# CHAPTER 7 - INTERIM REVISIONS

### 7.1.1 Purpose

This chapter establishes policy to be used by MassDOT and Consultant Rating Engineers in determining the safe load carrying capacity of newly built and existing bridges. The development of a bridge load rating requires engineering judgment and the implementation of sound engineering principles that are commonly accepted in the field of bridge engineering.

Load rating for a bridge shall be performed using the same methodology that was used for its design. The majority of existing bridges in the Commonwealth of Massachusetts were designed using the Allowable Stress Design (ASD) method. In general, the Central Artery bridges were designed using the Load Factor Design (LFD) method and more recently all bridges have been designed using the Load and Resistance Factor Design (LRFD) method. It is the responsibility of the Rating Engineer to determine the method that will be used for development of the load rating.

The initial load rating, typically submitted by the Design Engineer as the Rating Engineer, shall be performed in accordance with the methodologies of the MassDOT Bridge Manual that was in effect at the time of the completion of design, but submitted in accordance with the latest requirements. The Rating Engineer shall clearly identify which version of the Bridge Manual was used for reference in the Available Plans, Inspection Reports, and References section of the Load Rating Report. Any unique assumptions that were used during the design process shall be identified by the Rating Engineer so that future load ratings can include these assumptions, if MassDOT and the Rating Engineer agree that they are still valid.

Load ratings are performed to evaluate and identify substandard bridges requiring posting, and to assist in determining the bridges requiring rehabilitation or replacement. Load ratings shall not be used as the sole basis for project scoping, but shall be used in combination with the available inspection information, field verification, and engineering judgement. Additionally, FHWA requires reporting of bridge load ratings on an annual basis.

Massachusetts General Laws require the determination of the maximum weight of vehicle with load which a bridge will safely carry for the Rating Vehicles as defined in the sections that follow.

FHWA memoranda, specifically Load Rating of Specialized Hauling Vehicles, dated November 15, 2013, and Load Rating for the FAST Act's Emergency Vehicles, dated November 3, 2016, identify additional Rating Vehicles to be load rated. FHWA has information regarding the load rating of these vehicles on the Bridges and Structures page of their Program Policy & Guidance Center website. <https://www.fhwa.dot.gov/bridge/loadrating/>

### 7.1.2 Rating Specifications

All bridges shall be rated in accordance with the provisions of the current *AASHTO Manual for Bridge Evaluation*, including all Interims except where modified by this Bridge Manual.

Section 6 of the *AASHTO Manual for Bridge Evaluation* is divided into two parts. Part A of the *AASHTO Manual for Bridge Evaluation* incorporates provisions specific to the Load and Resistance Factor Rating (LRFR) methodology, whereas Part B provides rating criteria and procedures for the Allowable Stress Rating (ASR) and Load Factor Rating (LFR) methods of evaluation.

In the articles that follow a designation of “A” or “B” is used to differentiate between the LRFR methodology and the ASR/LFR methodologies, respectively.

7.1.7.2B In accordance with requirements of the December 1995 FHWA NBIS Coding Guide an Inventory and Operating Rating shall be obtained for the HS20 vehicle using the Load Factor Rating method. Timber and stone masonry structures are exempt from this requirement and shall be reported based upon the Allowable Stress Rating method. The gross tonnage is reported on the Summary of Bridge Rating of the rating report for Item 64 and Item 66. Since MassDOT reports these Items in metric units, the gross tonnage results from the rating calculations performed in U.S. Customary units shall be converted to metric units. The HS20 truck gross tonnage shall be converted to an MS18 gross tonnage using a conversion factor of 0.9, instead of the exact conversion of 0.907185 metric tons per U.S. short ton. An MS Equivalent to Items 64 and 66 shall be calculated by dividing the MS18 gross tonnage by 1.8. The resulting MS18 metric ton ratings and MS Equivalent shall be specified on the Summary of Bridge Rating in the spaces provided for Item 64 and Item 66. The header in the MS18 FHWA NBIS CODING GUIDE table in the Summary of Bridge Rating shall be revised to note Allowable Stress Rating rather than Load Factor Rating for timber and stone masonry structures.

Note that when using BrR to perform the load rating, the default rating method in the .XML file submitted shall be saved in accordance with the method used for rating and not the one required for MS18 reporting.

#### 7.2.2.7 Bridge Decks. Reinforced concrete decks and exodermic bridge decks supported by girders or floorbeams do not require load ratings unless their condition warrants investigation. If the Rating Engineer considers that the deck should be rated based upon condition or other concerns, he/she shall consult with the Bridge Section regarding the potential inclusion of the deck rating in their proposal.

In the event that the reinforced concrete deck needs to be rated, the Rating Engineer shall check punching shear under wheel loads and not check flexure, as discussed in *AASHTO Manual for Bridge Evaluation*, Article C6.1.5.1.

* Timber decks require a load rating.
* Vaulted sidewalks that have thin (less than 5½”) deck slabs shall require a punching shear check.
* Metal grid decks, including concrete filled or partially filled decks, do not require a load rating, but purlins supporting the metal grid decking shall be rated.

Rating alternative deck types (i.e. orthotropic, sandwich-plate, FRP, etc.) will be considered on a case-by-case basis and shall be discussed with the Bridge Section prior to developing a scope and fee for the rating.

