) is a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse. Secondary members and diaphragm or cross-frame members in horizontally-curved bridges should not be designated as FCMs. When designating some rolled shapes as FCMs, it may not be possible to secure shapes that are produced using fine-grained practices. In such cases, the fine-grained practices should be waived.

A steel primary member in tension, or with a tension element, that has redundancy based on the number of main supporting members between points of support, such that fracture of one cross-section of one member will not cause a portion of or the entire bridge to collapse is defined in FHWA (2019) and herein as a Load Path Redundant Member (LPRM). LPRMs are usually longitudinal and parallel, such as girders or trusses. Redundancy can be determined by engineering judgment or simple calculation. Primary members in tension, or with a tension element, in common girder bridges with three or more girders with spacing no greater than 12 ft are LPRMs.

A steel primary member in tension, or with a tension element, that is not qualified as an LPRM but has redundancy in the bridge system, such that fracture of one cross-section of the member will not cause a portion of or the entire bridge to collapse is defined in FHWA (2019) and herein as a System Redundant Member (SRM). SRMs are to be classified through calculation, analysis or other criteria supported by experimental verification and approved by the FHWA. One acceptable approach is to use refined analysis per the AASHTO Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members. Members or portions thereof satisfying the provisions of the Guide...
Specifications may be designated as SRMs (Connor et al., 2018). SRMs are to be fabricated in accordance with Clause 12 of the AASHTO/AWS D1.5M/D1.5 Bridge Welding Code and should not be subjected to the hands-on in-service inspection requirements described in 23 CFR 650 (FHWA, 2019).

A steel primary member in tension, or with a tension element, that is not qualified as an LPRM but has redundancy in the cross-section, such that fracture of one element will not propagate through the entire member, and is discoverable by the applicable inspection procedures, is defined in FHWA (2019) and herein as an Internally Redundant Member (IRM). IRMs are to be classified through calculation, analysis or other criteria supported by experimental verification and approved by the FHWA. One acceptable approach is given in the AASHTO Guide Specifications for Internal Redundancy of Mechanically-fastened Built-Up Steel Members. IRMs may be inspected using an appropriate technique at a special inspection interval established according to the provisions of the Guide Specifications (Hebdon et al., 2015; Lloyd et al., 2018).

➢ ...and delete the next two paragraphs of Article C6.6.2.2.


T-14 Agenda Item No. 32
Section 6, Various Articles (2)

• Item #7:
  ➢ Revise the 7th paragraph of Article C6.10.11.3.1 as follows:

For determining the nominal fatigue resistance of various longitudinal stiffener end details, refer to Condition 4.3 in Table 6.6.1.2.3-1. While the use of complete-or partial-penetration groove welds to attach the stiffener to the web or flange is permitted, the use of fillet welds is strongly encouraged as shown in Table 6.6.1.2.3-1. The use of groove welds does not enhance the nominal fatigue resistance of the end detail. Consideration should be given to wrapping the weld around the end of the stiffener for sealing. The weld and stiffener material should be ground to a smooth contour where the radiused stiffener end becomes tangent to the web or flange. Welded shop splices in longitudinal stiffeners should be complete-penetration groove welded. Where longitudinal stiffeners are discontinued at bolted field splices, consideration should be given to taking the stiffener to the free edge of the web where the normal stress is zero.

• Items #8 & #10:
  ➢ Delete the last two paragraphs of Articles 6.11.5 and C6.11.5.
• Item #9:
  ➢ Revise the 8\textsuperscript{th} paragraph of Article C6.11.5 as follows:

  Fatigue of the base metal at the net section of access holes or manholes should be considered. The fatigue resistance at the net section of large access holes is not currently specified satisfying specified geometric conditions and made to the requirements of the AASHTO/AWS D1.5M/D1.5 Bridge Welding Code is provided in Condition 1.6 of Table 6.6.1.2.3-1; however, base metal at the net section of open bolt holes has been shown to satisfy Category D (Brown et al., 2007). This classification of large access holes satisfying the specified conditions as fatigue detail Category C assumes a stress concentration, or ratio of the elastic tensile stress adjacent to the hole to the average stress on the net area, of 3.0 less than 2.4. A less severe fatigue category might be considered if the proper stress concentration at the edges of the access hole is evaluated.

• Items #11 thru #13:
  ➢ Revisions are proposed to Articles C6.13.3.6, 6.13.3.7, and C6.13.3.7 to indicate that the ends of \emph{longitudinal} web and flange stiffeners may also be wrapped for sealing.
## T-14 Editorial Items

<table>
<thead>
<tr>
<th>Location of Change</th>
<th>Current Text</th>
<th>Proposed Text</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Table 6.6.1.2.5-1</strong></td>
<td><strong>Table 6.6.1.2.5-1—Detail Category Constant, A</strong></td>
<td><strong>Table 6.6.1.2.5-1—Detail Category Constant, A</strong></td>
</tr>
<tr>
<td><strong>Detail Category</strong></td>
<td><strong>Constant, A \times 10^8 (ksi^3)</strong></td>
<td><strong>Detail Category</strong></td>
</tr>
<tr>
<td>A</td>
<td>250.0</td>
<td>A</td>
</tr>
<tr>
<td>B</td>
<td>120.0</td>
<td>B</td>
</tr>
<tr>
<td>B'</td>
<td>61.0</td>
<td>B'</td>
</tr>
<tr>
<td>C</td>
<td>44.0</td>
<td>C</td>
</tr>
<tr>
<td>C'</td>
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<td>ASTM F3125, Grades A325 and F1852 Bolts in Axial Tension</td>
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<td>Article C6.10.1.1.1a, 2nd paragraph</td>
<td>While shored construction is permitted according to these provisions, its use is not recommended. Unshored construction generally is expected to be more economical. Also, these provisions may not be sufficient for shored construction where close tolerances on the girder cambers are important. There has been limited research on the effects of concrete creep on composite steel girders under large dead loads. There have been only a very limited number of demonstration bridges built with shored construction in the U.S. Shored composite bridges that are known to have been constructed in Germany did not retain composite action. Furthermore, there is an increased likelihood of significant tensile stresses occurring in the concrete deck at permanent support points when shored construction is used.</td>
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| **Article 6.10.11.2.2** | **6.10.11.2.2—Projecting Width**  

The width, $b_p$, of each projecting stiffener element shall satisfy:  

$$b_p \leq 0.48t_p \sqrt{\frac{E}{F_{ys}}} \quad (6.10.11.2.2-1)$$  

where:  

- $F_{ys}$ = specified minimum yield strength of the stiffener (ksi)  
- $t_p$ = thickness of the projecting stiffener element (in.) | **6.10.11.2.2—Projecting Width Minimum Thickness**  

The width, $b_p$, thickness, $t_p$, of each projecting stiffener element shall satisfy:  

$$t_p \geq \frac{b_p}{0.48} \sqrt{\frac{E}{F_{ys}}} \quad (6.10.11.2.2-1)$$  

where:  

- $F_{ys}$ = specified minimum yield strength of the stiffener (ksi)  
- $t_p b_p$ = thickness-width of the projecting stiffener element (in.) |