THE MANUAL FOR BRIDGE EVALUATION

2019 Interim Revisions

American Association
of State Highway and
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AASHTO

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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

				Design Load			
Bridge Type	Limit State*	Dead Load	Dead Load	Inventory	Operating	Legal Load	Permit Load
		γ_{DC}	γ_{DW}	γ_{LL}	γ_{LL}	γ_{LL}	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
Prestressed 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 III	1.00	1.00	Table 6A.4.2.2-2	—	1.00	—
	Service I	1.00	1.00	—	—	—	1.00
Wood	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

* 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.

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

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:

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

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 $= CVp$ 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.

7.2.2.2.1—For Determination of Evaluation or Minimum Fatigue Life

Where field-measured strains are used to generate an effective stress range, R_s, for the determination of evaluation or minimum fatigue life, the stress-range estimate partial load factor shall be taken as 0.85.

7.2.2.2.2—For Determination of Mean Fatigue Life

Where field-measured strains are used to generate an effective stress range, R_s, for the determination of mean fatigue life, the stress-range estimate partial load factor shall be taken as 1.0.

7.2.3—Determining Fatigue-Prone Details

Bridge details are only considered prone to load-induced fatigue damage if they experience a net tensile stress. Thus, fatigue damage need only be evaluated if, at the detail under evaluation:

$$2.2\left(\Delta f\right)_{tension} > f_{dead\text{-}load\ compression} \qquad (7.2.3\text{-}1)$$

where:

$(\Delta f)_{tension}$ = Tensile portion of the effective stress range as specified in Article 7.2.2, and

$f_{dead\text{-}load\ compression}$ = Unfactored compressive stress at the detail due to dead load

7.2.4—Infinite-Life Check

If:

$$\left(\Delta f\right)_{max} \leq \left(\Delta F\right)_{TH} \qquad (7.2.4\text{-}1)$$

then:

$$Y = \infty \qquad (7.2.4\text{-}2)$$

where:

$(\Delta f)_{max}$ = The maximum stress range expected at the fatigue-prone detail, which may be taken as:

C7.2.3

The multiplier of two in the equation represents the assumed relationship between maximum stress range and effective stress range, as specified in the *AASHTO LRFD Bridge Design Specifications*.

When measured stress ranges are used to evaluate fatigue life, the multiplier of two in the equation should be reconsidered based upon the discussion of Article C7.2.2.2.

If the effective truck weight is significantly less than 54 kips, a multiplier more than two should be considered. Similarly, for a measured effective truck weight greater than 54 kips, a multiplier less than two would be appropriate.

C7.2.4

Theoretically, a fatigue-prone detail will experience infinite life if all of the stress ranges are less than the constant-amplitude fatigue threshold; in other words, if the maximum stress range is less than the threshold.

When measured stress ranges are used to evaluate fatigue life, the multiplier of 2.2 in the equation for $(\Delta f)_{max}$ should be reconsidered based upon the discussion in Article C7.2.2.2.

- R_p times the factored calculated stress range due to the passage of the fatigue truck as specified in LRFD Design Article 3.6.1.4; for Fatigue I Load Combination
- $2.2(\Delta f)_{eff}$; for calculated stress range due to a fatigue truck determined by a truck survey or weigh-in-motion study with R_s=1.0
- Larger of maximum (Δf_i), $2.2(\Delta f)_{eff}$, or other suitable value; for measured stress ranges $_{with}\ R_{s=1.0}$

$(\Delta F)_{TH}$ = The constant-amplitude fatigue threshold given in LRFD Design Table 6.6.1.2.5-3

Otherwise, the total fatigue life shall be estimated as specified in Article 7.2.5.

The load factor is 1.75 for live load specified for the Fatigue I limit state (infinite load-induced fatigue life) in LRFD Design Table 3.4.1-1.

When measured stress ranges are used to evaluate fatigue life, the maximum stress range should be taken as the larger value of 2.2 times the field measured effective stress range or the field measured maximum stress range, unless another suitable value is justified.

7.2.5—Estimating Finite Fatigue Life

7.2.5.1—General

Four levels of finite fatigue life may be estimated:

- The minimum expected fatigue life (which equals the conservative design fatigue life),
- Evaluation 1 fatigue life (which equals a conservative fatigue life for evaluation),
- Evaluation 2 fatigue life (which equals a less conservative fatigue life for evaluation), and
- The mean fatigue life (which equals the statistically most likely fatigue life).

The total ~~finite fatigue~~ estimated remaining life of a fatigue-prone detail, in years, shall be determined as follows:

- Check that there is remaining fatigue life at the present age; that is,

$$N_{av} > N_1 \qquad (7.2.5.1\text{-}1)$$

in which:

N_{av} = number of initially available fatigue stress cycles

$$= \frac{R_R A}{\left[(\Delta f_{eff})^3\right]} \qquad (7.2.5.1\text{-}2)$$

N_1 = number of fatigue stress cycles consumed over the present age of the detail

C7.2.5.1

Much scatter, or variability, exists in experimentally derived fatigue lives. For design, a conservative fatigue resistance of two standard deviations shifted below the mean fatigue resistance or life is assumed. This corresponds to the minimum expected finite fatigue life of this Article. This is approximately equivalent to a 97.5 percent probability that cracking will not be observed at the calculated minimum expected fatigue life. Limiting actual usable fatigue life to this design fatigue life is most cautious and can be costly since cracking may be predicted, but not observed. As such, means of estimating the two evaluation fatigue lives and the mean finite fatigue life are also included to aid the evaluator in the decision making. Evaluation 1 is equivalent to the evaluation life in the previous specification, while Evaluation 2 fatigue life provides an additional choice for the user midway between Evaluation 1 and the mean fatigue life values.

$$= \begin{cases} for\ \ (ADTT_{SL})_{PRESENT} = (ADTT_{SL})_0: \quad 365n(a+1)(ADTT_{SL})_{PRESENT} \\ \\ otherwise: \qquad 365n(ADTT_{SL})_{PRESENT}\left[\dfrac{1-\frac{(ADTT_{SL})_0}{(ADTT_{SL})_{PRESENT}}}{\left(\frac{(ADTT_{SL})_{PRESENT}}{(ADTT_{SL})_0}\right)^{\frac{1}{a}}-1}+1\right] \end{cases} \qquad (7.2.5.1\text{-}3)$$