#### 7.2.4.2A Bridges shall be rated for Inventory and Operating Level with the HL-93 design live load, as defined by Part A of the *AASHTO Manual for Bridge Evaluation*. The resulting rating factors for roadway beams shall be specified on the Summary of Bridge Rating in the spaces provided for Item 64 and Item 66. Sidewalk beam rating factors shall not be reported in the Summary. Due to limitations in NBIS coding, the rating factors shall not be reported at a value greater than 3.0 at Operating (Item 64) and 2.9 at Inventory (Item 66). If the calculated rating factors shown in the Breakdown of Bridge Rating exceed 3.0, report 3.0 in the Summary of Bridge Rating.

These bridges shall be rated for the H20, Type 3, Type 3S2, SU4, SU5, SU6, and SU7 vehicles outlined above. The ratings and the corresponding gross tonnage for each of these vehicles shall be determined based on the Load Factors for Design Load, so that they can be reported at both the Inventory and Operating Level for the Limit States contained in Table B6A-1 of the *AASHTO Manual for Bridge Evaluation*. The Fatigue Load limit state shall not be evaluated If limit states other than Strength I control, only report values if the rating factor is less than 1.0.

#### 7.2.4.2B Bridges shall be rated based on the method used for design. For most bridges the ratings will be performed using the Allowable Stress Rating method. There are existing bridges designed and constructed during the Central Artery timeframe that were designed using Load Factor Rating method. These bridges shall be rated using the Load Factor Rating method.

These bridges shall be rated for the H20, Type 3, Type 3S2, HS20, SU4, SU5, SU6, and SU7 vehicles outlined above. The ratings and the corresponding gross tonnage for each of these vehicles shall be reported at both the Inventory and Operating Level.

The MS18 gross tonnage for roadway beams, as specified in Paragraph 7.1.7.2B shall be specified on the Summary of Bridge Rating in the spaces provided for Item 64 and Item 66. Sidewalk beam gross tonnage shall not be reported in the Summary. Due to limitations in NBIS coding, the rating factors shall not be reported at a value greater than 99.9. If the calculated rating tonnages shown in the Breakdown of Bridge Rating equal or exceed 99.9, report 99.8 at Operating (Item 64) and 99.7 at Inventory (Item 66) in the Summary of Bridge Rating. The one exception to this rule according to the December 1995 FHWA NBIS Coding Guide is to code 99.9 for Items 64 and 66 “for a structure under sufficient fill such that, according to AASHTO design, the live load is insignificant in the structure load capacity.”

Both Inventory and Operating Ratings shall be calculated for the Rating Vehicles outlined above. In general, lane loadings shall not be used for the H20 and HS20 vehicles when the span length is less than 200 feet. However, if a component of a structure is rated for the H vehicle, and the rating is determined to be 12 tons or less, this component must also be rated using the lane loading.

The above 12-ton limitation is based upon the 1978 *AASHTO Manual for Maintenance Inspection of Bridges,* which states in 5.2.*2 “The probability of having a series of closely spaced vehicles of the maximum allowed weight becomes greater as the maximum allowed weight for each unit becomes less. That is, it is more likely to have a train of light-weight vehicles than it is to have a train of heavy-weight vehicles.”*

For spans greater than 200 feet in length, the load effect of the Type 3 and Type 3S2 vehicles shall be modeled as a lane-type loading, similar to that shown in the *AASHTO Manual for Bridge Evaluation*, Figure D6A-4. Each lane-type loading shall consist of a legal lane weight of 0.20 klf concurrent with a vehicle that is 75% of the weight of the applicable vehicle as shown in Figure 7.10B.  A single vehicle load of the same type shall be placed in the adjacent lane(s). For continuous span bridges, where at least one span is greater than 200 feet in length, an additional model for negative moment and interior reactions shall apply. The model for continuous span negative moment and interior reaction shall be a lane-type loading, similar to that shown in the *AASHTO Manual for Bridge Evaluation*, Figure D6A-5. Each continuous span lane-type loading shall consist of a legal lane weight of 0.20 klf concurrent with two vehicles that are 75% of the weight of the applicable vehicle as shown in Figure 7.10B, with the two vehicles spaced with 30 feet clear distance between vehicles. A single vehicle load of the same type shall be placed in the adjacent lane(s). For the H20 and HS20 vehicles, the standard lane loading with concentrated load(s), including the provisions for lane loads on continuous spans, as defined by the *AASHTO Standard Specifications*, shall be used. Lane-type loadings are not required to be considered for the SU or EV vehicles.

For bridges composed of adjacent precast beams, including prestressed deck slabs and box beams, with functioning shear keys, with or without a composite concrete slab, the equations from Article 3.23.4 of the *AASHTO Standard Specifications for Highway Bridges* shall be superseded by the following equations for live load bending moment distribution from the 13th Edition of the *AASHTO Standard Specifications for Highway Bridges*. In calculating the bending moments no longitudinal distribution of wheel load shall be assumed.

