

# THE MANUAL FOR BRIDGE EVALUATION

## 2020 Interim Revisions



AMERICAN ASSOCIATION  
OF STATE HIGHWAY AND  
TRANSPORTATION OFFICIALS  
**AASHTO**

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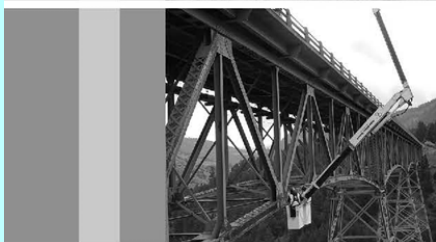
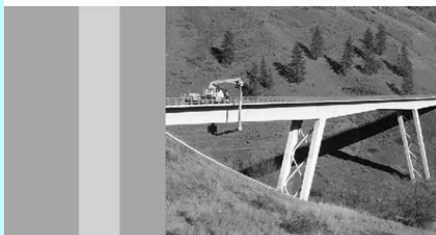
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## 2020 INTERIM REVISIONS

### INSTRUCTIONS AND INFORMATION

#### General

AASHTO has issued proposed interim revisions to the *Manual for Bridge Evaluation*, Third Edition (2018). This packet contains the revised pages. They are designed to replace the corresponding pages in the book.

#### Affected Articles

Underlined text indicates revisions that were approved in 2019 by the AASHTO Committee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Committee. A list of affected articles is included below.

All interim pages are displayed on a blue background to make the changes stand out when inserted in the third edition binder. They also have a page header displaying the page number affected and the interim publication year. Please note that these pages may also contain nontechnical (i.e., editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

#### 2020 Changed Articles

##### SECTION 6: LOAD RATING

6A.6.10

6B.5.2.1

6B.5.3.1

##### APPENDIX A

Example A1—*Replaced in its entirety*

Example A7—*Replaced in its entirety*

#### 2020 Added Articles

##### SECTION 6: LOAD RATING

6A.6.10

C6A.6.10

6A.6.10.1

6A.10

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Single angles and tees are commonly used as cross-frame members and are often subjected to axial forces and bending. They are almost always connected eccentrically at their ends with respect to the centroid of the cross-section. LRFD Design Article C6.12.2.2.4 refers the Engineer to AISC (2005) for additional guidance on determining the load-carrying capacity of these types of members.

### 6A.6.10—Evaluation for Shear

Shear resistance at the strength limit state is specified in the *AASHTO LRFD Bridge Design Specifications* for I-sections, box girders, and miscellaneous composite members. The nominal shear resistance of a stiffened web end panel may alternatively be determined as specified in Article 6A6.10.1.

#### 6A.6.10.1—End Panels

The nominal shear resistance of a stiffened web end panel may be determined as:

$$V_n = V_p \left[ C + \alpha \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}} \right] \quad (6A.6.10.1-1)$$

in which:

$\alpha$  = parameter to consider partial tension-field action

$$\alpha = \frac{2.8 \left( \sqrt{M_{pf} + M_{pm}} + \sqrt{M_{pb} + M_{pm}} \right)}{D \sqrt{t_w F_{yw} (1-C)}} \leq 1.0 \quad (6A.6.10.1-2)$$

$V_p$  = plastic shear force (kip)

$$= 0.58 F_{yw} D t_w \quad (6A.6.10.1-3)$$

where:

$C$  = ratio of the shear-buckling resistance to the shear yield strength determined as specified in LRFD Design Article 6.10.9.3.2

$d_o$  = transverse stiffener spacing (in.)

$D$  = web depth (in.)

$F_{yw}$  = specified minimum yield strength of the web (ksi)

$M_{pb}$  = plastic moment resistance of the bearing stiffeners (kip-in.)

$M_{pf}$  = plastic moment resistance of the top flange (kip-in.)

$M_{pm}$  = minimum value of  $M_{pb}$  and  $M_{pf}$  (kip-in.)

$t_w$  = web thickness (in.)

