AASHTO

Victoria F. Sheehan, *President*
Commissioner, New Hampshire Department of Transportation
Jim Tymon, *Executive Director*

ERRATA for
Manual for Bridge Evaluation, 3rd Edition (MBE-3)

November 2021

Dear Customer:

Recently, we were made aware of some technical revisions that need to be applied to the *Manual for Bridge Evaluation*, 3rd Edition. This file contains the previous errata from October 2018 (indicated in the summary by white backgrounds) as well as newer corrections (indicated in the summary by shaded backgrounds). Note that MBE-3 has also had two interim revisions between the first erratum and this publication, and all pages in this file will include all changes from these as well, so your replacement pages will have all the current information in the correct order.

Please scroll down to see the full erratum.

In the event that you need to download this file again, please download from AASHTO's online bookstore at:

http://downloads.transportation.org/MBE-3-Errata.pdf

Then, please replace the existing pages with the corrected pages to ensure that your edition is both accurate and current.

AASHTO staff sincerely apologizes for any inconvenience to our readers.

Sincerely,

Erin Grady

Publications Director

555 12th Street NW | Suite 1000 | Washington, DC 20004 | 202-624-5800 Phone | transportation.org

American Association of State Highway and Transportation Officials

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Summary of Errata for MBE-3

<table>
<tr><th>Page</th><th>Original Text</th><th>Corrected Text</th></tr>
<tr><td>6-18</td><td>Table 6A.2.2-1, Row “Prestressed Concrete Service III,” Column “Design Load Inventory:”

0.80</td><td>Table 6A.2.2-1, Row “Prestressed Concrete Service III,” Column “Design Load Inventory:”

~~0.80~~ Table 6A.4.2.2-2</td></tr>
<tr><td>6-18</td><td>(None)</td><td>Inserted Table 6A.4.2.2-2:

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

<table><tr><th>Component</th><th>γ_{LL}</th></tr><tr><td>Prestressed concrete components rated using the refined estimates of time-dependent losses as specified in LRFD Design Article 5.9.5.4 in conjunction with taking advantage of the elastic gain</td><td>1.0</td></tr><tr><td>All other prestressed concrete components</td><td>0.8</td></tr></table></td></tr>
<tr><td>6-21</td><td>**C6A.4.3.3**</td><td>(Deleted heading with no contents)</td></tr>
<tr><td>6-25</td><td>(None)</td><td>Added to end of Section C6A.4.4.2.1b:

“SHVs, as detailed in Figures D6A-7 and 6B.7.2-2, can create higher load effects on bridges with shorter span lengths and on transverse floorbeams and thus result in lower ratings. Some state laws may preclude certain SHVs. Load ratings are required only for SHVs that can legally operate in a state.
Some states may have heavier state legal loads or design loads that cause larger load effects than the SHVs; this could preclude the need to rate bridges for SHVs. Most existing bridges have ASR, LFR, or LRFR ratings for the AASHTO Design Truck (HS-20, for example) and the AASHTO legal trucks (Routine Commercial Vehicles Type 3, Type 3S2, and Type 3-3).
Because it is not possible to re-rate the entire inventory all at once for the SHVs, the available controlling ratings for the AASHTO design trucks or AASHTO legal trucks can be used to screen the existing inventory of bridges that need to be rated for SHVs. The following situations illustrate lower risk bridges or bridges where the SHVs will not control the load rating and are less a priority to rate for the SHVs.
Studies of load effects for simple and continuous spans, for both flexure and shear, show:
• Bridges having an HL-93 Operating RF>1.0 need not be rated for SHVs.</td></tr>
</table>

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<tr><td></td><td></td><td>Bridges having an HS20 Operating RF > 1.20 need not be rated for SHVs.
Bridges with a minimum Operating RF > 1.35 for the AASHTO legal trucks under ASR or LFR, or a RF > 1.35 for these trucks using LRFR, would have adequate load capacity for the SHVs as follows: SU4 and SU5 for all spans; SU6 for spans above 70 ft; and SU7 for spans above 80 ft.
Posting needs for SHVs for spans below these span limits should be verified by rating.
The 2013 Interims to the MBE made the LRFR live load factors the same for the AASHTO legal trucks and the SHVs, following the recommendations of NCHRP 12-78. LRFR ratings completed prior to that change would need to account for the different live load factors used in the ratings when using the aforementioned screening for SHVs.”</td></tr>
<tr><td>6-56</td><td>C6A.6.3

…
Users of this Manual and the L L will note some differences in the specified resistance factors for main truss member gusset plates.</td><td>C6A.6.3

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Users of this Manual and the <u>AASHTO LRFD Bridge Design Specifications</u> will note some differences in the specified resistance factors for main truss member gusset plates.</td></tr>
<tr><td>6-94</td><td>Table B6A-1, Row “Prestressed Concrete, Service III,” Column “Design Load: Inventory”: 0.80</td><td>Table B6A-1, Row “Prestressed Concrete, Service III,” Column “Design Load: Inventory”: <s>0.80</s> <u>Table B6A-2</u></td></tr>
<tr><td>6-94</td><td>(None)</td><td>Inserted Table B6A-2:

Table B6A-2—Load Factors for Live Load for the Service III Load Combination, γ_{LL}, at the Design-Load Inventory Level
(6A.4.2.2-2)

<table><tr><th>Component</th><th>γ_{LL}</th></tr><tr><td>Prestressed concrete components rated using the refined estimates of time-dependent losses as specified in LRFD Design Article 5.9.5.4 in conjunction with taking advantage of the elastic gain</td><td>1.0</td></tr><tr><td>All other prestressed concrete components</td><td>0.8</td></tr></table></td></tr>
<tr><td>6-94</td><td>“Table B6A-2”</td><td>“Table B6A-3” (Renamed table)</td></tr>
<tr><td>6-95</td><td>“Table B6A-3”</td><td>“Table B6A-4” (Renamed table)</td></tr>
<tr><td>6-95</td><td>“Table B6A-4”</td><td>“Table B6A-5” (Renamed table)</td></tr>
<tr><td>6-147</td><td>(None)</td><td>Added to Section 6B.7.1, after first paragraph:</td></tr>
</table>

Page	Original Text	Corrected Text
		"A concrete bridge with unknown reinforcement need not be posted for restricted loading when it has been carrying normal traffic for an appreciable length of time and shows no distress. In other cases, a concrete bridge with no visible signs of distress but whose calculated load rating indicates the bridge needs to be posted can be alternately evaluated through load testing. If a concrete culvert with depths of fill 2.0 ft or greater with known details or with unknown components (such as culverts without plans) has been carrying normal traffic for an appreciable period and is in fair or better condition, as determined by a physical inspection of the culvert by a qualified inspector and documented in the inspection report, the culvert may be assigned an inventory load rating factor of 1.0 and operating load rating factor of 1.67 for the HS-20 design load and need not be posted for restricted loading. The load rating shall be documented in the bridge file."
6-147	(None)	Added to Section C6B.7.1, after third paragraph: "The simplified modeling approach used for culvert ratings tends to produce conservative force demands. Buried structures carry vertical loads through a combination of internal capacity and soil arching around the structure; this is termed soil–structure interaction. Soil–structure interaction effects are neglected when establishing culvert load ratings. It is therefore not uncommon to observe satisfactory performance of in-service culverts even when analytical ratings may show insufficient capacity for normal traffic."
6-150	(None)	Added to the end of Section C6B.7.2: "SHVs, as detailed in Figures D6A-7 and 6B.7.2-2, can create higher load effects on bridges with shorter span lengths and on transverse floorbeams and thus result in lower ratings. Some state laws may preclude certain SHVs. Load ratings are required only for SHVs that can legally operate in a state. Some states may have heavier state legal loads or design loads that cause larger load effects than the SHVs; this could preclude the need to rate bridges for SHVs. Most existing bridges have ASR, LFR, or LRFR ratings for the AASHTO Design Truck (HS-20, for example) and the AASHTO legal trucks (Routine Commercial Vehicles Type 3, Type 3S2, and Type 3-3). Because it is not possible to re-rate the entire inventory all at once for the SHVs, the available controlling ratings for the AASHTO design trucks or AASHTO legal trucks can be used to screen the existing inventory of bridges that need to be rated for

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		SHVs. The following situations illustrate lower risk bridges or bridges where the SHVs will not control the load rating and are less a priority to rate for the SHVs. Studies of load effects for simple and continuous spans, for both flexure and shear, show: • Bridges having an HL-93 Operating RF > 1.0 need not be rated for SHVs. • Bridges having an HS20 Operating RF > 1.20 need not be rated for SHVs. • Bridges with a minimum Operating RF > 1.35 for the AASHTO legal trucks under ASR or LFR, or a RF > 1.35 for these trucks using LRFR, would have adequate load capacity for the SHVs as follows: SU4 and SU5 for all spans; SU6 for spans above 70 ft; and SU7 for spans above 80 ft. Posting needs for SHVs for spans below these span limits should be verified by rating. The 2013 Interims to the MBE made the LRFR live load factors the same for the AASHTO legal trucks and the SHVs, following the recommendations of NCHRP 12-78. LRFR ratings completed prior to that change would need to account for the different live load factors used in the ratings when using the aforementioned screening for SHVs."
A-6	In Article *A1A.1.4.1:* $e_g = {}^{1}/_{2}\,(7.25) + 19.02 = 22.43$ in.	$e_g = {}^{1}/_{2}\,(7.25) + 19.02 = 22.\underline{6}43$ in.
A-12	In Article *A1A.1.8.1a:* Flexure: = $RF = \dfrac{(1.0)(1.0)(1.0)(2{,}873.0) - (1.25)(439.9 + 129.4)}{(1.75)(954.10)}$ $= 1.29\cancel{754}$	Flexure: = $RF = \dfrac{(1.0)(1.0)(1.0)(2{,}873.0) - (1.25)(439.9 + 129.4)}{(1.75)(954.10)}$ $= 1.29\underline{449}$
A-13	In Article *A1A.1.8.1a:* Shear: $RF =$ $\dfrac{(1.0)(1.0)(1.0)(3\cancel{6}0.15) - (1.25)(27.1 + 8.0)}{(1.75)(78.9)}$	Shear: $RF =$ $\dfrac{(1.0)(1.0)(1.0)(3\underline{8}0.15) - (1.25)(27.1 + 8.0)}{(1.75)(78.9)}$

