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

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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 6A.6.10.1.

**6A.6.10.1—End Panels**

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

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

in which:

\[
\alpha = \text{parameter to consider partial tension-field action}
\]

\[
\alpha = \frac{2.8}{D} \left[ \sqrt{M_{tf} + M_{pm}} + \sqrt{M_{pb} + M_{pm}} \right] \tag{6A.6.10.1-2}
\]

\[
V_p = \text{plastic shear force (kip)}
\]

\[
V_p = 0.58 F_{yw} D t_w \tag{6A.6.10.1-3}
\]

where:

\[
C = \text{ratio of the shear-buckling resistance to the shear yield strength determined as specified in LRFD Design Article 6.10.9.3.2}
\]

\[
d_o = \text{transverse stiffener spacing (in.)}
\]

\[
D = \text{web depth (in.)}
\]

\[
F_{yw} = \text{specified minimum yield strength of the web (ksi)}
\]

\[
M_{pb} = \text{plastic moment resistance of the bearing stiffeners (kip-in.)}
\]

\[
M_{pf} = \text{plastic moment resistance of the top flange (kip-in.)}
\]

\[
M_{pm} = \text{minimum value of } M_{pb} \text{ and } M_{pf} (\text{kip-in.})
\]

\[
t_w = \text{web thickness (in.)}
\]

In the calculation of the plastic moment resistances,
$M_{pb}$ 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:

$$d_e = \frac{35t_w (0.8 - C)^2}{35}$$

(6A.6.10.1-4)

Otherwise:

$$d_e = 0$$

(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 \( M_{ab} \) shall not exceed \( 0.84 t_w \sqrt{E/F_{yw}} \).

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.

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.6.11.1

See Article C6A.6.9.7.

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

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

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. 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.

6A.6.12.5—Riveted Connections  
Riveted connections shall be evaluated as bearing-type connections. 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.

6A.6.12.5.1—Rivets in Shear  
The factored resistance of rivets in shear shall be taken as:

$$R_v = \phi_s F_{rv} = \phi_s F_u R_1 R_2 R_3 m A_v$$  \hspace{1cm} (6A.6.12.5.1-1)
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.

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


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, $K_L$, may be determined in accordance with the AASHTO Standard Specifications or taken as follows:

$$ KL = 75 \text{ percent of the total length of a column having riveted end connections} $$

$$ = 87.5 \text{ percent of the total length of a column having pinned end connections} $$

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

If the investigation of shear and stiffener spacing is desirable, such investigation may be based on the AASHTO Standard Specifications. The allowable shear stress, $F_{sv}$, of a stiffened web end panel may alternatively be determined from Eq. 6A.6.10.1-1, with $F_{c}$ replaced by $F_{y}/3$, 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, 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 $W_{18} \times 46$ 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.\(^2\) 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.

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

where:
- \(P\) = Total load, lb
- \(A\) = Cross-sectional area, in.\(^2\)
- \(E\) = 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, \( V_s \), of a stiffened web end panel may alternatively be taken as \( V_s \) 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, \( V_s \), of a stiffened web interior panel may alternatively be taken as \( V_s \) 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 propose 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, \( D/\ell_0 \), shall not exceed 150.” This statement may be interpreted to mean that webs with \( D/\ell_0 > 150 \) 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 \( D/\ell_0 > 150 \), 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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A1—SIMPLE SPAN COMPOSITE STEEL STRINGER BRIDGE

Editor’s Note: Since all of Example A1 was revised by the AASHTO Committee on Bridges and Structures at their 2019 Annual Meeting, the new text in this Section has not been underlined.

PART A—LOAD AND RESISTANCE FACTOR RATING METHOD

A1A.1—Evaluation of an Interior Stringer

Note: When reference is given as “LRFD Design" and "MBE-3," it refers to the 8th edition of the AASHTO LRFD Bridge Design Specifications and 3rd edition of the Manual for Bridge Evaluation respectively.

A1A.1.1—Bridge Data

Span: 65.00 ft
Year Built: 1964
Material: Structural Steel: A36 Steel
\[ F_y = 36 \text{ ksi} \]
Deck Concrete: \[ f'_c = 3.0 \text{ ksi} \]
Structure Condition: No deterioration (NBI Item 59 = 7)
Member is in good condition
Riding Surface: Minor surface deviations (Field verified and documented)
\[ ADTT \] (one direction): 700
\[ ADTT_{SL} \] 200 (ADTT at year 0)
\[ ADTT_{SL, LIMIT} \] 1,200 (roadway limit ADTT)
Average traffic grown rate: 1 percent
Skew: 0°
Additional Information: Diaphragms spaced at 16 ft 3 in.
Overlay Thickness: None
Bridge category: Interstate Ramp Structure
Bridge Geometry: Straight (No Curvature)

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 6.10.1.1.1b). The as-built section properties are used in this analysis as there is no deterioration.

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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.

Shape: $W_{33} \times 130$

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<th>Property</th>
<th>Value</th>
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<tbody>
<tr>
<td>$t_f$</td>
<td>0.855 in.</td>
</tr>
<tr>
<td>$b_f$</td>
<td>11.510 in.</td>
</tr>
<tr>
<td>$t_w$</td>
<td>0.580 in.</td>
</tr>
<tr>
<td>$d$</td>
<td>33.10 in.</td>
</tr>
<tr>
<td>$A$</td>
<td>38.26 in.$^2$</td>
</tr>
<tr>
<td>$I$</td>
<td>6,699 in.$^4$</td>
</tr>
</tbody>
</table>

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.

Bottom Cover Plate: $\frac{5}{8}$ in. $\times$ 10 1/2 in.

$A_{PL} = t_{PL} \times b_{PL} = 6.56$ in.$^2$

$I_{PL} = 0.21$ in.$^4$ $\approx 0$ in.$^4$ (negligible)

Distance to C.G. =

\[
\bar{y} = \frac{\frac{d}{2} + t_{PL}}{A_{PL} + A_{W}} \left( A_{W33\times130} \right) + \left( \frac{t_{PL}}{2} \right) \left( t_{PL} \times b_{PL} \right)
\]

\[
\bar{y} = \frac{(17.175)(38.26) + (0.313)(6.56)}{38.26 + 6.56}
\]

\[
\bar{y} = 14.707 \text{ in. from bottom of section to centroid}
\]

\[
I_x = 6,699 + 38.26(2.468)^2 + 0.21 + 6.56(14.395)^2
\]

\[
I_x = 8,291.6 \text{ in.$^4$}
\]

\[
S_t = \frac{8,291.6}{19.018} = 436.0 \text{ in.$^3$} \quad \text{Section Modulus at top of steel}
\]

\[
S_b = \frac{8,291.6}{14.707} = 563.8 \text{ in.$^3$} \quad \text{Section Modulus at bottom of steel}
\]

A1A.1.2.2—Composite Section Properties

Effective Flange Width, $b_n$, may be taken as the tributary width perpendicular to the axis of the member.

$b_n = 7'–4" = 88.0$ in.

Modular Ratio, $n$

\[
f'_c = 3.00 \text{ ksi}
\]

\[
E_{tuck} = 33,000(w'c)^{1.5} \sqrt{f'_c}
\]

\[
= 33,000(0.145 \text{ kcf})^{1.5} \sqrt{3.00} \text{ ksi}
\]

\[
= 3,155.9 \text{ ksi}
\]

then,

\[
n = \frac{E_g}{E_{tuck}} = \frac{29,000}{3,155.9} = 9.2 \quad \text{(rounded to nearest 1$^{st}$ decimal)}
\]

Typical Interior Stringer:

Short-Term Composite, $n$:

$W_{33} \times 130, PL \frac{5}{8}$ in. $\times$ 10 1/2 in. and Conc. 7 1/4 in. $\times$ 88 in.

Effective Flange Width, $b_n = \frac{88.0}{n} = 9.57$ in.
$$\bar{y} = \frac{(17.175)(38.26) + (0.3125)(6.56) + \left(\frac{88}{9.2} \times 7.25\right)(37.35)}{38.26 + 6.56 + \left(\frac{88}{9.2} \times 7.25\right)}$$

$$\bar{y} = 28.461 \text{ in. from bottom of section to centroid}$$

$$I_x = (6,699) + (38.26)(11.286)^2 + 0.21 + 6.56(28.149)^2 + \left(\frac{88}{9.2}\right)(7.25)^3 + \left(\frac{88}{9.2}\right) \times 7.25 \times (8.889)^2$$

$$I_x = 22,533.7 \text{ in}^4$$

$$S_t = \frac{22,533.7}{5.264} = 4,284.5 \text{ in}^3 \quad \text{Section Modulus at top of steel}$$

$$S_b = \frac{22,533.7}{28.461} = 792.4 \text{ in}^3 \quad \text{Section Modulus at bottom of steel}$$

Long-Term Composite, 3n:

W33 × 130, PL 5/8 in. × 10 1/2 in. and Conc. 7 1/4 in. × 88 in.

Effective Flange Width, \(b_e = \frac{88}{3 \times 9} = 3.26 \text{ in.}\)

$$\bar{y} = \frac{(17.175)(38.26) + (0.3125)(6.56) + \left(\frac{88}{27.6} \times 7.25\right)(37.35)}{38.26 + 6.56 + \left(\frac{88}{27.6} \times 7.25\right)}$$

$$\bar{y} = 22.411 \text{ in. from bottom of section to centroid}$$

$$I_x = (6,699) + (38.26)(5.236)^2 + 0.21 + 6.56(22.099)^2 + \left(\frac{88}{27.6}\right)(7.25)^3 + \left(\frac{88}{27.6}\right) \times 7.25 \times (14.939)^2$$

$$I_x = 16,211.9 \text{ in}^4$$

$$S_t = \frac{16,211.9}{11.314} = 1,432.9 \text{ in}^3 \quad \text{Section Modulus at top of steel}$$

$$S_b = \frac{16,211.9}{22.411} = 723.4 \text{ in}^3 \quad \text{Section Modulus at bottom of steel}$$

**A1A.1.2.3—Summary of Section Properties at Midspan**

**A1A.1.2.3a—Steel Section Only**

\(S_t = 436.0 \text{ in}^3\)
$S_b = 563.8 \text{ in.}^3$

*A1A.1.2.3b—Composite Section—Short Term, $n = 9.2$

$S_t = 4,284.5 \text{ in.}^3$

$S_b = 792.4 \text{ in.}^3$

*A1A.1.2.3c—Composite Section—Long Term, $3n = 27.6$

$S_t = 1,432.9 \text{ in.}^3$

$S_b = 723.4 \text{ in.}^3$

**A1A.1.3—Dead-Load Analysis—Interior Stringer**

*A1A.1.3.1—Components and Attachments, DC*

In general, attachments may include connection plates, stiffeners, diaphragms, bracing, and other miscellaneous components. A refined rating calculation accounts for major weight components; alternatively, a percentage of stringer weight can be used as an estimate. For this example, three interior diaphragms were taken into account and end diaphragms that are directly over the supports were neglected when estimating uniform span loads.

*A1A.1.3.1a—Noncomposite Dead Loads, DC1*

1. Dead load due to Deck
   \[
   = \left(7.333 \text{ ft} \right) \left( \frac{7.25 \text{ in.}}{12} \right) \left(0.150 \text{ kcf} \right)
   \]
   \[
   = 0.665 \text{ kip/ft}
   \]

2. Stringer (self-weight)
   \[
   = (0.130 \text{ kip/ft}) \times 1.06
   \]
   \[
   (\text{six percent increase for connections})
   \]
   \[
   = 0.138 \text{ kip/ft}
   \]

3. Cover Plate \((38 \text{ ft} \times 10.5 \text{ in.} \times 0.625 \text{ in.})\)
   \[
   = \frac{38 \text{ ft} \times 10.5 \text{ in.}}{12} \times \frac{0.625 \text{ in.}}{12} \times 0.49 \text{ kcf}
   \]
   \[
   = 0.8486 \text{ kip}
   \]
   \[
   \text{approx. uniform loading (over 65 ft stringer)}
   \]
   \[
   = (0.8486 \text{ kip}) \times (1.06) / (65 \text{ ft})
   \]
   \[
   = 0.014 \text{ kip/ft}
   \]

4. Diaphragms:
   \[
   = (3)(0.0427 \text{ kip/ft})(7.333 \text{ ft})
   \]
   \[
   = 0.9394 \text{ kip}
   \]
   \[
   \text{approx. uniform loading (over 65 ft stringer)}
   \]
   \[
   = (0.9394 \text{ kip}) \times (1.06)/(65 \text{ ft})
   \]
   \[
   = 0.016 \text{ kip/ft}
   \]

(Note: The Uniform diaphragm load was used for simplicity. Using the uniform load instead of point loads may result in slight unconservative results.)

So, Total dead load \((DC_1) / \text{Stringer}\)

\[
= 0.665 + 0.138 + 0.014 + 0.016
\]

\[
= 0.833 \text{ kip/ft}
\]

Dead Load Moment

\[
M_{DC1} = \frac{0.833 \text{ kip/ft} \times (65 \text{ ft})^2}{8} = 439.9 \text{ kip-ft at midspan}
\]

Dead Load Shear

\[
V_{DC1} = \frac{0.833 \text{ kip/ft} \times (65 \text{ ft})}{2} = 27.1 \text{ kip at bearing}
\]
A1A.1.3.1b—Composite Dead Loads, DC2

All permanent loads on the deck are uniformly distributed among the beams. LRFD Design 4.6.2.2.1

The unit weight of reinforced concrete is generally taken as .005 kcf greater than the unit weight of plain concrete; hence for estimating concrete load 0.150 kcf was assumed. LRFD Design C3.5.1

Barrier Weight

Curb = (1 ft) x (10 in/12) x (0.150 kcf) (2 curbs / 4 beams)
= 0.063 kip/ft

Parapet = [(6 in x 19 in) + (18 in x 12 in)]/144 x (0.150 kcf) (2 parapets / 4 beams)
= 0.172 kip/ft

Railing = Assume 0.020 kip/ft (2 Railings / 4 beams)
= 0.010 kip/ft

So, Total barrier weight/stringer = 0.063 + 0.172 + 0.010
= 0.245 kip/ft

Dead Load Moment = $M_{dc} = \frac{0.245(65)^2}{8}$ = 129.4 kip-ft at midspan

Dead Load Shear = $V_{dc} = \frac{0.245(65)}{2}$ = 8.0 kip at bearing

A1A.1.3.2—Wearing Surface

There is no wearing surface on the bridge.

As a result, $DW = 0.0$

A1A.1.4—Live Load Analysis—Interior Stringer (LRFD Design Table 4.6.2.2.1-1)

A1A.1.4.1—Compute Live Load Distribution Factors (Type (a) cross section) (LRFD Design Table 4.6.2.2.1-1)

Longitudinal Stiffness Parameter, $K_g$

$K_g = n \left( I + A e_g^2 \right)$

in which $n = \frac{E_B}{E_D}$

$E_D = 33,000 \left( w_c \right)^{1.5} \sqrt{f_c^2}$

= 33,000 (0.145)$^{1.5}$ $\sqrt{3}$

= 3,155.9 ksi

$E_B = 29,000$ ksi

Beam + Cover Plate

$I = 8,291.6$ in.$^4$
\( A = 44.82 \text{ in.}^2 \)  

Distance to centroid from top fiber = 19.018

\( e_g = \frac{1}{2} (7.25) + 19.02 = 22.43 \text{ in.} \)

\( K_g = \frac{29,000}{3,155.9} \left( 8,291.6 + 44.82 \times 22.643^2 \right) \)

\( K_g = 287,354.0 \text{ in.}^4 \)

**A1A.1.4.1a—Distribution Factor for Moment, \( g_m \) (LRFD Design Table 4.6.2.2b-1)**

**Range of Applicability Check:**

a. \( S = 7.3333 \text{ ft} \) (meets \( 3.5 \leq S \leq 16 \))

b. \( t_s = 7.25 \text{ in} \) (meets \( 4.5 \leq t_s \leq 12.0 \))

c. \( L = 65.00 \text{ ft} \) (meets \( 20.0 \leq L \leq 240 \))

d. \( N_b = 4 \) (meets \( N_b \geq 4 \))

e. \( K_g = 287,349 \) (meets \( 10000 \leq K_g \leq 7,000.000 \))

Since all the variables fall within the range of applicability given for Cross Section a, simplified LLDF will be established using the expressions given in the Table.

\[
\frac{K_g}{12.0L^3} = \frac{287,354.0}{12.0 \times 65 \times 7.25^3} = 0.967
\]

One Lane Loaded LLDF:

\[
g_{m1} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \frac{S}{L} \left[ \left( \frac{K_g}{12.0L^3} \right)^{0.1} \right]
\]

\[
= 0.06 + \left( \frac{7.3333 \text{ ft}}{14} \right)^{0.4} \left( \frac{7.3333 \text{ ft}}{65 \text{ ft}} \right)^{0.3} (0.967)^{0.1}
\]

\[
= 0.460
\]

Two or More Lanes Loaded LLDF:

\[
g_{m2} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \frac{S}{L} \left[ \left( \frac{K_g}{12.0L^3} \right)^{0.1} \right]
\]

\[
= 0.075 + \left( \frac{7.3333}{9.5} \right)^{0.6} \left( \frac{7.3333}{65} \right)^{0.2} (0.967)^{0.1}
\]

\[
= 0.627 > g_{m1} = 0.460
\]

So, use \( g_m = 0.627 \)
A1A.1.4.1b—Distribution Factor for Shear, \( g_v \) (LRFD Design 4.6.2.2.3a)

One Lane Loaded LLDF:

\[
g_{v1} = 0.36 + \frac{S}{25} \quad \text{LRFD Design Table 4.6.2.2.3a-1}
\]

\[
= 0.36 + \frac{7.3333}{25}
\]

\[
= 0.653
\]

Two or More Lanes Loaded LLDF:

\[
g_{v2} = 0.20 + \left( \frac{S}{12} \right) - \left( \frac{S}{35} \right)^2 \quad \text{LRFD Design Table 4.6.2.2.3a-1}
\]

\[
= 0.20 + \left( \frac{7.3333}{12} \right) - \left( \frac{7.3333}{35} \right)^2
\]

\[
= 0.767 > g_{v1} = 0.653
\]

So, use \( g_v = 0.767 \)

A1A.1.4.2—Compute Maximum Live Load Effects

A1A.1.4.2a—Maximum Design Live Load (HL-93) Moment at Midspan

The maximum moment effects are estimated to occur with the design live load centered on the span. Calculate moments by statics.