In the calculation of the plastic moment resistances,

#### C6A.6.10.1

The nominal shear resistance equations currently specified in LRFD Design Article 6.10.9 only allow for the consideration of tension-field action for interior web panels and conservatively limit the nominal shear resistance of end panels to the shear-yielding or shear-buckling resistance,  $V_{cr} = CV_p$ . This is based on the assumption that there is no effective anchor mechanism to resist the horizontal component of the tension-field force at the end panel. Research has indicated that whole or partial tension-field action does exist in stiffened web end panels based on contributions from the top flange and bearing stiffeners (Kim and Uang, 2015; Uang and Kim, 2018). The research also showed that the concrete deck slab contributes to the overall shear resistance for composite girders; however this contribution is conservatively ignored herein.



$M_{pb}$  and  $M_{pf}$ , a portion of the web area defined by an effective web depth,  $d_e$ , shall be considered.  $d_e$  shall be determined as follows:

If  $C \leq 0.8$ , then:

$$d_e = 35t_w(0.8 - C)^2 \quad (6A.6.10.1-4)$$

Otherwise:

$$d_e = 0 \quad (6A.6.10.1-5)$$

The effective web depths,  $d_e$ , to be considered in the calculation of the plastic moment resistances,  $M_{pb}$  and  $M_{pf}$ , are shown in Figure 6A.6.10.1-1.

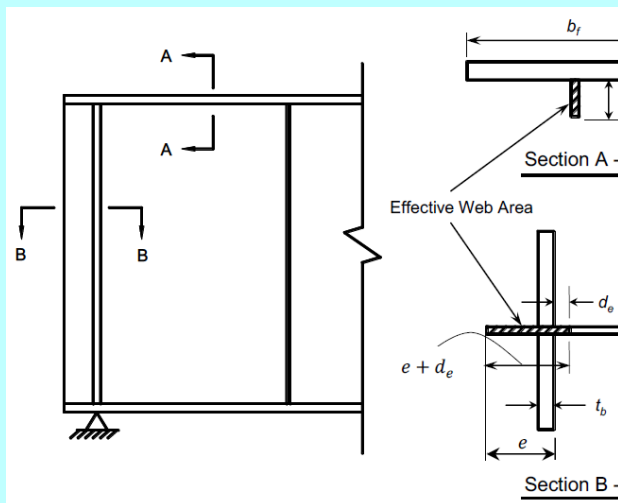


**Figure 6A.6.10.1-1—Effective Web Depth,  $d_e$ , for the Top Flange and Bearing Stiffeners (Kim and Uang, 2018)**

The effective sections to be considered in the plastic moment resistance calculations for the top flange and bearing stiffeners are shown in Figure 6A.6.10.1-2.

The web end distance,  $e$ , used in the calculation of  $M_{pb}$  shall not exceed  $0.84t_w\sqrt{E/F_{yw}}$ .

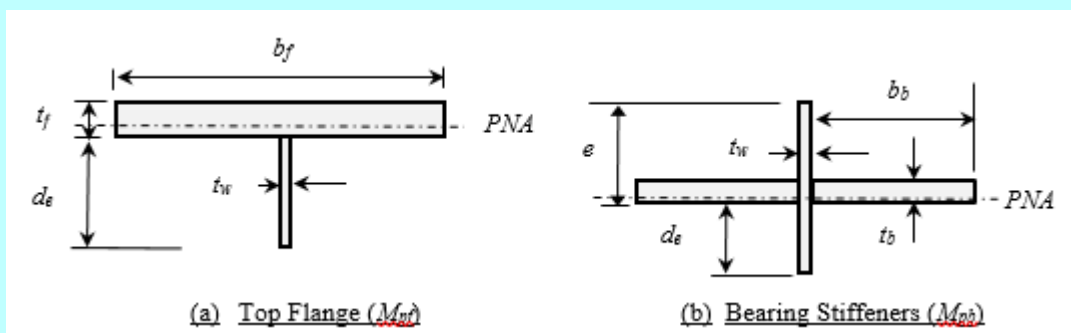


**Figure 6A.6.10.1-2—Effective Sections for the Top Flange and Bearing Stiffeners**

The web end distance,  $e$ , used in the calculation of  $M_{pb}$  shall not exceed  $0.84t_w\sqrt{E/F_{yw}}$ .

#### 6A.6.11—Box Sections in Flexure

The flexural resistance of straight or horizontally curved multiple or single box sections composite with a concrete deck at the strength limit state shall be determined as specified in LRFD Design Article 6.11.6.2. The provisions of LRFD Design Article 6.11.1.1 shall also apply.