DW = Dead load effect due to wearing surface and utilities

P = Permanent loads other than dead loads

LL = Live load effect

IM = Dynamic load allowance

γ_{DC} = LRFD load factor for structural components and attachments

γ_{DW} = LRFD load factor for wearing surfaces and utilities

γ_p = LRFD load factor for permanent loads other than dead loads = 1.0

γ_{LL} = Evaluation live load factor

φ_c = Condition factor

φ_s = System factor

φ = LRFD resistance factor

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

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

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

6A.4.2.2—Limit States

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

C6A.4.2.2

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

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

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

* Defined in the *AASHTO LRFD Bridge Design Specifications*

Notes:

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

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

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

6A.4.2.3—Condition Factor: φ_c

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

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

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

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

C6A.4.2.3

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

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

Improved inspections will reduce, but not totally eliminate, the increased scatter or resistance variability in deteriorated members. Improved inspection and field measurements will reduce the uncertainties inherent in identifying the true extent of deterioration for use in calculating the nominal member resistance. If section properties are obtained accurately, by actual field measurement of losses rather than by an estimated percentage of losses, the values specified for φ_c in Table 6A.4.2.3-1 may be increased by 0.05 ($\varphi_c \leq 1.0$).

The condition factor, φ_c, tied to the structural condition of the member, accounts for the member deterioration due to natural causes (e.g., atmospheric corrosion). Damage caused by accidents is specifically not considered here.

If condition information is collected and recorded in the form of NBI condition ratings only (not as element level data), then the following approximate conversion may be applied in selecting φ_c.

Table C6A.4.2.3-1—Approximate Conversion in Selecting φ_c

Superstructure Condition Rating (SI & A Item 59)	Equivalent Member Structural Condition
6 or higher	Good or Satisfactory
5	Fair
4 or lower	Poor

6A.4.2.4—System Factor: φ_s

System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factored member capacities reduced, and, accordingly, will have lower ratings.

System factors that correspond to the load factor modifiers in the *AASHTO LRFD Bridge Design Specifications* should be used. The system factors in Table 6A.4.2.4-1 are more conservative than the LRFD design values and may be used at the discretion of the evaluator until they are modified in the *AASHTO LRFD Bridge Design Specifications*; however, when rating nonredundant superstructures for legal loads using the generalized load factors given in Article 6A.4.4.2.3, the system factors from Table 6A.4.2.4-1 shall be used to maintain an adequate level of system safety.

The system factor for riveted and bolted gusset plates and their connections for all force effects shall be taken as 0.90.

C6A.4.2.4

Structural members of a bridge do not behave independently, but interact with other members to form one structural system. Bridge redundancy is the capability of a bridge structural system to carry loads after damage to or the failure of one or more of its members. Internal redundancy and structural redundancy that exists as a result of continuity are neglected when classifying a member as nonredundant.

Table 6A.4.2.4-1—System Factor: φ_s for Flexural and Axial Effects

Superstructure Type	φ_s
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing 6 ft	0.85
Four-Girder Bridges with Girder Spacing ≤4 ft	0.95
All Other Girder Bridges and Slab Bridges	1.00
Floorbeams with Spacing >12 ft and Noncontinuous Stringers	0.85
Redundant Stringer Subsystems between Floorbeams	1.00

If Table 6A.4.2.4-1 is used, the system factors are used to maintain an adequate level of system safety. Nonredundant bridges are penalized by requiring their members to provide higher safety levels than those of similar bridges with redundant configurations. The aim of φ_s is to add a reserve capacity such that the overall system reliability is increased from approximately an operating level (for redundant systems) to a more realistic target for nonredundant systems corresponding to inventory levels.

If the Engineer can demonstrate the presence of adequate redundancy in a superstructure system (Reference: NCHRP Report 406), then φ_s may be taken as 1.0. In some instances, the level of redundancy may be sufficient to utilize a value of φ_s greater than 1.0, but in no instance should φ_s be taken as greater than 1.2.

If the simplified system factors presented in Table 6A.4.2.4-1 are used, they should be applied only when checking flexural and axial effects at the strength limit state of typical spans and geometries.

A constant value of $\varphi_s = 1.0$ is to be applied when checking shear at the strength limit state.

For evaluating timber bridges, a constant value of $\varphi_s = 1.0$ is assigned for flexure and shear.

A more liberal system factor for nonredundant riveted sections and truss members with multiple eyebars has been provided. The internal redundancy in these members makes a sudden failure far less likely. An increased system factor of 0.90 is appropriate for such members.

Some agencies may consider all three-girder systems, irrespective of girder spacing, to be nonredundant. In such cases, φ_s may be taken as 0.85 for welded construction and 0.90 for riveted construction.

Subsystems that have redundant members should not be penalized if the overall system is nonredundant. Thus, closely spaced parallel stringers would be redundant even in a two-girder-floorbeam main system.

For narrow bridges (such as one-lane bridges) with closely spaced three-and four-girder systems, all the girders are almost equally loaded and there is no reserve strength available. Therefore, φ_s is decreased to 0.85.

For the purposes of determining system factors, each web of a box girder may be considered as an I-girder.

System factors are generally not appropriate for shear, as shear failures tend to be brittle, so system reserve is not possible. The design resistance, factored for shear, should be calibrated to reflect the brittle characteristics. Thus, in the evaluation, all the φ_s should be equal. A constant value of $\varphi_s = 1.0$ is assigned for evaluation.

More accurate quantification of redundancy is provided in NCHRP Report 406, *Redundancy in Highway Bridge Superstructures*. Tables of system factors are given in the referenced report for common simple-span and continuous bridges with varying number of beams and beam spacings. For bridges with configurations that are not covered by the tables, a direct redundancy analysis approach may be used, as described in NCHRP Report 406.

6A.4.3—Design-Load Rating

6A.4.3.1—Purpose

The design-load rating assesses the performance of existing bridges utilizing the LRFD-design loading (HL-93) and design standards. The design-load rating of bridges may be performed at the same design level

C6A.4.3.1

The design-load rating is performed using dimensions and properties for the bridge in its present condition, obtained from a recent field inspection.

No further evaluation is necessary for bridges that have

(Inventory level) reliability adopted for new bridges by the *AASHTO LRFD Bridge Design Specifications* or at a second lower-level reliability comparable to the operating level reliability inherent in past load-rating practice. The design-load rating produces inventory and operating level rating factors for the HL-93 loading.

The design-load rating serves as a screening process to identify bridges that should be load rated for legal loads, per the following criteria:

- Bridges that pass HL-93 screening at the inventory level will have adequate capacity for all AASHTO legal loads and state legal loads that fall within the exclusion limits described in the *AASHTO LRFD Bridge Design Specifications*.
- Bridges that pass HL-93 screening only at the operating level will have adequate capacity for AASHTO legal loads, but may not rate ($RF < 1$) for all state legal loads, specifically those vehicles significantly heavier than the AASHTO trucks.

The results are also suitable for use in NBI reporting, and bridge management and bridge administration, at a local or national level. The rating results for service and fatigue limit states could also guide future inspections by identifying vulnerable limit states for each bridge.

adequate capacity ($RF > 1$) at the Inventory level reliability for HL-93. Bridges that pass HL-93 screening only at the operating level reliability will not have adequate capacity for state legal loads significantly heavier than the AASHTO legal loads. Existing bridges that do not pass a design-load rating at the operating level reliability should be evaluated by load rating for AASHTO legal loads using procedures provided in this Section.

6A.4.3.2—Live Loads and Load Factors

6A.4.3.2.1—Live Load

The LRFD-design, live load HL-93 (see Appendix C6A) shall be used.

6A.4.3.2.2—Live Load Factors

The evaluation live load factors for the Strength I limit state shall be taken as shown in Table 6A.4.3.2.2-1.

Table 6A.4.3.2.2-1—Load Factors for Design Load: γ_L

Evaluation Level	Load Factor
Inventory	1.75
Operating	1.35

C6A.4.3.2.2

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

6A.4.3.3—Dynamic Load Allowance

The dynamic load allowance specified in the LRFD Specifications for new bridge design (LRFD Design Article 3.6.2) shall apply.

Dynamic load allowance need not be applied to wood components (LRFD Design Article 3.6.2.3).

6A.4.4—Legal Load Rating

6A.4.4.1—Purpose

Bridges that do not have sufficient capacity under the design-load rating shall be load rated for legal loads to establish the need for load posting or strengthening. Load rating for legal loads determines the safe load capacity of a

C6A.4.4.1

Evaluation procedures are presented herein to establish a safe load capacity for an existing bridge that recognizes a balance between safety and economics. The previous distinction of operating and inventory level ratings is no longer bridge for the AASHTO family of legal loads and state legal loads, using safety and serviceability criteria considered appropriate for evaluation. A single safe load capacity is obtained for a given legal load configuration.

maintained when load rating for legal loads.

Past load-rating practice defined two levels of load capacity: inventory rating and operating rating. Redundancy was not explicitly considered in load rating, and the inventory and operating ratings were generally taken to represent the lower and upper bounds of safe load capacity. Some Bridge Owners considered redundancy and condition of the structure when selecting a posting load level between inventory and operating levels.