Design Lane Load Moment = \( \frac{wL^2}{8} = \frac{0.640(65 \text{ ft})^2}{8} = 338 \text{ kip-ft at midspan} \)

Design Truck Moment with the middle axle located at midspan:

Design Truck Moment \( = \frac{P_{32}L}{4} + \frac{(P_8 + P_{32})xb}{\ell} \)

\[
= \frac{32 \times 65 \text{ ft}}{4} + \frac{(8 + 32)(32.5 \text{ ft} \times 18.5 \text{ ft})}{65 \text{ ft}}
\]

Design Truck Moment = 890 kip-ft (Governs)

Design Tandem Axles Moment with tandem axles located equidistant from midspan:

Tandem Axles Moment \( = P_{25a} = 25 \times 30.5 \text{ ft} = 762.5 \text{ kip-ft} \)

Dynamic allowance factor, \( IM = 33 \% \) LRFD Design Table 3.6.2.1-1

\[
M_{LL+IM} = 338 + 890 \times 1.33
\]

\[
= 1,521.7 \text{ kip-ft}
\]

A1A.1.4.2b—Maximum Design Live Load Shear at Beam Ends

The maximum shear effects occur with the heaviest axle located to create the maximum end reaction. Calculate shears by statics.
Design Lane Load Shear \[ \frac{w\ell}{2} = \frac{0.640 \text{ klf} \times 65 \text{ ft}}{2} = 20.8 \text{kips} \]

Design Truck Shear with the last axle located at support

\[
\text{Design Truck Shear} = P_{32} + P_{32} \left( \frac{\ell - x_{32}}{\ell} \right) + P_{8} \left( \frac{\ell - x_{8}}{\ell} \right)
\]

\[
= 32^k + 32^k \left( \frac{65 \text{ ft} - 14 \text{ ft}}{65 \text{ ft}} \right) + 8^k \left( \frac{65 \text{ ft} - 28 \text{ ft}}{65 \text{ ft}} \right)
\]

Design Truck Shear \(= 61.7 \text{kips} \) (Governs)

Design Tandem Axles shear with one tandem axle located at support

\[
\text{Tandem Axles Shear} = P_{25} + P_{25} \left( \frac{\ell - x_{25}}{\ell} \right) = 25^k + 25^k \left( \frac{65 \text{ ft} - 4 \text{ ft}}{65 \text{ ft}} \right) = 48.5 \text{kips}
\]

Dynamic allowance factor, \( IM = 33 \text{ percent} \)

\[ V_{LL + IM} = 20.8 \text{kips} + 61.7 \text{kips} \times 1.33 \]

\[ = 102.9 \text{kips} \]

_A1A.1.4.2c—Distributed Live Load Moments and Shears_

Design Live-Load HL-93:

\[ M_{LL + IM} = 1,521.7 \times g_m \]

\[ = 1,521.7 \times 0.627 \]

\[ = 954.1 \text{ kip-ft} \]

\[ V_{LL + IM} = 102.9 \times g_v \]

\[ = 102.9 \times 0.767 \]

\[ = 78.9 \text{kips} \]

_A1A.1.5—Compute Nominal Resistance of Section at Midspan (LRFD Design Appendix D6.1)_

Locate Plastic Neutral Axis PNA:

\[ t_f = 0.855 \text{ in.} \]

\[ b_f = 11.510 \text{ in.} \]

\[ t_w = 0.580 \text{ in.} \]

\[ d = 33.10 \text{ in.} \]

\[ A_g = 38.26 \text{ in.}^2 \]

Cov. PL Area \((PL \frac{5}{8} \text{ in.} \times 10\frac{1}{2} \text{ in.}) A_{PL} = t_{PL} \times b_{PL} = 6.56 \text{ in.}^2 \)

Web Depth:

\[ D = 33.10 \text{ in.} - 2 \times (0.855 \text{ in.}) = 31.39 \text{ in.} \]

Treat the bottom flange and the cover plate as one element.
Flange area \( A_{bt} = (11.51)(0.855) + (10.5)(0.625) = 16.404 \text{ in.}^2 \)

\[
\bar{y} = \frac{(11.51)(0.855) + (10.5)(0.625)\left(\frac{0.855 + 0.625}{2}\right)}{(11.51)(0.855) + (10.5)(0.625)}
\]

\( = 0.724 \text{ in. (from top of tension flange to centroid of flange and cover plate)} \)

**Plastic Forces**

LRFD Design Appendix D6.1

Note the forces in longitudinal reinforcement may be conservatively neglected.

Set \( P_{rb} = P_{rt} = 0.0 \)

\[
P_s = 0.85 f'_c b_{eff} t_s
\]

\( = 0.85 \times 3.00 \times 88 \times 7.25 \)

\( = 1,626.9 \text{ kips} \)

\[
c_{rb} = \frac{5.25}{t_s}
\]

where \( c_{rb} \) is the distance from the top of the concrete slab to the center of the bottom layer of the longitudinal concrete deck reinforcement and \( t_s \) is the thickness of the concrete deck. Assume cover + 1/2 bar diameter = 2 in., then \( c_{rb} \) equals 5.25 in.

\[
P_c = F_y A_c \text{ where } A_c = b d_t
\]

\( = 36 \times 11.51 \times 0.855 \)

\( = 354.3 \text{ kips} \)

\[
P_w = F_y D t_w
\]

\( = 36 \times 31.39 \times 0.58 \)

\( = 655.4 \text{ kips} \)

\[
P_t = F_y A_t \text{ where } F_y (b d_t + A_{pl})
\]

\( = 36(11.51 \times 0.855 + 6.56) \)

\( = 590.4 \text{ kips} \)

\[
P_t + P_w + P_c = 590.4 + 655.4 + 354.3 = 1,600.1 \text{ kips} < 1,626.9 \text{ kips}
\]

This means the PNA is within the slab.

\[
\frac{c_{rb}}{t_s} P_s + P_{rb} + P_{rt} = \frac{5.25}{7.25} - 1,626.9 + 0.0 + 0.0 \text{ kips} = 1,178.1 \text{ kips} < 1,600.1 \text{ kips}
\]

This means the PNA is below the bottom layer of deck rebars.

The PNA lies in the slab; only a portion of the slab (depth = \( \bar{y} \)) is required to balance the plastic forces in the steel beam.

\[
\bar{y} = (t_s) \left[ \frac{P_t + P_w + P_c - P_{rb}}{P_s} \right]
\]

\[
\bar{y} = (7.25) \frac{1,600.1}{1,626.9} = 0.98 \text{ in.}
\]
\( Y = 7.13 \text{ in. from the top of the concrete deck slab} \)

\( A1A.1.5.1 - \text{Classify Section (LRFD Design 6.10.7 and Figure C6.4.5-1)} \)

Following the I-Sections in Positive Flexure Flowchart

(Section is considered to be Constant Depth)

\( A1A.1.5.1a - \text{Check Web Slenderness (LRFD Design 6.10.6.2.2)} \)

Since PNA is in the slab, the web slenderness requirement is automatically satisfied.

For composite sections in positive bending, the remaining stability criteria are automatically satisfied. The section is compact.

\( A1A.1.5.1b - \text{Check Ductility Requirement (LRFD Design 6.10.7.1.2)} \)

\[ D_p = Y = 7.13 \text{ in.} \]

\[ D_i = \text{Depth of Composite Section} \]

\[ = d + t_{\text{cover plate}} + t_s = 33.10 + 0.625 + 7.25 \]

\[ = 40.975 \text{ in.} \]

If \( D_p \leq 0.1D_i \), then \( M_n = M_p \)

Otherwise, \( M_n = M_p \left( 1.07 - 0.7\frac{D_p}{D_i} \right) \)

\[ 0.1D_i = 0.1 \times 40.975 = 4.098 \text{ in.} \]

\[ D_p = 7.13 \text{ in.} > 0.1D_i = 4.098 \text{ in.} \text{ therefore } M_n < M_p \]

\( A1A.1.5.2 - \text{Plastic Moment, } M_p \)

Moment arms about the PNA:

Compression Flange:

\( d_c = \left( t_s - Y \right) + \frac{t_c}{2} \)

\[ = (7.25 - 7.13) + \frac{0.855}{2} \]

\[ = 0.55 \text{ in.} \]

\[ d_w = \left( t_s - Y \right) + t_c + \frac{D}{2} \]

\[ = (7.25 - 7.13) + 0.855 + \frac{31.39}{2} \]

\[ = 16.670 \text{ in.} \]

Tension Flange with cover plate:

\[ d_t = \left( t_s - Y \right) + t_c + D + \frac{t_s}{2} \]

\[ = (7.25 - 7.13) + 0.855 + 31.39 + 0.724 \]

\[ = 33.089 \text{ in.} \]

(0.724 in. is the distance to the centroid of the bottom flange and cover plate from the top of the flange)

The plastic moment, \( M_p \), is the sum of the moments of the plastic forces about the PNA.
APPENDIX A: ILLUSTRATIVE EXAMPLES

\[ M_p = \left( \frac{27.13}{2} \right) \left[ P_{\alpha}d_{\alpha} + P_{\beta}d_{\beta} + P_{\gamma}d_{\gamma} + P_{\delta}d_{\delta} \right] \]

LRFD Design Table D6.1-1

\[ = \left( \frac{7.13^2 \times 1626.9}{2 \times 7.25} \right) + \left[ 0 + 0 + 354.3 \times 0.5475 + 655.4 \times 16.670 + 590.4 \times 33.089 \right] \]

\[ = 36,359.1 \text{ kip-in. or } 3,030.0 \text{ kip-ft} \]

Therefore, \( D_p \neq 0.1D_t \)

LRFD Design Eq. 6.10.7.1.2-1

Therefore, \( M_n = M_p \left( 1.07 - 0.7 \frac{D_p}{D_t} \right) \)

LRFD Design Eq. 6.10.7.1.2-2

\[ = 3,030 \times \left( 1.07 - 0.7 \frac{7.13}{40.975} \right) \]

\[ = 2,873.0 \text{ kip-ft} \]

A1A.1.5.3—Nominal Shear Resistance, \( V_n \) (LRFD Design 6.10.9.2)

\( W33 \times 130 \) Rolled section, no stiffeners.

\( D = d - 2t_f \) (Clear distance between flanges)

\[ = 33.1 - 2 \times 0.855 \]

\[ = 31.39 \text{ in.} \]

\( t_w = 0.580 \text{ in.} \)

\( F_{yw} = 36.00 \text{ ksi} \)

Unstiffened web and therefore,

The shear buckling coefficient, \( k = 5.00 \)

LRFD Design 6.10.9.2

\[ \frac{D}{t_w} = 54.10 \]

\[ 1.12 \sqrt{\frac{E_k}{F_{yw}}} = 1.12 \sqrt{\frac{29,000 \times 5.00}{36.0}} = 71.08 \]

LRFD Design Eq. 6.10.9.3.2-4

So, \( \frac{D}{t_w} \leq 1.12 \sqrt{\frac{E_k}{F_{yw}}} \) and therefore \( C = 1.00 \)

Shear Capacity \( V_n = V_r = CV_p \)

LRFD Design Eq. 6.10.9.2-1

\( V_p = 0.58F_{yw}D_{tw} \)

LRFD Design Eq. 6.10.9.2-2

\( V_p = 0.58 \times 36.0 \times 31.39 \times 0.580 \)

\[ = 380.15 \text{ kip} \]
Shear Capacity at the End panel 
\[ CV_r = 1.00 \times 380.15 = 380.15 \text{ kip} \]

### A1A.1.5.4—Demand Summary for Interior Stringer

#### Table A1A.1.5.4-1

<table>
<thead>
<tr>
<th></th>
<th>Dead Load $DC_1$</th>
<th>Dead Load $DC_2$</th>
<th>Live Load Distribution Factor</th>
<th>Dist. Live Load + Impact</th>
<th>Nominal Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment, kip-ft</td>
<td>439.90</td>
<td>129.40</td>
<td>0.627</td>
<td>954.10</td>
<td>2,873.0</td>
</tr>
<tr>
<td>Shear, kips</td>
<td>27.10</td>
<td>8.0</td>
<td>0.767</td>
<td>78.90</td>
<td>380.15</td>
</tr>
</tbody>
</table>

#### A1A.1.6—General Load-Rating Equation

\[
RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)} \quad \text{Eq. 6A.4.1-1}
\]

#### A1A.1.7—Evaluation Factors (for Strength Limit States)

1. Resistance Factor, $\varphi$
   \[ \varphi = 1.00 \text{ for flexure and shear} \]

2. Condition Factor, $\varphi_c$
   \[ \varphi_c = 1.00 \quad \text{Member is in good condition. NBI Item 59 = 7.} \]

3. System Factor, $\varphi_s$
   \[ \varphi_s = 1.00 \quad \text{4-girder bridge, spacing > 4 ft (for flexure and shear).} \]

#### A1A.1.8—Design Load Rating (6A.4.3)

**A1A.1.8.1—Strength I Limit State (6A.6.4.1)**

Capacity $C = (\varphi)(\varphi_c)(\varphi_s)R_n$

\[
RF = \frac{(\varphi_c)(\varphi_s)(\varphi)(\varphi_s)R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL + IM)}
\]

**A1A.1.8.1a—Inventory Level**

<table>
<thead>
<tr>
<th>Load Factors</th>
<th>Table 6A.4.2.2-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{DC}$</td>
<td>1.25</td>
</tr>
<tr>
<td>$\gamma_{DW}$</td>
<td>1.50</td>
</tr>
<tr>
<td>$\gamma_{LL}$</td>
<td>1.75</td>
</tr>
</tbody>
</table>

The dead load demands established for load cases $DC_1$ and $DC_2$ are permanent loads and therefore the load factor for these loads will be taken from the load case $DC$.

Flexure: \[
RF = \frac{1.00(1.0)(1.0)(2,873.0) - (1.25)(439.9+129.4)}{(1.75)(954.10)}
\]

\[ = 1.29754 \]

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Note: The general rule for simple spans carrying moving concentrated loads states: the maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support. In a refined analysis with the HL-93 truck located in such a manner, the resulting rating factor for flexure is $RF = 1.2922$ for this stringer. It should be understood that locating the precise critical section and load position for rating depends on the combined influence of dead load, live load, member capacity, and load factors that make up the general rating factor equation.

Shear: $RF = \frac{(1.0)(1.0)(1.0)(360.15) - (1.25)(27.1 + 8.0)}{(1.75)(78.9)}$

$= 2.435$

**A1A.1.8.1b—Operating Level**

<table>
<thead>
<tr>
<th>Load</th>
<th>Load Factor $\gamma$</th>
<th>Table 6A.4.2.2-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$DC$</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td>$DW$</td>
<td>1.50</td>
<td></td>
</tr>
<tr>
<td>$LL$</td>
<td>1.35</td>
<td></td>
</tr>
</tbody>
</table>

For Strength I Operating Level, only the live-load factor changes; therefore, the rating factor can be calculated by direct proportions.

Flexure: $RF = 1.294 \times \frac{1.75}{1.35}$

$= 1.677$

Shear: $RF = 2.435 \times \frac{1.75}{1.35}$

$= 3.156$

**A1A.1.8.2—Service II Limit State (6A.6.4.1)**

Capacity $C = f_R$

$RF = \frac{f_R - (\gamma_{DC})(f_{DC}) - (\gamma_{DW})(f_{DW}) \pm (\gamma_{P})(f_P)}{(\gamma_{LL})(f_{LL+IM})}$

Eq. 6A.6.4.2.1-1

For this example, the terms:

$(\gamma_{DW})(f_{DW}) \pm (\gamma_{P})(f_P)$

do not contribute and the general equation reduces to:

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

**A1A.1.8.2a—Inventory Level**

Allowable Flange Stress for tension flange $f_R = 0.95R_nF_{sy}$ ($f_t = 0$) LRFD Design Eq. 6.10.4.2.2-2

Checking the tension flange as compression flanges typically do not govern for composite sections.

$R_n = 1.0$ for non-hybrid sections LRFD Design 6.10.1.10.1

$f_R = 0.95 \times 1.0 \times 36$


\[ f_{DC} = f_{DC_1} + f_{DC_2} \]

\[ f_{DC} = \frac{M_{DC1}}{S_b} + \frac{M_{DC2}}{S_{b3n}} \]

\[ = \frac{439.9 \times 12}{563.8} + \frac{129.4 \times 12}{723.4} \]

\[ = 9.363 + 2.147 = 11.510 \text{ ksi} \]

\[ f_{LL+IM} = \frac{M_{LL+IM}}{S_{b3n}} \]

\[ f_{LL + IM} = \frac{954.1 \times 12}{792.4} = 14.449 \text{ ksi} \]

\[ \gamma_{LL} = 1.30 \quad \gamma_{DC} = 1.00 \quad \text{Table 6A.4.2.2-1} \]

\[ RF = \frac{34.2 - (1.0)(11.510)}{(1.3)(14.449)} \]

\[ = 1.208 \]

A1A.1.8.2b—Operating Level

\[ \gamma_{LL} = 1.00 \quad \gamma_{DC} = 1.00 \quad \text{Table 6A.4.2.2-1} \]

For Service II Operating Level, only the live-load factor changes; therefore, the rating factor can be calculated by direct proportions as well.

\[ RF = 1.208 \times \frac{1.30}{1.00} \]

\[ = 1.570 \]

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 thickness = 0.855 in. is greater than 0.8 in.

