

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

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Summary of Errata for MBE-3

Page	Original Text	Corrected Text						
6-18	Table 6A.2.2-1, Row “Prestressed Concrete Service III,” Column “Design Load Inventory:” 0.80	Table 6A.2.2-1, Row “Prestressed Concrete Service III,” Column “Design Load Inventory:” ~~0.80~~ Table 6A.4.2.2-2						
6-18	(None)	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><u>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</u></td><td><u>1.0</u></td></tr><tr><td><u>All other prestressed concrete components</u></td><td><u>0.8</u></td></tr></table>	Component	γ_{LL}	<u>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</u>	<u>1.0</u>	<u>All other prestressed concrete components</u>	<u>0.8</u>
Component	γ_{LL}							
<u>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</u>	<u>1.0</u>							
<u>All other prestressed concrete components</u>	<u>0.8</u>							
6-21	C6A.4.3.3	(Deleted heading with no contents)						
6-25	(None)	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.						

Page	Original Text	Corrected Text						
		• 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.”						
6-56	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.	C6A.6.3 ... Users of this Manual and the <i>AASHTO LRFD Bridge Design Specifications</i> will note some differences in the specified resistance factors for main truss member gusset plates.						
6-94	Table B6A-1, Row “Prestressed Concrete, Service III,” Column “Design Load: Inventory”: 0.80	Table B6A-1, Row “Prestressed Concrete, Service III,” Column “Design Load: Inventory”: ~~0.80~~ <u>Table B6A-2</u>						
6-94	(None)	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>	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
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							
6-94	“Table B6A-2”	“Table B6A-3” (Renamed table)						
6-95	“Table B6A-3”	“Table B6A-4” (Renamed table)						
6-95	“Table B6A-4”	“Table B6A-5” (Renamed table)						
6-147	(None)	Added to Section 6B.7.1, after first paragraph:						

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 <i>A1A.1.4.1</i>: $e_g = \frac{1}{2} (7.25) + 19.02 = 22.43 \text{ in.}$	$e_g = \frac{1}{2} (7.25) + 19.02 = 22.\underline{6}43 \text{ in.}$
A-12	In Article <i>A1A.1.8.1a</i>: Flexure: = $RF = \frac{(1.0)(1.0)(1.0)(2,873.0) - (1.25)(439.9 + 129.4)}{(1.75)(954.10)}$ $= 1.297\underline{5}4$	Flexure: = $RF = \frac{(1.0)(1.0)(1.0)(2,873.0) - (1.25)(439.9 + 129.4)}{(1.75)(954.10)}$ $= 1.294\underline{4}9$
A-13	In Article <i>A1A.1.8.1a</i>: Shear: $RF =$ $\frac{(1.0)(1.0)(1.0)(3\u>60.15) - (1.25)(27.1 + 8.0)}{(1.75)(78.9)}$	Shear: $RF =$ $\frac{(1.0)(1.0)(1.0)(3\u>60.15) - (1.25)(27.1 + 8.0)}{(1.75)(78.9)}$

- 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		Legal Load γ_{LL}	Permit Load γ_{LL}
				Inventory γ_{LL}	Operating γ_{LL}		
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.80	—	—	—
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service I	1.00	1.00	—	—	—	1.00
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 ($\phi_c \leq 1.0$).

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.4.2.3, 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 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 $\phi_s = 1.0$ is to be applied when checking shear at the strength limit state.

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

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.

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 $\phi_s = 1.0$ 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.

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

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

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.

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).

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