The single safe load capacity produced by the procedures presented in this Manual considers redundancy and bridge condition in the load-rating process. The load and resistance factors have been calibrated to provide uniform levels of reliability and permit the introduction of bridge- and site-specific data in a rational and consistent format. It provides a level of reliability corresponding to the operating level reliability for redundant bridges in good condition. The capacity of nonredundant bridges and deteriorated bridges should be reduced during the load-rating process by using system factors and condition factors. The safe load capacity may approach or exceed the equivalent of operating rating for redundant bridges in good condition on low traffic routes, and may fall to the equivalent of inventory levels or below for heavily deteriorated, nonredundant bridges on high traffic routes.

6A.4.4.2—Live Loads and Load Factors

6A.4.4.2.1—Live Loads

6A.4.4.2.1a—Routine Commercial Traffic

The AASHTO legal vehicles and lane-type load models shown in Figures D6A-1 through D6A-5 shall be used for load rating bridges for routine legal commercial traffic.

For all span lengths the critical load effects shall be taken as the larger of the following:

- For all load effects, AASHTO legal vehicles (Type 3, Type 3S2, Type 3-3; applied separately) or state legal loads.
- For negative moments and reactions at interior supports, a lane load of 0.2 klf combined with two AASHTO Type 3-3 vehicles or state legal loads multiplied by 0.75 heading in the same direction separated by 30 ft.

Take the largest of Type 3, Type 3S2, Type 3-3 vehicles, or state legal loads plus lane loads. The lane load model is common to all truck types. In addition, for span lengths greater than 200 ft, critical load effects shall be created by:

- AASHTO Type 3-3 or state legal load multiplied by 0.75 and combined with a lane load of 0.2 klf.

Dynamic load allowance shall be applied to the AASHTO legal vehicles and state legal loads but not the lane loads. If the *ADTT* is less than 500, the lane load may be excluded and the 0.75 factor changed to 1.0 if, in the Engineer's judgment, it is warranted.

C6A.4.4.2.1

C6A.4.4.2.1a

Usually bridges are load rated for all three AASHTO trucks and lane loads to determine the governing loading and governing load rating. A safe load capacity in tons may be computed for each vehicle type (see Article 6A.4.4.4). When the lane type, load model governs the load rating, the equivalent truck weight for use in calculating a safe load capacity for the bridge shall be taken as 80 kips.

AASHTO legal vehicles, designated as Type 3, Type 3S2, and Type 3-3 are sufficiently representative of average truck configurations in use today, and are used as vehicle models for load rating. These vehicles are also suitable for bridge posting purposes. Load ratings may also be performed for state legal loads that have only minor variations from the AASHTO legal loads using the live load factors provided in Table 6A.4.4.2.3a-1 for the AASHTO vehicles. It is unnecessary to place more than one vehicle in a lane for spans up to 200 ft because the load factors provided have been modeled for this possibility.

The federal bridge formula (Reference: TRB Special Report 225, *Truck Weight Limits Issues and Options*, 1990) restricts truck weights on interstate highways through (a) a total, or gross, vehicle weight limit of 80 kips; (b) limits on axle loads (20 kips for single axles, 34 kips for tandem axles); and (c) a bridge formula that specifies the maximum allowable weight on any group of consecutive axles based on the number of axles in the group and the distance from first to the last axles. Grandfather provisions in the federal statutes allow states to retain higher limits than these if such limits were in effect when the applicable federal statutes were first enacted.

The objective of producing new *AASHTO LRFD Bridge Design Specifications* that will yield designs having uniform reliability required as its basis a new live load model with a consistent bias when compared with the exclusion vehicles. The model consisting of either the HS-20 truck plus the uniform lane load or the tandem plus the uniform lane load (designated as HL-93 loading) resulted in a tight clustering of data around a 1.0 bias factor for all force effects over all span lengths. This combination load was therefore found to be an adequate basis for a notional design load in the *AASHTO LRFD Bridge Design Specifications*.

While this notional design load provides a convenient and uniform basis for design and screening of existing bridges against new bridge safety standards, it has certain limitations when applied to evaluation. The notional design load bears no resemblance or correlation to legal truck limits on the roads and poses practical difficulties when applied to load rating and load posting of existing bridges.

A characteristic of the AASHTO family of legal loads (Type 3, Type 3S2, and Type 3-3) is that the group satisfies the federal bridge formula. The AASHTO legal loads model three portions of the bridge formula which control short, medium, and long spans. Therefore, the combined use of these three AASHTO legal loads results in uniform bias over all span lengths, as achieved with the HL-93 notional load model (see Figure C6A-1). These vehicles are presently widely used for load rating and load posting purposes. These AASHTO vehicles model many of the configurations of present truck traffic. They are appropriate for use as rating vehicles as they satisfy the major aim of providing uniform reliability over all span lengths. They are also widely used as truck symbols on load posting signs. Additionally, these vehicles are familiar to engineers and provide continuity with current practice.

6A.4.4.2.1b—Specialized Hauling Vehicles

The notional rating load (NRL) shown in Figure D6A-6, which envelopes the load effects of the Formula B specialized hauling vehicle configurations (see Figure D6A-7) weighing up to 80 kips, should be used for legal load ratings.

C6A.4.4.2.1b

The vehicles referred to as specialized hauling vehicles (SHV) are legal single-unit, short-wheelbase, multiple-axle trucks commonly used in the construction, waste management, bulk cargo, and commodities hauling industries.

Trucks weighing up to 80 kips are typically allowed unrestricted operation and are generally considered "legal" provided they meet weight guidelines of Federal Bridge Formula B (Formula B). In the past, the maximum legal weight for short-wheelbase trucks was usually controlled by Formula B rather than by the 80 kips gross weight limit. Since the adoption of the AASHTO family of three legal loads, the trucking industry has introduced specialized single-unit trucks with closely spaced multiple axles that make it possible for these short-wheelbase trucks to carry the maximum load of up to 80,000 lb and still meet Formula B. The AASHTO family of three legal loads selected at the time to closely match the Formula B in the short, medium, and long truck length ranges do not represent these newer axle configurations. These SHV trucks cause force effects that exceed the stresses induced by HS-20 in bridges by up to 22 percent and by the Type 3, 3S2, or 3-3 posting vehicles by over 50 percent, in certain cases. The shorter bridge spans are most sensitive to the newer SHV axle configurations.

The NRL represents a single load model that will envelop the load effects on simple and continuous span bridges of the worst possible Formula B single-unit truck configurations with multiple axles up to 80 kips. It is called "notional" because it is not intended to represent any particular truck. Vehicles considered to be representative of the newer Formula B configurations were investigated through the analysis of weigh-in-motion data and other truck and survey data obtained from the states (refer to NCHRP Project 12-63 Final Report). Bridges that rate for the NRL loading will have adequate load capacity for all legal Formula B truck configurations up to 80 kips. Bridges that do not rate for the NRL loading should be investigated to determine posting needs using the single-unit posting loads SU4, SU5, SU6, and SU7, specified in Article 6A.8.2. These SU trucks were developed to model the extreme loading effects of single-unit SHVs with four or more axles.

The Federal tandem axle weight limit on the Interstate System is 34,000 pounds. Although tandem axles are generally defined as an axle group consisting of two axles, CFR Title 23 658.5 defines tandem axle in terms of the distance between the outer axles. That is, two or more consecutive axles whose centers may be spaced more than 40 in. and not more than 96 in. apart.

By this definition, the total weight on three axles spaced at 8 ft between the outer axles would be limited to 34,000 pounds. Spacings even slightly above 8 ft would be governed by Formula B with a maximum weight of 42,000 pounds.

The SHVs referenced in Article 6A.8.2 are load models for rating and posting that show three axle groups with a spacing of 8 ft between the outer axles and a total weight on the three axles of 42,000 lb. This is not in strict compliance with the tandem axle definition in the CFR. However, these load models provide analysis efficiency in load ratings by serving as envelope vehicles and are not meant as examples of real trucks, where the actual axle spacings may be slightly different.

In context of the previous paragraph, for all practical purposes, the SHVs are compliant with Formula B.

In the NRL loading, axles that do not contribute to the maximum load effect under consideration should be neglected. For instance, axles that do not contribute to the maximum positive moments need to be neglected or they will contribute to bending in the opposite (negative) direction. This requirement may only affect certain continuous bridges, usually with short span lengths. The drive axle spacing of 6 ft may also be increased up to 14 ft to maximize load effects. Increasing the drive axle spacing to 14 ft could result in a slight increase in moments, again in continuous span bridges.

It is unnecessary to consider more than one NRL loading per lane. Load ratings may also be performed for state legal loads that have only minor variations from the AASHTO legal loads using the live load factors provided in Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1.

SHVs, as detailed in Figures D6A-7 and 6B.7.2-2, can create higher load effects on bridges with shorter span lengths and on transverse floorbeams and thus result in lower ratings. Some state laws may preclude certain SHVs. Load ratings are required only for SHVs that can legally operate in a state.

Some states may have heavier state legal loads or design loads that cause larger load effects than the SHVs; this could preclude the need to rate bridges for SHVs. Most existing bridges have ASR, LFR, or LRFR ratings for the AASHTO Design Truck (HS-20, for example) and the AASHTO legal trucks (Routine Commercial Vehicles Type 3, Type 3S2, and Type 3-3).

Because it is not possible to re-rate the entire inventory all at once for the SHVs, the available controlling ratings for the AASHTO design trucks or AASHTO legal trucks can be used to screen the existing inventory of bridges that need to be rated for SHVs. The following situations illustrate lower risk bridges or bridges where the SHVs will not control the load rating and are less a priority to rate for the SHVs.

Studies of load effects for simple and continuous spans, for both flexure and shear, show:

- Bridges having an HL-93 Operating RF>1.0 need not be rated for SHVs.
- Bridges having an HS20 Operating RF > 1.20 need not be rated for SHVs.
- Bridges with a minimum Operating RF > 1.35 for the AASHTO legal trucks under ASR or LFR, or a RF > 1.35 for these trucks using LRFR, would have adequate load capacity for the SHVs as follows: SU4 and SU5 for all spans; SU6 for spans above 70 ft; and SU7 for spans above 80 ft.