The provisions of LRFD Design Articles 6.11.2.1 and 6.11.2.2 pertaining to cross-section proportion limits need not be considered during evaluation.

The constructibility requirements specified in LRFD Design Article 6.11.3 need not be considered during evaluation.

The fatigue requirements for webs specified in LRFD Design Article 6.10.5.3 need not be considered during evaluation.

#### 6A.6.11.1—Diaphragms and Cross-Frames

Diaphragm and cross-frame members in horizontally curved bridges shall be considered to be primary members and should be load rated accordingly at the discretion of the Owner.

#### C6A.6.11.1

See Article C6A.6.9.7.

#### 6A.6.12—Evaluation of Critical Connections

##### 6A.6.12.1—General

External connections of nonredundant members shall be evaluated during a load rating analysis in situations where the evaluator has reason to believe that their capacity may govern the load rating of the entire bridge. Evaluation of critical connections shall be performed in accordance with the provisions of these articles.

##### C6A.6.12.1

External connections are connections that transfer calculated load effects at support points of a member. Nonredundant members are members without alternate load paths whose failure is expected to cause the collapse of the bridge.

It is common practice to assume that connections and splices are of equal or greater capacity than the members they adjoin. With the introduction of more accurate evaluation procedures to identify and use increased member load capacities, it becomes increasingly important to also closely scrutinize the capacity of connections and splices to ensure that they do not govern the load rating.

**6A.6.12.2—Bearing-Type Connections**

Bearing-type connections shall be evaluated for the strength limit state (at the operating level when checking for HL-93), for flexural moment, shear, or axial force due to the factored loadings at the point of connection.

**6A.6.12.3—Slip-Critical Connections**

High-strength bolted joints designed as slip-critical connections shall be evaluated as slip-critical connections. Slip-critical connections shall be checked (at the operating level when checking for HL-93) for slip under the Service II load combination and for bearing, shear, and tensile resistance at the strength limit state. Provisions of LRFD Design Article 6.13.2.2 shall apply.

The friction value shall be based on a value of  $K_s = 0.33$  where the condition of the faying surface is unknown.

**6A.6.12.4—Pinned Connections**

Pins shall be evaluated for combined flexure and shear as specified in LRFD Design Article 6.7.6.2.1 and for bearing as specified in LRFD Design Article 6.7.6.2.2.

**6A.6.12.5—Riveted Connections**

Riveted connections shall be evaluated as bearing-type connections.

*6A.6.12.5.1—Rivets in Shear*

The factored resistance of rivets in shear shall be taken as:

$$R_n = \phi_s F_{uv} = \phi_s F_u R_1 R_2 R_3 m A_r \quad (6A.6.12.5.1-1)$$

**C6A.6.12.2**

See Table 6A.4.2.2-1 for load factors.

**C6A.6.12.3**

See Table 6A.4.2.2-1 for load factors.

**C6A.6.12.4**

Pinned connections are used both in trusses and at expansion joints of truss and girder suspended spans. Pins are short cylindrical beams and shall be evaluated for: 1) bending, 2) shear, and 3) bearing. Pin analyses should be performed during the load-rating analyses of pin-connected bridges because the pins may not necessarily be of equal or greater capacity than the members they adjoin.

The alignment of adjoining members relative to the pin could have a significant effect on the load capacity of the pin as the movement of a member changes the point of application of the member force on the pin. This is especially important on bridges without spacer collars between individual components at a pin. The relative positions of all members that connect to a pin should be ascertained in the field.

The pin size should be measured in the field to ascertain any reduction due to corrosion and wear.

**C6A.6.12.5**

Refer to the *AASHTO LRFD Design Specifications* Article 6.13.6.1.4—Fillers and commentary for more information regarding filler plates.

If rivets are of unknown origin or if more rigorous testing is necessary to determine the Ultimate Tensile Strength of the rivets, the use of chemical testing of the rivet may be considered to determine the carbon equivalent and corresponding ASTM specification or grade.

traffic. A speed posting should not be considered as a basis for increasing the weight limit in areas where enforcement will be difficult and frequent violations can be anticipated.