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

\[ f_{\text{dead-load compression}} = 0.0 \text{ at cover plate at all locations because beam is a simple span and cover plate is located in the tension zone} \]

\[ \therefore \text{ must consider fatigue; determine if the detail possesses infinite life.} \]

Composite section properties without cover plate:
APPENDIX A: ILLUSTRATIVE EXAMPLES

\[
\bar{y} = \frac{(38.26)(16.55) + \left(\frac{88}{9.2} \times 7.25\right)(36.725)}{(38.26) + \left(\frac{88}{9.2} \times 7.25\right)}
\]

\[
= 29.552 \text{ in. from bottom of section to centroid}
\]

\[
I_x = 6,699 + \left(\frac{88}{9.2}\right)^2 + \frac{(7.25)^3}{12} - \frac{88}{9.2}(7.25)(7.173)^2
\]

\[
= 17,038.8 \text{ in.}^4
\]

\[
S_b = \frac{17,038.8}{29.552} = 576.57 \text{ in.}^3 \text{ Section Modulus at bottom of steel}
\]

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.

Influence line ordinates for moment at 13.5 ft from support

\[
M_{LL} = (32 \text{ kips}) (10.696 \text{ ft}) + (32 \text{ kips}) (4.465 \text{ ft}) + (8 \text{ kips}) (1.558 \text{ ft})
\]

\[
= 497.62 \text{ kip-ft} = 5,971.0 \text{ kip-in.}
\]

\[
IM = 15 \text{ percent}
\]

Using influence lines.

\[
M_{LL} + IM = (1.15)(5,971) = 6,866.7 \text{ kip-in.}
\]

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

\[
g_{Fatigue} = \frac{g_{ml}}{1.20}
\]

\[
= \frac{0.460}{1.20}
\]

\[
= 0.383
\]

Distributed Live-Load Moment:

\[
gM_{LL + IM} = (0.383)(6866.7)
\]

\[
= 2,629.9 \text{ kip-in.}
\]

Fatigue Load Stress Range:

\[
\Delta f_{LL + IM} = \frac{2,629.9}{576.57}
\]

\[
= 4.561 \text{ ksi at the cover plate weld}
\]
Nominal fatigue resistance for infinite life:

\[(\Delta F)_{TH} = 2.60 \text{ ksi for Detail Category E'}\]

Infinite-Life Fatigue Check:

\[(ADTT_{\text{present}}) = 700\]

\[\text{Span Length (L)} = 65.00 \text{ ft}\]

\[\text{Number of lanes } (n_L) = 2\]

\[R_p = 0.988 + 6.87 \times 10^{-5} \text{ Span Length} + 4.01 \times 10^{-6} (ADTT)_{\text{present}} + \frac{0.0107}{\text{Number of lanes}}\]

\[= 0.988 + 6.87 \times 10^{-5} \times 65 + 4.01 \times 10^{-6} \times 700 + 0.0107/2\]

\[= 1.00062\]

\[(\Delta f)_{\text{max}} = (R_p)(\Delta f_{\text{FATIGUE-I}}) = (1.00)(1.75)(4.56)\]

\[= 1.00062 \times 1.75 \times 4.561\]

\[= 7.987 \text{ ksi} > 2.6 \text{ ksi}\]

Fatigue Rating Factor for Infinite Life

\[RF_{\text{infinite}} = \frac{(\Delta F)_{TH}}{(\Delta f)_{\text{max}}} = \frac{2.60}{7.987} = 0.326\]

And, \((\Delta f)_{\text{max}} > (\Delta F)_{TH}\)

Therefore, the detail does not possess infinite fatigue life.

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

Fatigue Rating Factor for Finite Life

\[(\Delta f)_{\text{max}} = R_p \times \Delta f_{\text{FATIGUE-II}}\]

\[= R_p \times Y_{\text{LL-fatigue-II}} \times \Delta f_{\text{LL+IM}}\]

\[= 1.00062 \times 0.80 \times 4.561\]

\[= 3.651 \text{ ksi} > 2.6 \text{ ksi}\]

\[RF_{\text{infinite}} = \frac{(\Delta F)_{TH}}{(\Delta f)_{\text{max}}} = \frac{2.60}{3.651} = 0.712\]

A1A.1.8.3b—Calculation of Finite Fatigue Life

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

\[ADTT \text{ (One Direction)} = 700 \text{ (present value)}\]

\[\text{[ADTTSL]}_{\text{present}} = 0.85(700) = 595\]

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

Bridge Age \(a\): (2019–1964) = 55 years

Assume Evaluation 1 Life to be used for bridge assessment.

Hence, \(R_p = 1.30\)

Calculate effective stress range:

\[R_p = 1.00062\]

\[R_{sa} = 1.000\]

\[R_{sa} = 1.000\]

\[R_s = R_{sa} \times R_{st} = 1.000\]
\[ \Delta f_{\text{eff}} = (R_p)(R_f)(\Delta f_{\text{FATIGUE II}}) = (1.0062)(1.000)(0.80)(4.561) = 3.651 \text{ ksi} \]

\[ A = 3.90 \times 10^8 \text{ ksi}^3 \]

\[ n = 1.00 \quad \text{simple span girders} \]

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

\[ N_{av} > N_1 \]

\[ N_{av} = \frac{R_g A}{(\Delta f_{\text{eff}})^n} = \frac{1.3 \times 3.9 \times 10^8}{3.651^n} = 10,417,718 \text{ cycles} \]

\[ N_1 = 365n \left( ADTT_{SL} \right)_{\text{PRESENT}} \left[ 1 - \frac{(ADTT_{SL})_o}{(ADTT_{SL})_{\text{PRESENT}}} + 1 \right] \left[ \left( \frac{(ADTT_{SL})_{\text{PRESENT}}}{(ADTT_{SL})_o} \right)^{1/7} - 1 \right] \]

\[ N_1 = 365(1)(595) \left[ 1 - \frac{200}{595} \right] + 1 = 7,418,583 \text{ cycles} < N_{av}, \text{ OK} \]

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

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

\[ = \log \left[ \frac{0.01}{1 + 0.01} \left( \frac{10,417,718 - N_1}{365 \times 1 + 595} \right) + 1 \right] \log(1 + 0.01) = 12.8 \text{ years} \]

Check the following:

\[ (ADTT_{SL})_{\text{FUTURE}} \leq (ADTT_{SL})_{\text{LIMIT}} \]

\[ (ADTT_{SL})_{\text{FUTURE}} = (ADTT_{SL})_{\text{FUTURE}} \left( 1 + g \right)^{12.8} \]

\[ = (595)(1 + 0.01)^{12.8} \]

\[ = 676 < (ADTT_{SL})_{\text{LIMIT}} = 1,200 \text{ OK} \]

A1A.1.8.3c—Calculation of Fatigue Serviceability Index

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

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

\( G = 1.00 \) \hspace{1cm} \text{Table 7.2.6.1-1}

No. of Spans = 1

\( R = 0.90 \) \hspace{1cm} \text{Table 7.2.6.1-2}
\[
Y = a + Y_{REM} = 54 + 12.8 = 66.8 \text{ years}
\]
\[
N = \text{(larger of 100 or } Y) = 100.0
\]

Since this bridge is on an Interstate Highway,

\[
I = 0.9
\]
\[
Q = \left(\frac{66.8 - 54}{100}\right)(1.0)(0.9)(0.9) = 0.1040
\]

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, \(R_a = 1.60\)

Calculate effective stress range:
\[
(Af)_{eff} = 3.65 \text{ ksi}
\]
\[
A = 3.90 \times 10^8
\]
\[
n = 1.0 \text{ simple span girders}
\]
\[
N_{av} = \frac{R_a A}{(Af)_{eff}^3} = \frac{1.6 \times 3.9 \times 10^8}{(3.65)^3} = 12,821,803 \text{ cycles}
\]
\[
Y = \log \left[ \frac{0.01 + 1.01}{1 + 0.01} \left( \frac{12,821,803 - 7,418,583}{365(1.0)(595)} \right) + 1 \right] = 22.1 \text{ years}
\]

**FATIGUE SERVICEABILITY INDEX**

Fatigue Serviceability Index \[Q = \left(\frac{Y - a}{N}\right)^{GRI}\]

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

\[G = 1.00\]

**No. of Spans = 1**

\[R = 0.90\]

\[N = \text{(larger of 100 or } Y) = 100\]

\[Y = Y_{REM} = 25.2 + 48 = 73.2\]

Assuming that the bridge is on an Interstate Highway, \(I = 0.9\)

\[Q = \left(\frac{76.1 - 54}{100}\right)(1.0)(0.9)(0.9) = 0.179\]

Note that the Fatigue Serviceability Index, \(Q\), has increased from 0.104 to 0.179 as a result of accepting a greater risk of fatigue cracking at the critical detail.

**AIA.1.9—Legal Load Rating**

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 (as stated in LRFD Design Article C3.6.1.2.1) 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.
A1A.1.9a—Live Load: AASHTO Legal Loads—“Routine Commercial Traffic”—Type 3, 3S2, 3-3 (rate for all three)

From previous calculations, \( g_m = 0.627 \)
From previous calculations, \( g_v = 0.767 \)

\( IM = 20 \text{ percent} \) Please note that 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.

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.

Since shear demands for simple spans are not listed in the MBE, the shear demands (without impact) are established using statics and listed below.

Table A1A.1.9-1—AASHTO Routine Legal Load Demands for Interior Stringer

<table>
<thead>
<tr>
<th></th>
<th>Type 3</th>
<th>Type 3S2</th>
<th>Type 3-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{LL} ) interpolated</td>
<td>660.70</td>
<td>707.20</td>
<td>654.40</td>
</tr>
<tr>
<td>( M_{LL} ) statics</td>
<td>660.77</td>
<td>707.03</td>
<td>644.68</td>
</tr>
<tr>
<td>( g_m ) ( M_{LL} + IM )</td>
<td>497.2</td>
<td>532.0</td>
<td>485.1</td>
</tr>
<tr>
<td>( V_{LL} ) statics</td>
<td>44.28</td>
<td>51.38</td>
<td>50.58</td>
</tr>
<tr>
<td>( g_v ) ( M_{LL} + IM )</td>
<td>40.75</td>
<td>47.29</td>
<td>46.55</td>
</tr>
</tbody>
</table>

A1A.1.9b—Live Load: AASHTO Legal Loads—Specialized Hauling Vehicles and Notional Rating Load—SU4, SU5, SU6, SU7, and NRL

Interpolated values are used for the Specialized Hauling Vehicles in this example for illustrative purposes and to familiarize the reader with the Appendix A tables.

The moment demands are established by interpolating demands listed in Table 6A-2; the shear demands are established using statics.

Table A1A.1.9-2—AASHTO Specialized Hauling Vehicles Load Demands for Interior Stringer

<table>
<thead>
<tr>
<th></th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
<th>NRL</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{LL} + IM ) interpolated</td>
<td>744.7</td>
<td>821.2</td>
<td>913.5</td>
<td>994.1</td>
<td>1,037</td>
<td>kip-ft</td>
</tr>
<tr>
<td>( g_m ) ( M_{LL} + IM )</td>
<td>560.3</td>
<td>617.9</td>
<td>687.3</td>
<td>748</td>
<td>780.2</td>
<td>kip-ft</td>
</tr>
<tr>
<td>( V_{LL} + IM ) statics</td>
<td>48.65</td>
<td>54.43</td>
<td>58.31</td>
<td>62.04</td>
<td>63.01</td>
<td>kip</td>
</tr>
<tr>
<td>( g_v ) ( V_{LL} + IM )</td>
<td>44.78</td>
<td>50.10</td>
<td>53.67</td>
<td>57.10</td>
<td>57.99</td>
<td>kip</td>
</tr>
</tbody>
</table>

A1A.1.9.1—Strength I Limit State

For Types 3, 3S2, and 3-3

Dead Load \( DC \): \( \gamma_{DC} = 1.25 \)

\( ADTT \) (One Direction) = 700

Generalized Live-Load Factor for Legal Loads, \( \gamma_{LL} = 1.30 \)
Flexure: 
\[ RF = \frac{(1.0)(1.0)(1.0)(2,873.0) - (1.25)(439.9 + 129.4)}{(1.30)(M_{LL+IM})} \]

Shear: 
\[ RF = \frac{(1.0)(1.0)(380.15) - (1.25)(27.1 + 8.0)}{(1.30)(V_{LL+IM})} \]

Table A1A.1.9.1-1—(Strength I) Rating Factors for AASHTO SHV Vehicles

<table>
<thead>
<tr>
<th></th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
<th>NRL</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF (Flexure)</td>
<td>2.967</td>
<td>2.691</td>
<td>2.419</td>
<td>2.223</td>
<td>2.131</td>
</tr>
<tr>
<td>RF (Shear)</td>
<td>5.777</td>
<td>5.163</td>
<td>4.820</td>
<td>4.530</td>
<td>4.461</td>
</tr>
</tbody>
</table>

6A.6.4.2.2

For Types 3, 3S2, and 3-3, and for Specialized Hauling Units and NRL

Generalized Live-Load Factor for Legal Loads: 
\[ \gamma_{LL} = 1.3 \]
\[ \gamma_{DC} = 1.0 \]

Dead Load DC:

\[ f_r = 34.200 \text{ ksi} \]

\[ f_d = f_{DC_1} + f_{DC_2} = 11.510 \text{ ksi} \]

\[ f_{LL+IM} = \frac{M_{LL+IM} \times 12}{792.4} \]

\[ RF = \frac{34.2 - (1.0)(11.510)}{(1.3)(f_{LL+IM})} \]

Table A1A.1.9.2-1—(Service) Rating Factors for AASHTO Routine Legal Vehicles

<table>
<thead>
<tr>
<th></th>
<th>Type 3</th>
<th>Type 3S2</th>
<th>Type 3-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{LL+IM} )</td>
<td>7.530</td>
<td>8.057</td>
<td>7.346</td>
</tr>
<tr>
<td>RF (Service II)</td>
<td>2.318</td>
<td>2.166</td>
<td>2.376</td>
</tr>
</tbody>
</table>

Table A1A.1.9.2-2—(Service) Rating Factors for AASHTO SHV Legal Vehicles

<table>
<thead>
<tr>
<th></th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
<th>NRL</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{LL+IM} )</td>
<td>8.485</td>
<td>9.357</td>
<td>10.408</td>
<td>11.328</td>
<td>11.815</td>
</tr>
<tr>
<td>RF (Service II)</td>
<td>2.057</td>
<td>1.865</td>
<td>1.677</td>
<td>1.541</td>
<td>1.477</td>
</tr>
</tbody>
</table>

No posting required as \( RF > 1.0 \).

1A.1.9.3—Summary

Safe Load Capacity (tons), \( RT = RF \times W \)

<table>
<thead>
<tr>
<th>Truck</th>
<th>Type 3</th>
<th>Type 3S2</th>
<th>Type 3-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight (tons)</td>
<td>25</td>
<td>36</td>
<td>40</td>
</tr>
<tr>
<td>RF (Service II—Controls)</td>
<td>2.318</td>
<td>2.166</td>
<td>2.376</td>
</tr>
<tr>
<td>Safe Load Capacity (tons)</td>
<td>58</td>
<td>78</td>
<td>95</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Truck</th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
<th>NRL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight (tons)</td>
<td>27</td>
<td>31</td>
<td>34.75</td>
<td>38.75</td>
<td>40</td>
</tr>
<tr>
<td>RF (Service II—Controls)</td>
<td>2.057</td>
<td>1.865</td>
<td>1.677</td>
<td>1.541</td>
<td>1.477</td>
</tr>
<tr>
<td>Safe Load Capacity (tons)</td>
<td>56</td>
<td>58</td>
<td>58</td>
<td>60</td>
<td>59</td>
</tr>
</tbody>
</table>
The NRL rating demonstrates Article C6A.4.4.2.1b: “Bridges that rate for the NRL loading will have adequate load capacity for all legal Formula B truck configurations up to 80 kips.” Example A1 shows this holding true: NRL RF > 1 and all SU RF > 1, while Example A2 shows when NRL RF < 1, RF for the SUs may or may not be >1 and need to be checked on an individual basis.