Posting needs for SHVs for spans below these span limits should be verified by rating.

The 2013 Interims to the MBE made the LRFR live load factors the same for the AASHTO legal trucks and the SHVs, following the recommendations of NCHRP 12-78. LRFR ratings completed prior to that change would need to account for the different live load factors used in the ratings when using the aforementioned screening for SHVs.

6A.4.4.2.2—Live Load Factors

The LRFR provisions provide generalized live load factors for load rating that have been calibrated to provide a uniform and acceptable level of reliability. Load factors appropriate for use with the AASHTO and state legal vehicles are defined based on the traffic data available for the site.

Traffic conditions at bridge sites are usually characterized by traffic volume. The *ADTT* at a site is usually known or can be estimated. Generalized load factors are representative of bridges nationwide with similar traffic volumes.

C6A.4.4.2.2

FHWA requires an *ADTT* to be recorded on the Structural Inventory and Appraisal (SI&A) form for all bridges. In cases where site traffic conditions are unavailable or unknown, worst-case traffic conditions should be assumed.

The HS-20 truck may be substituted in place of the three AASHTO legal trucks for load rating purposes. This does not mean that the HS-20 is the worst loading. The SHVs and exclusion vehicles are more severe than HS-20.

Live load varies from site to site. More refined site-specific load factors appropriate for a specific bridge site may be estimated if more detailed traffic and truck load data are available for the site. *ADTT* and truck loads through weigh-in-motion measurements recorded over a period of time allow the estimation of site-specific load factors that are characteristic of a particular bridge site.

6A.4.4.2.3—Generalized Live Load Factors: γ_L

C6A.4.4.2.3

6A.4.4.2.3a—Generalized Live Load Factors for Routine Commercial Traffic

C6A.4.4.2.3a

Generalized live load factors for the Strength I limit state are specified in Table 6A.4.4.2.3a-1 for routine commercial traffic on structures other than buried structures. If in the Engineer's judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in this Manual when determining the safe legal load, the Engineer may increase the factors in Table 6A.4.4.2.3a-1, not to exceed the value of the factor multiplied by 1.3.

Table 6A.4.4.2.3a-1—Generalized Live Load Factors, γ_L, for Routine Commercial Traffic

Traffic Volume (One direction)	Load Factor
Unknown	1.45
$ADTT \geq 5{,}000$	1.45
$ADTT \leq 1{,}000$	1.30

Linear interpolation is permitted for *ADTT* values between 5,000 and 1,000.

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

The generalized live load factors are intended for AASHTO legal loads and state legal loads that have only minor variations from the AASHTO legal loads. Legal loads of a given jurisdiction having gross vehicle rates that are significantly greater than the AASHTO legal loads should preferably be load rated using load factors provided for routine permits in this Manual.

The generalized live load factors were derived using methods similar to that used in the *AASHTO LRFD Bridge Design Specifications*. The load factor is calibrated to the reliability analysis in the *AASHTO LRFD Bridge Design Specifications* with the following modifications:

- Reduce the reliability index from the design level to the operating (evaluation) level.
- Reduced live load factor to account for a 5-year instead of a 75-year exposure.
- The multiple presence factors herein are derived based on likely traffic situations rather than the most extreme possible cases used in the *AASHTO LRFD Bridge Design Specifications*.

The load factors listed in Table 6A.4.4.2.3a-1 were developed under the NCHRP 12-78 project and are based on a target reliability index of 2.5. Reduced load factors have been recommended based on the reliability index to live load factor comparison studies completed in NCHRP 12-78, which showed that the original live load factors included in Table 6A.4.4.2.3a-1 were producing a higher reliability index than the target reliability index of 2.5. Results of this study may be found in NCHRP Report 700.

The NCHRP 12-78 study was based on the data analysis of 1,500 bridges with redundant superstructure systems that included steel beam and girder bridges, P/S I-beam and box beam bridges, R/C T-beam, and slab bridges. When rating nonredundant superstructures, the use of these reduced load factors should be done in conjunction with the use of the system factors given in Article 6A.4.2.4 to maintain an adequate level of system safety.

Site-Specific Live Load Factors

Consideration should be given to using site-specific load factors when a bridge on a low-volume road may carry unusually heavy trucks or industrial loads due to the proximity of the bridge to an industrial site.

When both truck weight and truck traffic volume data are available for a specific bridge site, appropriate load factors can be derived from this information. Truck weights at a site should be obtained by generally accepted weigh-in-motion technology. In general, such data should be obtained by systems able to weigh all trucks without allowing heavy overweight vehicles to bypass the weighing operation.

To obtain an accurate projection of the upper tail of the weight histogram, only the largest 20 percent of all truck weights are considered in a sample for extrapolating to the largest loading event. A sufficient number of truck samples need to be taken to provide accurate parameters for the weight histogram.

For a two- or more than two-lane loading case, the live load factor for the Strength I limit state shall be taken as:

$$\gamma_L = 1.8\left[\frac{2W^* + t_{(ADTT)}1.41\sigma^*}{240}\right] > 1.30 \quad \text{(C6A.4.4.2.3a-1)}$$

For the single-lane loading case, the live load factor for the Strength I limit state shall be taken as:

$$\gamma_L = 1.8\left[\frac{W^* + t_{(ADTT)}\sigma^*}{120}\right] > 1.80 \quad \text{(C6A.4.4.2.3a-2)}$$

where:

W^* = Mean truck weight for the top 20 percent of the weight sample of trucks (kips)

σ^* = Standard deviation of the top 20 percent of the truck weight sample (kips)

$t_{(ADTT)}$ = Fractile value appropriate for the maximum expected loading event—given below in Table C6A.4.4.2.3a-1

The measured site parameters, W^* and σ^*, should be substituted into the equations for the load factors. Both single and two or more lanes (where present) shall be checked to determine the lower rating factor.

Table C6A.4.4.2.3a-1—$t_{(ADTT)}$

ADTT	Two or More Lanes	One Lane
5000	4.3	4.9
1000	3.3	4.5
100	1.5	3.9

A simplified procedure for calculating load factors suggested follows the same format used in the derivation of live load factors contained in NCHRP Report 368, *Calibration of LRFD Bridge Design Code*.

Among the variables used in evaluation, the uncertainty associated with live loads is generally the greatest. It is, therefore, a logical candidate for closer scrutiny. Much of the total uncertainty about bridge loads represents site-to-site uncertainty rather than inherent randomness in the truck-loading process itself. In design, conservative load factors are assigned to encompass all likely site-to-site variabilities in loads to maintain a uniform and satisfactory reliability level. In evaluation, much of the conservatism associated with loads can be eliminated by obtaining site-specific information. The reduction in uncertainty could result in reduced load factors for evaluation. However, if site investigation shows greater overloads, the load factor may be increased rather than reduced.

For a specific bridge with a low-load rating using generalized load factors, further investigation of site-specific loading could result in improved load rating. In many cases, assessing the site-specific loading will require additional load data collection. Advances in weigh-in-motion technology have significantly lowered the cost of collecting load and traffic data. The cost of additional data collection should be weighed against the potential benefit that may result from improved load ratings.

Permit vehicles should be removed from the stream, if possible, when estimating statistical parameters. WIM data on trucks should be unbiased and should capture any seasonal, weekly, or daily fluctuations. The data collection period should be sufficient to capture at least 400 trucks in the upper 20 percent of the weight sample for the site. Additional guidance on determining site-specific load factors can be found in the NCHRP Report 454.

Alternate Approach to Deriving Site-Specific Load Factors from WIM Data

The commentary above on site specific live load factors describes a simplified procedure for calculating load factors using statistics for the heaviest 20 percent of the truck weight spectra to model the maximum load effects expected on typical bridges. It assumes that the heaviest trucks follow a Normal distribution and that 1 in 15 trucks will cross the bridge side-by-side. Studies performed in NCHRP 12-76 have shown that these simplifying assumptions may not be valid in all cases. NCHRP Project 12-76 has proposed a more consistent approach for using WIM data for live load modeling, which takes into consideration the actual distribution of the truck traffic data, including the actual configurations and the actual percentage of side-by-side crossings.

Calculating Maximum Load Effect, L_{max}

The estimation of the maximum load effect, L_{max}, expected over a 5-year bridge evaluation period can be executed through a variety of methods. The one implemented herein is based on the assumption that the tail end of the histogram of the maximum load effect over a given return period approaches a Gumbel distribution as the return period increases. The method assumes that the WIM data is assembled over a sufficiently long period of time to ensure that the data is representative of the tail end of the truck weight histograms. Equations in closed form for statistical projections can be utilized provided the tail of the load effect histogram for the original population of trucks approaches the tail end of a Normal distribution. A Normal distribution fit can usually be obtained for the top five percent of data points.

The process begins by assembling the measured load effects histograms (moment effect or shear force effect) for single lane events and two-lane events for a suite of simple and continuous spans. Then calculate cumulative distribution function for each load effect and obtain the standard deviate of the cumulative function. A plot is made of the upper five percent of the values of the normal deviate versus the load effect, X. The slope, m, and intercept, n, of the best fit regression line provides the statistics for the normal distribution that best fits the tail end of the distribution.