---

where:

a = Current age of the detail in years

A = Detail-category constant given in LRFD Design Table 6.6.1.2.5-1

$(ADTT_{SL})_0$ = Average number of trucks per day in a single lane in the first year the detail was in service

$(ADTT_{SL})_{PRESENT}$ = Average number of trucks per day in a single lane in the present year

$(\Delta f)_{eff}$ = The effective stress range as specified in Article 7.2.2 (ksi)

N = Number of stress-range cycles per truck passage estimated according to Article 7.2.5.2

R_R = Resistance factor specified for evaluation, minimum, or mean fatigue life as given in Table 7.2.5.1-1

Y = Total fatigue life calculated as $Y = Y_{REM} + a$

- If N_{av} is greater than N_1, calculate the estimated remaining fatigue life, Y_{REM}, of the fatigue-prone detail as follows:

$$Y_{REM} = \frac{\log\left[\left(\dfrac{g}{1+g}\right)\left(\dfrac{N_{av}-N_1}{365n\left(ADTT_{SL}\right)_{PRESENT}}\right)+1\right]}{\log(1+g)} \qquad (7.2.5.1\text{-}4)$$

where:

g = Expected annual growth rate of the average number of trucks per day in a single lane. Input in decimal form, e.g., if growth rate is 2 percent, input 0.02.

The methodology specified herein for the calculation of the estimated remaining fatigue life of a fatigue-prone detail allows for the input of the average number of trucks per day in a single lane averaged over the first year the detail was in service, $(ADTT_{SL})_0$, in order to avoid unrealistically low estimates of the number of fatigue stress cycles consumed over the present age of the detail, N_1. Additionally, to avoid the risk of considering unrealistically high $ADTT_{SL}$ values in the future, the methodology allows the user to input a limit on the maximum average number of trucks per day in a single lane, $(ADTT_{SL})_{LIMIT}$, for the roadway under consideration for use in the evaluation. It is well known that there are physical upper limits to the $ADTT_{SL}$. Assuming that this limit occurs at some time in the future, it must be checked to ensure that the number of years from the present day until $(ADTT_{SL})_{LIMIT}$ is reached, $(Y_{ADTT})_{LIMIT}$, is less than the remaining fatigue life, Y, calculated by Eq. 7.2.5.1-4. When this limit is reached before the estimated remaining fatigue life is exhausted, either a lower assumed value of the expected annual growth rate, g, must be used, if justified, to ensure that $(ADTT_{SL})_{LIMIT}$ is not exceeded, or else the estimated remaining fatigue life must be revised to account for this limitation. In the latter case, the remaining fatigue life is calculated assuming the number of cycles per year is held constant at $(ADTT_{SL})_{LIMIT}$ for all future years beyond which $(ADTT_{SL})_{LIMIT}$ is reached. A revised estimate of the remaining fatigue life is then calculated from Eq. 7.2.5.1-8 as the sum of the number of years calculated based on the preceding assumption and $(Y_{ADTT})_{LIMIT}$.

The fatigue life calculation approach is based on the periodic compounding formulas. A single growth rate is assumed for the entire life of the structure. The approach is slightly modified so that the Engineer has to input the ADTT at year zero, and the limiting ADTT in addition to the current ADTT, the expected ADTT growth rate, and the age of the structure.

~~Recent research has made it possible to obtain a closed-form solution for the total finite fatigue life using an estimated traffic growth rate and the present (ADTT)~~$_{SL}$~~.~~ The ~~estimated~~ expected annual traffic growth rate, g, can be obtained using available information in the agency's bridge inventory or the NBI. For cases with ~~zero traffic growth, a very small value of g should be selected (less than 0.01 percent) for use in the expressions for Y~~

zero or negative growth rate, i.e., $g \leq 0$, a very small positive value of g should be conservatively selected (less than 0.0001 percent) for use in the expressions given in these articles to avoid erroneous results. In some cases, $(ADTT_{SL})_{FUTURE}$ may be found to be greater than $(ADTT_{SL})_{LIMIT}$ if overly conservative estimates of the expected annual growth rate are used. In these cases, it may be reasonable to reduce the expected growth rate.

- Check the following:

$$(ADTT_{SL})_{FUTURE} \leq (ADTT_{SL})_{LIMIT} \qquad (7.2.5.1\text{-}5)$$

in which:

$(ADTT_{SL})_{FUTURE}$
= Average number of trucks per day in a single lane in the year when the life corresponding to year Y_{REM} is reached

$$= \left[(ADTT_{SL})_{PRESENT}\right](1+g)^{Y_{REM}} \qquad (7.2.5.1\text{-}6)$$

where:

$(ADTT_{SL})_{LIMIT}$
= Highway design maximum average number of trucks per day in a single lane for the roadway under consideration

- If $(ADTT_{SL})_{FUTURE} > (ADTT_{SL})_{LIMIT}$,:
    - Calculate the number of years from the present day until $(ADTT_{SL})_{LIMIT}$ is reached, $(Y_{ADTT})_{LIMIT}$, and the estimated remaining fatigue life, Y_{REM}, of the fatigue-prone detail as follows:

$$(Y_{ADTT})_{LIMIT} = \frac{\log\left[\dfrac{(ADTT_{SL})_{LIMIT}}{(ADTT_{SL})_{PRESENT}}\right]}{\log(1+g)}$$

(7.2.5.1-7)

$$Y_{REM} = \frac{N_{av} - N_1}{365n(ADTT_{SL})_{LIMIT}} - \frac{(1+g)^{(Y_{ADTT})_{LIMIT}} - 1}{g(1+g)^{(Y_{ADTT})_{LIMIT}-1}} + (Y_{ADTT})_{LIMIT}$$

(7.2.5.1-8)

~~g = Estimated annual traffic-growth rate, percent, expressed as a decimal; i.e., 5 percent = 0.05~~

~~a = Present age of the detail in years~~

~~$[(ADTT)_{SL}]_{PRESENT}$ = Present average number of trucks per day in a single lane~~

~~$(\Delta f)_{eff}$ = The effective stress range as specified in Article 7.2.2~~

The resistance factors for fatigue life, specified in Table 7.2.5.1-1, represent the variability of the fatigue life of the various detail categories, A through E′. The minimum life, Evaluation 1 Life and Evaluation 2 Life fatigue-life curves are shifted from the mean fatigue-life S-N curves in log-log space. Scatter of the fatigue lives at given stress range values from controlled laboratory testing provides statistical information on fatigue behavior of bridge details under cyclic loading. Accordingly, the probability of failure associated with each level of fatigue life approaches 2 percent, 16 percent, 33 percent and 50 percent for the minimum Evaluation 1, Evaluation 2 and mean fatigue lives, respectively. Typically, the minimum life or Evaluation 1 Life is used to evaluate the fatigue serviceability. If concerns are encountered regarding the computed fatigue serviceability, then the serviceability index can be revised according to Article 7.2.7.2.