## 6A.9—SPECIAL TOPICS

### 6A.9.1—Evaluation of Unreinforced Masonry Arches

#### 6A.9.1.1—General

The predominant type of unreinforced masonry bridge is the filled spandrel arch. Materials may be unreinforced concrete, brick, and ashlar or rubble stone masonry. Mortar used to bind the individual masonry units should be classified in accordance with ASTM C270.

The total load-carrying capacity of an unreinforced masonry arch should be evaluated by the Allowable Stress method (Article 6B.5.2.6) based on limitation of the tensile and compressive stresses developed in the extreme fiber when axial and bending stresses are combined, and on failure modes due to instability.

#### 6A.9.1.2—Method of Analysis

Internal stresses of masonry arches are usually analyzed by regarding the arch as an elastic redundant structure. When evaluating masonry arches, three types of failures are generally investigated: 1) overturning of two adjacent masonry units of the arch, 2) sliding or shear failure, and 3) compressive failure of the masonry.

There may be instances in which the capacity of the arch based on approximate analysis methods may be inadequate or the behavior of the arch under traffic is not consistent with that predicted by evaluation. In these situations load tests or more refined analysis may be helpful in establishing a more accurate safe load capacity.

#### C6A.9.1.2

Failure due to crushing of the masonry material is not common. In classical arch analysis, the stability of the arch masonry units is ensured by keeping the line of resistance (or the resultant of the moment and thrust at a given point) within the middle third of the arch ring (or within the kern). Keeping the resultant within the kern will ensure that no part of the arch is subjected to tension.

Classical analysis of filled arches tends to greatly under-estimate their true capacity. The filled arch is a very complex structure composed of both the arch ring and the surrounding fill. A rigorous solution to establish the load capacity of masonry arches should consider the soil-structure interaction including the effects of lateral earth pressure. Classical arch analysis neglects the effects of lateral earth pressure on arch behavior. In filled arches the passive restraint of the fill is sufficient to greatly limit the distortion of the arch under live load. A large portion of the composite stiffness of the arch and fill is due to the restraint of the fill.

A number of simple empirical methods and computer-based analysis methods have been developed to assess masonry arch bridges in the United Kingdom, where a significant portion of the bridge stock is said to consist of masonry arches. Details of these methods are contained in *The Assessment of Highway Bridges and Structures BD 21/97 & BA 16/93, Department of Transport, UK.*

#### 6A.9.1.3—Allowable Stresses in Masonry

The allowable stresses in masonry materials shall be as specified in Article 6B.5.2.6 of this Manual.

### 6A.9.2—Historic Bridges

Most states have undertaken historic bridge surveys to identify which of their bridges that were built more than 50 years ago are historic. Historic bridge survey information is generally maintained by the state Department of Transportation, and it may be in a master database and/or may have been entered into the state's BMS database. This information is frequently part of the bridge record, and it offers guidance on why the bridge is noteworthy. The survey data may also contain useful information about original design details.

Historic bridges are defined as those that meet the National Register of Historic Places' criteria for evaluation. The criteria establish a measure of consideration to evaluate which bridges have the significance and integrity to be determined historic and thus worthy of preservation. Many types of bridges, from stone arch and metal truss bridges to early continuous stringer and prestressed beam bridges have been determined to be historic for their technological significance. Other bridges are historic because they are located in historic districts or are associated with historic transportation routes, such as rail lines or parkways.

Historic bridges, like all other National Register-listed or eligible resources, are affected by federal laws intended to strengthen the governmental commitment to preservation. This means that all work needs to be done in compliance with the applicable federal, and often state, regulations and procedures. They require consideration of the historic significance of the bridge when developing maintenance, repair, and/or rehabilitation methodologies. The goal is to avoid having an adverse effect on the historic bridge. Guidance on how to develop successful approaches for working on historic bridges can be found in *The Secretary of the Interior's Standards for Rehabilitation* and *The Secretary of the Interior's Standards for the Treatment of Historic Properties 1992*. Both offer approaches for considering ways to upgrade structures while maintaining their historic fabric and significance, and they are available from the National Park Service Preservation Assistance Division or the state historic preservation office.