Calculation of L_{max} for each span using equations in closed form for statistical projections can be performed as follows:

- The mean of Normal that best fits the tail end of the distribution: $\mu_{event} = -n/m$.
- The standard deviation of the best fit Normal: $\sigma_{event} = 1/m$.
- Let n_{day} = total number of trucks per day
- For 5 years: $N = n_{day} \times 365 \times 5$
- The most probable value, u_N, for the Gumbel distribution that models the maximum value in 5 years L_{max} is given as:

$$u_N = \mu_{event} + \sigma_{event} \times \left[\sqrt{2\ln(N)} - \frac{\ln(\ln(N)) + \ln(4\pi)}{2\sqrt{2\ln(N)}} \right] \quad \text{(C6A.4.4.2.3a-3)}$$

- The dispersion coefficient for the Gumbel distribution that models the maximum load effect L_{max} is given as:

$$a_N = \frac{\sqrt{2\ln(N)}}{s_{event}} \quad \text{(C6A.4.4.2.3a-4)}$$

- The mean value of L_{max} is given as:

$$L_{max} = \mu_{max} = u_N + \frac{0.577216}{\alpha_N} \quad \text{(C6A.4.4.2.3a-5)}$$

The next step in the derivation of live load factors applies the projected maximum load effect, L_{max}, from the

WIM data in Equations 30 and 34 contained in NCHRP Report 454, *Calibration of Load Factors for LRFR Evaluation*. The general expressions for site-specific live load factors for the Strength I limit state, following the same format as the derivation of the LRFR load factors are as follows:

Two or more lanes loading case:

$$\gamma_L = \left[\frac{L_{\max 2}}{\mathrm{LE}_2}\right] 1.8 > 1.3 \qquad \text{(C6A.4.4.2.3a-6)}$$

One lane loading case:

$$\gamma_L = \left[\frac{L_{\max 1}}{\mathrm{LE}_1}\right] 1.8 > 1.8 \qquad \text{(C6A.4.4.2.3a-7)}$$

where:

$L_{\max 1}$ = Maximum single-lane load effect expected over a 5-year period

$L_{\max 2}$ = Maximum two or more lane load effect expected over a 5-year period

LE_1 = Maximum load effect from one 120 K, 3S2 truck

LE_2 = Maximum load effect from two 120 K, 3S2 trucks side by side

6A.4.4.2.3b—Generalized Live Load Factors for Specialized Hauling Vehicles

Generalized live load factors for the Strength I limit state are given in Table 6A.4.4.2.3b-1 for the NRL rating load and posting loads for specialized hauling vehicles satisfying Formula B specified in Article 6A.8.2 on structures other than buried structures. If in the Engineer's judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in this Manual when determining the safe legal load, the Engineer may increase the factors in Table 6A.4.4.2.3b-1, not to exceed the value of the factor multiplied by 1.3.

Table 6A.4.4.2.3b-1—Generalized Live Load Factors, γ_L ,for Specialized Hauling Vehicles

Traffic Volume (One Direction)	Load Factor
Unknown	1.45
ADTT ≥ 5,000	1.45
ADTT = 1,000	1.30

Linear interpolation is permitted for *ADTT* values between 1,000 and 5,000.

C6A.4.4.2.3b

The live load factors provided in these specifications account for the multiple-presence of two heavy trucks side-by-side on a multi-lane bridge as well as the probability that trucks may be loaded in such a manner that they exceed the corresponding legal limits. Using the reliability analysis and data applied in AASHTO LRFD and LRFR Specifications show that the live load factor should increase as the ADTT increases. The increase in γ_L with ADTT is provided in Table 6A.4.4.2.3b-1 for routine commercial traffic. The same consideration for SHVs using field data and assumptions for the percent of SHVs in the traffic stream led to the γ_L factors in Table 6A.4.4.2.3b-1 for SHVs.

The load factors listed in Table 6A.4.4.2.3b-1 were developed under the NCHRP 12-78 project and are based on a target reliability index of 2.5. Reduced load factors have been recommended based on the reliability index to live load factor comparison studies completed in NCHRP 12-78, which showed that the original live load factors included in Table 6A.4.4.2.3b-1 were producing a higher reliability index than the target reliability index of 2.5. Results of this study may be found in NCHRP Report 700.

The NCHRP 12-78 study was based on the data analysis of 1,500 bridges with redundant superstructure systems that included steel beam and girder bridges, P/S I-beam and box beam bridges, R/C T-beam, and slab bridges. When rating nonredundant superstructures, the use of these reduced load factors should be done in conjunction with the use of the system factors given in Article 6A.4.2.4 to maintain an adequate level of system safety.

6A.6—STEEL STRUCTURES

6A.6.1—Scope

The provisions of Article 6A.6 shall apply to the evaluation of steel and wrought-iron components of bridges. The provisions of Article 6A.6 apply to components of straight or horizontally curved I-girder bridges and straight or horizontally curved single or multiple closed-box or tub girder bridges.

C6A.6.1

LRFD Design Article 6.10 provides a unified approach for consideration of combined major-axis bending and flange lateral bending from any source in I-sections. In load rating, flange lateral bending effects from wind and deck placement need not be considered.

Bridges containing both straight and curved segments are to be treated as horizontally curved bridges.

6A.6.2—Materials

6A.6.2.1—Structural Steels

The minimum mechanical properties of structural steel given in Table 6A.6.2.1-1 may be assumed based on the year of construction of the bridge when the specification and grade of steel are unknown.

Table 6A.6.2.1-1—Minimum Mechanical Properties of Structural Steel by Year of Construction

Year of Construction	Minimum Yield Point or Minimum Yield Strength, F_y, ksi	Minimum Tensile Strength, F_u, ksi
Prior to 1905	26	52
1905 to 1936	30	60
1936 to 1963	33	66
After 1963	36	66

Where it is possible to identify the designation (AASHTO or ASTM) and grade of the steel from available records, it is possible to determine the minimum yield and tensile strengths to be used for evaluation by reviewing the designation specification.

In cases where the initial evaluation suggests load capacity inadequacies, or there is doubt about the nature and quality of a particular material, the mechanical properties can be verified by testing. Mechanical properties of the material should be determined based on coupon tests. The nominal values for yield and tensile strength are typically taken as the mean test value minus 1.65 standard deviation to provide a 95 percent confidence limit. Average test values should not be used for evaluation. Guidance on material sampling for bridge evaluation is provided in Article 5.3.

Actual values of yield and ultimate tensile stresses reported on mill certificates should not be used for evaluation. Instead, the strength used should be the guaranteed minimum value as specified for the grade of steel shown. The resistance factors account for the fact that the mean strength of the actual material supplied usually exceeds the minimum specified strength.

C6A.6.2.1

Mechanical properties of eyebars, high-strength eyebars, forged eyebars, and cables vary depending on manufacturer and year of construction. When information from records is not available, microstructural and chemical analyses and hardness testing are helpful in classifying the material. In the absence of material tests, the Engineer should carefully investigate the material properties using manufacturer's data and compilation of older steel properties before establishing the yield point and tensile strength to be used in load rating the bridge.

6A.6.2.2—Pins

If the material designation for pins is unknown, the yield strength may be selected from Table 6A.6.2.2-1, based on the year of construction.

Table 6A.6.2.2-1—Minimum Yield Point of Pins by Year of Construction

Year of Construction	Minimum Yield Point, F_y, ksi
Prior to 1905	25.5
1905 through 1935	30
1936 through 1963	33
After 1963	36

6A.6.2.3—Wrought Iron

When the material designation is unknown for wrought iron, the minimum tensile strength, F_u, should be taken as 48 ksi and the minimum yield point, F_y, should be taken as 26 ksi.

Where practical, coupon tests should be performed to confirm the minimum mechanical properties used in the evaluation.

6A.6.3—Resistance Factors

Except as specified herein, resistance factors, φ, for steel members, for the strength limit state, shall be taken as specified in LRFD Design Article 6.5.4.2.

If the year of construction is prior to 1991, the resistance factor for axial compression for steel, φ_c, shall be taken as 0.90 for built-up compression members, unless it can be established that the member has not been fabricated from universal mill plate, in which case φ_c may be taken as 0.95.

For load rating of main truss member gusset plates, the resistance factors shall be taken as follows:

- Gusset plate compression $\varphi_{cg} = 0.95$
- Gusset plate chord splices $\varphi_{cs} = 0.85$
- Gusset plate shear yielding $\varphi_{vy} = 1.00$
- Gusset plate block shear rupture $\varphi_{bs} = 1.00$
- Gusset plate shear fracture $\varphi_{vu} = 0.80$
- Tension, fracture in net section $\varphi_u = 0.80$
- Tension, yielding in gross section $\varphi_y = 0.95$
- A325 and A490 bolts in shear $\varphi_s = 0.80$
- A307 bolts in shear $\varphi_s = 0.75$
- Fasteners bearing on material $\varphi_{bb} = 0.80$

C6A.6.3

For service limit states, $\varphi = 1.0$.

Users of this Manual and the ~~L L~~ *AASHTO LRFD Bridge Design Specifications* will note some differences in the specified resistance factors for main truss member gusset plates. The differences are due to the fact that a higher acceptable level of reliability can be tolerated more readily in design than in rating. In addition, the determination of the resistance factors in the two specifications was based on different dead-to-live load ratios in order to provide more lenient factors for use in rating. The resistance factors are based on the findings from NCHRP Project 12-84 (Ocel, 2013), which did not obtain sufficient data for all possible gusset-plate modes of failure to justify a difference in some of the factors that are provided in the two specifications.