Table 7.2.5.1-1—Resistance Factor for Evaluation, Minimum or Mean Fatigue Life, R_R

Detail Category (from Table 6.6.1.2.5-1 of the *LRFD Specifications*)	R_R			
	Minimum Life	Evaluation 1 Life	Evaluation 2 Life	Mean Life
A	1.0	1.5	2.2	2.9
B	1.0	1.3	1.7	2.0
B’	1.0	1.3	1.6	1.9
C	1.0	1.3	1.7	2.1
C’	1.0	1.3	1.7	2.1
D	1.0	1.3	1.7	2.0
E	1.0	1.2	1.4	1.6
E’	1.0	1.3	1.6	1.9

7.2.5.2—Estimating the Number of Cycles per Truck Passage

The number of stress-range cycles per truck passage may be estimated (in order of increasing apparent accuracy and complexity):

- Through the use of LRFD Design Table 6.6.1.2.5-2,
- Through the use of influence lines, or
- By field measurements.

7.2.6—Fatigue Serviceability Index

7.2.6.1—Calculating the Fatigue Serviceability Index

The fatigue serviceability index shall be calculated as:

$$Q = \left(\frac{Y - a}{N}\right) GRI \qquad (7.2.6.1\text{-}1)$$

where:

Y = Calculated fatigue life, as given in Section 7.2.5.1

a = Present age of detail, in years

N = Greater of Y or 100 years

G = Load Path Factor, as given in Table 7.2.6.1-1

R = Redundancy Factor, as given in Table 7.2.6.1-2

I = Importance Factor, as given in Table 7.2.6.1-3

Table 7.2.6.1-1—Load Path Factor, *G*

Number of Load Path Members	*G*
1 or 2 members	0.8
3 members	0.9
4 or more members	1

Table 7.2.6.1-2—Redundancy Factor, *R*

Type of Span	*R*
Simple	0.9
Continuous	1

Table 7.2.6.1-3—Importance Factor, *I*

Structure or Location	Importance Factor, *I*
Interstate Highway Main Arterial State Route Other Critical Route	0.90
Secondary Arterial Urban Areas	0.95
Rural Roads Low *ADTT* Routes	1.00

C7.2.6.1

The fatigue serviceability index is a dimensionless relative measure of the performance of a structural detail, at a particular location in the structure, with respect to the overall fatigue resistance of the member. The numerical values for Q vary between 1.0 and 0.

The load path, redundancy, and importance factors are risk factors that modify the fatigue serviceability index. They reduce the index from its base value, i.e. based on fatigue resistance alone, to a reduced value that reflects greater consequences from the lack of ability to redistribute the load (load path factor), lack of redundancy (redundancy factor), or use of the structure (importance factor). The net effect of a reduction in the index will be to move the composite index value to a lower value that may initiate a lower fatigue rating. These risk factors are similar to the ductility, redundancy, and operational classification factors in the *AASHTO LRFD Bridge Design Specifications*. Improved quantification with time will possibly modify these factors.

The number of members that carry load when a fatigue truck is placed on the bridge is used to select the load path factor; e.g., two members for a two-girder bridge and for a typical truss structure; four or more members for a multi-beam or multi-girder bridge; etc. For floorbeams, consider the number of floorbeam members loaded by the fatigue truck. For diaphragms and secondary members, use $G = 1$.

The redundancy factor is to be applied for the specific member or detail being considered. Redundant subsystems need not be penalized when supported by a non-redundant main system.

7.2.6.2—Recommended Actions Based on Fatigue Serviceability Index

The fatigue serviceability index may be used as a guide for actions that may be undertaken based on fatigue ratings and assessments. Bridge owners should develop a uniform methodology for recommended actions based upon the fatigue serviceability index.

C7.2.6.2

In the recommended actions provided, it is expected that based upon increasing risk, the inspection frequency of the bridge shall be increased on a case-by-case assessment by the bridge owner. Actions may include increased inspection frequency, reassessment as given in Article 7.2.7, or retrofit.

7.2.7—Strategies to Increase Fatigue Serviceability Index

7.2.7.1—General

If the fatigue serviceability index is deemed unacceptable, the strategies of Articles 7.2.7.2 and 7.2.7.3 may be applied to enhance the fatigue serviceability index.

C7.2.7.1

Retrofit, increased inspection, or load-restriction decisions should be made based upon the evaluation fatigue life unless the physical condition or fabrication quality of the bridge is poor. In general, it is uneconomical to limit the useful fatigue life of in-service bridges to the minimum (design) fatigue life.

If the estimated fatigue serviceability index based upon the evaluation fatigue life is deemed unacceptable, a fatigue life approaching the mean fatigue life can be used for evaluation purposes if the additional risk of fatigue cracking is acceptable.

7.2.7.2—Recalculate Fatigue Serviceability Index

7.2.7.2.1—Through Accepting Greater Risk

In general, Evaluation 1 Life of Article 7.2.5 is used in determining the fatigue serviceability index of a bridge detail according to Article 7.2.6. If the evaluator is willing to accept greater risk of fatigue cracking due to:

- Long satisfactory cyclic performance of the detail to date,
- A high degree of redundancy, and/or
- Increased inspection effort, (e.g., decreased inspection interval),
- Some combination of the above,

the fatigue serviceability index may be determined using a fatigue life approaching the mean fatigue life of Article 7.2.5.

C7.2.7.2.1

Greater risk of fatigue cracking can be realized by using either the Evaluation 2 or mean fatigue life values to compute the fatigue serviceability index.

7.2.7.2.2—Through More Accurate Data

The calculated fatigue serviceability index may be refined by using more accurate data as input to the fatigue-life estimate. Sources of improvement of the estimate include:

- Field measurement of stress ranges at the fatigue-prone detail under construction
- 3-D finite element analysis for stresses at the fatigue-prone detail under consideration
- Weigh-in-motion data of truck weights at or near the bridge site,
- Site-specific data on average daily truck traffic (*ADTT*) at or near the bridge site

This strategy is based upon achieving a better estimate of the fatigue life.