**A1A.1.10—Permit Load Rating**

Permit Type: Special (Single-Trip, Escorted)
Permit Weight: 220 kips
Permit Vehicle: Shown in Figure A1A.1.10-1

ADTT (one direction): 700

Demand from one percent permit truck without impact from live load analysis by computer program:

Maximum \( M_{LL} = 2,115.0 \text{ kip-ft} \)

Maximum \( V_{LL} = 143.5 \text{ kips} \)

**A1A.1.10.1—Strength II Limit State**

\[ \gamma_{LL} = 1.10 \]

Use one-lane distribution factor and divide out the 1.2 multiple presence factor.

\[ g_{mt-permit} = \frac{g_{ml}}{1.20} = \frac{0.460}{1.20} = 0.383 \]

\[ g_{sl-permit} = \frac{g_{sl}}{1.20} = \frac{0.653}{1.20} = 0.544 \]

\[ IM = 20 \text{ percent (no speed control, minor surface deviations)} \]

Distributed Live-Load Effects:

\[ M_{LL+IM} = 2,115 \times 0.383 \times 1.2 \]

\[ = 972.1 \text{ kip-ft} \]

\[ V_{LL+IM} = (143.5) (0.544) (1.20) \]

\[ = 94.90 \text{ kips} \]

Flexure: \[ RF = \frac{(1.0)(1.0)(1.0)(2,873.0) - (1.25)(439.9 + 129.4)}{(1.10)(972.1)} = 2.021 \]

Shear: \[ RF = \frac{(1.0)(1.0)(1.0)(380.15) - (1.25)(27.1 + 8.0)}{(1.10)(94.9)} = 3.221 \]

**A1A.1.10.2—Service II Limit State (Optional)**

\[ RF = \frac{f_s - (\gamma_{DC})(f_{DC})}{\gamma_L (f_{LL+IM})} \]

\[ IM = 20 \text{ percent (no speed control, minor surface deviations)} \]

Generalized Live-Load Factor: \( \gamma_L = 1.00 \)
Dead Load DC: \( \gamma_D = 1.00 \)
Live-load effects for the Service II permit rating of vehicles that mix with traffic are calculated using the LRFD distribution analysis methods. This check is based on past practice and does not use the one-lane distribution with permit load factors that have been calibrated for the Strength II permit rating. For escorted permits, a one-lane distribution factor can be used as the permit crosses the bridge with no other vehicles allowed on the bridge at the same time.

\[ g_{m-permit} = 0.383 \quad (MPF = 1.2 \text{ has been divided out}) \]

\[ M_{LL+IM} = (2,115)(0.383)(1.2) = 972.1 \text{ kip-ft.} = 11,665 \text{ kip-in.} \]

\[ f_{LL+IM} = \frac{M_{LL+IM}}{S_b} = \frac{11,665}{792.4} = 14.72 \text{ ksi} \]

\[ RF = \frac{34.2 - (1.0)(11.510)}{(1.0)(14.721)} = 1.541 \]

Figure A1A.1.10-1—Permit Truck Loading Configuration
A1A.2—Evaluation of an Exterior Stringer

Note: The same given bridge data as for interior stringers applies.

A1A.2.1—Section Properties

A1A.2.1.1—Noncomposite Section Properties

$W \times 33 \times 130$ and $PL \frac{3}{4} \text{in.} \times 10 \frac{1}{2} \text{in.}$

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 the *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.

$W \times 33 \times 130$ Bottom Cover Plate: $PL \frac{3}{4} \text{in.} \times 10 \frac{1}{2} \text{in.}$

$t_f = 0.855 \text{in.}$

$b_f = 11.510 \text{in.}$

$t_w = 0.580 \text{in.}$

$d = 33.10 \text{in.}$

$A_g = 38.26 \text{in.}^2$

$I_g = 6,699 \text{in.}^4$

$A_{PL} = t_{PL} \times b_{PL} = 7.875 \text{in.}^2$

$I_{PL} = 0.37 \text{in.}^4$

Distance to C.G.

$$\bar{y} = \frac{\left(\frac{d}{2} + t_{PL}\right)A_{W33x130} + \left(\frac{t_{PL}}{2}\right) + (t_{PL} \times b_{PL})}{A_{W33x130} + A_{PL}}$$

$$\bar{y} = \frac{(17.300)(38.26) + (0.375)(7.875)}{38.26 + 7.875}$$

$\bar{y} = 14.411 \text{in.}$ from bottom of section to centroid

$I_x = 6.699 + 38.26(2.889)^2 + 0.37 + 7.875(14.036)^2$

$I_x = 8,570.1 \text{in.}^4$

$S_t = \frac{8,570.1}{19.439} = 440.9 \text{in.}^3$

Section modulus at top of steel

$S_b = \frac{8,570.1}{14.411} = 594.7 \text{in.}^3$

Section modulus at bottom of steel

A1A.2.1.2—Composite Section Properties

Effective Flange Width, $b_c$, may be taken as one-half the distance to the adjacent stringer or girder plus the full overhang width.

Effective Flange Width $b_c = \frac{1}{2}(88 \text{in.}) + 12 \text{in.} = 56.0 \text{in.}$

Modular Ratio, $n$

$$f'c = 3.00 \text{ksi}$$

$$E_{Deck} = 33,000 \left( w_e \right)^{1.5} \sqrt{f'c}$$

$$= 33,000 \left( 0.145 \text{ksi} \right)^{1.5} \sqrt{300 \text{ksi}}$$

$$= 3,155.9 \text{ksi}$$

Then, $n = \frac{E_B}{E_{Deck}} = \frac{29,000}{3,155.9} = 9.2$

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Typical Exterior Stringer:

Short-Term Composite, \( n \):

\[ W^{33} \times 130, \text{PL} \frac{3}{4} \text{in.} \times 10^{1/2} \text{in. and Conc.} \ 7^{1/4} \text{in.} \times 56 \text{in.} \]

Effective Flange Width, \( b_e = \frac{56.0}{n} = 6.09 \text{in.} \)

\[
\bar{y} = \frac{(17.30)(38.26) + (0.375)(7.875) + \left(\frac{56}{9}\right)(7.25)(37.475)}{38.26 + 7.875 + \left(\frac{56}{9.2}\right)(7.25)}
\]

\[
\bar{y} = 25.687 \text{ in. from bottom of section to centroid}
\]

\[
I_x = 6,699 + 38.26(8.387)^2 + 0.37 + (7.875)(25.312)^2
\]

\[
+ \left(\frac{56}{9.2}\right)(7.25)^3 + \left(\frac{56}{9.2}\right)(7.25)(11.788)^2
\]

\[
I_x = 20,761.7 \text{ in.}^4
\]

\[
S_t = \frac{20,761.7}{8.163} = 2,543.4 \text{ in.}^3 \text{ Section Modulus at top of steel}
\]

\[
S_b = \frac{20,761.7}{25.81} = 808.3 \text{ in.}^3 \text{ Section Modulus at bottom of steel}
\]

Long-Term Composite, \( 3n \):

\[ 3n = 3 \times 9 = 27 \]

\[ W^{33} \times 130, \text{PL} \frac{3}{4} \text{in.} \times 10^{1/2} \text{in. and Conc.} \ 7^{1/4} \text{in.} \times 56 \text{in.} \]

Effective Flange Width, \( b_e = \frac{56/3n}{n} = 2.029 \text{ in.} \)

\[
\bar{y} = \frac{(17.30)(38.26) + (0.375)(7.875) + \left(\frac{56}{27.6}\right) \times 7.25)(37.475)}{38.26 + 7.875 + \left(\frac{56}{27.6}\right) \times 7.25}}
\]

\[
\bar{y} = 19.987 \text{ in. from bottom of section to centroid}
\]

\[
I_x = 6,699 + 38.26(2.687)^2 + 0.37 + (7.875)(19.612)^2
\]

\[
+ \left(\frac{56}{27.6}\right)(7.25)^3 + \left(\frac{56}{27.6}\right)(7.25)(17.488)^2
\]

\[
I_x = 14,567.8 \text{ in.}^4
\]

\[
S_t = \frac{14,567.8}{13.863} = 1,050.8 \text{ in.}^3 \text{ Section Modulus at top of steel}
\]

\[
S_b = \frac{14,567.8}{19.987} = 728.9 \text{ in.}^3 \text{ Section Modulus at bottom of steel}
A1A.2.1.3—Summary of Section Properties at Midspan

A1A.2.1.3a—Steel Section Only

\[ S_{\text{TOP}} = 440.9 \text{ in.}^3 \]
\[ S_{\text{BOT}} = 594.7 \text{ in.}^3 \]

A1A.2.1.3b—Composite Section—Short Term, \( n = 9.2 \)

\[ S_{\text{TOP steel}} = 2,599 \text{ in.}^3 \]
\[ S_{\text{BOT}} = 809 \text{ in.}^3 \]

A1A.2.1.3c—Composite Section—Long Term, \( 3n = 27.6 \)

\[ S_{\text{TOP steel}} = 1,050.8 \text{ in.}^3 \]
\[ S_{\text{BOT}} = 728.9 \text{ in.}^3 \]

A1A.2.2—Dead Load Analysis—Exterior Stringer

A1A.2.2.1—Components and Attachments, DC

A1A.2.2.1a—Noncomposite Dead Loads, DC₁

Dead Load Due to Deck: \[
= \left(1 + \frac{7.33}{2}\right)\left(\frac{7.25}{12}\right)(0.150 \text{ kip/ft})
\]
\[ = (4.666 \text{ ft})(7.25 \text{ in.}/12) \times (0.150 \text{ kcf}) \]
\[ = 0.423 \text{ kip/ft} \]

Stringer: (self-weight) \[ = (0.130 \text{ kip/ft}) \times (1.06) \]

(six percent increase for connection)
\[ = 0.138 \text{ kip/ft} \]

Cover Plate (40 ft \times 10.5 in. \times 0.750 in.): \[ = (40 \text{ ft})\times(10.5 \text{ in.}/12)\times(0.75 \text{ in.}/12)\times(0.49 \text{ kcf}) \]
\[ = 1.0719 \text{ kip} \]

Approx. uniform loading (over 65 ft stringer) = 1.0719 kip
\[ = 0.018 \text{ kip/ft} \]

Diaphragms: \[ = (3)\times(0.0427 \text{ kip/ft})\times(3.6667 \text{ ft}) \]
\[ = 0.4697 \text{ kip} \]

approx. uniform loading (over 65 ft stringer) \[ = 0.4697 \text{ kip} \times (1.06)/65 \text{ ft.} \]
\[ = 0.008 \text{ kip/ft} \]

So, total dead load \((DC₁)/\text{stringer} = 0.423 + 0.138 + 0.018 + 0.008\)
\[ = 0.587 \text{ kip/ft} \]

\[ M_{DC₁} = \frac{(0.587)(65)^2}{8} = 310.0 \text{ kip-ft at midspan} \]

\[ V_{DC₁} = (0.587)\left(\frac{65}{2}\right) = 19.1 \text{ kips at bearing} \]

A1A.2.2.1b—Composite Dead Loads, DC₂

From interior girder calcs, barrier weight / stringer = 0.245 kip/ft
Dead Load Moment = \( M_{DC2} = \frac{0.245(65)^2}{8} = 129.4 \text{ kip-ft at midspan} \)

Dead Load Shear = \( V_{DC2} = \frac{0.245(65)}{2} = 8.0 \text{ kips at bearing} \)

**A1A.2.2.2—Wearing Surface**

There is no wearing surface on the bridge.

As a result, \( DW = 0.0 \)

**A1A.2.3—Live Load Analysis—Exterior Stringer**

**A1A.2.3.1—Compute Live Load Distribution Factors (Type (a) cross section)**

**A1A.2.3.1a—Distribution Factor for Moment, \( g_m \) (LRFD Design Table 4.6.2.2.2d-1)**

One Lane Loaded LLDF:

**Lever Rule Approach**

For one lane loaded, the multiple presence factor, \( m = 1.2 \)

For:

\[
S + d_e = 7.3333 \text{ ft} + 0 \text{ ft} < 8.00 \text{ ft} \quad \text{one wheel acting upon the girder}
\]

\[
g_{m1} = m \left( \frac{S + d_e}{2S} \right) = 1.2 \left( \frac{7.3333 + 0 - 2}{2 \times 7.3333} \right) = 0.436
\]

Two or More Lanes Loaded LLDF:

Since the spacings at the exterior girder bay and interior girder bays are the same, \( S \) will be taken as 7.3333 ft when establishing the LLDF for interior girder.

\[
g_{interior} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12.0L(t)} \right)^{0.1}
\]

\[
= 0.075 + \left( \frac{7.3333}{9.5} \right)^{0.6} \left( \frac{7.3333}{L} \right)^{0.2} (0.967)^{0.1}
\]

\[
= 0.627 > g_{ml} = 0.436
\]

\[
g_{m2} = e g_{interior} \quad \text{where} \quad e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{0}{9.1} = 0.77 +
\]

\[
g_{m2} = (0.77) (0.627) = 0.483 > g_{ml} = 0.436 \quad \text{So, use} \ g_m = 0.483
\]

**A1A.2.3.1b—Distribution Factor for Shear, \( g_v \) (LRFD Design Table 4.6.2.2.3b-1)**

One Lane Loaded LLDF:

**Lever Rule Approach**

\[
g_{v1} = g_{m1} = 0.436
\]

Two or More Lanes Loaded LLDF:

\[
g_{interior} = 0.20 + \left( \frac{S}{12} \right)^2 \left( \frac{S}{35} \right)^2
\]

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\[
= 0.20 + \left( \frac{7.3333}{12} \right) - \left( \frac{7.3333}{35} \right)^2
\]

\[
= 0.767
\]

\[
g_{v_2} = e \times g_{\text{interior}} \quad \text{where} \quad e = 0.60 + \frac{d}{10} = 0.60 + \frac{0}{10} = 0.60
\]

\[
= 0.60 \times 0.767
\]

\[
= 0.460 > g_{v_1} = 0.436
\]

So, use \( g_v = 0.460 \)

**A1A.2.3.1c—Special Analysis for Exterior Girders with Diaphragms or Cross-Frames (LRFD Design 4.6.2.2.2d)**

Roadway Layout: two 11-ft wide lanes

\[
R = \frac{N_L}{N_b} + \frac{X_{\text{int}} \sum e}{\sum x^2}
\]

\[
g_{\text{special}} = (m)(R)
\]

One Lane Loaded:

\[
R = \frac{1}{4} + \frac{(11)(6)}{11^2 + 3.67^2 + (-3.67)^2 + (-11)^2} = 0.495
\]

\[
g_{\text{special1}} = 1.2(0.495) = 0.595
\]

Two Lanes Loaded:

\[
R = \frac{2}{4} + \frac{(11)[6+(-5)]}{11^2 + 3.66667^2 + (-3.66667)^2 + (-11)^2} = 0.5409
\]

\[
g_{\text{special2}} = 1.0(0.5409) = 0.541
\]

**A1A.2.3.1d—Summary of Distribution Factors for the Exterior Girders**

**Moment, \( g_m \)**

<table>
<thead>
<tr>
<th>1 Lane</th>
<th>2 or More Lanes</th>
<th>Special Analysis (1 Lane)</th>
<th>Special Analysis (2 Lanes)</th>
<th>Governs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.436</td>
<td>0.482</td>
<td>0.595</td>
<td>0.541</td>
<td></td>
</tr>
</tbody>
</table>

\( g_m = 0.595 \)

**Shear, \( g_v \)**

<table>
<thead>
<tr>
<th>1 Lane</th>
<th>2 or More Lanes</th>
<th>Special Analysis (1 Lane)</th>
<th>Special Analysis (2 Lanes)</th>
<th>Governs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.436</td>
<td>0.460</td>
<td>0.595</td>
<td>0.541</td>
<td></td>
</tr>
</tbody>
</table>

\( g_v = 0.595 \)
A1A.2.3.2—Compute Maximum Live Load Effects for HL-93

Same as for interior girder

Midspan: \( M_{LL + IM} = 1,521.7 \text{ kip-ft} \)

Bearing: \( V_{LL + IM} = 102.9 \text{ kips} \)

A1A.2.3.2a—Distributed Live Load Moments and Shears

Design Live Load HL-93

\[
M_{LL + IM} = 1,521.7 \times g_m = (1,521.7) (0.595) = 905.4 \text{ kip-ft}
\]

\[
V_{LL + IM} = 102.9 \text{ kips} \times g_v = (102.9) (0.595) = 61.2 \text{ kips}
\]

A1A.2.4—Compute Nominal Resistance of Section at Midspan

Locate Plastic Neutral Axis, PNA:

\[
d = 33.10 \text{ in.}
\]

\[
t_f = 0.855 \text{ in.}
\]

\[
t_w = 0.580 \text{ in.}
\]

\[
b_f = 11.510 \text{ in.}
\]

\[
A_g = 38.26 \text{ in.}^2
\]

Cover PL Area (10½ in. × ¾ in.) \( A_{PL} = t_{PL} \times b_{PL} = 7.875 \text{ in.}^2 \)

Web Depth \( D = 33.1 \text{ in.} - 2 \times (0.855 \text{ in.}) = 31.39 \text{ in.} \)

Treat the bottom flange and the cover plate as one element.

Flange area \( A_{f} = (11.51) (0.855) + (10.5) (0.75) = 17.716 \text{ in.}^2 \)

\[
t_f = \frac{(11.51)(0.855)(0.855) + (10.5)(0.75)(0.855 + \frac{0.75}{2})}{(11.51)(0.855) + (10.5)(0.75)} = 0.784 \text{ in.} \text{ (from top of tension flange to centroid of flange and cover plate)}
\]
Plastic Forces

Note the forces in longitudinal reinforcement can be conservatively neglected.

Set $P_{th} = P_{rt} = 0$

$P_s = 0.85f'_c b_{eff}$

$= 0.85 \times (3.00) \times (56) \times (7.25)$

$= 1,035.3$ kips

$t_s = 7.25$ in.

$P_c = F_y A_c = F_y b f$

$= (36) \times (11.51) \times (0.855)$

$= 354.3$ kips

$P_w = F_y A_w = F_y D t_w$

$= (36) \times (31.39) \times (0.58)$

$= 655.4$ kips

$P_t = F_y A_t = F_y (b f + A_{PL})$

$= 36 \times (11.51 \times 0.855 + 7.875)$

$= 637.8$ kips

$P_t + P_w < P_c + P_s + P_{th} + P_{rt} \therefore$ Conditions for Case I are not met

$P_t + P_w + P_c \geq P_s + P_{th} + P_{rt} \therefore$ The PNA lies in the top flange (meets the conditions of Case II)

The PNA lies in the top flange; only a portion of the top flange (depth = $\overline{Y}$) is required to balance the plastic forces in the steel beam.

$$\overline{Y} = \left( \frac{t_c}{2} \right) \left[ \frac{P_w + P_t - P_s}{P_c} \right] + 1$$

$$= 0.855 \left( \frac{655.4 + 637.8 - 1,035.3}{354.3} \right) + 1$$

$$= 0.739$$ in. from the top of the top flange

**A1A.2.4.1—Classify Section (LRFD Design 6.10.7 and Appendix C6 Figure C6.4.5-1)**

Following the I-Sections in Flexure Flowchart (section is considered to be constant depth).