Load Fraction = S/D

Where:

For

For

Where:

|  |  |  |
| --- | --- | --- |
| Nl | = | Number of traffic lanes. Note that this number may vary from 1 to the total number of lanes to allow for calculation of the Force Effect due to Adjacent Vehicle |
| Ng | = | Total number of longitudinal beams |
| C | = | K(W/L), a stiffness parameter |
| W | = | Overall width of the bridge (ft) |
| L | = | Span length (ft) |

Values of K to be used in C = K(W/L):

Table 7.2.4-1: Live Load Distribution Constant for Prestressed Adjacent Beams

|  |  |  |
| --- | --- | --- |
| Bridge Type | Beam Type and Deck Material | K |
| Multi-Beam | Nonvoided rectangular beams | 0.7 |
|  | Rectangular beams with circular voids | 0.8 |
|  | Box section beams | 1.0 |
|  | Channel Beams | 2.2 |

7.2.4.3A The Type EV2 and Type EV3 shall be rated and the corresponding gross tonnage for each of these vehicles shall be reported at the Strength I Limit State for Operating Level only, using the load factors for dead loads contained in Table B6A-1 of the *AASHTO Manual for Bridge Evaluation* and a load factor of 1.3 for live load as advised by FHWA. The live load distribution factor should be the One Design Lane Loaded factor from the *AASHTO LRFD Bridge Design Specifications* with the built-in multiple presence factor of 1.2 divided out. As of the issuance of this Bridge Manual, BrR adequately handles these changes when the element to be rated includes both single and multi-lane live load distribution factors.Interior beams shall be rated with the Type EV2 or Type EV3 in one lane, and a Type 3S2 vehicle in an adjacent lane(s). The Type 3S2 vehicle is the only adjacent Rating Vehicle that needs to be considered.

When using BrR to determine the rating for the Type EV2 and Type EV3 vehicles, the rating vehicle shall be defined as a Permit Load Rating and the Type 3S2 shall be defined as the Adjacent Vehicle. Under the Advanced Vehicle Properties, check “override” and input 1.3 for the Permit Live Load Factor. Additionally, the Adjacent Vehicle Live Load Factor shall be set as 1.3. As of the issuance of this Bridge Manual, BrR cannot run culvert analyses for Permit Load Rating or Legal Load Rating. For reinforced concrete culvert type structures, run the EV2 and EV3 in the Operating case with the other vehicles, edit the Advanced Vehicle Properties, check Single Lane Loaded.

When not using BrR, use the modified *AASHTO LRFD Bridge Design Specifications* Article 4.6.2.2.5 equations below to determine the force effect on the structural member being rated for the Type EV2 or EV3 and Type 3S2 adjacent vehicle.

Where:

|  |  |  |
| --- | --- | --- |
| LL | = | Final Force Effect due to Rating Vehicle Live Load (EV2 or EV3) to be applied in rating equation |
|  | = | Force Effect due to Emergency Vehicle |
|  | = | Single lane live load distribution factor |
|  | = | A factor taken as 1.20 where the lever rule was not utilized, and 1.0 where the lever rule was used for a single lane live load distribution factor |
| AdjLL | = | Force Effect due to Adjacent Vehicle (Type 3S2) to be applied in rating equation |
|  | = | Force Effect due to Adjacent Vehicle (Type 3S2) |
|  | = | Multiple lane live load distribution factor |

In all cases the Rating Factor for the Type EV2 and EV3 vehicles shall be determined using the following formula:

Where:

|  |  |  |
| --- | --- | --- |
| RF | = | Rating Factor |
| C | = | Capacity |
|  | = | Load Factor for components and attachments |
| DL | = | Force Effect due to components and attachments |
|  | = | Load Factor for wearing surfaces and utilities |
| DW | = | Force Effect due to wearing surfaces and utilities |
|  | = | Load Factor for Adjacent Vehicle = 1.3 |
| AdjLL | = | Force Effect due to Adjacent Vehicle |
|  | = | Load Factor for Rating Vehicle = 1.3 |
| LL | = | Force Effect due to Rating Vehicle Live Load |

#### 7.2.4.3B The Type EV2 and Type EV3 shall be rated and the corresponding gross tonnage for each of these vehicles shall be reported at the Operating Level only with the exception that stone masonry arches, and timber piles with only one capacity value provided on the plans, shall be reported at the Inventory Level as specified in Paragraph 7.2.7.11B. If the Load Factor Rating method is required, the Operating Level shall be obtained by using the load factor for live load (βL or A2) equal to 1.30. The live load distribution factor shall be the “Bridge Designed for One Traffic Lane” taken from the *AASHTO Standard Specifications for Highway Bridges*.

Interior beams shall be rated with the Type EV2 or Type EV3 in one lane, and a Type 3S2 vehicle in an adjacent lane(s). The Type 3S2 vehicle is the only adjacent Rating Vehicle that needs to be considered.

When using BrR to determine the rating for the Type EV2 and EV3 vehicles, the rating vehicle shall be defined as Permit Operating and the Type 3S2 shall be defined as the Adjacent Vehicle. Under the Advanced Vehicle Properties, the Adjacent vehicle live load factor shall be set as 1.0 for an Allowable Stress Rating and 1.3 for a Load Factor Rating. Under the Live Load Distribution tab for each Member Alternative check the box for “Allow Distribution factors to be used to compute effects of permit loads with routine traffic”.