7.2.7.2.3—Through Truncated Fatigue Life Distribution

When a negative fatigue serviceability index is obtained according to Article 7.2.6, the detail's fatigue serviceability index may be updated using the equations below for mean, evaluation, and minimum lives, provided a field inspection finds no evidence of fatigue cracking at the detail.

$$Y'_{mean} = 2.19 Y_{mean} e^{0.73\Phi^{-1}[0.18(1-P)+P]-0.27} \tag{7.2.7.2.3-1}$$

$$Y'_{eval2} = 2.19 Y_{mean} e^{0.73\Phi^{-1}[0.12(1-P)+P]-0.27} \tag{7.2.7.2.3-2}$$

$$Y'_{eval1} = 2.19 Y_{mean} e^{0.73\Phi^{-1}[0.074(1-P)+P]-0.27} \tag{7.2.7.2.3-3}$$

$$Y'_{minimum} = 2.19 Y_{mean} e^{0.73\Phi^{-1}[0.039(1-P)+P]-0.27} \tag{7.2.7.2.3-4}$$

where:

Y'_{mean} = updated mean life in years

Y_{mean} = mean life in years without updating based on no detection of cracking at detail in question

Y'_{eval1} = updated Evaluation 1 Life in years

Y'_{eval2} = updated Evaluation 2 Life in years

$Y'_{minimum}$ = updated minimum life in years

Φ^{-1} = inverse of the standard normal variable's cumulative probability function (Table 7.2.7.2-1)

C7.2.7.2.3

The fatigue life of a structural detail is modeled using a log normally distributed random variable, as shown in Figure C7.2.7.2.3-1. When the estimated life using Article 7.2.5 is smaller than the present age, the remaining life becomes negative as illustrated.

In this situation, if field inspection finds no evidence of cracking, the estimated life is an overly-conservative estimate. The low tail of the total life distribution is truncated up to the present life. The eliminated probability, *P*, is computed, and the resulting probability density function is divided by (1 – *P*) to ensure that the total probability under the distribution curve is still 1.0 as shown in Figure C7.2.7.2.3-2. Then the updated life is determined to maintain the same reliability level for fatigue life distribution. Functions Φ (.) and Φ^{-1}(.) are commonly available in commercial spreadsheet programs.

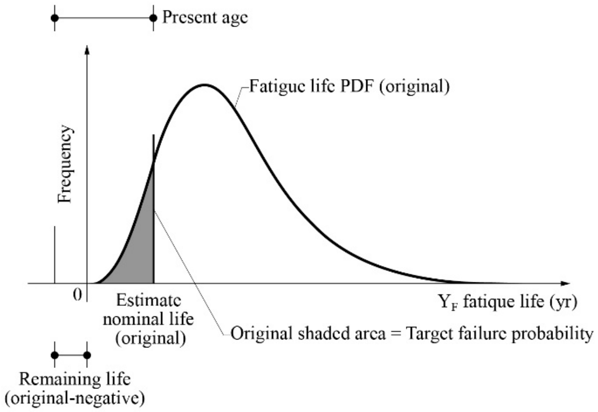


Figure C7.2.7.2.3-1—Probability Density Function of Fatigue Life and Estimated Life as a Value on Horizontal Axis

P = probability of fatigue life being shorter than current age before updating based on no crack found

$$= \Phi\left[\frac{Ln\left(\frac{a}{2.19Y_{mean}}\right)+0.27}{0.73}\right] \qquad (7.2.7.2.3\text{-}5)$$

where:

a = present age in years

Φ = standard normal variable's cumulative probability function (Table 7.2.7.2-1)

Since it is not possible to reliably determine if fatigue cracks exist under the heads of fasteners, such as in riveted structures, this Article shall not be applied when evaluating the internal redundancy of a member using the *Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-up Steel Members* unless further investigation using destructive or nondestructive testing has confirmed with a reasonable degree of certainty that fatigue cracking is not present. It is acceptable to utilize this Article for mechanically fastened fracture critical members when the hands-on inspection interval is intended to remain at 24 months.

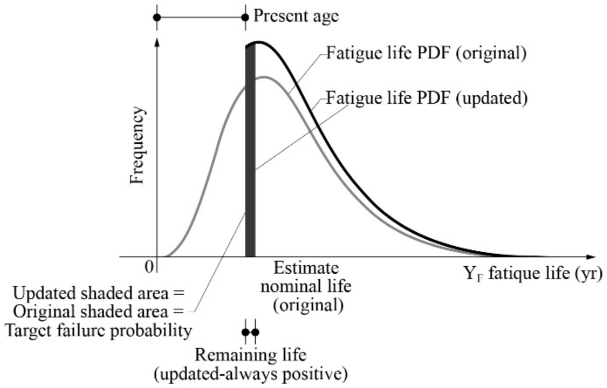


Figure C7.2.7.2.3-2—Truncated Probability Density Function of Fatigue Life and Updated Life as a Value on the Horizontal Axis

Significant caution should be exercised when invoking this Article as it is entirely based on the presumption that no cracks exist on the structure as reported during field inspection, though the fatigue life calculations suggest otherwise. The provisions found in the *Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-up Steel Members* requires that positive remaining fatigue life exists in order to exploit the internal redundancy of built-up members. Since cracks that could be present under a fastener head can't be found using typical visual inspection techniques, it is not permissible to utilize Article 7.2.7.2.3 when evaluating internal redundancy.

Consideration should also be given to the probability of detection (POD) associated with other crack types and locations. For example, weld toe cracks at cover plates and other details can be very difficult to find unless some visual aids are used, such as a 10x magnifying glass. During the development of the fatigue categories for cover plates for example, such a visual aid was needed in many cases.