**A1A.2.4.1a—Check Web Slenderness**

Since PNA is in the top flange, the web slenderness requirement is automatically satisfied. For composite sections in positive bending, the remaining stability criteria are automatically satisfied. The section is compact.

**A1A.2.4.1b—Check Ductility Requirement (LRFD Design 6.10.7.1.2)**

$$D_p = t_s + \overline{Y} = 7.25 + 0.739 = 7.989$$ in.

$$D_l = d + t_{coverplate} + t_s = 33.10 + 0.75 + 7.25 = 41.100$$ in.

If $D_p \leq 0.1 D_l$, then $M_n = M_p$

Otherwise, $M_n = M_p \left( 1.07 - 0.7 \frac{D_p}{D_l} \right)$

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0.1D_t = 0.1 \times 41.100 = 4.110\text{ in.}

D_P = 7.989\text{ in.} > 0.1D_t, \text{ therefore } M_n \text{ will be less than } M_p

**A1A.2.4.2—Plastic Moment, } M_p^\prime

Moment arms about the PNA.

**Compression Flange:**

\begin{align*}
d_s &= \left( \frac{t_c}{2} + \bar{y} \right) = \left( \frac{7.25}{2} + 0.739 \right) = 4.364\text{ in.} \\
d_w &= D - \frac{t_c}{2} - \bar{y} = \frac{31.39}{2} + 0.855 - 0.739 = 15.811\text{ in.}
\end{align*}

**Tension Flange:**

\begin{align*}
d_t &= t_c - D + t_t \\
    &= (0.855 - 0.739) + 31.39 + 0.784 \\
    &= 32.290\text{ in.}
\end{align*}

The plastic moment, } M_p^\prime \text{, is the sum of the moments of the plastic forces about the PNA.}

\[
M_p^\prime = \frac{P_c}{2t_c}\left[\left(\frac{t_c - \bar{y}}{2}\right)^2 + (t_c - \bar{y})^2\right] + P_d d_s + P_{ta} d_{ta} + P_{ta} d_{ta} + P_{ta} d_{ta} + P_{ta} d_{ta} + P_{ta} d_{ta}
\]

\[
= \left\{ \frac{354.3}{2(0.855)} \left[ (0.739)^2 + (0.855 - 0.739)^2 \right] \right\} \\
+ \left( 1035.3 \right) \left( 4.364 \right) + 0 + 0 + (655.4)(15.811) + (637.8)(32.29)
\]

\[
= 35,591\text{ kip-in.} = 2,966\text{ kip-ft}
\]

\[
D_P > 0.1D_t
\]

Therefore, } M_n = M_p^\prime \left( 1.07 - 0.7 \frac{D_P}{D_t} \right)

\[
= 2,966.0 \left( 1.07 - 0.7 \times \frac{7.989}{41.10} \right) \\
= 2,770.0 \text{ kip-ft}
\]

**A1A.2.4.3—Nominal Shear Resistance, } V_n \text{ (LRFD Design 6.10.9.2)}

W33 × 130 Rolled section, no stiffeners.

Classification and Resistance same as for interior stringer

\[
V_n = 380.15\text{ kip}
\]
**Table A1A.2.4.4-1**

<table>
<thead>
<tr>
<th>Load</th>
<th>Load Factor, γ</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC</td>
<td>1.25</td>
</tr>
<tr>
<td>LL</td>
<td>1.75</td>
</tr>
</tbody>
</table>

**A1A.2.5—General Load-Rating Equation**

\[
RF = \frac{C - \gamma_{DC}(DC) - \gamma_{DW}(DW)(PM) \pm \gamma_{P}(P)}{(\gamma_{LL})(LL + IM)}
\]

**Eq. 6A.4.2.1-1**

**A1A.2.6—Evaluation Factors (for Strength Limit States)**

1. Resistance Factor, φ
   \[\phi = 1.0\] for flexure and shear

2. Condition Factor, \(\varphi_c\)
   Member is in good condition. NBI Item 59 = 7.
   \[\varphi_c = 1.0\]

3. System Factor, \(\varphi_s\)
   \[\varphi_s = 1.0\] 4-girder bridge, spacing > 4 ft (for flexure and shear).

**A1A.2.7—Design Load Rating (6A.4.3)**

**A1A.2.7.1—Strength I Limit State (6A.6.4.1)**

\[
RF = \frac{(\varphi_c)(\varphi_s)(\varphi)(R_s - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) + (\gamma_{P})(P))}{(\gamma_{LL})(LL + IM)}
\]

**A1A.2.7.1a—Inventory Level**

The dead load demands established for load cases \(DC_1\) and \(DC_2\) are permanent loads and therefore the load factor for these loads will be taken from the load case \(DC\).

**Flexure:**

\[
RF = \frac{(1.0)(1.0)(1.0)(2.770.0) - (1.25)(310.0 + 129.4)}{(1.75)(905.4)}
\]

\[= 1.402\]

**Shear:**

\[
RF = \frac{(1.0)(1.0)(1.0)(380.15) - (1.25)(19.1 + 8.0)}{(1.75)(61.2)}
\]

\[= 3.233\]
A1A.2.7.1b—Operating Level

<table>
<thead>
<tr>
<th>Load</th>
<th>Load Factor, γ</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC</td>
<td>1.25</td>
</tr>
<tr>
<td>LL</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Table 6A.4.2.2-1

For Strength I Operating Level, only the live-load factor changes; therefore, the rating factor can be calculated by direct proportions.

Flexure:

\[
RF = 1.402 \times \frac{1.75}{1.35} = 1.817
\]

Shear:

\[
RF = 3.233 \times \frac{1.75}{1.35} = 4.191
\]

A1A.2.7.2—Service II Limit State (6A.6.4.1)

Capacity \( C = f_R \)

\[
RF = \frac{f_R - (\gamma_{DC})(f_{DC}) - (\gamma_{DW})(f_{DW}) \pm (\gamma_P)(f_P)}{(\gamma_{LL})(f_{LL+IM})}
\]

For this example, the terms:

\((\gamma_{DW})(f_{DW}) \pm (\gamma_P)(f_P) = 0\)

Therefore:

\[
RF = \frac{f_R - (\gamma_{DC})(f_{DC})}{(\gamma_{LL})(f_{LL+IM})}
\]

A1A.2.7.2a—Inventory Level

Allowable Flange Stress for tension flange:

\[
f_R = 0.95 R_h F_{sf} \quad (f_i = 0)
\]

Checking the tension flange as a compression flange typically does not govern for composite sections.

\[
R_h = 1.00 \text{ for non-hybrid sections}
\]

\[
f_R = 0.95 \times 1.00 \times 36 = 34.200 \text{ ksi}
\]

\[
f_{DC} = f_{DC1} + f_{DC2}
\]

\[
f_{DC} = \frac{M_{DC1}}{S_b} + \frac{M_{DC2}}{S_{num}}
\]
\[ f_{DC} = \frac{(310.0)(12)}{594.7} + \frac{(129.4)(12)}{728.9} = 6.255 + 2.130 = 8.385 \text{ ksi} \]

\[ f_{LL+IM} = \frac{M_{LL+IM}}{S_{m}} \]

\[ f_{LL+IM} = \frac{(905.4)(12)}{808.3} = 13.442 \text{ ksi} \]

\[ \gamma_{LL} = 1.30 \quad \gamma_{DC} = 1.00 \]

\[ RF = \frac{34.2 - (1.0)(8.386)}{1.3(13.442)} = 1.477 \]

\[ A1A.2.7.2b—Operating Level \]

\[ \gamma_{LL} = 1.00 \quad \gamma_{DC} = 1.00 \] Table 6A.4.2.2-1

For Service II Operating Level, only the live-load factor changes; therefore, the rating factor can be calculated by direct proportions as well.

\[ RF = 1.477 \times \frac{1.30}{1.00} = 1.920 \]

\[ A1A.2.7.3—Fatigue Limit State \]

The calculations are not done. See the calculations for interior stringers for guidance.

\[ A1A.2.8—Legal Load Rating (6A.6.4.2) \]

Note: The Inventory Design Load Rating produced rating factors greater than 1.0. 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.

\[ A1A.2.8.1—Live Load Demand \]

\[ A1A.2.8.1a—Live Load: AASHTO Legal Loads—Routine Commercial Traffic—Types 3, 3S2, and 3-3 \]

From previous calculations, \( g_m = 0.595 \)

From previous calculations, \( g_v = 0.595 \)

\( IM = 20 \text{ percent} \) Table C6A.4.4.3-1

(Please note that 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.)

Since shear demand for simple spans is not listed in the MBE, both the moment and shear demand (without impact) are established using statics and listed below. Hand Calculations (not shown)
Table A1A.2.8.1a—AASHTO Routine Legal Load Demands for Exterior Stringer

<table>
<thead>
<tr>
<th>Truck</th>
<th>Type 3</th>
<th>Type 3S2</th>
<th>Type 3-3</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{LL}$ (interpolated)</td>
<td>660.77</td>
<td>707.03</td>
<td>644.68</td>
<td>kip-ft</td>
</tr>
<tr>
<td>$g_{M}M_{LL+IM}$</td>
<td>471.8</td>
<td>504.8</td>
<td>460.3</td>
<td>kip-ft</td>
</tr>
<tr>
<td>$V_{LL}$ (statics)</td>
<td>44.28</td>
<td>51.38</td>
<td>50.58</td>
<td>kip</td>
</tr>
<tr>
<td>$g_{V}V_{LL+IM}$</td>
<td>31.62</td>
<td>36.68</td>
<td>36.12</td>
<td>kip</td>
</tr>
</tbody>
</table>

A1A.2.8.1b—Live Load: AASHTO Legal Loads—Specialized Hauling Vehicles (SHVs) and Notional Rating Load—SU4, SU5, SU6, SU7, and NRL

Interpolated values are used for the Specialized Hauling Vehicles in this example for illustrative purposes and to familiarize the reader with the Appendix A tables.

Table E6A-2 (with 33 percent $IM$)

The moment demands are established by interpolating demands listed in Table E6A-2 and the shear demands are established using statics.

Table A1A.2.8.1b—AASHTO Specialized Hauling Vehicles Load Demands for Exterior Stringer

<table>
<thead>
<tr>
<th>Truck</th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
<th>NRL</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{LL+IM}$ (interpolated)</td>
<td>744.7</td>
<td>821.2</td>
<td>913.5</td>
<td>994.1</td>
<td>1,037.0</td>
<td>kip-ft</td>
</tr>
<tr>
<td>$g_{M}M_{LL+IM}$</td>
<td>531.7</td>
<td>586.3</td>
<td>652.2</td>
<td>709.8</td>
<td>740.4</td>
<td>kip-ft</td>
</tr>
<tr>
<td>$V_{LL+IM}$ (statics)</td>
<td>48.65</td>
<td>54.43</td>
<td>58.31</td>
<td>62.04</td>
<td>63.01</td>
<td>kip</td>
</tr>
<tr>
<td>$g_{V}M_{LL+IM}$</td>
<td>34.74</td>
<td>38.86</td>
<td>41.63</td>
<td>44.30</td>
<td>44.99</td>
<td>kip</td>
</tr>
</tbody>
</table>

A1A.2.8.2—Strength I Limit State

For Types 3, 3S2, and 3-3

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

$ADTT$ (one direction) = 700

Generalized Live-Load Factor for Legal Loads:

$\gamma_{LL} = 1.30$ Table 6A.4.4.2.3b-1

Flexure:

$$RF = \frac{(1.0)(1.0)(1.0)(2,770.0)\quad -(1.25)(310.0+129.4)}{(1.30)(M_{LL+IM})}$$

Shear:

$$RF = \frac{(1.0)(1.0)(1.0)(380.15)\quad -(1.25)(19.1+8.0)}{(1.30)(V_{LL+IM})}$$

Table A1A.2.8.2—(Strength I) Rating Factors for AASHTO SHV Vehicles

<table>
<thead>
<tr>
<th>Truck</th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
<th>NRL</th>
</tr>
</thead>
<tbody>
<tr>
<td>$RF$ (Flexure)</td>
<td>3.213</td>
<td>2.914</td>
<td>2.619</td>
<td>2.407</td>
<td>2.307</td>
</tr>
<tr>
<td>$RF$ (Shear)</td>
<td>7.667</td>
<td>6.854</td>
<td>6.398</td>
<td>6.013</td>
<td>5.921</td>
</tr>
</tbody>
</table>

A1A.2.8.3—Service II Limit State

For Types 3, 3S2, and 3-3, and for Specialized Hauling Units and NRL

Dead Load $DC$: $\gamma_{DC} = 1.00$ Table 6A.4.2.2-1

Generalized Live-Load Factor for Legal Loads, $\gamma_{LL} = 1.30$ Table 6A.4.2.2-1

$f_{R} = 34,200$ ksi

$f_{DC} = f_{DC1} + f_{DC2} = 8.385$ ksi

See Calcs A1A.2.7.2a
\[ f_{LL+IM} = \frac{M_{LL+IM} \times 12}{808.3} \]
\[ RF = \frac{34.2 - (1.0)(8.385)}{(1.3)(f_{LL+IM})} \]

Table A1A.2.8.3-1—(Service) Rating Factors for AASHTO Routine Legal Vehicles

<table>
<thead>
<tr>
<th>Truck</th>
<th>Type 3</th>
<th>Type 3S2</th>
<th>Type 3-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{LL+IM} )</td>
<td>7.004</td>
<td>7.494</td>
<td>6.834 ksi</td>
</tr>
<tr>
<td>RF (Service II)</td>
<td>2.835</td>
<td>2.650</td>
<td>2.906</td>
</tr>
</tbody>
</table>

Table A1A.2.8.3-2—(Service) Rating Factors for AASHTO SHV Legal Vehicles

<table>
<thead>
<tr>
<th>Truck</th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
<th>NRL</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{LL+IM} )</td>
<td>7.894</td>
<td>8.704</td>
<td>9.683</td>
<td>10.538</td>
<td>10.992 ksi</td>
</tr>
<tr>
<td>RF (Service II)</td>
<td>2.516</td>
<td>2.281</td>
<td>2.051</td>
<td>1.884</td>
<td>1.807</td>
</tr>
</tbody>
</table>

No posting required as \( RF > 1.0 \) 6A.8.3

A1A.2.8.4—Summary (6A.4.4.4)

Safe Load Capacity (tons), \( RT = RF \times W \)  Eq. 6A.4.4.4-1

Table A1A.2.8.4-1—Safe Load Capacity for AASHTO Routine Legal Vehicles

<table>
<thead>
<tr>
<th>Truck</th>
<th>Type 3</th>
<th>Type 3S2</th>
<th>Type 3-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight (tons)</td>
<td>25</td>
<td>36</td>
<td>40</td>
</tr>
<tr>
<td>RF (Service II—Controls)</td>
<td>2.835</td>
<td>2.650</td>
<td>2.906</td>
</tr>
<tr>
<td>Safe Load Capacity (tons)</td>
<td>71</td>
<td>95</td>
<td>116</td>
</tr>
</tbody>
</table>

Table A1A.2.8.4-2—Safe Load Capacity for AASHTO SHV Legal Vehicles

<table>
<thead>
<tr>
<th>Truck</th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
<th>NRL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight (tons)</td>
<td>27</td>
<td>31</td>
<td>34.75</td>
<td>38.75</td>
<td>40</td>
</tr>
<tr>
<td>RF (Service II Controlling)</td>
<td>2.516</td>
<td>2.281</td>
<td>2.051</td>
<td>1.884</td>
<td>1.806</td>
</tr>
<tr>
<td>Safe Load Capacity (tons)</td>
<td>68</td>
<td>71</td>
<td>71</td>
<td>73</td>
<td>72</td>
</tr>
</tbody>
</table>

The NRL rating demonstrates Article C6A.4.4.2.1b: “Bridges that rate for the NRL loading will have adequate load capacity for all legal Formula B truck configurations up to 80 kips.” Example A1 shows this holding true NRL \( RF > 1 \) and all SHV \( RF > 1 \), while Example A2 shows when NRL \( RF < 1 \), RF for the SUs may or may not be > 1 and need to be checked on an individual basis.

A1A.2.9—Permit Load Rating (6A.6.4.2)

Permit Type: Special (Single Trip, Escorted)
Permit Weight: 220 kips
Permit Vehicle: Shown in Figure A1A.1.10-1.