When not using BrR, use *AASHTO Standard Specifications for Highway Bridges* Table 3.23.1 to calculate the live load distribution factors for a Bridge Designed for One Traffic Lane and Bridge Designed for Two or more Traffic Lanes. The Live Load Distribution Factor for the adjacent vehicle shall be the difference between these two live load distribution factors. The live load distribution factor for the Type EV2 and EV3 vehicles shall be that for a Bridge Designed for One Traffic Lane.

Where:

|  |  |  |
| --- | --- | --- |
| LL | = | Final Force Effect due to Rating Vehicle Live Load (EV2 or EV3) to be applied in rating equation |
|  | = | Force Effect due to Emergency Vehicle |
|  | = | Live load distribution factor for Bridge Designed for One Traffic Lane |
| AdjLL | = | Force Effect due to Adjacent Vehicle (Type 3S2) to be applied in rating equation |
|  | = | Force Effect due to Adjacent Vehicle (Type 3S2) |
|  | = | Live load distribution factor for Bridge Designed for Two or more Traffic Lanes |

In all cases the Rating Factor for the Type EV2 and EV3 vehicles shall be determined using the following formula:

Where:

|  |  |  |
| --- | --- | --- |
| RF | = | Rating Factor |
| C | = | Capacity |
| A1 | = | Dead Load Factor |
| DC | = | Force Effect due to Stage 1 Dead Load |
| DW | = | Force Effect due to Stage 2 Dead Load |
| A3 | = | Adjacent Vehicle Live Load Factor |
|  | = | 1.0 for Allowable Stress Rating |
|  | = | 1.3 for Load Factor Rating |
| AdjLL | = | Force Effect due to Adjacent Vehicle |
| A2 | = | Live Load Factor |
|  | = | 1.0 for Allowable Stress Rating |
|  | = | 1.3 for Load Factor Rating |
| LL | = | Force Effect due to Rating Vehicle Live Load |

#### 7.2.4.8 Curbs with height less than 12 inches shall be considered mountable. The beams supporting a mountable sidewalk, mountable median, or mountable safety walk with a width greater than 2 feet measured from the face of the bridge rail to the curb line shall be rated by placing a wheel line 2 feet from the face of the bridge rail. If the above referenced width is 2 feet or less, the wheel line shall be placed 2 feet from the face of the curb. This rating shall be performed at the Operating Level. Since traffic on sidewalk should not be a routine occurrence, Service level ratings for LFR or LRFR shall not be developed for the curb mounted case. The Inventory Rating shall always be calculated with the wheel line located in the travelway 2 feet from the face of the curb. Refer to Paragraph 7.2.4.5 for Alternative Load Rating using Actual Lane Location procedures. Refer to Chapter 3, Paragraph 3.5.3.11, Case II for guidance regarding the application of the HL-93 loading for this situation.

#### 7.2.5.1 Any request for clarification of, or deviation from, these guidelines must be submitted in writing via email to the State Bridge Engineer. Written responses will be provided.

#### 7.2.5.4 Pile bent structures constructed of steel piles, timber piles, or concrete piles, including their pile caps, shall be rated. Other non-reinforced concrete substructures, such as steel frames or substructures that include steel cross girder members, shall also be rated.

Typically, reinforced concrete substructures such as multi-column piers, single column hammerhead piers, solid wall piers and concrete abutments, do not need to be rated because they have sufficient capacity. However, in cases where these types of substructures have undergone deterioration in critical areas that has reduced their load carrying capacity significantly enough to influence the overall rating of the bridge, then the Rating Engineer shall consult with the MassDOT Ratings and Overload Engineer regarding the need for rating these substructures. This deterioration shall include deterioration of bridge seats and pedestals which has undermined the bridge bearings.

In either case, the report shall contain a statement noting the Rating Engineer’s judgment with regards to the substructure.

7.2.5.8B All timber structures designed using the Allowable Stress Design methodology shall be rated using the Allowable Stress Rating methodology.  Where the actual species and grade of lumber are unknown, the Rating Engineer shall determine the species and grade by field observation and/or testing.

The Allowable Inventory Stresses for various timber species and grades shall be taken from Section 8 of the *AASHTO LRFD Bridge Design Specifications* and the appropriate adjustment factors shall be taken from Section 13 of the *AASHTO Standard Specifications for Highway Bridges.*  The values used for Allowable Operating Unit Stresses shall be equal to 1.33 times the values determined for the Allowable Inventory Unit Stresses.

#### 7.2.5.9 Tire Contact Area Dimensions. The Tire Contact Area for a given rating vehicle wheel shall be calculated by dividing the reaction of the wheel by an assumed tire pressure of 80 psi. The length of this Tire Contact Area shall be taken as 10” for all vehicle wheels and the width shall be calculated by dividing the calculated Tire Contact Area by this length.

#### 7.2.5.10 BrR can only model parabolic and linear varying web depths for reinforced concrete T-beam superstructures. If a beam’s web depth varies along a circular curve, the concrete T-beams can only be modeled in BrR using cross sections and cross-sectional ranges with linear varying web depths.

BrR can model parabolic, circular, and linear varying web depths for steel girder superstructures.