Because historic bridges require demonstrated consideration of ways to avoid adverse effects, evaluations should be complete, encompassing the relevant parts of this Manual. Nondestructive testing methods should be considered to verify components and system performance. Repair rather than replacement of original elements should be considered, and any replacement should be in kind where feasible. Strengthening should be done in a manner that is respectful to the historic bridge.

### 6A.10—REFERENCES

Kim, D. W. and C. M. Uang. Shear Resistance of End Panels in Steel and Steel-Concrete Composite Plate Girders. Structural Systems Research Project, Report No. SSRP-15/02, Department of Structural Engineering, University of California, San Diego, 2015.

Uang, C. M. and D. W. Kim. Evaluation of Shear Strength of Stiffened Flexural Members. Report to AISC TC 4 Committee on Member Design. Chicago, IL. November 2018.

### 6B.4.2—Allowable Stress

For the allowable stress method,  $A_1 = 1.0$  and  $A_2 = 1.0$  in the general rating equation.

The capacity,  $C$ , depends on the rating level desired, with the higher value for  $C$  used for the Operating level. The determination of the nominal capacity of a member is discussed in Article 6B.5.2.

### 6B.4.3—Load Factor

For the load factor method,  $A_1 = 1.3$  and  $A_2$  varies depending on the rating level desired. For inventory level,  $A_2 = 2.17$  and for operating level,  $A_2 = 1.3$ .

The nominal capacity,  $C$ , is the same regardless of the rating level desired (see Article 6B.5.3).

## 6B.5—NOMINAL CAPACITY: $C$

### 6B.5.1—General

The nominal capacity to be used in the rating equation depends on the structural materials, the rating method, and rating level used. Nominal capacities based on the Allowable Stress method are discussed in Article 6B.5.2 and those based on the Load Factor method are discussed in Article 6B.5.3.

The Bridge Owner is responsible for selecting the rating method. The method used should be identified for future reference.

### 6B.5.2—Allowable Stress Method

In the allowable stress method, the capacity of a member is based on the rating level evaluated: inventory level-allowable stress, or operating level-allowable stress. The properties to be used for determining the allowable stress capacity for different materials follow. For convenience, the tables provide, where appropriate, the inventory, operating, and yield stress values. Allowable stress and strength formulas should be those provided herein or those contained in the AASHTO Standard Specifications. When situations arise that are not covered by these specifications, then rational strength of material formulae should be used consistent with data and plans verified in the field investigation. Deviations from the AASHTO Standard Specifications should be fully documented.

When the bridge materials or construction are unknown, the allowable stresses should be fixed by the Engineer, based on field investigations and/or material testing conducted in accordance with Section 5, and should be substituted for the basic stresses given herein.

### 6B.5.2.1—Structural Steel

The allowable unit stresses used for determining safe load capacity depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine a nominal yield point. When information on specifications of the steel is not available, allowable stresses should be taken from the applicable “Date Built” column of Tables 6B.5.2.1-1 and 6B.5.2.1-2.

Table 6B.5.2.1-1 gives allowable inventory stresses and Table 6B.5.2.1-2 gives the allowable operating stresses for structural steel. The nominal yield stress,  $F_y$ , is also shown in Tables 6B.5.2.1-1 and 6B.5.2.1-2. Tables 6B.5.2.1-3 and 6B.5.2.1-4 give the allowable inventory and operating stresses for bolts and rivets. For compression members, the effective length,  $KL$ , may be determined in accordance with the AASHTO Standard Specifications or taken as follows:

$$KL = \begin{aligned} &75 \text{ percent of the total length of a column having} \\ &\text{riveted end connections} \\ &= 87.5 \text{ percent of the total length of a column having} \\ &\text{pinned end connections} \end{aligned}$$

The modulus of elasticity,  $E$ , for steel should be 29,000,000 lb/in.<sup>2</sup>

If the investigation of shear and stiffener spacing is desirable, such investigation may be based on the AASHTO Standard Specifications. The allowable shear stress,  $F_v$ , of a stiffened web end panel may alternatively be determined from Eq. 6A.6.10.1-1, with  $V_p$  replaced by  $F_v/3$ , and with  $C$  determined as specified in Article 10.34.4.2 of the AASHTO Standard Specifications.

### C6B.5.2.1

When nonspecification materials are encountered, standard coupon testing procedures may be used to establish the nominal yield point. To provide a 95 percent confidence limit, the nominal yield point would typically be the mean coupon test value minus 1.65 standard deviations.