\( ADTT \) (One Direction): 700

Demand from one Permit Truck without impact from Live-Load Analysis by Computer Program:
Maximum \( M_{LL} = 2,115.0 \) kip-ft
Maximum \( V_{LL} = 145.3 \) kips
### A1A.2.9.1—Strength II Limit State

\[ \gamma_{LL} = 1.10 \]  

Use the one-lane distribution factor and divide out the 1.2 multiple presence factor.  

\[ g_{m-\text{Permit}} = \frac{g_{ml}}{1.20} = \frac{0.595}{1.20} = 0.496 \]  

\[ g_{v-\text{Permit}} = \frac{g_{vl}}{1.20} = \frac{0.595}{1.20} = 0.496 \]  

\[ IM = 20 \text{ percent (no speed control, minor surface deviations)} \]

**Distributed Live-Load Effects:**

\[ M_{LL+IM} = (2,115.0)(0.496)(1.2) \]  

\[ = 1,258.8 \text{ kip-ft} \]

\[ V_{LL+IM} = (145.3)(0.496)(1.2) \]  

\[ = 86.50 \text{ kips} \]

**Flexure:**

\[ RF = \frac{(1.0)(1.0)(1.0)(2,770.0) - (1.25)(310.0 + 129.4)}{(1.10)(1,258.8)} \]  

\[ = 1.604 \]

**Shear:**

\[ RF = \frac{(1.0)(1.0)(1.0)(380.15) - (1.25)(19.1 + 8.0)}{(1.10)(86.50)} \]  

\[ = 3.639 \]

### A1A.2.9.2—Service II Limit State (Optional)

**Dead Load DC:**

\[ \gamma_{DC} = 1.00 \]  

**Generalized Live-Load Factor, \( \gamma_{LL} \):**

\[ IM = 20 \text{ percent (no speed control, minor deviations)} \]  

\[ f_{R} = 34.200 \text{ ksi} \]  

\[ f_{D} = 8.385 \text{ ksi} \]

Live-load effects for the Service II permit rating of an escorted permit are calculated using the same one-lane-loaded procedures as for the Strength II rating.  

\[ g_{m-\text{Permit}} = 0.496 \text{ (MPF 1.2 has already been divided out)} \]

\[ M_{LL+IM} = (2,115.0)(0.496)(1.2) = 1,258.8 \text{ kip-ft} \]  

\[ = 15,106 \text{ kip-in.} \]

\[ f_{LL+IM} = \frac{M_{LL+IM}}{S_b} = \frac{15,106}{808.3} = 18.688 \text{ ksi} \]

\[ RF = \frac{34.2(1.0)(8.385)}{1.0(18.688)} = 1.381 \]
### Table A1A.3-1—Summary of Rating Factors for Load and Resistance Factor Rating Method—Interior Stringer

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Design Load Rating</th>
<th>Legal Load Rating</th>
<th>Permit Load Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inventory</td>
<td>Operating</td>
<td>Type 3</td>
</tr>
<tr>
<td>Strength I</td>
<td>Flexure</td>
<td>1.294</td>
<td>1.677</td>
</tr>
<tr>
<td>Strength II</td>
<td>Flexure</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Service II</td>
<td>Strength I</td>
<td>1.208</td>
<td>1.570</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Fatigue</td>
<td>Strength I</td>
<td>0.326</td>
<td>—</td>
</tr>
</tbody>
</table>

NP—Calculations are not performed

### Table A1A.3-2—Summary of Rating Factors for Load and Resistance Factor Rating Method—Exterior Stringer

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Design Load Rating</th>
<th>Legal Load Rating</th>
<th>Permit Load Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inventory</td>
<td>Operating</td>
<td>Type 3</td>
</tr>
<tr>
<td>Strength II</td>
<td>Flexure</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Service II</td>
<td>Strength I</td>
<td>1.477</td>
<td>1.920</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Fatigue</td>
<td>Strength I</td>
<td>NP</td>
<td>—</td>
</tr>
</tbody>
</table>

NP—Calculations are not performed
This page intentionally left blank.
PART B—ALLOWABLE STRESS AND LOAD FACTOR RATING METHODS

A1B.1—EVALUATION OF AN INTERIOR STRINGER


A1B.1.1—Bridge Data

Refer to Article A1A.1.1, Simple Span Composite Steel Stringer Bridge Data.

A1B.1.2—Section Properties

In unshored construction, the 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 $3n$ (AASHTO 10.38.1). The as-built section properties are used in this analysis.

A1B.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 the 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.

Shape—W33×130

Bottom Cover Plate $5/8$ in. × $10 1/2$ in.

$t_f = 0.855$ in.

$b_f = 11.510$ in.

$t_w = 0.580$ in.

$d = 33.10$ in.

$A_g = 38.26$ in.$^2$

$I_g = 6,699$ in.$^4$

$t_{PL} = 0.625$ in.

$b_{PL} = 10.500$ in.

$A_{PL} = t_{PL} \times b_{PL} = 6.56$ in.$^2$

$I_{PL} = 0.21$ in.$^4$

Figure A1B.1.2.1-1—Cross Section—Interior Stringer, Noncomposite
Distance to C.G. = $\bar{y} = \frac{d \left( \frac{d}{2} + t_{PL} \right) A_{W33×130} + (t_{PL}^2 + t_{PL} \times b_{PL})}{A_{W33×130} + A_{PL}}$

$\bar{y} = \frac{(17.175)(38.26) + (0.313)(6.56)}{38.26 + 6.56}$

$\bar{y} = 14.707$ in. from bottom of section to centroid

$I_x = 6,699 + 38.26(2.468)^2 + 0.21 + 6.56(14.395)^2$

$I_x = 8,291.5$ in.$^2$

$S_t = \frac{8,291.5}{19.018} = 436.0$ in.$^3$ Section Modulus at top of steel

$S_b = \frac{8,291.5}{14.707} = 563.8$ in.$^3$ Section Modulus at bottom of steel

**A1B 1.2.2—Composite Section Properties**

Effective Flange Width, $b_e$

Smaller of $\frac{1}{4}d(65)(12) = 195$ in.

$= (7.3333)(12) = 88.0$ in.

$= (7.250)(12) = 87.0$ in. (Controls)

$b_e = 87.0$ in.

Modular Ratio $n$

$f_c' = 3.00$ ksi

for $f_c' = 3,000$ psi $- n = 9.0$

**Typical Interior Stringer:**

**Short-Term Composite, (n):**

W33×130, PL $\frac{5}{8}$ in. × 10 $\frac{1}{2}$ in. and Conc. 7 $\frac{1}{4}$ in. × 87 in.

Effective Flange Width, $b_e = 87/n = 9.67$ in
Figure A1B.1.2.2-1—Cross Section—Interior Stringer, Composite $n = n$

\[
\bar{y} = \frac{(17.175)(38.26) + (0.313)(6.56) + \left(\frac{87}{9} \times 7.25\right)(37.35)}{38.26 + 6.56 + \left(\frac{87}{9} \times 7.25\right)}
\]

\[
\bar{y} = 28.518 \text{ in. from bottom of section to centroid}
\]

\[
I_x = 6.699 + (38.26)(11.343)^2 + 0.21 + 6.56(28.2055)^2 + \frac{\left(\frac{87}{9} \times 7.25\right)^3}{12} + \frac{\left(\frac{87}{9} \times 7.25\right)(8.832)^2}{12}
\]

\[
I_x = 22,614.5 \text{ in.}^4
\]

Note: $I_x$ for the bottom cover plate is negligible, however, its $A d^2$ term makes a significant contribution.

\[
S_{tn} = \frac{22,614.5}{5.207} = 4,343.1 \text{ in.}^3 \text{ Section modulus at top of steel}
\]

\[
S_{bn} = \frac{22,614.5}{28.518} = 793.0 \text{ in.}^3 \text{ Section modulus at bottom of steel}
\]

Use with Live Load.

**Long-Term Composite, 3n:**

$W \times 33 \times 130, PL \times \frac{5}{8} \text{ in.} \times 10^{1/2} \text{ in. and Conc. } 7^{1/4} \text{ in.} \times 87 \text{ in.}$

Effective Flange Width, $b_e = \frac{87}{3n} = 3.22 \text{ in.}$
Figure A1B.1.2.2-2—Cross Section—Interior Stringer, Composite $n = 3n$

$$
\bar{y} = \frac{(17.175)(38.26) + (0.313)(6.56) + \left(\frac{87}{27}\times 7.25\right)(37.35)}{38.26 + 6.56 + \left(\frac{87}{27}\times 7.25\right)}
$$

$$
\bar{y} = 22.465 \text{ in. from bottom of section to centroid}
$$

$$
I_x = 6,699 + (38.26)(5.290)^2 + 0.21(6.56)(22.1525)^2 + \frac{\left(\frac{87}{27}\times 7.25\right)(7.25)^3}{12} + \left(\frac{87}{27}\times 7.25\right)(14.885)^2
$$

$$
I_x = 16,267.4 \text{ in.}^4
$$

$$
S_{x3n} = \frac{16,267.4}{11.260} = 1,444.7 \text{ in.}^3 \text{ Section Modulus at top of steel}
$$

$$
S_b = \frac{16,267.4}{22.465} = 724.1 \text{ in.}^3 \text{ Section Modulus at bottom of steel}
$$

Use with Superimposed Dead Load (SDL).

A1B.1.3—Dead Load Analysis—Interior Stringer

A1B.1.3.1—Dead Loads (Includes an Allowance of Six Percent of Steel Weight for Connections)

1. Dead load due to Deck
   
   $$(7.3333 \text{ ft}) (7.25 \text{ in/12}) \times (150 \text{ pcf}) = 664.6 \text{ lbs/ft}$$

2. Stringer (self-weight)
   
   $$(130 \text{ lbs/ft}) \times (1.06) = 137.8 \text{ lbs/ft}$$
   (six percent increase for connection)

3. Cover Plate (38 ft x 10.5 in x 0.625 in)
   
   $$(38 \text{ ft} \times (10.5 \text{ in/12}) \times (0.625 \text{ in/12}) \times (490 \text{ pcf}) = 848.6 \text{ lbs/ft}$$
   
   approx. uniform loading (over 65 ft stringer) $= (848.6 \text{ lbs/ft}) \times (1.06)/(65 \text{ ft})$
4. Diaphragms

\[ \text{approx. uniform loading (over 65 ft stringer)} = (939.4 \text{ lbs}) \times (1.06)/(65 \text{ ft}) \]

\[ = 15.4 \text{ lbs/ft} \]

So, Total dead load \( (DC_1) / \text{Stringer} \)

\[ = 664.6 + 137.8 + 13.9 + 15.4 \]

\[ = 831.7 \text{ lbs/ft} \]

**A1B.1.3.2—Superimposed Dead Loads (AASHTO 3.23.2.3.1.1)**

1. Barrier Weight

Curb

\[ = (1 \text{ ft}) \times (10 \text{ in./12}) \times (150 \text{ pcf}) \]

\[ = 62.5 \text{ lbs/ft} \]

Parapet

\[ = \left[ (6 \text{ in.} \times 19 \text{ in.}) + (18 \text{ in.} \times 12 \text{ in.}) \right]/144 \times (150 \text{ pcf}) \]

\[ = 171.9 \text{ lbs/ft} \]

Railing

\[ = \text{Assume 20 lbs/ft} \]

\[ = 10.0 \text{ lbs/ft} \]

2. Wearing Surface

\[ = 0.0 \text{ lbs/ft} \]

So, total barrier weight / stringer

\[ = 62.5 + 171.9 + 10.0 + 0.0 \]

\[ = 244.4 \text{ lbs/ft} \]

**A1B.1.4—Live Load Analysis—Interior Stringer**

Moments:

\[ w_{\text{SDL}} = 0.2444 \text{ kip/ft.} \]

\[ w_{\text{DL}} = 0.8317 \text{ kip/ft.} \]

**Figure A1B.1.4-1—Load Diagram—Interior Stringer, Dead Load, and Superimposed Dead Load**

Dead Load Moment, \( M_{DL} = \frac{0.8317(65)^2}{8} = 439.2 \text{ kip-ft at midspan} \)

Dead Load Moment, \( M_{SDL} = \frac{0.2444(65)^2}{8} = 129.1 \text{ kip-ft at midspan} \)

\[ M_{LL} = \frac{403.3 + 492.8}{2} \]

\[ \Leftarrow 65 \text{ ft} \]

\[ 70 \]

\[ 492.8 \]

\[ M_{LL} = 448 \text{ kip-ft} \]

(without impact and without LL Distribution)
Note the moments given in the MBE are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MBE value.

Maximum $M_{LL}$ without impact for 65 ft span, with exact values determined by statics, is 448.02 kip-ft. Nevertheless, judgment should be exercised whether to interpolate tabulated values. The general rule for simple spans carrying moving concentrated loads states that the maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support. It should be understood that locating the precise critical section and load position for rating depends on the combined influence of dead load, live load, and member capacity that make up the general Rating Factor equation.

A1B.1.5—Allowable Stress Rating (6B.3.1, 6B.4.2, and 6B.5.2)

Consider Maximum Moment Section only for this example.

A1B.1.5.1—Impact (Use Standard AASHTO) (6B.6.4, AASHTO 3.8.2.1)

\[
I = \frac{50}{L + 125} \leq 0.3
\]

\[
I = \frac{50}{65 + 125} = 0.26
\]

A1B.1.5.2—Distribution (Use Standard AASHTO) (6B.6.3, AASHTO 3.23.2.2, and Table 3.23.1)

Since this bridge carries two or more lanes,

\[
DF = \frac{S_s}{5.5} = \frac{7.33333}{5.5} = 1.3333
\]

\[
M_{LL+I} = M_{LL}(1 + I) \times DF = 448.1(1 + 0.263)(1.3333)
\]

\[
M_{LL+I} = 754.6 \text{ kip-ft}
\]

A1B.1.5.3—Inventory Level (Bottom Tension Controls) (6B.5.2.1, Table 6B.5.2.1-1)

For steel with $F_y = 36 \text{ ksi} \rightarrow f_I = 0.55 f_y$

Thus:

\[
f_I = 0.55(36) = 19.8 \text{ ksi}
\]

The Resisting Capacity ($M_{RI}$) = $f_I S_{wb} = f_I S_w$

\[
M_{RI} = 19.8 \text{ ksi} \times 793.0 \text{ in.}^3
\]

\[
= 15,701.4 \text{ kip-in}
\]

\[
= 1,308.5 \text{ kip-ft}
\]

Then:

\[
RF_I = \frac{M_{RI} - M_{DL} \frac{S_w^{L}}{S_{s}^{DL}} - M_{SDL} \frac{S_{wb}^{L}}{S_{s}^{SDL}}}{M_{LL+I}}
\]
\[ RF_I = \frac{M_{RI} - M_{DL} \frac{S^L_{bn}}{S^L_b} - M_{SDL} \frac{S_{bn}}{S_{bn}}}{M_{LL+1}} \]

\[ = \frac{1,308.5 - 439.2}{563.8} - \frac{129.1}{724.1} = 0.728 \]

Alternatively, in terms of stress:

\[ RF_I = \frac{f_I \frac{M_{DL}}{S_{DL}^L} - M_{SDL} \frac{S_{DL}}{S_{DL}^L}}{M_{LL+1}} \]

\[ = \frac{19.8 \text{ ksi} \times 439.2 \text{ kip-ft} \times 12 \text{ in./ft}}{563.8 \text{ in.}^3} - \frac{129.1 \text{ kip-ft} \times 12 \text{ in./ft}}{724.1 \text{ in.}^3} = 0.728 \] as above

A1B.1.5.4—Operating Level (6B.5.2.1, Table 6B.5.2.1-2)

For steel with \( f_y = 36 \text{ ksi} \rightarrow f_O = 0.75 f_y \)

Thus:

\[ f_O = 0.75(36) = 27.00 \text{ ksi} \]

And

\[ \text{The Resisting Capacity (} M_{RO} \text{)} = f_O S^L \]

\[ M_{RO} = 27.00 \text{ ksi} \times (793.0 \text{ in.}^3) \]

\[ = 21,411 \text{ kip-in.} \]

\[ = 1,784.3 \text{ kip-ft} \]

Then

\[ RF_0 = \frac{1,784.3 - 439.2 \times 793.0}{563.8} - \frac{129.1 \times 793.0}{724.1} \]

\[ = 0.728 \] or \( 0.728 \times 36 \text{ tons} = 26.2 \text{ tons} \)
$RF_0 = \frac{1,025.17}{754.6}$

$= 1.359$ or $1.359 \times 36$ tons $= 48.9$ tons

**A1B.1.5.5—Summary of Ratings for Allowable Stress Rating Method**

<table>
<thead>
<tr>
<th></th>
<th>$RF$</th>
<th>Tons</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inventory</td>
<td>0.728</td>
<td>26.2</td>
</tr>
<tr>
<td>Operating</td>
<td>1.359</td>
<td>48.9</td>
</tr>
</tbody>
</table>

**A1B.1.6—Load Factor Rating (6B.3.2, 6B.4.3, and 6B.5.3)**

Consider maximum moment section only for this example. See general notes.

**A1B.1.6.1—Impact (Use Standard AASHTO) (6B.6.4)**

From Allowable Stress Rating, $I = 0.26$  
See Calcs A1B.1.5.1

**A1B.1.6.2—Distribution (Use Standard AASHTO) (6B.6.3)**

From Allowable Stress Rating $DF = 1.3333$  
See Calcs A1B.1.5.2

$M_{LL+I} = M_{LL} (1+I) \times DF$

$= 448.1 \times (1 + 0.263)(1.3333)$

$= 754.6$ kip-ft (as for AS rating)

**A1B.1.6.3—Capacity of Section, $M_R$ (6B.5.3.1)**

For braced, compact, composite sections:

$M_R = M_u$

AASHTO 10.50.1.1

where $M_u$ is found in accordance with applicable load factor provisions of AASHTO.

Check assumptions:

1. Section is fully braced along top flange by composite deck (for Live Load and SDL).
2. To check if section is compact, need to apply provisions of AASHTO 10.50.1.1.1.

These checks follow.