#### 7.2.6.2B The Allowable Stresses at Inventory and Operating Levels for prestressed concrete members shall be calculated using the formulas presented in the *AASHTO Manual for Bridge Evaluation*, Article 6B.5.3.3. All Allowable Stress values used in the preparation of the rating report must be clearly stated in the Rating Analysis Assumptions and Criteria section of the rating report.

The *AASHTO Manual for Bridge Evaluation* provides one set of rating factor formulas for the rating of prestressed concrete members that consider both strength and serviceability together. Therefore, when calculating the Load Factor Rating of prestressed concrete members, the flexural and shear strength rating factors for both Inventory and Operating Levels shall be obtained using the formulas as specified in Article 6B.5.3.3 of the *AASHTO Manual for Bridge Evaluation*. The rating factor formulas make no provisions for serviceability for Operating Ratings and thus serviceability Operating Rating values need not be calculated.

#### 7.2.7.1 The *AASHTO Manual for Bridge Evaluation*, Article 6A.9.1 states that unreinforced stone masonry arches should be evaluated by the Allowable Stress Rating method. An acceptable method of analysis is outlined below.

#### 7.2.7.10B In the load rating of stone masonry arches, the concept of a "kern" or middle third section is used to determine whether any portion of the masonry is in tension. The kern points are located above and below the neutral axis of the arch at a distance r2/c, where “r” is the radius of gyration and “c” is the distance from the neutral axis to the extreme fiber.

In cases where the combined dead and live load thrust falls outside the kern points, resulting in tension in the masonry, a pressure wedge analysis shall be used to calculate the maximum compressive stress. The portion of the arch masonry in tension shall be effectively ignored by redistributing the pressure over a smaller depth.

If the eccentricity (e) of the combined thrust is located below the bottom kern point, the maximum compressive stress shall be determined as follows:

|  |  |  |
| --- | --- | --- |
| ft | = | 0 (no tension assumed at top of masonry) |
| fb | = |  |
|  |  |  |

Where:

|  |  |  |
| --- | --- | --- |
| e | = | Combined Moment/Combined Thrust |
| ft | = | Stress at the top of the section |
| fb | = | Stress at the bottom of the section |
| P | = | Compressive reaction |
| A | = | (Unit Width) |
| d | = | Depth of Arch Section |
| c | = | Distance from neutral axis to extreme fiber |

If the eccentricity (e) of the combined thrust is located above the top kern point, the maximum compressive stress shall be similarly determined as follows:

|  |  |  |
| --- | --- | --- |
| ft | = |  |
| fb | = | 0 (no tension assumed at bottom of masonry) |

If the eccentricity (e) of the combined thrust is located between the kern points, the maximum compressive stress shall be determined as follows:

|  |  |  |
| --- | --- | --- |
| fb or ft | = |  |

Where:

|  |  |  |
| --- | --- | --- |
| A | = | Cross sectional area |

#### 7.2.7.11B The Inventory Allowable Compressive Stresses for stone masonry shall be determined in accordance with Article 6B.5.2.6 of the *AASHTO Manual for Bridge Evaluation*. Professional judgment based upon field observations and testing is pivotal to the proper determination of Inventory Allowable Compressive Stresses for stone masonry. Based upon the Rating Engineer's judgment, Allowable Compressive Stresses may be lowered for low quality masonry, or raised, if justified by testing of samples taken from the bridge. Ratings for stone masonry arches shall only be provided at the Inventory Stress Level. Ratings for the EV2 and EV3 vehicles shall also be calculated and reported at the Inventory Stress Level . Report the same values for Items 64 and 66 in accordance with Paragraph 7.2.4.2B of this Bridge Manual.

#### 7.2.9.2 Based on typically observed beam-end deterioration, as well as the anticipated failure mechanism, nominal capacities shall be determined based on the average remaining thickness of the web within the bottom 4” of the web height. Any holes shall be considered ineffective for the full 4” height. Engineering judgement shall be used in situations where advanced section loss occurs outside of the 4” height.

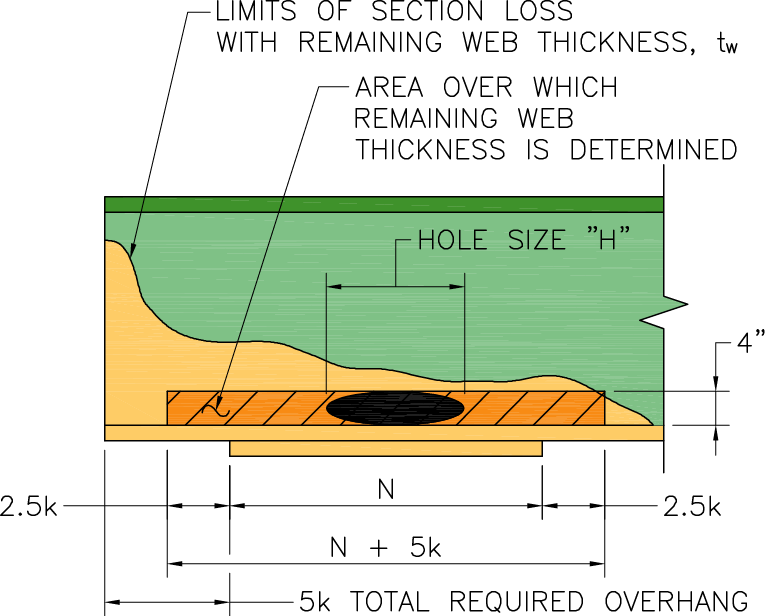


Figure 7.2.9-1: Graphic for Corroded Steel Beam Webs

Previous editions of this Bridge Manual included a check of the buckling capacity at beam ends where advanced corrosion had occurred. It was found that often this check was very conservative and not representative of the actual failure mechanism. This is typically the result of the presence of concrete encased end diaphragms; localized corrosion along bottom of the web, rather than over the full height; conservative boundary conditions, ignoring that the web is supported along three sides; and neglecting the length of the bearing, among other factors. Therefore, this check has been deleted. This section has been further revised based on the completion of research performed by UMass for MassDOT. The remaining provisions of these Subsections have been revised based on the final report, which may be found here:

[https://www.mass.gov/doc/development-of-load-rating-procedures-for-deteriorated-steel-beam-end/download](https://www.mass.gov/doc/development-of-load-rating-procedures-for-deteriorated-steel-beam-end/download" \t "_blank)It shall be noted that additional revisions to the approach have been made following issuance of the final report. These revisions are not readily available online.

The UMass research developed these current provisions through extensive finite element modeling, which was calibrated through testing of simple span ends of deteriorated beams. The beams were single beam segments without diaphragms, stiffeners, or a slab. The prior provisions were based on the *AISC Steel Construction Manual* for similar, while non-deteriorated, end conditions.

Also note, that based on the tests, that these capacities are often achieved with a large amount of deformation of the section. This is especially true when the section losses at the end exceeded 65% of the original section.

When determining what section loss to apply to the beam for regular shear capacity (BrR input), a weighted average over the entire beam depth shall be used. For example, if the bottom half of a beam web has 50% section loss, the overall deterioration input will be 25%. This approach shall not be used when considering these local effects.

#### 7.2.9.3 The nominal web beam end capacities for beam ends with section loss and without bearing stiffeners shall be calculated using the following procedure.

The resistance factors for the LRFR method are as given in the *AASHTO LRFD Bridge Design Specifications,* whereas the Allowable Stress safety factors (Ω) are from the *AISC Steel Construction Manual* and are used for the Inventory level capacities. The Operating safety factor is taken as the Inventory safety factor multiplied by 55/75. For Load Factor Ratings the safety factors (Ω) are modified as described in Paragraph 7.2.9.4B.

The nominal web local yielding capacity, in kips (Rn,(1)) shall be determined from the minimum value calculated as follows:

For beam ends without bearing stiffeners, the average web thickness, tave, used for beam-end deterioration analysis at end supports with a beam overhang of at least 5k, shall be based on the equation below

DIf an overhang past the bearing of less than 5k is provided, then the “5k” term in the equation shall be substituted with “2.5k”.

Where:

|  |  |  |
| --- | --- | --- |
| tave | = | average remaining web thickness (in.) |
| N | = | bearing length (in.) |
| k | = | distance from outer face of flange to toe of web fillet for a rolled shape, or toe of web to flange weld for a plate girder (in.) |
| H | = | total length of hole(s) along length used for capacity within (N+5k or N+2.5k) (in.) |
| tw | = | remaining web thickness (in.) |

At beam end reactions where an overhang past the bearing of at least 5k is provided

|  |  |  |
| --- | --- | --- |
| Rn,(1) | = |  |

At beam end reactions where an overhang of less than 5k is provided

|  |  |  |
| --- | --- | --- |
| Rn,(1) | = |  |

Where:

|  |  |  |
| --- | --- | --- |
| Rn,(1) | = | nominal web local yielding capacity (kips) |
| F­y | = | minimum yield strength (ksi) |
| t­­­ave | = | the average remaining thickness within the bottom 4” of the web height (in.) |
| k | = | distance from outer face of flange to toe of web fillet for a rolled shape, or toe of web to flange weld for a plate girder (in.) |

The nominal web beam end capacity, based on the UMass equation, in kips (Rn,(2)) shall be calculated as follows:

For beam ends without bearing stiffeners, the average web thickness, tave, used for beam-end deterioration analysis at end supports shall be based on the equation below. The UMass study determined that one of the controlling factors regarding beam end capacity is the initial imperfection amplitude, the amount that the web is out of plane. This amplitude is measured as a fraction of the original web thickness, i.e. 1 tweb Imperfection Amplitude is equal to 1 web thickness out of plane. MassDOT’s Inspection section has been made aware that this value has become critical to determining the capacity and is developing procedures to measure this value. When these measurements are not available in the inspection report the Rating Engineer shall assume the values associated with 0.5 tweb ≥ i > 0.1 tweb. However, the Rating Engineer shall review the report to ensure that there are no imperfections clearly exceeding the 0.5 tweb limit.