Mechanical properties of eyebars, high-strength eyebars, forged eyebars, and cables vary depending on manufacturer and year of construction. In the absence of material tests, the Engineer should carefully investigate the material properties using manufacturer's data and compilations of older steel properties before establishing the yield and allowable stresses to be used in load rating the bridge.

The formulas for the allowable bending stress in partially supported or unsupported compression flanges of beams and girders, given in Tables 6B.5.2.1-1 and 6B.5.2.1-2 are based on the corresponding formula given in Table 10.32.1A of the Allowable Stress Design portion of the AASHTO Standard Specifications. The equation in Table 6B.5.2.1-1 is to be used for an inventory rating and the equation in Table 6B.5.2.1-2 is to be used for an operating rating.

The previously used formulas are inelastic parabolic formulas which treat the lateral torsional buckling of a beam as flexural buckling of the compression flange. This is a very conservative approach for beams with short unbraced lengths. The flexural capacity is reduced for any unbraced length greater than zero. This does not reflect the true behavior of a beam. A beam may reach  $M_p$  with unbraced lengths much greater than zero. In addition, the formula neglects the St. Venant torsional stiffness of the cross-sections. This is a significant contribution to the lateral torsional buckling resistance of rolled shapes, particularly older “I” shapes. The previous formulas must also be limited to the values of  $I/b$  listed. This limit is the slenderness ratio when the estimated buckling stress is equal to half the yield strength or  $0.275 F_y$  in terms of an allowable stress. Many floor stringers will have unbraced lengths beyond this limit. If the formulas are used beyond these limits, negative values of the allowable stress can result.

The new formulas have no upper limit which allows the determination of allowable stresses for all unbraced lengths. In addition, the influence of the moment gradient upon buckling capacity is considered using the modifier,  $C_b$ , in the new formulas.

The specification formulas are based on the exact formulations of the lateral torsional buckling of beams. They are currently used in the AISC LRFD Specifications and other specifications throughout the world. They are also being used to design and rate steel bridges by the load factor method. Figures 6B.5.2.1-1 and 6B.5.2.1-2 show a comparison between the specification formulas and the previous specification formulas for two sections. Figure 6B.5.2.1-1 compares results for a  $W18 \times 46$  rolled section. The new specification gives a much higher capacity than the previous specification. The difference is due to the inclusion of the St. Venant torsional stiffness,  $J$ , in the proposed specification. Figure 6B.5.2.1-2 shows a similar comparison for a plate-girder section. The section

**6B.5.2.7—Timber**

Determining allowable stresses for timber in existing bridges will require sound judgment on the part of the Engineer making the field investigation.

*(1) Inventory Stress*

The inventory unit stresses should be equal to the allowable stresses for stress-grade lumber given in the AASHTO Standard Specifications.

Allowable inventory unit stresses for timber columns should be in accordance with the applicable provisions of the AASHTO Standard Specifications.

*(2) Operating Stress*

The maximum allowable Operating unit stresses should not exceed 1.33 times the allowable stresses for stress-grade lumber given in the current AASHTO Standard Specifications. Reduction from the maximum allowable stress will depend upon the grade and condition of the timber and should be determined at the time of the inspection.

Allowable operating stress in lb/in.<sup>2</sup> of cross-sectional area of simple solid columns should be determined by the following formulas but the allowable operating stress should not exceed 1.33 times the values for compression parallel to grain given in the design stress table of the AASHTO Standard Specifications.

$$\frac{P}{A} = \frac{4.8E}{(\ell/r)^2} \quad (6B.5.2.7-1)$$

where:

$P$  = Total load, lb

$A$  = Cross-sectional area, in.<sup>2</sup>

$E$  = Modulus of elasticity

$\ell$  = Unsupported overall length between points of lateral support of simple columns, in.

$r$  = Least radius of gyration of the section, in.

For columns of square or rectangular cross-section, this formula becomes:

$$\frac{P}{A} = \frac{0.40E}{(1/d)^2} \quad (6B.5.2.7-2)$$

where:

$d$  = Dimension of the narrowest face, in.

The above formula applies to long columns with  $\ell/d$  over 11, but not greater than 50.