The resistance factor, φ_c, for built-up members subject to axial compression is reduced from 0.95 to 0.90 if the year of construction is prior to 1991 to appropriately reflect the fact that steel built-up compression members may have been fabricated from universal mill plate. Such columns are contained in the data band of lowest strength reflected by SSRC Column Category 3P. Since only one column curve based on SSRC Column Category 2P is used for all columns, earlier versions of LRFD Design specified a

APPENDIX A6A—LOAD AND RESISTANCE FACTOR RATING FLOW CHART

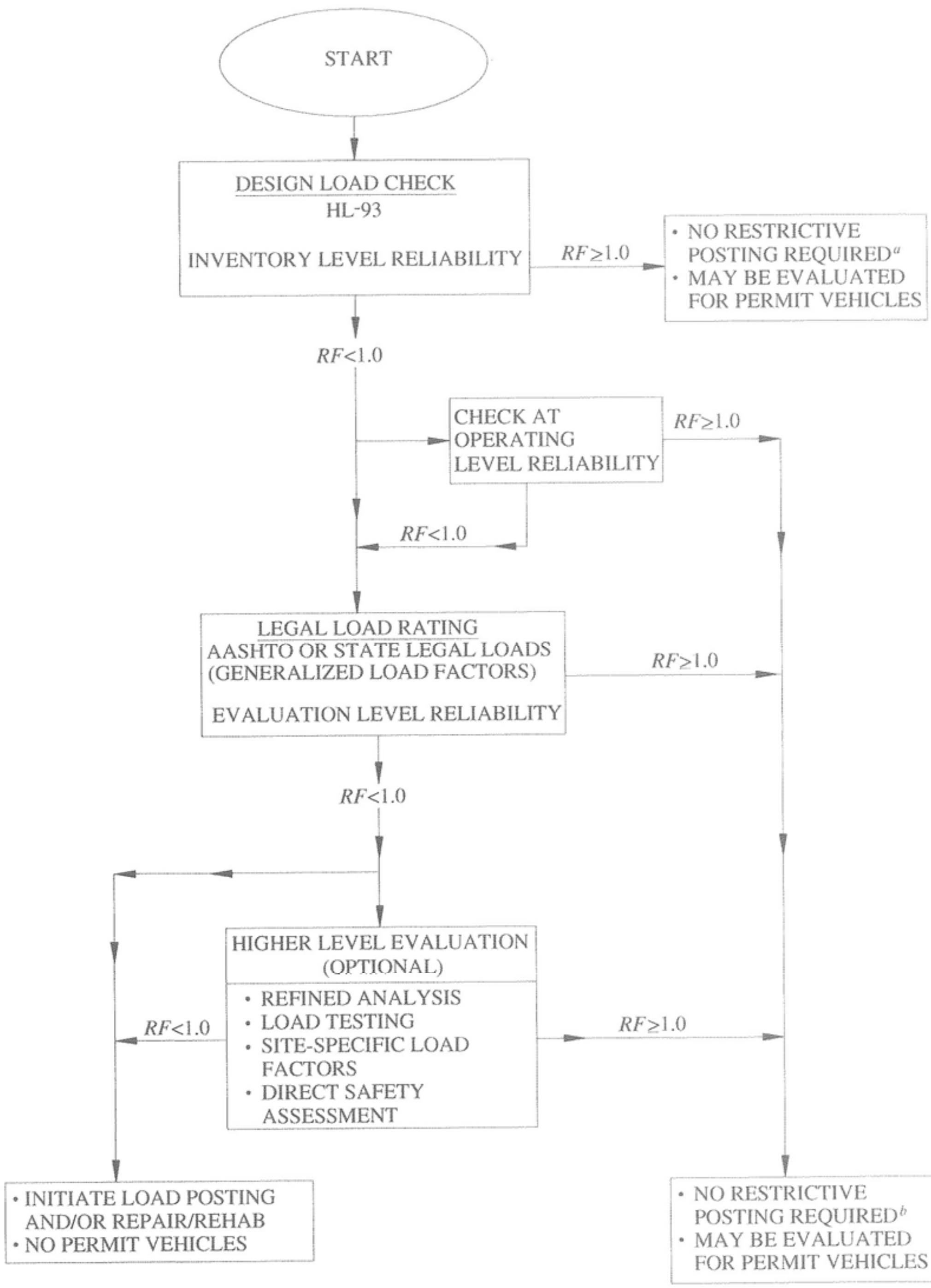

Figure A6A-1—Load and Resistance Factor Rating Flow Chart

[a] For routinely permitted commercial traffic on highways of various states under grandfather exclusions to federal weight laws

[b] For legal loads that comply with federal weight limits and Formula B

APPENDIX B6A—LIMIT STATES AND LOAD FACTORS FOR LOAD RATING

Table B6A-1—Limit States and Load Factors for Load Rating (6A.4.2.2-1)

Bridge Type	Limit State*	Dead Load *DC*	Dead Load *DW*	Design Load Inventory *LL*	Design Load Operating *LL*	Legal Load *LL*	Permit Load *LL*
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.80	—	—	—
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service I	1.00	1.00	—	—	—	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service III	1.00	1.00	~~0.80~~ Table B6A-2	—	1.00	—
	Service I	1.00	1.00	—	—	—	1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	–	Table 6A.4.5.4.2a-1

* Defined in the *AASHTO LRFD Bridge Design Specifications*.

Shaded cells of the table indicate optional checks.

Service I is used to check the $0.9F_y$ stress limit in reinforcing steel.

Load factor for *DW* at the strength limit state may be taken as 1.25 where thickness has been field measured.

Fatigue limit state is checked using the LRFD fatigue truck (see Article 6A.6.4.1).

Table B6A-2—Load Factors for Live Load for the Service III Load Combination, γ_{LL}, at the Design-Load Inventory Level (6A.4.2.2-2)

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

Table B6A-~~2~~3—Generalized Live Load Factors for Legal Loads: γ_L for Routine Commercial Traffic (6A.4.4.2.3a-1)

Traffic Volume (one direction)	Load Factor
Unknown	1.45
$ADTT \geq 5{,}000$	1.45
$ADTT = 1{,}000$	1.30

Note: Linear interpolation is permitted for other *ADTT* values between 1,000 and 5,000.

Table B6A-~~3~~4—Generalized Live Load Factors, γ_L for Specialized Hauling Vehicles (6A.4.4.2.3b-1)

Traffic Volume (one direction)	Load Factor
Unknown	1.45
$ADTT \geq 5{,}000$	1.45
$ADTT = 1{,}000$	1.30

Note: Linear interpolation is permitted for other *ADTT* values between 1,000 and 5,000**.**

Table B6A-~~4~~5—Permit Load Factors: γ_L (6A.4.5.4.2a-1)

Permit Type	Frequency	Loading Condition	*DF*[a]	*ADTT* (one direction)	Load Factor by Permit Weight Ratio[b]		
					GVW / AL < 2.0 (kip/ft)	2.0 < GVW/AL < 3.0 (kip/ft)	GVW/AL > 3.0 (kip/ft)
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5,000	1.40	1.35	1.30
				=1,000	1.35	1.25	1.20
				<100	1.30	1.20	1.15
					All Weights		
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.10		
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	All *ADTT*s	1.20		
	Multiple Trips (less than 100 crossings	Mix with traffic (other vehicles may be on the bridge)	One lane	All *ADTT*s	1.40		

Notes:

[a] *DF* = LRFD distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

[b] Permit Weight Ratio = GVW/AL where GVW = Gross Vehicle Weight and AL = Front axle to rear axle length. Use only axles on the bridge.

APPENDIX C6A—LRFD DESIGN LIVE LOAD (HL-93) (LRFD DESIGN ARTICLE 3.6.1)

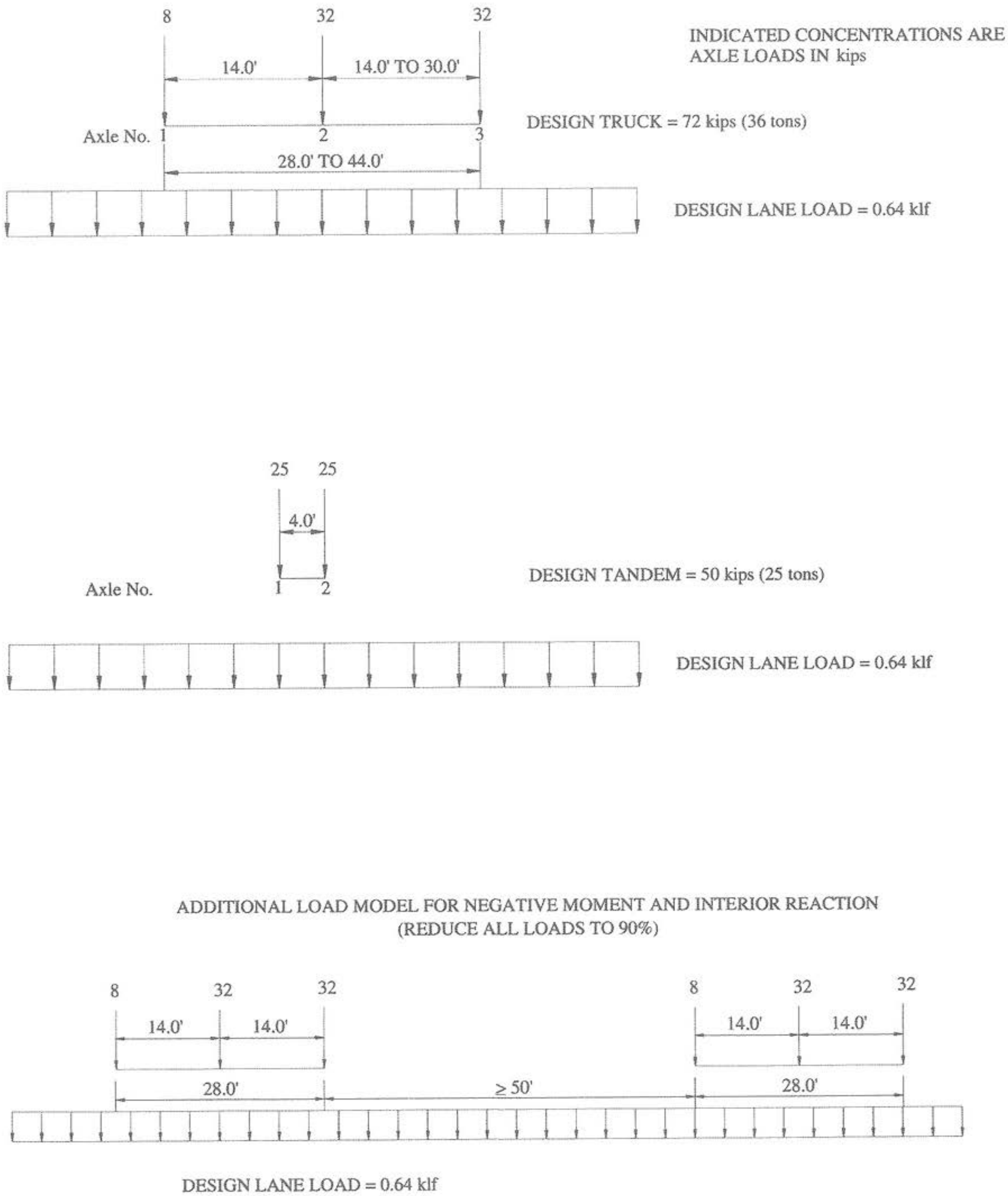

Figure C6A-1—LRFD Design Live Load (HL-93)

girders, which would result in reduction in shear capacity of reinforced concrete girders. Temperature gradient loading (*TG*) could induce significantly higher bending moments in framed structures.