Where:

|  |  |  |
| --- | --- | --- |
| t­­­ave | = | average remaining web thickness (in.) |
| N | = | bearing length (in.) |
| m | = | factor specified in Table 7.2.9-1 |
| d | = | beam depth (in.) |
| H | = | total length of hole(s) along length used for capacity within (N + m d) (in.) |
| tw | = | remaining web thickness (in.) |

Table 7.2.9-1: Values of Factor (m) - for Average Web Thickness Calculation

|  |  |  |  |
| --- | --- | --- | --- |
|  | Imperfection Amplitude *(i)*\* | | |
|  | i > 0.5 tweb | 0.5 tweb ≥ i > 0.1 tweb | i ≤ 0.1 tweb |
| N/d > 0.2 | 0.2 | 0.2 | 0.1 |
| N/d ≤ 0.2 | 0.1 | 0.1 | 0.0 |

\*Values shall not be interpolated

For beam end reactions when N/d > 0.2

Rn,(2)

Where:

|  |  |  |
| --- | --- | --- |
| Rn,(2) | = | nominal web beam end capacity, based on the UMass equation (kips) |
| E | = | elastic modulus of steel (ksi) |
| Fy | = | yield strength of steel (ksi) |
| tf | = | corroded bottom flange thickness (in.) Use the actual measured thickness, except that tf shall not be less than 20% of the original thickness () |
| t­­­ave | = | average remaining web thickness (in.) |
| b | = | factor specified in Table 7.2.9-2 |
| N | = | bearing length (in.) |
| H | = | total length of hole(s) along length used for capacity within (N + md) (in.) |
| CL | = | corrosion length within length over which average remaining web thickness is determined (in.) CL shall not be less than N/2 nor more than N+md. For smaller corrosion lengths, use a minimum of N/2. (CLmin = N/2). For larger corrosion lengths, use a maximum of N+m d. (CLmax = N+m d) |
| m | = | factor specified in Table 7.2.9-1 |

Table 7.2.9-2: Factors for Calculating Rn,(2) when N/d > 0.2

|  |  |  |  |
| --- | --- | --- | --- |
|  | Imperfection Amplitude *(i)*\* | | |
|  | i > 0.5 tweb | 0.5 tweb ≥ i > 0.1 tweb | i ≤ 0.1 tweb |
| a | 0.37 | 0.32 | 0.57 |
| b | 0.17 | 0.50 | 0.23 |

In \*Values shall not be interpolated

For beam end reactions when N/d ≤ 0.2

Rn,(2)

Where:

|  |  |  |
| --- | --- | --- |
| Rn,(2) | = | nominal web beam end capacity, based on the UMass equation (kips) |
| E | = | elastic modulus of steel (ksi) |
| Fy | = | yield strength of steel (ksi) |
| tf | = | bottom flange thickness (in.) |
| t­­­ave | = | average remaining web thickness (in.) |
| N | = | bearing length (in.) |
| H | = | total length of hole(s) along length used for capacity within (N + m d) (in.) |
| d | = | beam depth (in.) |

Table 7.2.9-3: Factors for Calculating Rn,(2) when N/d ≤ 0.2

|  |  |  |  |
| --- | --- | --- | --- |
|  | Imperfection Amplitude *(i)*\* | | |
|  | i > 0.5 tweb | 0.5 tweb ≥ i > 0.1 tweb | i ≤ 0.1 tweb |
| a | 0.33 | 0.32 | 0.38 |
| b | 0.00 | 0.17 | 0.00 |
| h | 0.40 | 0.20 | 0.15 |

#### 7.2.9.5 For beam ends with bearing stiffeners, the web beam end capacity (R(3)) shall be calculated using the following derived resistances. This section, originally developed based on the provisions of the *AASHTO LRFD Bridge Design Specifications* has been further revised based on the completion of research performed by UMass for MassDOT. The remaining provisions of these Subsections have been revised based on the final report, which may be found here:

<https://www.mass.gov/doc/improved-load-rating-procedures-for-deteriorated-steel-beam-ends-with-deteriorated-stiffeners-final-report/download>

The UMass research developed these current provisions through extensive finite element modeling, which was calibrated through testing of simple span ends of deteriorated beams. The beams were single beam segments without diaphragms or a slab.

When determining what section loss to apply to the beam and stiffener for regular shear capacity (BrR input), a weighted average over the entire beam and/or stiffener depth shall be used. For example, if the bottom half of a beam web has 50% section loss, the overall deterioration input will be 25%. For the stiffener input, the section thickness in BrR will need to be reduced to model the deterioration. This approach shall not be used when considering these local effects.

The current provisions completely supersede the prior provisions, providing an empirical formula with associated parameters determined from the corrosion patterns defined as follows.

#### 7.2.10.1 Concrete deterioration and loss of prestressing is a significant issue in the load rating of prestressed concrete beams. This issue is of particular concern with adjacent box and deck beam bridges, as these structures are impossible to completely inspect, with only the bottom flange and the exterior web of the fascia beams visible and available for tactile inspection. However, in many instances evidence of leakage of salt laden roadway runoff through the grouted joints is visible, indicating possible deterioration of unknown levels in locations unavailable for inspection.

Often this deterioration will progress to the underside of the beam (bottom flange), spalling off large pieces of concrete and exposing the prestressing strands to the environment, eventually leading to their deterioration. There is no uniformly accepted guidance on how to estimate loss of prestressing force, if any, and its effect on load carrying capacity.

The following guidelines for calculating a reduced prestressing force are based primarily upon research conducted by the University of Illinois at Urbana-Champaign and Illinois DOT (IDOT), and are to be used in evaluating prestressed concrete beams. This sectionhas been further revised based on the completion of research performed by UMass for MassDOT. The remaining provisions of these Subsections have been revised based on the final report, which may be found here:

<https://www.mass.gov/doc/revised-load-rating-procedures-for-deteriorated-prestressed-concrete-beams-final-report/download>

Refer to Figures 7.2.10-1 below for guidance.

