

THE MANUAL FOR BRIDGE EVALUATION

2019 Interim Revisions



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American Association of State Highway and Transportation Officials
444 North Capitol Street, NW, Suite 249
Washington, DC 20001
202-624-5800 phone/202-624-5806 fax
www.transportation.org

Cover photos: **Top Left:** Photo of a demonstration of Olson Instruments Bridge Deck Scanner system on a bridge taken by Larry Olson, Olson Engineering, Inc. **Second Left:** Photo of a through truss bridge taken by Thomas Drda, FHWA. **Third Left:** Photo courtesy of Idaho Department of Transportation. **Bottom Left:** Photo of an inspection of a deck truss bridge using an under bridge inspection truck. Taken by John Thiel, FHWA. **Right:** Leonard P. Zakim Bunker Hill Bridge in Boston, MA. Courtesy of Shay Burrows, FHWA.

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2019 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. Please note that these changes come after the 2019 Errata for the *Manual for Bridge Evaluation*, Third Edition, also issued this month.

Affected Articles

Underlined text indicates revisions that were approved in 2018 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 pink background to make the changes stand out when inserted in the first edition binder. They also have a page header displaying the section 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.

2019 Changed Articles

SECTION 6: LOAD RATING

6B.5.3.1

SECTION 7: FATIGUE EVALUATION OF STEEL BRIDGES

7.2.5.1

7.2.7.2.3

APPENDIX A

A1A.1.1

A1A.1.8.3

A1A.1.8.3a

A1A.1.8.3b

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- DW = Dead load effect due to wearing surface and utilities
- P = Permanent loads other than dead loads
- LL = Live load effect
- IM = Dynamic load allowance
- γ_{DC} = LRFD load factor for structural components and attachments
- γ_{DW} = LRFD load factor for wearing surfaces and utilities
- γ_p = LRFD load factor for permanent loads other than dead loads = 1.0
- γ_{LL} = Evaluation live load factor
- ϕ_c = Condition factor
- ϕ_s = System factor
- ϕ = LRFD resistance factor

The load rating shall be carried out at each applicable limit state and load effect with the lowest value determining the controlling rating factor. Limit states and load factors for load rating shall be selected from Table 6A.4.2.2-1.

Components subjected to combined load effects shall be load rated considering the interaction of load effects (e.g., axial-bending interaction or shear-bending interaction), as provided in this Manual under the sections on resistance of structures.

Secondary effects from prestressing of continuous spans and locked-in force effects from the construction process should be included as permanent loads other than dead loads, P (see Articles 6A.2.2.2. and 6A.2.2.3).

6A.4.2.2—Limit States

Strength is the primary limit state for load rating; service and fatigue limit states are selectively applied in accordance with the provisions of this Manual. Applicable limit states are summarized in Table 6A.4.2.2-1.

C6A.4.2.2

Service limit states that are relevant to load rating are discussed under the articles on resistance of structures (see Articles 6A.5, 6A.6, and 6A.7).

Table 6A.4.2.2-1—Limit States and Load Factors for Load Rating

Bridge Type	Limit State*	Dead Load γ_{DC}	Dead Load γ_{DW}	Design Load		Legal Load γ_{LL}	Permit Load γ_{LL}
				Inventory γ_{LL}	Operating γ_{LL}		
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.80	—	—	—
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service I	1.00	1.00	—	—	—	1.00
	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
Prestressed Concrete	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service III	1.00	1.00	Table 6A.4.2.2-2	—	1.00	—
	Service I	1.00	1.00	—	—	—	1.00
	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
Wood	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1

* Defined in the *AASHTO LRFD Bridge Design Specifications*

Notes:

- Gray shaded cells of the table indicate optional checks.
- Service I is used to check the $0.9 F_y$ stress limit in reinforcing steel.
- Load factor for DW at the strength limit state may be taken as 1.25 where thickness has been field measured.
- Fatigue limit state is checked using the LRFD fatigue truck (see Article 6A.6.4.1).

Table 6A.4.2.2-2—Load Factors for Live Load for the Service III Load Combination, γ_{LL} , at the Design-Load Inventory Level

Component	γ_{LL}
Prestressed concrete components rated using the refined estimates of time-dependent losses as specified in LRFD Design Article 5.9.5.4 in conjunction with taking advantage of the elastic gain	1.0
All other prestressed concrete components	0.8

6A.4.2.3—Condition Factor: ϕ_c

Use of Condition Factors as presented below may be considered optional based on an agency's load-rating practice.

The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles.

Table 6A.4.2.3-1—Condition Factor: ϕ_c

Structural Condition of Member	ϕ_c
Good or Satisfactory	1.00
Fair	0.95
Poor	0.85

C6A.4.2.3

The uncertainties associated with the resistance of an existing intact member are at least equal to that of a new member in the design stage. Once the member experiences deterioration and begins to degrade, the uncertainties and resistance variabilities are greatly increased (scatter is larger).

Additionally, it has been observed that deteriorated members are generally prone to an increased rate of future deterioration when compared to intact members. Part of ϕ_c relates to possible further section losses prior to the next inspection and evaluation.

Improved inspections will reduce, but not totally eliminate, the increased scatter or resistance variability in deteriorated members. Improved inspection and field measurements will reduce the uncertainties inherent in

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.² 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.²

E = Modulus of elasticity

ℓ = 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 ℓ/d over 11, but not greater than 50.

For short columns, ℓ/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.

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.

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.

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/tw , shall not exceed 150.” This statement may be interpreted to mean that webs with $D/tw > 150$ 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 stiffened 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 $D/tw > 150$, 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.