Table 7.2.7.2-1—Cumulative Distribution Function Φ(*x*) for Standard Normal Variable *x*

x	**0.00**	**0.01**	**0.02**	**0.03**	**0.04**	**0.05**	**0.06**	**0.07**	**0.08**	**0.09**
0.0	0.5000	0.5040	0.5080	0.5120	0.5160	0.5199	0.5239	0.5279	0.5319	0.5359
0.1	0.5398	0.5438	0.5478	0.5517	0.5557	0.5596	0.5636	0.5675	0.5714	0.5753
0.2	0.5793	0.5832	0.5871	0.5910	0.5948	0.5987	0.6026	0.6064	0.6103	0.6141
0.3	0.6179	0.6217	0.6255	0.6293	0.6331	0.6368	0.6406	0.6443	0.6480	0.6517
0.4	0.6554	0.6591	0.6628	0.6664	0.6700	0.6736	0.6772	0.6808	0.6844	0.6879
0.5	0.6915	0.6950	0.6985	0.7019	0.7054	0.7088	0.7123	0.7157	0.7190	0.7224
0.6	0.7257	0.7291	0.7324	0.7357	0.7389	0.7422	0.7454	0.7486	0.7517	0.7549
0.7	0.7580	0.7611	0.7642	0.7673	0.7704	0.7734	0.7764	0.7794	0.7823	0.7852
0.8	0.7881	0.7910	0.7939	0.7967	0.7995	0.8023	0.8051	0.8078	0.8106	0.8133
0.9	0.8159	0.8186	0.8212	0.8238	0.8264	0.8289	0.8315	0.8340	0.8365	0.8389
1.0	0.8413	0.8438	0.8461	0.8485	0.8508	0.8531	0.8554	0.8577	0.8599	0.8621
1.1	0.8643	0.8665	0.8686	0.8708	0.8729	0.8749	0.8770	0.8790	0.8810	0.8830
1.2	0.8849	0.8869	0.8888	0.8907	0.8925	0.8944	0.8962	0.8980	0.8997	0.9015
1.3	0.9032	0.9049	0.9066	0.9082	0.9099	0.9115	0.9131	0.9147	0.9162	0.9177
1.4	0.9192	0.9207	0.9222	0.9236	0.9251	0.9265	0.9279	0.9292	0.9306	0.9319
1.5	0.9332	0.9345	0.9357	0.9370	0.9382	0.9394	0.9406	0.9418	0.9429	0.9441
1.6	0.9452	0.9463	0.9474	0.9484	0.9495	0.9505	0.9515	0.9525	0.9535	0.9545
1.7	0.9554	0.9564	0.9573	0.9582	0.9591	0.9599	0.9608	0.9616	0.9625	0.9633
1.8	0.9641	0.9649	0.9656	0.9664	0.9671	0.9678	0.9686	0.9693	0.9699	0.9706
1.9	0.9713	0.9719	0.9726	0.9732	0.9738	0.9744	0.9750	0.9756	0.9761	0.9767
2.0	0.9772	0.9778	0.9783	0.9788	0.9793	0.9798	0.9803	0.9808	0.9812	0.9817
2.1	0.9821	0.9826	0.9830	0.9834	0.9838	0.9842	0.9846	0.9850	0.9854	0.9857

7.2.7.3—Retrofit

If the recalculated fatigue serviceability index is not ultimately acceptable, the actual fatigue serviceability index may be increased by retrofitting the critical details to improve the detail category, and thus increase the fatigue serviceability index. This strategy increases the actual fatigue serviceability index when further enhancement of the calculated fatigue serviceability index, through improved input, is no longer possible or practical.

7.3—DISTORTION-INDUCED FATIGUE EVALUATION

Distortion-induced fatigue is typically caused by the out-of-plane deformation of the web plate, which causes fatigue crack formation on details that are prone to such cracking under cyclic loading. The cracks tend to form in the member web at locations where there is a geometrical discontinuity, such as a vertical gap between a stiffener or connection plate and the girder flange or a horizontal gap between a gusset plate and a connection plate. Distortion-induced fatigue is a stiffness problem (more precisely the lack thereof) versus a load problem.

C7.2.7.3

In certain cases, Owners may wish to institute more intensive inspections, in lieu of more costly retrofits, to assure adequate safety. Restricting traffic to increase the fatigue serviceability index is generally not considered cost effective. If the fatigue serviceability index is deemed inadequate, the appropriate option to increase the fatigue serviceability index should be determined based upon the economics of the particular situation.

C7.3

Often, distortion-induced fatigue cracks initiate after relatively few stress-range cycles at fatigue-prone details. However, depending upon the magnitude of the out-of-plane distortion and the geometry of the web gap detail, the crack growth may be slow and a significant period of time may be required before they become large enough to be detected visually. Therefore, existing bridges should not be assumed to be insensitive to distortion-induced cracking if fatigue cracks do not appear after a short period of time. Experience has shown that in some cases,

APPENDIX A:

ILLUSTRATIVE EXAMPLES

A1—SIMPLE SPAN COMPOSITE STEEL STRINGER BRIDGE

PART A—LOAD AND RESISTANCE FACTOR RATING METHOD

A1A.1—Evaluation of an Interior Stringer

A1A.1.1—Bridge Data

Span: 65 ft
Year Built: 1964
Material: A36 Steel
F_y = 36 ksi
f'_c = 3 ksi
Condition: No deterioration (NBI Item 59 = 7)
Member is in good condition
Riding Surface: Minor surface deviations (Field verified and documented)
ADTT (one direction): ~~1,000~~ 700
$[ADTT_{SL}]_0$ 200 (ADTT at year 0)
$[ADTT_{SL}]_{LIMIT}$ 1,200 (roadway limit ADTT)

Skew: 0°
Additional Information: Diaphragms spaced at 16 ft 3 in.

A1A.1.2—Section Properties

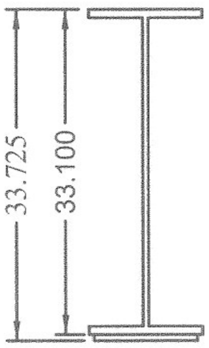
In unshored construction, the noncomposite steel stringer must support its own weight plus the weight of the concrete slab. For the composite section, the concrete is transformed into an equivalent area of steel by dividing the area of the slab by the modular ratio. Live load plus impact stresses are carried by the composite section using a modular ratio of n. To account for the effect of creep, superimposed dead-load stresses are carried by the composite section using a modular ratio of $3n$ (LRFD Design 6.10.1.1.1.b). The as-built section properties are used in this analysis as there is no deterioration.

A1A.1.2.1—Noncomposite Section Properties

Section properties of rolled shapes are subject to change with changes in rolling practices of the steel industry. Identify steel components from available records, construction date, and field measurements. The section properties for this beam were determined from *AISC Manual of Steel Construction*, Sixth Edition, printed during the period from July 1963 to March 1967, which is consistent with the "Year Built" date for this bridge.