The compressive force in the slab, $C$, is equal to the smallest value given by the following equations:

$C = 0.85 f'c b_t s + (A F_y)_c$

AASHTO 10.50.1.1(a)

AASHTO Eq. 10-123

Neglecting that part of the reinforcement that lies in the compressive zone, the equation reduces to:

$C_{CONC} = 0.85 f'c b_t s = 0.85 (3 \text{ ksi})(87 \text{ in.})(7.25 \text{ in.}) = 1,608.4$ kip

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All rights reserved. Duplication is a violation of applicable law.
\[ C = \left( AF_y \right)_{bf} + \left( AF_y \right)_{sf} + \left( AF_y \right)_{w} \]

where \( (AF_y)_{bf} \) includes cover plate, this equation reduces to:

\[ C_{STL} = \left( 38.26 \text{ in.}^2 + 6.56 \text{ in.}^2 \right) (36 \text{ ksi}) = 1,613.5 \text{ kip} \]

\[ C_{CONC} < C_{STL} \Rightarrow C_{CONC} = 1,608.4 \text{ kip (controls)} \]

Capacity:

\[ C' = \frac{\sum (AF_y)}{2} - \frac{C}{2} = \frac{1,613.5 - 1,608.4}{2} = 2.55 \text{ kip} \]

\[ (AF_y)_{gf} = (11.51 \times 0.855)(36) = 354 \text{ kip} > 2.55 \text{ kip} \Rightarrow \text{NA in top flange of the I-girder} \]

\[ \bar{y} = \frac{C'}{(AF_y)_{gf}} = \frac{2.75}{354}(0.855) = 0.0062 \text{ in.} \]

Since the PNA is within the top of the flange, the depth of the web in compression at the plastic moment, \( D_{CP} \), is equal to zero. Hence, the web slenderness requirement given by Eq. 10-129 in AASHTO Article 10.50.1.1.2 is automatically satisfied.

Check the ductility requirement given by Eq. 10-129a in AASHTO Article 10.50.1.1.2:

\[ \left( \frac{D_p}{D'} \right) \leq 5 \]

\[ D' = \beta \left( \frac{d + t_s + t_b}{7.5} \right) \]

\[ \beta = 0.9 \text{ for } F_y = 36,000 \text{ psi} \]

\[ D' = 0.9 \left( \frac{33.725 + 7.25 + 0.00}{7.5} \right) = 4.917 \]

\[ D_p = 7.25 + 0.0062 \text{ in.} = 7.2562 \text{ in.} \]

\[ \left( \frac{D_p}{D'} \right) = \frac{7.2562}{4.917} = 1.4757 \leq 5 \text{ OK} \]

Since the top flange is braced by shear studs anchored in the hardened concrete deck, local and lateral buckling requirements need not be checked. The capacity of composite beams in simple spans satisfying the preceding web slenderness and ductility requirements is given by Eq. 10-129c in AASHTO 10.50.1.1.2 when \( D_p \) exceeds \( D' \):

\[ D' < D_p \leq 5D' \]

\[ 4.917 \text{ in.} < 7.2562 \text{ in.} \leq 5 \times 4.917 \text{ in.} = 24.585 \text{ in.} \]

Therefore:

\[ C = M_c = M_U = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4} \left( \frac{D_p}{D'} \right) \]

\[ \text{AASHTO Eq. 10.129c} \]
Compute the plastic moment capacity, $M_p$

$M_p = C \times \text{arm} = (1,608.4 + 2.55)(22.643) = 36,476.7 \text{ kip-in.} = 3,039.7 \text{ kip-ft}$

$M_R = \frac{5(3,039.7) - 0.85(2,379.0)}{4} + \frac{0.85(2,379.0) - 3,035}{4}(1.4757) = 2,918.7 \text{ kip-ft}$

Figure A1B.1.6.3-1—Cross Section—Interior Stringer, for Determining Plastic Moment Capacity, $M_p$

$RF_I^{\text{LF}} = \frac{M_R - A_1M_D}{A_2M_{L+1}}$  
Eq. 6B.4.1-1

where:

$A_1 = 1.3$  and  $A_2 = 2.17$

Thus:

$RF_I^{\text{LF}} = \frac{(2918.7) - 1.3 \times (439.2 + 129.1)}{2.17(754.6)}$

$RF_I^{\text{LF}} = 1.331$ or $1.331 \times 36 \text{ tons} = 47.9 \text{ tons}$

A1B.1.6.5—Operating Level (6B.4.3)

Only change is $A_2 = 1.3$

Thus:
APPENDIX A: ILLUSTRATIVE EXAMPLES

\[
RF_{O}^{LF} = \frac{2.17}{1.30} \quad RF_{j}^{LF} = \frac{2.17}{1.30}(1.33) \\
RF_{O}^{LF} = 2.222 \text{ or } 2.222 \times 36 \text{ tons} = 80.0 \text{ tons}
\]

A1B.1.6.6—Check Serviceability Criteria

For HS loadings, overload is defined as \(D + 5(L + I)/3\) \hspace{1cm} AASHTO 10.57

\[
f_{DL} + f_{SDL} + 1.67\left(f_{LL+1}\right) \leq \text{Serv. Strength} = 0.95F_{y} \hspace{1cm} \text{AASHTO 10.57.2}
\]

Thus \(A_1 = 1.0\) and \(A_2 = 1.67\) for service rating:

\[
RF_{i}^{LF} = \frac{0.95F_{y} - (1.0)f_{DL} - (1.0)f_{SDL}}{(1.67)f_{LL+1}}
\]

\[
= \frac{0.95(36 \text{ ksi}) - (1.0)\frac{439.2 \times 12}{563.8} - (1.0)\frac{129.1 \times 12}{724.1}}{1.67\frac{754.6 \times 12}{793.0}}
\]

\[
= \frac{34.20 - 9.438 - 2.139}{1.67(11.419)}
\]

\[
= 1.191 \text{ or } 1.191 \times 36 \text{ tons} = 42.9 \text{ tons}
\]

Check the web compressive stress:

\[
C = F_{cr} = \frac{26,200,000ak}{D} \left(\frac{D}{t_{w}}\right)^{2} \hspace{1cm} \text{AASHTO Eq. 10-173}
\]

where:

\[k = 9\left(\frac{D}{D_{c}}\right)^{2}\] and \(\alpha = 1.3\)

Since \(D_{c}\) is a function of the dead-to-live-load stress ratio according to the provisions of AASHTO 10.50(b), an iterative procedure may be necessary to determine the rating factor:

Compute the compressive stresses at the top of the web:

\[
f_{DL} = \frac{439.2(12)(18.163)}{8,291.5} = 11.545 \text{ ksi}
\]

\[
f_{ADL} = \frac{129.1(12)(10.405)}{16,267.4} = 0.991 \text{ ksi}
\]

\[
f_{LL+1} = \frac{(754.6)(12)(4.352)}{22,614.5} = 1.743 \text{ ksi}
\]

Total compressive stress = 14.279 ksi
Compute the tensile stresses at the bottom of the web:

\[
f_{DL} = \frac{439(12)(13.227)}{8,291} = 8.408 \text{ ksi}
\]

\[
f_{ADL} = \frac{129.1(12)(20.985)}{16,267.4} = 1.998 \text{ ksi}
\]

\[
f_{LL+I} = \frac{4.6(12)(27.038)}{22,614.5} = 10.826 \text{ ksi}
\]

Total tensile stress = 21.232 ksi

\[
D_c = 31.39 \times \left( \frac{14.279}{14.279 + 21.232} \right) = 12.622 \text{ in.}
\]

\[
k = 9 \left( \frac{D}{D_c} \right)^2 = 9 \left( \frac{31.39}{12.622} \right)^2 = 55.66
\]

\[
C = F_{cr} = \frac{26,200,000(1.3)(55.66)}{(31.39)^2(1,000)} = 647.23 \text{ ksi} > F_{yw}
\]

\[
C = F_{cr} = F_{yw} = 36 \text{ ksi}
\]

Thus, rating factor based on compressive stress to top of the web:

\[
RF_f = \frac{36.00 - 11.545 - 0.991}{1.67(1.743)} = 8.061 \text{ or } 8.061 \times 36 \text{ tons} = 290.2 \text{ tons}
\]

Since the computed rating factor would cause the total stresses in the tension flange to far exceed \(F_y\) (causing the neutral axis to be higher on the web), further iterations are not necessary in this case. The web compressive stress does not govern the serviceability rating.

A1B.1.6.6b—At Operating Level

\[
f_{DL} + f_{SDL} + (f_{LL+I}) \leq \text{Serv. Strength} = 0.95F_y \quad \text{AASHTO 10.57.2}
\]

\[
f_{DL} + f_{SDL} + RF_O (f_{LL+I}) \leq \text{Serv. Strength} = 0.95F_y
\]

Thus \(A_1 = 1.0\) and \(A_2 = 1.00\) for service rating at operating level.

Thus:

\[
RF_o = \frac{1.67}{1.0} \times RF_f = 1.67 \times 1.191
\]

\[
= 1.989 \text{ or } 1.989 \times 36 \text{ tons} = 71.6 \text{ tons}
\]
APPENDIX A: ILLUSTRATIVE EXAMPLES

A1B.1.7.6—Summary of Ratings for Load Factor Rating Method

Table A1B.1.6.7-1—Summary of Ratings for Load Factor Rating Method—Interior Stringer

<table>
<thead>
<tr>
<th></th>
<th>RF</th>
<th>Tons</th>
<th>Controlled</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inventory</td>
<td>1.191</td>
<td>42.9</td>
<td>Serviceability</td>
</tr>
<tr>
<td>Operating</td>
<td>1.989</td>
<td>71.6</td>
<td>Serviceability</td>
</tr>
</tbody>
</table>

A1B.1.7—Load Factor Rating—Rate for Single-Unit Formula B Loads

\[ M_{LL+I} \text{ from Appendix C6B:} \]

<table>
<thead>
<tr>
<th>Span (ft)</th>
<th>HS-20</th>
<th>NRL</th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>512.2</td>
<td>595.1</td>
<td>430.2</td>
<td>472.5</td>
<td>525.0</td>
<td>569.9</td>
</tr>
<tr>
<td>70</td>
<td>619.2</td>
<td>714.2</td>
<td>510.2</td>
<td>564.4</td>
<td>628.3</td>
<td>685.4</td>
</tr>
</tbody>
</table>

By interpolation:

| 65        | 565.7 | 654.7 | 470.2 | 518.5 | 576.7 | 627.7 |

Apply distribution factor \( DF = 1.333 \)

<p>| | | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>65</td>
<td>754.2</td>
<td>872.9</td>
<td>626.9</td>
<td>691.3</td>
<td>768.9</td>
<td>836.9</td>
</tr>
</tbody>
</table>

Capacity of Section \( M_R = 2,918.7 \text{ kip-ft} \)

Dead Load \( M_{DL} = 439.2 \text{ kip-ft} \)

Superimposed Dead Loads \( M_{SDL} = 129.1 \text{ kip-ft} \)

Inv. \( RF = \frac{2,918.7 - 1.30(439.2 + 129.1)}{2.17(M_{LL+I})} \)

Opr. \( RF = \frac{2,918.7 - 1.30(439.2 + 129.1)}{1.30(M_{LL+I})} \)

Strength Rating Factors:

<table>
<thead>
<tr>
<th></th>
<th>HS-20</th>
<th>NRL</th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inventory</td>
<td>1.332</td>
<td>1.151</td>
<td>1.602</td>
<td>1.453</td>
<td>1.306</td>
<td>1.200</td>
</tr>
<tr>
<td>Operating</td>
<td>2.223</td>
<td>1.921</td>
<td>2.675</td>
<td>2.426</td>
<td>2.181</td>
<td>2.004</td>
</tr>
</tbody>
</table>

Check Serviceability Criteria:

\[ RF_i = \frac{0.95F_y - f_{DL} - f_{SDL}}{1.67f_{LL+I}} \]

Capacity of Section, \( f_R = 0.95F_y = 34.20 \text{ ksi} \)

Dead Load \( f_D = 9.348 \text{ ksi} \)

Superimposed Dead Loads \( f_{SDL} = 2.139 \text{ ksi} \)

\[ RF_i = \frac{34.2 - 9.348 - 2.139}{1.67(M_{LL+I} \times 12)} \]

\[ RF_0 = \frac{34.2 - 9.348 - 2.139}{1.00(M_{LL+I} \times 12)} \]
Serviceability Rating Factors:

<table>
<thead>
<tr>
<th></th>
<th>HS20</th>
<th>NRL</th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inventory</td>
<td>1.191</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Operating</td>
<td>1.989</td>
<td>1.719</td>
<td>2.394</td>
<td>2.171</td>
<td>1.952</td>
<td>1.793</td>
</tr>
</tbody>
</table>

As the Notional Rating Load, NRL, $RF > 1.0$ for strength and serviceability, the bridge has adequate capacity for all legal loads, including the single-unit Formula B trucks.
### PART C—SUMMARY

#### A1C.1—Summary of All Ratings for Example A1

Table A1C.1-1—Summary of Rating Factors for All Rating Methods—Interior Stringer

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Design Load Rating (HL-93)</th>
<th>Legal Load Rating</th>
<th>Permit Load Rating</th>
<th>HS-20 Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inventory</td>
<td>Operating</td>
<td>Type 3</td>
<td>Type 3S2</td>
</tr>
<tr>
<td>Strength I</td>
<td>Flexure</td>
<td>1.294</td>
<td>1.677</td>
<td>3.344</td>
</tr>
<tr>
<td>Strength II</td>
<td>Flexure</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Service II</td>
<td>—</td>
<td>1.208</td>
<td>1.570</td>
<td>2.318</td>
</tr>
<tr>
<td>Fatigue I</td>
<td>0.326</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Allowable Stress Method</td>
<td>—</td>
<td>—</td>
<td>NP</td>
<td>NP</td>
</tr>
<tr>
<td>Load Factor Method</td>
<td>Strength</td>
<td>—</td>
<td>—</td>
<td>NP</td>
</tr>
<tr>
<td>Serviceability</td>
<td>—</td>
<td>—</td>
<td>NP</td>
<td>NP</td>
</tr>
</tbody>
</table>

* Rating Factors for LF method corresponds to operating level are listed in this table.
NP—Calculations are not performed

Table A1C.1-2—Summary of Rating Factors for Load and Resistance Factor Rating Method—Exterior Stringer

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Design Load Rating (HL-93)</th>
<th>Legal Load Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inventory</td>
<td>Operating</td>
</tr>
<tr>
<td>Strength II</td>
<td>Flexure</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>—</td>
</tr>
<tr>
<td>Service II</td>
<td>—</td>
<td>1.477</td>
</tr>
<tr>
<td>Fatigue</td>
<td>NP</td>
<td>—</td>
</tr>
</tbody>
</table>

NP—Calculations are not performed
A1C.2—References


The reinforced concrete slab bridge design and legal load check is detailed in Example A7, which was revised by the AASHTO Committee on Bridges and Structures at their 2019 Annual Meeting.

Editor's Note: Since all of Example A7 was revised by the AASHTO Committee on Bridges and Structures at their 2019 Annual Meeting, the new text in this Section has not been underlined.

Note: Evaluation of this bridge was performed in accordance to the 3rd Edition of the MBE and 8th edition of the AASHTO LRFD Bridge Design Specifications (LRFD Design.)

### A7.1—Bridge Data

- **Span Length:** 21.50 ft (simple span, distance from bearing to bearing). 0.50 ft End of the slab beyond the CL support
- **Year Built:** 1963
- **Material:**
  - Deck Concrete: $f'_c = 3.00$ ksi
  - Reinforced Steel: $f'_y = 40.00$ ksi
- **Structure Condition:** No deterioration. NBI Item 59 = 7. Member is in good condition
- **Riding Surface:** Not field verified and documented
- **ADTT (one direction):** Unknown
- **Skew:** 0°
- **Bridge category:** Interstate Bridge
- **Overlay Thickness:** 3.5 in. (field verified)

![Figure A7.1-1—Reinforced Concrete Slab Bridge](image)

**Cross Section**

Dist. from edge to center of rebar = [Clear cover (1.5 in.)+1/2 the rebar diameter]
A7.2—Dead Load Analysis

A7.2.1—Interior Strip—Unit (One Foot) Width

A7.2.1.1—Components, DC

Concrete slab:

\[
\left( \frac{14}{12} \right) (1.0) = 0.175 \text{ kip/ft}
\]

Parapet and curb:

\[
\frac{2 \left[ (1.5)(1.5) + (2.33)(1.0) \right] (1.0)(0.150)}{43} = 0.032 \text{ kip/ft}
\]

Total Dead Load/unit width DC = 0.207 kip/ft

Dead Load Moment = \( M_{DC} \) = \( \frac{1}{8} \times 0.207 \times 21.5^2 \)

= 12.0 kip-ft at mid span

A7.2.1.2—Wearing Surface, DW

Asphalt Thickness = 3.50 in. (field measured)

Asphalt Overlay = \( \left( \frac{3.5}{12} \right) (1.0)(0.144) = 0.042 \text{ kip/ft} \)

Dead Load Moment = \( M_{DW} \) = \( \frac{1}{8} \times 0.042 \times 21.5^2 \)

= 2.4 kip-ft at mid span

A7.3—Live Load Analysis (Design Load Check)

(a) Equivalent strip width for slab type bridges (Interior Strip)

A7.3.1—One Lane Loaded

\[ E = 10.0 + 5.0 \sqrt{L_1 W_1} \]

Where \( L_1 \) = Lesser of the actual span length or 60 ft

\( L_1 = 21.50 \text{ ft} < 60 \text{ ft}, \) use 21.50 ft

\( W_1 = \) Lesser of Bridge width or 30.0 ft

\( = 43.0 \text{ ft} > 30 \text{ ft}, \) use 30.00 ft

\[
E = 10.0 + 5.0 \sqrt{21.5 \times 30}
\]

= 137.0 in.