Bearings' becoming nonfunctional generally leads to thermal forces being applied onto bridge elements that were not designed for such loads. Keeping bearings in good working order could prevent temperature and shrinkage forces from occurring.

6B.6.7.4—Stream Flow

Forces caused by water movements should not be considered in calculating the load rating. However, remedial action should be considered if these forces are especially critical to the structure's stability.

6B.6.7.5—Ice Pressure

Forces caused by ice pressure should be considered in the evaluation of substructure elements in those regions where such effect can be significant. If these forces are especially important, then corrective action should be recommended.

6B.6.7.6—Permanent Loads Other Than Dead Loads

Secondary effects from post-tensioning shall be considered as permanent loads.

C6B.6.7.6

In continuous post-tensioned bridges, secondary moments are introduced as the member is stressed.

6B.7—POSTING OF BRIDGES

6B.7.1—General

Weight limitations for the posted structure should conform to local regulations or policy within the limits established by this Manual. A bridge should be capable of carrying a minimum gross live load weight of three tons at Inventory or Operating level. When deciding whether to close or post a bridge, the owner may particularly want to consider the volume of traffic, the character of traffic, the likelihood of overweight vehicles, and the enforceability of weight posting. A Bridge Owner may close a structure at any posting threshold, but bridges not capable of carrying a minimum gross live load weight of three tons must be closed.

A concrete bridge with unknown reinforcement need not be posted for restricted loading when it has been carrying normal traffic for an appreciable length of time and shows no distress. In other cases, a concrete bridge with no visible signs of distress but whose calculated load rating indicates the bridge needs to be posted can be alternately evaluated through load testing.

If a concrete culvert with depths of fill 2.0 ft or greater with known details or with unknown components (such as culverts without plans) has been carrying normal traffic for an appreciable period and is in fair or better condition, as determined by a physical inspection of the culvert by a qualified inspector and documented in the inspection report, the culvert may be assigned an inventory load rating factor of 1.0 and operating load rating factor of 1.67 for the HS-20 design load and

C6B.7.1

Most structures which require weight limits below statutory limits are old and designed for light loads, and/or are weak as a result of damage. With some exceptions, the weaker elements of older bridges are usually in the superstructure, not in the piers or abutments.

The simplified modeling approach used for culvert ratings tends to produce conservative force demands. Buried structures carry vertical loads through a combination of internal capacity and soil arching around the structure; this is termed soil–structure interaction. Soil–structure interaction effects are neglected when establishing culvert load ratings. It is therefore not uncommon to observe satisfactory performance of in-service culverts even when analytical ratings may show insufficient capacity for normal traffic.

There may be circumstances where the Bridge Owner may utilize load levels higher than those used for Inventory rating, in order to minimize the need for posting of bridges. In no case shall the load levels used be greater than those permitted by the Operating Rating.

For those bridges supporting large dead loads, the use of the Load Factor or Load and Resistance Factor rating methods may result in a live load capacity greater than that determined based on the allowable stress rating method.

Bridges which use a load level above the Inventory Level should be subject to more frequent, competent inspections. Several factors may influence the selection of the load level. For instance:

need not be posted for restricted loading. The load rating shall be documented in the bridge file.

The total load on any member caused by dead load, live load, and such other loads deemed applicable to the structure, should not exceed the member capacity as set forth in this Manual or in the rating report. When it becomes necessary to reduce the allowable live loads in order to conform to the capacity of a structure, such a reduction should be based on the assumption that each axle load maintains a proportional relation to the total load of the vehicle or vehicle combination.

1. The factor of safety commonly used in the design or Inventory level rating may have provided for an increase in traffic volume, a variable amount of deterioration and extreme conditions of live loading.
2. The factor of safety used in rating existing structures must provide for unbalanced distribution of vehicle loads, and possible overloads. For both design and rating, factors of safety must provide for lack of knowledge as to the distribution of stresses, possible minimum strength of the materials used as compared to quoted average values, possible differences between the strength of laboratory test samples and the material under actual conditions in the structure, and normal defects occurring in manufacture or fabrication.
3. A higher safety factor for a bridge carrying a large volume of traffic may be desirable as compared with the safety factor for a structure carrying few vehicles, especially if the former includes a high percentage of heavy loads.
4. The probability of having a series of closely spaced vehicles of the maximum allowed weight should be considered. This effect becomes greater as the maximum allowed weight for each unit becomes less.
5. Lower load levels may be warranted for nonredundant metal bridge elements due to the consequences of failure. Exceptions may be elements of riveted construction and all floor beams, provided they are in good condition. Examples of nonredundant elements are welded or rolled two-girder bridges, truss members, or pinned eyebar trusses and truss members on welded trusses.
6. Bridges with extensive material losses may warrant a lower load level because of the greater uncertainty in evaluating present strength capacity. This is especially true if the loss in material is in a highly stressed area.
7. Sites for which it is suspected that there are frequent truck overloads should be considered for lower load levels unless enforcement methods are put in place.
8. The ratio of dead load to live load may have an influence on the selection of appropriate load level. Structures with high ratios of dead to live load and for which there are no visible signs of distress may be considered for the higher load levels.

6B.7.2—Posting Loads

The live load to be used in the rating Eq. 6B.4.1-1 for posting considerations should be any of the three typical legal loads shown in Figure 6B7.2-1, any of the four single-unit legal loads shown in Figure 6B7.2-2 or State legal loads. For spans over 200 feet in length, the selected legal load should be spaced with 30 feet clear distance between vehicles to simulate a train of vehicles in one lane and a single vehicle load should be applied in the adjacent lanes(s). When the maximum legal load under state law

C6B.7.2

Trucks weighing up to 80,000 lb are typically allowed unrestricted operation and are generally considered "legal" provided they meet weight guidelines of Federal Bridge Formula B (Formula B). In the past, the maximum legal weight for short wheelbase trucks was usually determined by Formula B rather than by the 80,000-lb gross weight limit. Since the adoption of the AASHTO family of three legal loads, the trucking industry has introduced specialized single-unit trucks with closely spaced multiple axles that exceeds the safe load capacity of a bridge, restrictive posting shall be required.

make it possible for these short wheelbase trucks to carry the maximum load of up to 80,000 lb and still meet Formula B. The current AASHTO legal loads selected at the time to closely match the Formula B in the short, medium, and long truck length ranges do not represent these newer axle configurations. These specialized hauling vehicles cause force effects that exceed the stresses induced by HS-20 by up to 22 percent and by the Type 3, 3S2, or 3-3 posting vehicles by over 50 percent in certain cases. The shorter spans are most sensitive to axle configurations.

The Notional Rating Load, *NRL*, shown in Figure 6B7.2-3 may be used as a screening load model for single-unit trucks that meet Formula B. Bridges that result in $RF \geq 1.0$ for the *NRL* loading will have adequate load capacity for all legal single-unit Formula B truck configurations up to 80,000 lb.

The *NRL* loading represents a single load model that will envelop the load effects on simple and continuous span bridges of the worst possible Formula B single-unit truck configurations up to 80,000 lb. It is called "notional" because it is not intended to represent any particular truck. Vehicles considered to be representative of the newer Formula B configurations were obtained through the analysis of weigh-in-motion data and other truck and survey data obtained from the States. The single *NRL* load model with a maximum gross weight of 80,000 lb produces moments and shears that exceed the load effects for a series of 3- to 8-axle single-unit trucks allowed to operate under current federal weight laws (NCHRP Report 575).

In the *NRL* loading, axles that do not contribute to the maximum load effect under consideration shall be neglected. For instance, axles that do not contribute to the maximum positive moments need to be neglected or they will contribute to bending in the opposite (negative) direction. This requirement may only affect certain continuous bridges, usually with short span lengths. The drive axle spacing of 6 ft may also be increased up to 14 ft to maximize load effects. Increasing the drive axle spacing to 14 ft could result in a slight increase in moments for continuous bridges.

For bridges with $RF < 1.0$ for the *NRL* loading, a posting analysis should be performed to resolve posting requirements for single-unit multi-axle trucks. While a single envelope *NRL* loading can provide considerable simplification of load-rating computations, additional legal loads for posting are needed to give more accurate posting values. Certain multi-axle Formula B configurations that cause the highest load effects appear to be common only in some states, and they should not lead to reduced postings in all states.

Setting weight limits for posting often requires the evaluator to determine safe load capacities for legal truck types that operate within a given state, in accordance with State posting practices. The four single-unit Formula B legal loads shown in Figure 6B.7.2-2 include the worst 4-axle (SU4), worst 5-axle (SU5), worst 6-axle (SU6), and worst 7-axle (SU7) trucks (7-axle is also representative of 8-axle trucks) identified in the NCHRP 12-63 study. This series of loads affords the evaluator the flexibility of selecting only posting loads that model commercial

Formula B trucks in a particular state or jurisdiction.