$W\,33 \times 130$
t_f = 0.855 in.
b_f = 11.51 in.
t_w = 0.58 in.
A = 38.26 in.^{2}
I = 6,699 in.^{4}

$PL\ ^5/_8$ in. × 10 $^1/_2$ in.
t = 0.625 in.
b = 10.5 in.
$A = t \times b$ = 6.56 in.^{2}
I ~0 in.^{4} (negligible)



$$\bar{y} = \frac{\left(\frac{D_{W33\times130}}{2} + t_{PL}\right)\left(A_{W33\times130}\right) + \left(\frac{t_{PL}}{2}\right)\left(t_{PL} \times b_{PL}\right)}{A_{W33\times130} + \left(t_{PL} \times b_{PL}\right)}$$

$$\bar{y} = \frac{(17.175)(38.26) + (0.313)(6.56)}{38.26 + 6.56} \quad \text{Distance to } C.G.$$

$$\bar{y} = 14.71 \text{ in. from bottom of section to centroid}$$

$$I_x = 6{,}699 + 38.26(2.47)^2 + 6.56(14.40)^2$$

$$I_x = 8{,}293 \text{ in.}^4$$

$$S_t = \frac{8{,}293}{19.02} = 436.0 \text{ in.}^3 \quad \text{Section Modulus at top of steel}$$

$$S_b = \frac{8{,}293}{14.71} = 563.7 \text{ in.}^3 \quad \text{Section Modulus at bottom of steel}$$

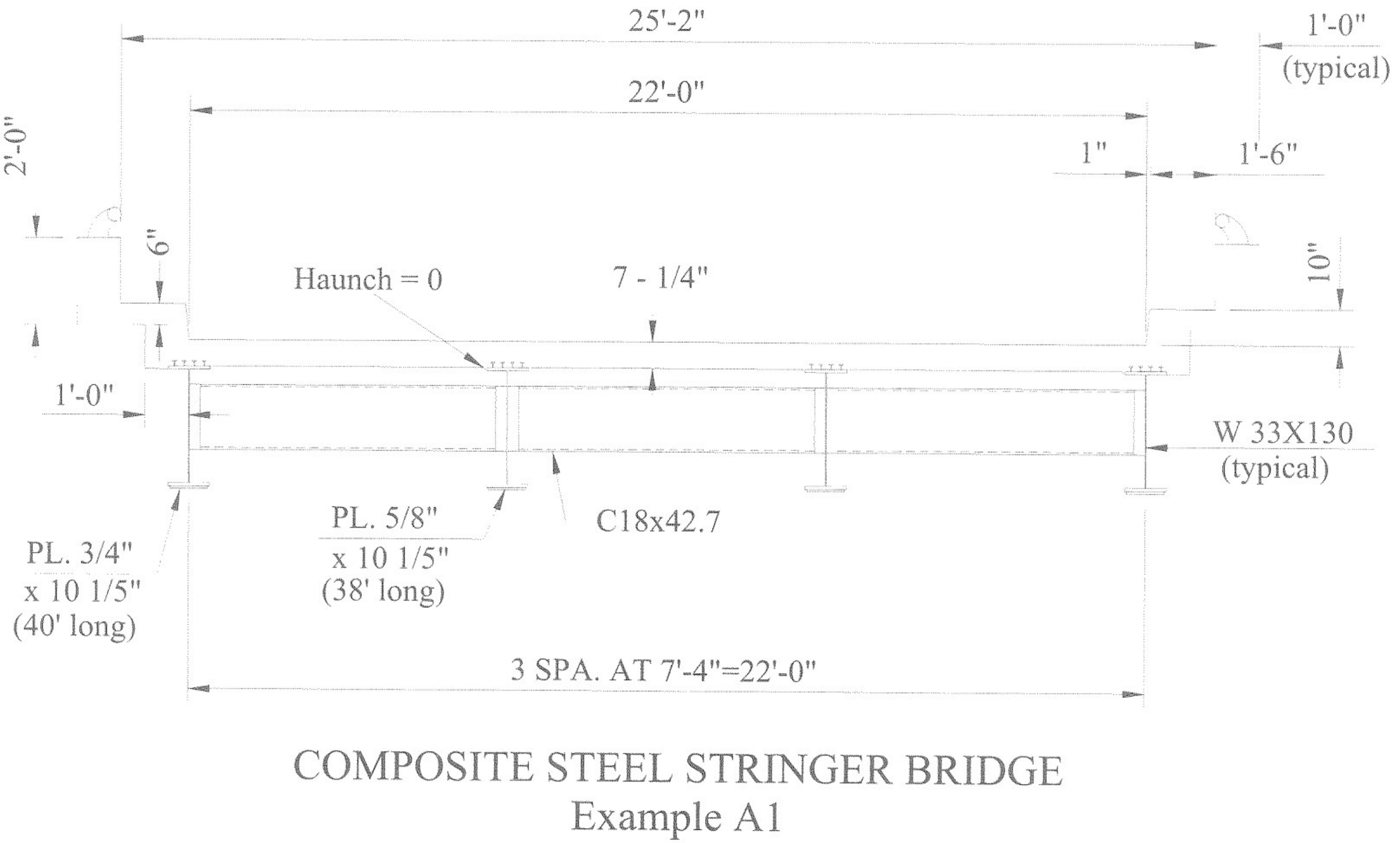


Figure A1A.1.2.1-1—Composite Steel Stringer Bridge

$$RF = \frac{34.2-(1.0)(11.49)}{(1.3)(14.42)}$$

$$= 1.21$$

A1A.1.8.2b—Operating Level

γ_{LL} = 1.0 γ_{DC} = 1.0 Table 6A.4.2.2-1

$$RF = \frac{34.2-(1.0)(11.49)}{(1.0)(14.42)}$$

$$= 1.57$$

A1A.1.8.3—Fatigue Limit State (6A.6.4.1)

Determine if the bridge has any fatigue-prone details (Category C or lower).

The transverse welds detail connecting the ends of cover plates to the flange are fatigue-prone details. Use Category E' details because the flange thicknesss = 0.855 in. is greater than 0.8 in. LRFD Design Table 6.6.1.2.3-1

If $2.2(\Delta f)_{tension} > f_{dead\text{-}load\ compression}$, the detail may be prone to fatigue.