= 11.42 ft
A7.3.2—More than One Lane Loaded

\[ E = 84.0 + 1.44 \sqrt{L_1 W_1} \leq \frac{12.0 \, W}{N_L} \]

Where \( L_1 \) = Lesser of the actual span length or 60 ft

\( L_1 = 21.50 \, \text{ft} < 60 \, \text{ft}, \text{use} \ 21.50 \, \text{ft} \)

\( W_1 = \) Lesser of Bridge width or 60.0 ft

\( W_1 = 43.0 \, \text{ft} > 60 \, \text{ft}, \text{use} \ 43.00 \, \text{ft} \)

\( W_1 = 43.0 \, \text{ft} < 60 \, \text{ft}, \text{use} \ 43.0 \, \text{ft} \).

\( W = \) Physical edge to edge bridge width = 43.0 ft

\( N_L = \frac{40.0}{12} = 3.333 \, \text{ft}, \text{use} \ 3 \, \text{Lanes} \)

\[ E = 84.0 + 1.44 \sqrt{21.50 \times 43.0} \leq \frac{12.0 \, W}{N_L} \]

= 127.8 in.

= 11.41 ft

\[ \frac{12.0 \, W}{N_L} = \frac{12 \times 43}{3} = 172 \, \text{in.} > 127.8 \, \text{in.} \quad \text{OK} \]

So, use \( E = 127.8 \, \text{in.} = 10.65 \, \text{ft} \)

(b) Longitudinal "Edge" Strip

For longitudinal edge strips, the effective strip width is:

Sum of:

1. the distance between the edge of the deck and the inside face of the barrier

2. one-quarter the strip width specified in LRFD Design Article 4.6.2.1.3, 4.6.2.3, or 4.6.2.10, as appropriate

3. 12.0 in.

but, the effective edge strip width shall not exceed either one-half the full strip width or 72.0 in.

So, \( E_{\text{edge}} = 18.0 \, \text{in.} + 0.25 \times 137.0 \, \text{in.} + 12.0 \, \text{in.} = 64.25 \, \text{in.} \)

but, limited to = 0.5 \times 137.0 \, \text{in.} = 68.5 \, \text{in.} or 72.00 \, \text{in.}

So, use \( E = 64.25 \, \text{in.} = 5.3542 \, \text{ft} \)

LRFD Design Article 4.6.2.1.4b assumes the longitudinal edge strip supports one wheel line and a tributary portion of the design lane load where appropriate.

By comparison of the ratios of the tributary design lane load width to effective slab width, the edge strip is estimated not to govern for this bridge. Note that parapet dead load was assumed to be uniformly distributed across the full bridge width and that parapet width can play an influential role when determining the governing case.
Figure A7.3.2-1—Longitudinal Edge Strip Comparison

From the Figure A7.3.2-1, one-inch edge slab strip will carry

\[ \frac{1}{6} \times 42.5 + \frac{46.25}{64.25} \text{ lane} = 0.0156 \text{ wheel} + 0.7198 \text{ lane} \]

From the Figure A7.3.2-1, one-inch interior slab strip will carry

\[ \frac{2}{1} \times 27.8 + \frac{120}{127.8} \text{ lane} = 0.01565 \text{ wheel} + 0.9390 \text{ lane} \]

load carried by 1in. edge strip

Since the rebar pattern within the edge strip and the interior strip is the same, the rating of the interior strip will control; as a result, only the rating of the interior strip width is performed in this example.

A7.3.2.1—Midspan Live Load Force Effects (HL-93)

Dynamic Load Allowance = 33 percent
Equivalent Strip Width = 10.65 ft

Live Load Moment per unit width of slab:

\[ M_{LL} = \frac{328.0}{10.65} = 30.8 \text{ kip-ft/ft slab} \]

A7.4—Compute Nominal Resistance of Unit Width (1 ft)

Flexural Resistance:

Rectangular Section = \( b_w = b = 12 \text{ in.} \)

\[ c = \frac{A_s f_y}{\alpha_1 f_c' B_1 b} \]

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\[ A_s = 0.79 \times 2 \quad \#8 \text{ bars at 6 in. CC} \]
\[ = 1.58 \text{ in.}^2/\text{ft} \]
\[ \alpha_i = 0.85 \quad \text{(for } f'_c < 10 \text{ ksi}) \quad \text{LRFD Design 5.6.2.2} \]
\[ \beta_i = 0.85 \quad \text{(for } f'_c < 4 \text{ ksi}) \quad \text{LRFD Design 5.6.2.2} \]
\[ f'_c = 40.00 \text{ ksi} \]
\[ f_c = 3.00 \text{ ksi} \]
\[ c = \frac{1.58^2 \times 40}{0.85 \times 3 \times 0.85 \times 12} \]
\[ = 2.430 \text{ in.} \]
\[ a = c\beta_i \quad \text{LRFD Design 5.6.2.2} \]
\[ = 2.43 \times 0.85 \]
\[ = 2.066 \text{ in.} \]
\[ d_t = 14.00 - 2.00 = 12.000 \text{ in.} \quad \text{LRFD Design 5.6.3.2.2} \]
\[ M_n = A_s f_y \left( d_s - \frac{a}{2} \right) \quad \text{LRFD Design Eq. 5.6.3.2.2-1} \]
\[ = 1.58 \times 40 \left( 12.0 - \frac{2.066}{2} \right) \times \frac{1}{12} \]
\[ = 57.76 \text{ kip-ft} \]

**A7.5—Maximum Reinforcement (6A.5.5, LRFD Design 5.6.2.1)**

Current provisions of the LRFD specification have eliminated the check for maximum reinforcement. Instead, the factored resistance (\( f \)) of compression-controlled sections shall be reduced in accordance with LRFD Design Article 5.5.4.2. This approach limits the capacity of over-reinforced (compression-controlled) sections.

The net tensile strain, \( \varepsilon_t \), is the tensile strain at nominal strength and determined by strain compatibility using similar triangles.

Given an allowable concrete strain of 0.003 and depth to neutral axis \( c = 2.43 \) in.

\[ \frac{\varepsilon_c}{c} = \frac{\varepsilon_t}{d_t - c} \quad \text{LRFD Design Figure C5.6.2.1-1} \]

\[ \frac{0.003}{2.430 \text{ in.}} = \frac{\varepsilon_t}{12.00 \text{ in.} - 2.430 \text{ in.}} \]

\[ \varepsilon_t = 0.0118 \]

The tension-controlled strain limit shall be taken as 0.005 for nonprestressed reinforcement with \( f_y \leq 75.0 \text{ ksi} \).
For $\varepsilon_t = 0.0118 > 0.005$, the section is tension controlled and Resistance Factor, $f_r$, shall be taken as 0.90.

**A7.6—Minimum Reinforcement (6A.5.6, LRFD Design 5.6.3.3)**

Amount of reinforcement must be sufficient to develop $M_r$ equal to the lesser of $1.33M_u$ or $M_{cr}$.

$$M_r = \varphi M_n = 0.90 \times 57.76 \text{ kip-ft} = 51.98 \text{ kip-ft}$$

1. $1.33M_u = 1.33 \times M_u = 1.33 \times (1.25 \times 12.0 + 1.25 \times 2.4 + 1.75 \times 30.8)$

   $= 95.6 \text{ kip-ft} > 51.98 \text{ kip-ft}$ No Good

2. $M_{cr} = \gamma_3 \left( (1 + \gamma_1 f_r + \gamma_2 f_{cpe}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right)$

where

$$\gamma_1 = 1.60$$

$$\gamma_3 = \frac{F_u}{F_a} = \frac{40.00 \text{ ksi}}{70.00 \text{ ksi}}$$

Ultimate tensile strength of 40 ksi rebar is 70 ksi

$$= 0.571$$

$$S_{nc} = \frac{I}{\gamma_i}$$

where:

$I$ = moment of inertia of uncracked section (neglecting reinforcement steel)

$$I = \frac{1}{12} \times 12 \text{ in.} \times (14 \text{ in.})^3$$

$y_t$ = distance from neutral axis of the uncracked section to the extreme tension fiber

$$= \frac{14.00}{2} = 7.00 \text{ in.}$$

$$S_{nc} = \frac{2,744.00 \text{ in.}^4}{700 \text{ in.}} = 392.0 \text{ in.}^3$$

$$S_c = S_{nc}$$

$$f_r = 0.24 \lambda \sqrt{f_c}$$

$\lambda = 1$ for normal weight concrete

$$f_r = 0.24 \times 1 \times \sqrt{3.00 \text{ ksi}} = 0.416 \text{ ksi}$$

$$M_{cr} = \gamma_3 \left( (1 + \gamma_1 f_r + \gamma_2 f_{cpe}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right)$$
Dead Load Moment = \( M_{dmc} = \frac{0.175 \text{kip/ft} \times (21.5 \text{ ft})^2}{8} = 10.1 \text{ kip-ft at mid span} \)

\[
M_{cr} = 0.571 \left[ (1.6 \times 0.416 \text{ ksi} + 0.4 \times 0) \times 392.0 \text{ in}^3 - 10.1 \left( \frac{392}{392} - 1 \right) \right] 
\]

= 148.98 kip-in = 12.42 kip-ft

12.42 kip-ft < \( M_r = 51.98 \) kip-ft So, OK

Therefore, the section meets the requirements for minimum reinforcement.

**A7.7—Shear**

Concrete slabs and slab bridges designed in conformance with AASHTO Specifications LRFD Design 5.12.2.1 Article 4.6.2.3 may be considered satisfactory for shear.

In service concrete bridges that show no visible signs of shear distress need not be checked for shear when rating for the design load or legal load ratings.

**A7.8—General Load-Rating Equation (6A.4.2)**

\[
RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_{P})(P)}{\gamma_{L}}(LL + IM) 
\]

**Eq. 6A.4.2.1-1**

**A7.9—Evaluation Factors (for Strength Limit States)**

**A7.9.1—Resistance Factor, \( \phi \) (LRFD Design 5.5.4.2)**

\( \phi = 0.90 \) For flexure

**A7.9.2—Condition Factor, \( \phi_c \) (6A.4.2.3)**

\( \phi_c = 1.00 \), since the member is in good condition. NBI Item 59 = 7.

**Table 6A.4.2.3-1**

**A7.9.3—System Factor, \( \phi_s \) (6A.4.2.4)**

\( \phi_s = 1.00 \), since the bridge is a Slab bridge

**Table 6A.4.2.4-1**

**A7.10—Design Load Rating (6A.4.3)**

**A7.10.1—Strength I Limit State (6A.5.4.1)**

Capacity, \( C = \phi_c \phi_{n} \phi_{R_s} \)

\[
RF = \frac{(\phi_c)(\phi_s)(\phi_n)R_s - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{\gamma_{L}}(LL + IM) 
\]

**Eq. 6A.4.2.1-2**

Load Factors are:

<table>
<thead>
<tr>
<th>Load</th>
<th>Inventory</th>
<th>Operating</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC</td>
<td>1.25</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td>DW</td>
<td>1.25</td>
<td>1.25</td>
<td>Asphalt thickness was field verified</td>
</tr>
<tr>
<td>LL + IM</td>
<td>1.75</td>
<td>1.35</td>
<td></td>
</tr>
</tbody>
</table>

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A7.10.1a—Inventory Level

Flexure: \( RF = \frac{(1.0)(1.0)(0.9)(57.76) - (1.25)(12.00 + 2.40)}{(1.75)(30.8)} \)

\[ RF = 0.631 \]

A7.10.1b—Operating Level

Flexure: \( RF = 0.631 \times \frac{1.75}{1.35} \) (Since the capacity is independent of demand)

\[ RF = 0.818 \]

As \( RF < 1.0 \) for HL-93, bridge needs to be evaluated for legal loads.

A7.10.2—Service Limit State

No service limit states apply to reinforced concrete bridge members.

A7.11—Legal Load Rating (6A.4.4)

A7.11.1—Live Load Demand

A7.11.1a—AASHTO Legal Loads—Routine Commercial Traffic—Type 3, 3S2, 3-3 (Rate for all 3)

From previous calculations, \( E = 10.650 \) ft

\( IM = 33 \) percent (Unknown riding surface conditions)

Moment demands for live load with 33 percent impact for 21.5 ft span were established by interpolating the values given in Table E6A-1.

<table>
<thead>
<tr>
<th>( M_{LL} ) (interpolated) (kip-ft)</th>
<th>Type 3</th>
<th>Type 3S2</th>
<th>Type 3-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>200.00</td>
<td>182.35</td>
<td>164.65</td>
<td></td>
</tr>
</tbody>
</table>

| \( \frac{M_{LL}+IM}{E} \) (kip-ft/ft) | 18.78 | 17.12 | 15.46 |

A7.11.1b—Live Load: AASHTO Legal Loads—Specialized Hauling Vehicles (SHVs) and Notional Rating Load—SU4, SU5, SU6, SU7, and NRL

Moment demands for live load with 33 percent impact for 21.5 ft span were established by interpolating the values given in Table E6A-2.

<table>
<thead>
<tr>
<th>( M_{LL}+IM ) (interpolated) (kip-ft)</th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
<th>NRL</th>
</tr>
</thead>
<tbody>
<tr>
<td>234.25</td>
<td>248.45</td>
<td>263.20</td>
<td>263.20</td>
<td>263.20</td>
<td>263.20</td>
</tr>
</tbody>
</table>

| \( \frac{M_{LL}+IM}{E} \) (kip-ft/ft) | 22.00 | 23.33 | 24.71 | 24.71 | 24.71 |

A7.11.2—Strength I Limit State (6A.5.4.2.1)

A7.11.2a—For Types 3, 3S2, and 3-3

Dead Load \( DC: \gamma_{DC} = 1.25 \)  
Dead Load \( DW: \gamma_{DW} = 1.25 \)  
\( ADTT \) (One Direction) = Unknown

Generalized Live-Load Factor for Legal Loads: \( \gamma_{L} = 1.45 \)
Flexure:

\[
RF = \frac{(1.0)(1.0)(0.90)(57.76) - (1.25)12.00 + 2.40}{(1.45)(M_{LL+IM})}
\]

<table>
<thead>
<tr>
<th></th>
<th>Type 3</th>
<th>Type 3S2</th>
<th>Type 3-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>(M_{LL+IM} / E) (kip-ft/ft)</td>
<td>18.78</td>
<td>17.12</td>
<td>15.46</td>
</tr>
<tr>
<td>(RF) (Flexure)</td>
<td>1.248</td>
<td>1.369</td>
<td>1.516</td>
</tr>
</tbody>
</table>

No posting required as \(RF > 1.0\) for all AASHTO Legal Loads. 6A.8.3

A7.11.2b—For Specialized Hauling Vehicles (SHVs) and NRL

Dead Load DC: \(\gamma_{DC} = 1.25\)  
Dead Load DC: \(\gamma_{DW} = 1.25\)

\(ADTT\) (One Direction) = Unknown

Generalized Live-Load Factor for Legal Loads, \(\gamma_{LL} = 1.45\)  

<table>
<thead>
<tr>
<th></th>
<th>SU4</th>
<th>SU5</th>
<th>SU6</th>
<th>SU7</th>
<th>NERL</th>
</tr>
</thead>
<tbody>
<tr>
<td>(M_{LL+IM} / E) (kip-ft/ft)</td>
<td>22.00</td>
<td>23.33</td>
<td>24.71</td>
<td>24.71</td>
<td>24.71</td>
</tr>
<tr>
<td>(RF) (Flexure)</td>
<td>1.065</td>
<td>1.005</td>
<td>0.948</td>
<td>0.948</td>
<td>0.948</td>
</tr>
</tbody>
</table>

Comparison of the above safe capacities for the SU4, SU5, SU6, and SU7 to the NRL Safe Load Capacity demonstrates that for bridges that do not rate for the NRL Load, a posting analysis should be performed to resolve posting requirements for single-unit multi-axle trucks. The above results show that the Safe Load Capacity for the SU4 and SU5 vehicles is adequate; however, posting may be required for SU6 and SU7 vehicles. 6A.8.2 and C6A.8.2

The decision to post a bridge should be made by the Bridge Owner. When for any legal truck the Rating Factor (\(RF\)) is between 0.3 and 1.0, then the following equation should be used to establish the safe posting load for that vehicle type.

\[
\text{Safe Posting Load} = \frac{W}{0.7}(RF - 0.3)
\]

Where \(W\) = Weight of rating vehicle. Eq. 6A.8.3-1

Therefore, for SU6 and SU7, the recommended safe posting loads are:

<table>
<thead>
<tr>
<th>Truck</th>
<th>SU6</th>
<th>SU7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight (Tons)</td>
<td>34.75</td>
<td>38.75</td>
</tr>
<tr>
<td>Safe Posting Loads (Tons)</td>
<td>32</td>
<td>35</td>
</tr>
</tbody>
</table>

(Rounded down)

A7.11.3—Service Limit State

No service limit states apply to reinforced concrete bridge members at the legal load rating. Table 6A.4.2.2-1

A7.11.4—Shear

Concrete slabs and slab bridges designed in conformance with AASHTO Specifications may be considered satisfactory for shear. LRFD Design 5.12.2.1

A7.11.5—Summary

Safe Load Capacity (tons), \(RT = RF \times W\)  

Eq. 6A.4.4.4-1
The NRL rating demonstrates Article C6A.4.4.2.1b: “Bridges that rate for the NRL loading will have adequate load capacity for all legal Formula B truck configurations up to 80 kips.” Example A1 shows this holding true NRL RF > 1.00 and all SU RF > 1.00, while Examples A2 and A7 show when NRL RF < 1.00, RF for the SHVs may or may not be >1.00 and needs to be checked on an individual basis.

A7.12—Summary of Rating Factors

Table A7.12-1 Summary of Rating Factors—Concrete Slab Interior Strip

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Design Load Rating</th>
<th>Legal Load Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inventory</td>
<td>Operating</td>
</tr>
<tr>
<td>Strength I</td>
<td>Flexure</td>
<td>0.63</td>
</tr>
</tbody>
</table>

A7.13—Reference