For short columns,  $\ell/d$  not over 11, use the allowable design unit stress in compression parallel to grain times 1.33 for the grade of timber used.

**C6B.5.2.7**

The material and member properties based on as-built information may need to be adjusted for field conditions such as weathering or decay. The Engineer's judgment and experience are required in assessing actual member resistance.

Eq. 6B.5.2.7-1 is based on the Euler long-column formula with two adjustments as follows. First,  $E$  is reduced by dividing by 2.74. This corresponds to a safety factor of 1.66 for solid timber members according to the National Design Specifications for Wood Construction (2005). Then the Euler allowable stress is multiplied by 1.33 to provide an operating level allowable stress as shown in Eq. 6B.5.2.7-1.

For square and rectangular columns, substituting  $d/\sqrt{12}$  for the radius of gyration,  $r$ , in Eq. 6B.5.2.7-1 results in Eq. 6B.5.2.7-2.

### 6B.5.3—Load Factor Method

Nominal capacity of structural steel, reinforced concrete and prestressed concrete should be the same as specified in the load factor sections of the AASHTO Standard Specifications. Nominal strength calculations should take into consideration the observable effects of deterioration, such as loss of concrete or steel-sectional area, loss of composite action or corrosion.

Allowable fatigue strength should be checked based on the AASHTO Standard Specifications. Special structural or operational conditions and policies of the Bridge Owner may also influence the determination of fatigue strength.

#### 6B.5.3.1—Structural Steel

The yield stresses used for determining ratings should depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine yield characteristics. The nominal yield value should be substituted in strength formulas and is typically taken as the mean test value minus 1.65 standard deviations. When specifications of the steel are not available, yield strengths should be taken from the applicable “date built” column of Tables 6B.5.2.1-1 to 6B.5.2.1-4.

The capacity of structural steel members should be based on the load factor requirements stated in the AASHTO Standard Specifications. The capacity,  $C$ , for typical steel bridge members is summarized in Appendix L6B. For beams, the overload limitations of Article 10.57 of the AASHTO Standard Specifications should also be considered.

If the investigation of shear and stiffener spacing for a straight web is desirable, such investigation may be based on the AASHTO Standard Specifications. The shear capacity,  $V_u$ , of a stiffened web end panel may alternatively be taken as  $V_n$  determined from Eq. 6A.6.10.1-1, with  $C$  determined as specified in Article 10.48.8.1 of the AASHTO Standard Specifications.

If the investigation of shear and stiffener spacing for a curved web is desirable, such investigation may be based on the AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges, 2003. The shear capacity,  $V_u$ , of a stiffened web interior panel may alternatively be taken as  $V_n$  for a stiffened interior web panel determined as specified in LRFD Design Article 6.10.9, and the shear capacity of a stiffened web end panel may alternatively be taken as  $V_n$  determined from Eq. 6A.6.10.1-1, with  $C$  determined as specified in Article 10.48.8.1 of the AASHTO Standard Specifications.

Curved steel beams with a web slenderness ratio exceeding the limits in Article 6.3 of the AASHTO 2003 *Guide Specifications for Horizontally Curved Girder Highway Bridges*, but with actual transverse stiffener spacing within the limits given in Article 6.3 may be considered sufficiently stiffened.

### C6B.5.3

Nominal capacities for members in the proposed guidelines are based on AASHTO’s Standard Specifications contained in the load factor section. This resistance depends on both the current dimensions of the section and the nominal material strength.

Different methods for considering the observable effects of deterioration were studied. The most reliable method available still appears to be a reduction in the nominal resistance based on measured or estimated losses in cross-sectional area and/or material strengths.

At the present time, load factor methods for determining the capacity of timber and masonry structural elements are not available.

#### C6B.5.3.1

Guidance on considering the effects of deterioration on load rating of steel structures can be found in Article C6A.6.5.

Specifications and guidance for determining the capacity of gusset plates can be found in Appendix L6B.