The more compact four- and five-axle trucks that produce the highest moment or shear per unit weight of truck will often govern the posting value (result in the lowest weight limit). States that post bridges for a single tonnage for all single-unit trucks may consider it desirable to reduce the number of new posting loads that need to be evaluated. Here it would be appropriate to use truck SU5 as a single representative posting load for the series of Formula B truck configurations with 5 to 8 axles. This simplification will introduce added conservatism in posting, especially for short-span bridges. It should be noted that situations could arise where a bridge may have a $RF > 1.0$ for SU5 but may not rate ($RF < 1.0$) for SU6 or SU7. Here the SU5 load model is being utilized to determine a single posting load for a bridge that has adequate capacity for SU5 but not for the heavier trucks.

SHVs, as detailed in Figures D6A-7 and 6B.7.2-2, can create higher load effects on bridges with shorter span lengths and on transverse floorbeams and thus result in lower ratings. Some state laws may preclude certain SHVs. Load ratings are required only for SHVs that can legally operate in a state.

Some states may have heavier state legal loads or design loads that cause larger load effects than the SHVs; this could preclude the need to rate bridges for SHVs. Most existing bridges have ASR, LFR, or LRFR ratings for the AASHTO Design Truck (HS-20, for example) and the AASHTO legal trucks (Routine Commercial Vehicles Type 3, Type 3S2, and Type 3-3).

Because it is not possible to re-rate the entire inventory all at once for the SHVs, the available controlling ratings for the AASHTO design trucks or AASHTO legal trucks can be used to screen the existing inventory of bridges that need to be rated for SHVs. The following situations illustrate lower risk bridges or bridges where the SHVs will not control the load rating and are less a priority to rate for the SHVs.

Studies of load effects for simple and continuous spans, for both flexure and shear, show:

- Bridges having an HL-93 Operating RF > 1.0 need not be rated for SHVs.
- Bridges having an HS20 Operating RF > 1.20 need not be rated for SHVs.
- Bridges with a minimum Operating RF > 1.35 for the AASHTO legal trucks under ASR or LFR, or a RF > 1.35 for these trucks using LRFR, would have adequate load capacity for the SHVs as follows: SU4 and SU5 for all spans; SU6 for spans above 70 ft; and SU7 for spans above 80 ft. Posting needs for SHVs for spans below these span limits should be verified by rating.

The 2013 Interims to the MBE made the LRFR live load factors the same for the AASHTO legal trucks and the SHVs, following the recommendations of NCHRP 12-78. LRFR ratings completed prior to that change would need to account for the different live load factors used in the ratings when using the aforementioned screening for SHVs.

A1A.1.3.1b—Composite Dead Loads, DC_2

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) × (0.150 kcf) (2 curbs / 4 beams)
= 0.063 kip/ft

Parapet = [(6 in x 19 in) + (18 in x 12 in)]/144 × (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 $= M_{DC2} = \dfrac{0.245(65)^2}{8}$ =129.4 kip-ft at midspan

Dead Load Shear $= V_{DC2} = \dfrac{0.245(65)}{2}$ = 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 LRFD Design 4.6.2.2.1

$$K_g = n\left(I + Ae_g^2\right)$$ LRFD Design Eq. 4.6.2.2.1-1

in which $n = \dfrac{E_B}{E_D}$ LRFD Design Eq. 4.6.2.2.1-2

$$E_D = 33{,}000\left(w_c\right)^{1.5}\sqrt{f_c'}$$ LRFD Design Eq. C5.4.2.4-2

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

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

$E_B = 29{,}000$ ksi

Beam + Cover Plate

I = 8,291.6 in.^{4}

A = 44.82 in.^{2} Distance to centroid from top fiber = 19.018

$$e_g = \frac{1}{2}(7.25) + 19.02 = 22.\underline{6}43 \text{ in.}$$

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

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

A1A.1.4.1a—Distribution Factor for Moment, g_m (LRFD Design Table 4.6.2.2.2b-1)

Range of Applicability Check:

a.	*S = 7.3333 ft*	*(meets $3.5 \le S \le 16$)*
b.	*t_s = 7.25 in*	*(meets $4.5 \le t_s \le 12.0$)*
c.	*L = 65.00 ft*	*(meets $20.0 \le L \le 240$)*
d.	*N_b = 4*	*(meets $N_b \ge 4$)*
e.	*K_g = 287,349*	*(meets $10000 \le K_g \le 7{,}000.000$)*

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.

$$\frac{K_g}{12.0Lt_s^3} = \frac{287{,}354.0}{12.0 \times 65 \times 7.25^3} = 0.967$$

One Lane Loaded LLDF:

$$g_{m1} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

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

$$= 0.460$$

Two or More Lanes Loaded LLDF:

$$g_{m2} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

$$= 0.075 + \left(\frac{7.33333}{9.5}\right)^{0.6}\left(\frac{7.33333}{65}\right)^{0.2}\left(0.967\right)^{0.1}$$

$$= 0.627 > g_{ml} = 0.460$$

So, use $g_m = 0.627$

$$M_p = \left(\frac{\overline{Y}^2 P_s}{2t_s}\right) + [P_{rt}d_{rt} + P_{rb}d_{rb} + P_c d_c + P_w d_w + P_t d_t]$$ LRFD Design Table D6.1-1

$$= \left(\frac{7.13^2 \times 1,626.9}{2 \times 7.25}\right) + \left[0 + 0 + 354.3 \times 0.5475 + 655.4 \times 16.670 + 590.4 \times 33.089\right]$$

= 36,359.1 kip-in. or 3,030.0 kip-ft

$D_p \not\leq 0.1D_t$ 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 kip-ft

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

$W33 \times 130$ Rolled section, no stiffeners.

$D = d - 2tf$ (Clear distance between flanges)
= 33.1 – 2 × 0.855
= 31.39 in.
t_w = 0.580 in.
F_{yw} = 36.00 ksi

Unstiffened web and therefore,

The shear buckling coefficient, $k = 5.00$ LRFD Design 6.10.9.2

$$\frac{D}{t_w} = \frac{31.39}{0.580} = 54.10$$

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{29,000 \times 5.00}{36.0}} = 71.08$$ LRFD Design Eq. 6.10.9.3.2-4

So, $\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_{yw}}}$ and therefore $C = 1.00$

Shear Capacity $V_n = V_r = CV_p$ LRFD Design Eq. 6.10.9.2-1

$V_p = 0.58F_{yw}Dt_w$ LRFD Design Eq. 6.10.9.2-2

$V_p = 0.58 \times 36.0 \times 31.39 \times 0.580$

= 380.15 kip

Shear Capacity at the End panel $= CV_P$

$=1.00 \times 380.15$

$= 380.15$ kip

A1A.1.5.4—Demand Summary for Interior Stringer

Table A1A.1.5.4-1

	Dead Load DC_1	Dead Load DC_2	Live Load Distribution Factor	Dist. Live Load + Impact	Nominal Capacity
Moment, kip-ft	439.90	129.40	0.627	954.10	2,873.0
Shear, kips	27.10	8.0	0.767	78.90	380.15

A1A.1.6—General Load-Rating Equation

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DN})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$ Eq. 6A.4.2.1-1

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

1. Resistance Factor, φ LRFD Design 6.5.4.2
   $\varphi = 1.00$ for flexure and shear

2. Condition Factor, φ_c 6A4.2.3
   $\varphi_c = 1.0\ 0$ Member is in good condition. NBI Item 59 = 7.

3. System Factor, φ_s 6A.4.2.4
   $\varphi_s = 1.00$ 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 $C = (\varphi_c)(\varphi_s)(\varphi)R_n$

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

A1A.1.8.1a—Inventory Level

Load Factors Table 6A.4.2.2-1

γ_{DC} 1.25

γ_{DW} 1.50

γ_{LL} 1.75

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

$= 1.29\cancel{754}\underline{449}$

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: $$RF = \frac{(1.0)(1.0)(1.0)(3\cancel{6}\underline{8}0.15) - (1.25)(27.1 + 8.0)}{(1.75)(78.9)}$$

$$= 2.435$$

A1A.1.8.1b—Operating Level

Load	Load Factor γ
DC	1.25
DW	1.50
LL	1.35

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

$$= 1.677$$

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 $f_R = 0.95 R_h F_{yf}$ $(f_\ell = 0)$ LRFD Design Eq. 6.10.4.2.2-2

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

R_h = 1.0 for non-hybrid sections LRFD Design 6.10.1.10.1

f_R = 0.95 × 1.0 × 36

$= 34.2$ ksi

$f_{DC} = f_{DC_1} + f_{DC_2}$

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

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

$= 9.363 + 2.147 = 11.510$ ksi

$$f_{LL+IM} = \frac{M_{LL+IM}}{S_b n}$$

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

$\gamma_{LL} = 1.30 \qquad \gamma_{DC} = 1.00$ — 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

$\gamma_{LL} = 1.00 \qquad \gamma_{DC} = 1.00$ — 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.

$$RF = 1.208 \times \frac{1.30}{1.00}$$

$= 1.570$

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 thicknesss = 0.855 in. is greater than 0.8 in. — LRFD Design Table 6.6.1.2.3-1

If $2.2(\Delta f)_{tension} > f_{dead\text{-}load\ compression}$, the detail may be prone to fatigue. — Eq. 7.2.3-1

$f_{dead\text{-}load\ compression}$ — 7.2.3

= 0.0 at cover plate at all locations because beam is a simple span and cover plate is located in the tension zone

$\therefore$ must consider fatigue; determine if the detail possesses infinite life.

Composite section properties without cover plate:

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MBE3E2