Eq. 7.2.3-1

7.2.3

$f_{dead\text{-}load\ compression}$

= 0 at cover plate at all locations because beam is a simple span and cover plate is located in the tension zone

∴ must consider fatigue; ~~compute *RF* for fatigue load for infinite life.~~ determine if the detail possesses infinite life.

$$\xcancel{RF = \frac{f_R-(\gamma_{DC})(f_{DC})}{(\gamma_{LL})(\Delta f_{LL+IM})_{max}}}$$

~~f_R = $(\Delta F)_{TH}$~~

~~γ_{LL} = 0.80 γ_{DC} = 0.00~~ ~~Table 6A.4.2.2-1~~

Composite section properties without cover plate:

$$\overline{y} = \frac{\sum A\times\overline{y}}{\sum A} = \frac{(38.26)(16.55)+\left(\frac{88}{9}\times7.25\right)(36.725)}{(38.26)+\left(\frac{88}{9}\times7.25\right)}$$

= 29.65 in. from bottom of flange

$$I_x = 6,699+(38.26)(13.10)^2+\frac{\left(\frac{88}{9}\right)(7.25)^3}{12}+\frac{88}{9}(7.25)(7.07)^2$$

$$= 17,119 \text{ in.}^4$$

$$S_b = \frac{17,119}{29.65} = 577 \text{ in.}^3$$

Live Load at Cover Plate Cut-Off (13.5 ft. from centerline of bearing)

Fatigue Load: Design truck with a spacing of 30 ft between 32 kip axles. — LRFD Design 3.6.1.4.1 and Figure 3.6.1.2.2-1

M_{LL} = (32 kips) (10.69 ft) + (32 kips) (4.46 ft) + (8 kips) (1.56 ft)

= 497 kip-ft = 5,967 kip-in. — Using influence lines.

IM = 15 percent — LRFD Design Table 3.6.2.1-1

M_{LL+IM} = (1.15) (5,967) = 6,862 kip-in.

A1A.1.8.3a—Load Distribution for Fatigue — LRFD Design 3.6.1.4.3b

The single-lane distribution factor will be used for fatigue. — LRFD Design 3.6.1.1.2

Remove multiple presence factor from the single-lane distribution. — LRFD Design C3.6.1.1.2

$$g_{Fatigue} = \frac{1}{1.2}\left(g_{m1}\right)$$

$$= \frac{1}{1.2}\left(0.46\right)$$

$$= 0.383$$

Distributed Live-Load Moment:

gM_{LL+IM} = (0.383) (6862)

= 2,628 kip-in.

Fatigue Load Stress Range:

$$\Delta f_{LL+IM} = \frac{2,628}{577}$$

= 4.56 ksi at the cover plate weld

Nominal fatigue resistance for infinite life:

$(\Delta F)_{TH}$ = 2.6 ksi for Detail Category E′ — LRFD Design Table 6.6.1.2.5-3

Infinite-Life Fatigue Check: — 7.2.4

~~ADTT = 1,000~~ $(ADTT)_{PRESENT}$ = 700

Span Length = 65 ft

Number of lanes = 2

R_p = 0.988 + 6.87x10^{-5} Span Length + 4.01x10^{-6} ~~ADTT~~ $(ADTT)_{PRESENT}$ + 0.0107 / Number of lanes — 7.2.2.1

= 0.988 + 6.87x10^{-5}*65 + 4.01x10^{-6}*~~1,000~~700+0.0107/2

= 1.00~~18~~

$(\Delta f)_{max} = (R_p)(\Delta f_{FATIGUE\ I})$ = (1.00~~18~~)(1.75)(4.56) — 7.2.4

= 7.99 ksi > 2.6 ksi

Thus, $(\Delta f)_{max} > (\Delta F)_{TH}$.

The detail does not possess infinite fatigue life.

Evaluate the estimated remaining fatigue life using procedures given in Section 7.

A1A.1.8.3b—Calculation of Estimated Remaining Fatigue Life

Fatigue life determination will be based upon the finite fatigue life.

$$\xcancel{Y = \frac{\log\left[\dfrac{R_R A}{365n\left[(ADTT)_{SL}\right]_{PRESENT}\left((\Delta f)_{eff}\right)^3} g(1+g)^{a-1}+1\right]}{\log(1+g)}}$$

~~7.2.5.1~~

$ADTT$ (One Direction) = ~~1,000~~ 700 (present value)

LRFD Design Table 3.6.1.4.2-1

$[ADTT_{SL}]_{PRESENT}$ = 0.85(~~1,000~~700) = ~~850~~595 (call 600)

Traffic Growth Rate g: ~~2~~1.0 percent is applied over the life of the bridge (input as 0.010)

Bridge Age a: 48 years

Assume Evaluation 1 Life to be used for bridge assessment.

Table 7.2.5.1-1

Hence, $R_R = 1.3$

Calculate effective stress range:

R_p = 1.00~~18~~

$R_{sa} = 1.0$

$R_{st} = 1.0$

$R_s = R_{sa} \times R_{st} = 1.0$

$\Delta f_{eff} = (R_p)(R_s)(\Delta f_{FATIGUE\ II})$ = (1.00~~18~~)(1.0)(0.80)(4.56) = 3.65 ksi

$A = 3.9 \times 10^8$ ksi^3

$n = 1.0$ simple span girders ~~with $L > 40$ ft~~

Table 7.2.2.1-1

7.2.2

LRFD Design Table 6.6.1.2.5-1

LRFD Design Table 6.6.1.2.5-2

$$\xcancel{Y = \frac{\log\left[\dfrac{(1.3)\left(3.9\times10^8\right)}{(365)(1.0)(850)\left(3.65^3\right)}(0.02)(1+0.02)^{48-1}+1\right]}{\log\left(1+0.02\right)} = 50 \text{ years}}$$

Check that there is remaining fatigue life at the present age. Noting that $(ADTT_{SL})_{PRESENT} \neq (ADTT_{SL})_0$), that is,

7.2.5.1

$$N_{av} > N_1$$

$$N_{av} = \frac{R_R A}{\left(\Delta f_{eff}\right)^3} = \frac{1.3\left(3.9x10^8\right)}{(3.65)^3} = 10426280\ cycles$$

$$N_1 = 365n(ADTT_{SL})_{PRESENT}\left[\frac{1-\dfrac{(ADTT_{SL})_0}{(ADTT_{SL})_{PRESENT}}}{\left(\dfrac{(ADTT_{SL})_{PRESENT}}{(ADTT_{SL})_0}\right)^{\frac{1}{a}}-1}+1\right]$$

$$N_1 = 365(1.0)(600)\left[\frac{1-\frac{200}{600}}{\left(\frac{600}{200}\right)^{\frac{1}{48}}-1}+1\right] = 6525235\ cycles < N_{av}\quad ok$$