In Article 6.3 (Transversely Stiffened Webs), of the 2003 AASHTO *Guide Specifications for Horizontally Curved Girder Highway Bridges*, the first sentence states “Web slenderness,  $D/t_w$ , shall not exceed 150.” This statement may be interpreted to mean that webs with  $D/t_w > 150$  are considered unstiffened and the shear capacity is computed as per 6.2 (Unstiffened Webs). This statement refers to handling requirements for new design and should not be considered when determining if the web is stiffener in a rating. Furthermore, this article defines “ $d$ ” as the “required stiffener spacing” for use in Eq. 6-9. In cases of new designs, the required stiffener spacing is used to determine the smallest possible value of the buckling coefficient,  $k$ . This is conservative when actual stiffener spacings are less than the required spacing. In a rating, the actual stiffener spacing should be used to determine  $k$  in order to calculate the actual shear capacity of each panel. If the  $D/t_w > 150$ , longitudinal web stiffeners are required according to the specification (see Article 6.4). However, the shear capacity is equal to the shear buckling capacity =  $CV_p$  with no dependency on web slenderness.

ASTM F3125 has replaced ASTM A325 and A490 specifications for high strength bolts. The designations A325 and A490 will be retained in Table 6B.5.3.1-1 as this designation shows on many plans and specifications and was used in existing bridges. It should be noted in footnote c the tensile strength of M164(A325) bolts decreases for diameters greater than 1.0 in., while the tensile strength of ASTM F3125 Grade A325 and A490 do not decrease for diameters greater than 1.0 in.

Except as specified in Appendix L6B.2.6.1, the Operating rating for welds, bolts, and rivets should be determined using the maximum strengths from Table 6B.5.3.1-1.

The Operating rating for friction joint fasteners (ASTM A325 bolts) should be determined using a stress of 21 ksi.  $A_1$  and  $A_2$  should be taken as 1.0 in the basic rating equation.

Where rivets carrying loads pass through undeveloped fillers 0.25 in. or more in thickness in axially loaded connections, refer to Article 6A.6.12.5.1 and AASHTO LRFD Design Article 6.13.6.1.4 for a potential capacity reduction factor.

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APPENDIX A: ILLUSTRATIVE EXAMPLES

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|         | Span                                                         | Type                                                                 | Rated Members                               |                   |                                     |                         |       |
| A1      | Simple Span<br>65 ft                                         | Composite Steel Stringer Bridge<br>(Interior and Exterior Stringers) | Interior and Exterior Stringer              | Design            | Strength I<br>Service II<br>Fatigue | LRFR<br><br>ASR and LFR | A-1   |
|         |                                                              |                                                                      |                                             | Legal             | Strength I<br>Service II            |                         | A-39  |
|         |                                                              |                                                                      |                                             | Permit            | Strength II<br>Service II           |                         |       |
| A2      | Simple Span<br>26 ft                                         | Reinforced Concrete T-Beam Bridge                                    | Interior Beam                               | Design            | Strength I                          | LRFR<br><br>ASR and LFR | A-53  |
|         |                                                              |                                                                      |                                             | Legal             | Strength I                          |                         | A-71  |
|         |                                                              |                                                                      |                                             | Permit            | Strength II<br>Service I            |                         |       |
| A3      | Simple Span<br>80 ft                                         | Prestressed Concrete I-Girder Bridge                                 | Interior Girder                             | Design            | Strength I<br>Service III           | LRFR                    | A-87  |
|         |                                                              |                                                                      |                                             | Permit            | Strength II<br>Service I            |                         |       |
| A4      | Simple Span<br>17 ft 10 in.                                  | Timber Stringer Bridge                                               | Interior Stringer                           | Design            | Strength I                          | LRFR<br><br>ASR and LFR | A-121 |
|         |                                                              |                                                                      |                                             | Legal             | Strength I                          |                         | A-129 |
| A5      | Four-Span Continuous<br>112 ft<br>140 ft<br>140 ft<br>112 ft | Welded Steel Plate Girder Bridge                                     | Interior Girder                             | Design            | Strength I                          | LRFR                    | A-137 |
|         |                                                              |                                                                      |                                             | Legal             | Service II                          |                         |       |
|         |                                                              |                                                                      |                                             | Permit            | Strength II                         |                         |       |
| A6      | Single Span<br>175 ft                                        | Steel Through Pratt Truss Bridge                                     | Top Chord, Bottom Chord, Diagonal, Vertical | Design            | Strength I                          | LRFR                    | A-165 |
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