Calculate the estimated remaining fatigue life, Y_{REM}, of the fatigue-prone detail as follows:

$$Y_{REM} = \frac{\log\left[\left(\frac{g}{g+1}\right)\left(\frac{N_{av}-N_1}{365n(ADTT_{SL})_{PRESENT}}\right)+1\right]}{\log(1+g)}$$

$$= \frac{\log\left[\left(\frac{0.01}{1+0.01}\right)\left(\frac{10426280-6525235}{365*1*600}\right)+1\right]}{\log(1+0.01)} = 16.3\ years$$

Check the following:

$$(ADTT_{SL})_{FUTURE} \le (ADTT_{SL})_{LIMIT}$$

$$(ADTT_{SL})_{FUTURE} = [(ADDT_{SL})_{PRESENT}](1+g)^{Y_{REM}}$$

$$= (600)(1+0.01)^{16.3} = 706 < (ADTT_{SL})_{LIMIT} = 1{,}200\ ok$$

A1A.1.8.3c—Calculation of Fatigue Serviceability Index

Fatigue Serviceability Index $Q = \left(\frac{Y-a}{N}\right)GRI$ 7.2.6.1

No. of load paths (in this case, girders) = 4

$G = 1.0$ Table 7.2.6.1-1
No. of Spans = 1 (Simple Span)

$R = 0.90$ Table 7.2.6.1-2

N = (larger of 100 or Y) = 100
Assuming that the bridge is on an Interstate Highway, $I = 0.9$ Table 7.2.6.1-3

~~$Q = \left(\frac{55-48}{100}\right)(1)(0.9)(0.9) = 0.06$~~ $Q = \left(\frac{64.3-48}{100}\right)(1.0)(0.9)(0.9) = 0.13$

Based on the value of the Fatigue Serviceability Index, the bridge owner will need to define the inspection frequency based upon the importance of the structure. Note, however, that the Fatigue Serviceability Index value could be increased if the owner decided to accept a greater risk of fatigue cracking and use an Evaluation 2 Life estimate instead of the Evaluation 1 Life estimate. This is illustrated below for the same example.

Assume that Evaluation 2 Life is used for the bridge fatigue assessment.

Hence, $R_R = 1.6$ — Table 7.2.5.1-1

$(\Delta f)_{eff} = 3.65$ ksi

$A = 3.9 \times 10^8$ ksi^3 — LRFD Design Table 6.6.1.2.5-1

$n = 1.0$ simple span girders ~~with $L > 40$ ft~~ — LRFD Design Table 6.6.1.2.5-2

$$\xcancel{Y = \frac{\log\left[\frac{(1.6)(3.9\times10^8)}{(365)(1.0)(850)(3.65^3)}(0.02)(1+0.02)^{48-1}+1\right]}{\log(1+0.02)} = 57 \text{ years}}$$

$$N_{av} = \frac{R_R A}{[(\Delta f_{eff})^3]} = \frac{1.6(3.9x10^8)}{(3.65)^3} = 12{,}832{,}344\ cycles$$

$$Y = \frac{\log\left[\left(\frac{0.01}{1+0.01}\right)\left(\frac{12{,}832{,}344 - 6{,}525{,}235}{365(1.0)(600)}\right)+1\right]}{\log(1+0.01)} = 25.2\ years$$

CALCULATION OF FATIGUE SERVICEABILITY INDEX

Fatigue Serviceability Index $Q = \left(\frac{Y-a}{N}\right)GRI$ — 7.2.6.1

No. of load paths (in this case, girders) = 4

$G = 1.0$ — Table 7.2.6.1-1

No. of Spans = 1 (Simple Span)

$R = 0.90$ — Table 7.2.6.1-2

$N =$ (larger of 100 or Y) = 100

$\underline{Y = Y_{REM} = 25.2 + 48 = 73.2}$

Assuming that the bridge is on an Interstate Highway, $I = 0.9$ — Table 7.2.6.1-3

$$\cancel{Q = \left(\frac{63-48}{100}\right)(1)(0.9)(0.9) = 0.12}\ \underline{Q = \left(\frac{73.2-48}{100}\right)(1.0)(0.9)(0.9) = 0.20}$$

Note that the Fatigue Serviceability Index, Q, has increased from ~~0.06~~ 0.13 to ~~0.13~~ 0.20 as a result of accepting a greater risk of fatigue cracking at the critical detail.

A1A.1.9—Legal Load Rating

6A.6.4.2

Appendix A6A

Note: The Inventory Design Load Rating produced rating factors greater than 1.0 (with the exception of fatigue). This indicates that the bridge has adequate load capacity to carry all legal loads within LRFD exclusion limits and need not be subject to legal load ratings.

The load rating computations that follow have been done for illustrative purposes. Shear ratings have not been illustrated.

Live Load: AASHTO Legal Loads—Type 3, 3S2, 3-3 (rate for all three)

g_m = 0.626

IM = 20 percent The standard dynamic load allowance of 33 percent is decreased based on a field evaluation verifying that the approach and bridge riding surfaces have only minor surface deviations or depressions. Table C6A.4.4.3-1

Table E6A-1

The following table compares interpolating to determine M_{LL} without impact for 65 ft span with exact values determined by statics. Note that for the Type 3-3, interpolating M_{LL} results in a value that is 1.5 percent greater than the true value. Judgement should be exercised whether to interpolate tabulated values.

	Type 3	Type 3S2	Type 3-3	
M_{LL} interpolated =	660.7	707.2	654.5	kip-ft
M_{LL} statics	660.77	707.03	644.68	kip-ft
gM_{LL+IM}	496.3	531.2	484.3	kip-ft

Live Load: AASHTO Legal Loads—Specialized Hauling Units and Notional Rating Load—SU4, SU5, SU6, SU7, and NRL

Interpolated values shall be used for the Specialized Hauling Units in this example for illustrative purposes and to familiarize the reader with the Appendix A tables. Table E6A-2

Interpolating to determine M_{LL} without impact for 65 ft span

	SU4	SU5	SU6	SU7	NRL	
M_{LL} interpolated =	744.7	821.2	913.5	994.1	1037.0	kip-ft
gM_{LL+IM}	559.4	616.9	686.2	746.8	779.0	kip-ft

A1A.1.9.1—Strength I Limit State

6A.6.4.2.1

For Types 3, 3S2, and 3-3

Dead Load *DC*: $\gamma_{DC} = 1.25$ Table 6A.4.2.2-1