#### 7.2.9.2 Based on typically observed beam-end deterioration, as well as the anticipated failure mechanism, nominal capacities shall be determined based on the average remaining thickness of the web within the bottom 4” of the web height. Any holes shall be considered ineffective for the full 4” height. Engineering judgement shall be used in situations where advanced section loss occurs outside of the 4” height.

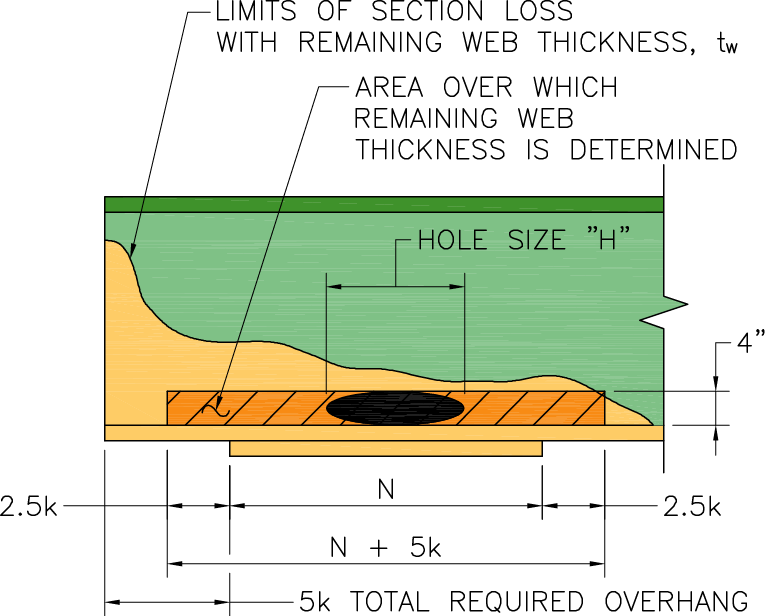


Figure 7.2.9-1: Graphic for Corroded Steel Beam Webs

Previous editions of this Bridge Manual included a check of the buckling capacity at beam ends where advanced corrosion had occurred. It was found that often this check was very conservative and not representative of the actual failure mechanism. This is typically the result of the presence of concrete encased end diaphragms; localized corrosion along bottom of the web, rather than over the full height; conservative boundary conditions, ignoring that the web is supported along three sides; and neglecting the length of the bearing, among other factors. Therefore, this check has been deleted. This section has been further revised based on the completion of research performed by UMass for MassDOT. The remaining provisions of these Subsections have been revised based on the final report, which may be found here:

[https://www.mass.gov/doc/development-of-load-rating-procedures-for-deteriorated-steel-beam-end/download](https://www.mass.gov/doc/development-of-load-rating-procedures-for-deteriorated-steel-beam-end/download" \t "_blank)

It shall be noted that additional revisions to the approach have been made following issuance of the final report. These revisions are not readily available online.

The UMass research developed these current provisions through extensive finite element modeling, which was calibrated through testing of simple span ends of deteriorated beams. The beams were single beam segments without diaphragms, stiffeners, or a slab. The prior provisions were based on the *AISC Steel Construction Manual* for similar, while non-deteriorated, end conditions.

Also note, that based on the tests, that these capacities are often achieved with a large amount of deformation of the section. This is especially true when the section losses at the end exceeded 65% of the original section.

When determining what section loss to apply to the beam for regular shear capacity (BrR input), a weighted average over the entire beam depth shall be used. For example, if the bottom half of a beam web has 50% section loss, the overall deterioration input will be 25%. This approach shall not be used when considering these local effects.

#### 7.2.10.3 In the vicinity of exposed prestressing strands deduct 100% of the strand area within the limits where they are exposed. Deduct 50% of the area of the strands in the next row directly above the limits of the exposed strands. Deduct 50% of the area of the strand(s) in the bottom row next to the limits of the exposed prestressing strands, unless strands are located in an area of concrete delamination.

#### 7.2.10.4 In areas of concrete delamination without exposed reinforcing stirrups or prestressing strands deduct 100% of the area of the prestressing strands located in the row directly above the limits of the delamination. Deduct 10% of the area of the prestressing strand(s) in the bottom row next to the limits of the delamination.

#### 7.2.10.5 Longitudinal and/or transverse cracks shall be considered as evidence of a potentially delaminated area. The delaminated area shall be estimated by tapping the concrete using a masonry hammer. The loss of prestressing force within this area shall be calculated in accordance with Paragraph 7.2.10.4 above.

#### 7.2.10.6 The reduced prestressing force due to losses as calculated in the paragraphs above shall only apply to the area of the deterioration. Strands shall be considered partially developed until they reach a distance equal to their development length outside the deteriorated regions. Development of strands shall only be considered to occur within sound concrete. For example, a strand with 100% loss from deterioration will require 100% of its development length before it is considered fully effective again. Likewise, a strand with 50% section loss will require 50% of its development length, in sound concrete, before it is considered fully effective again.

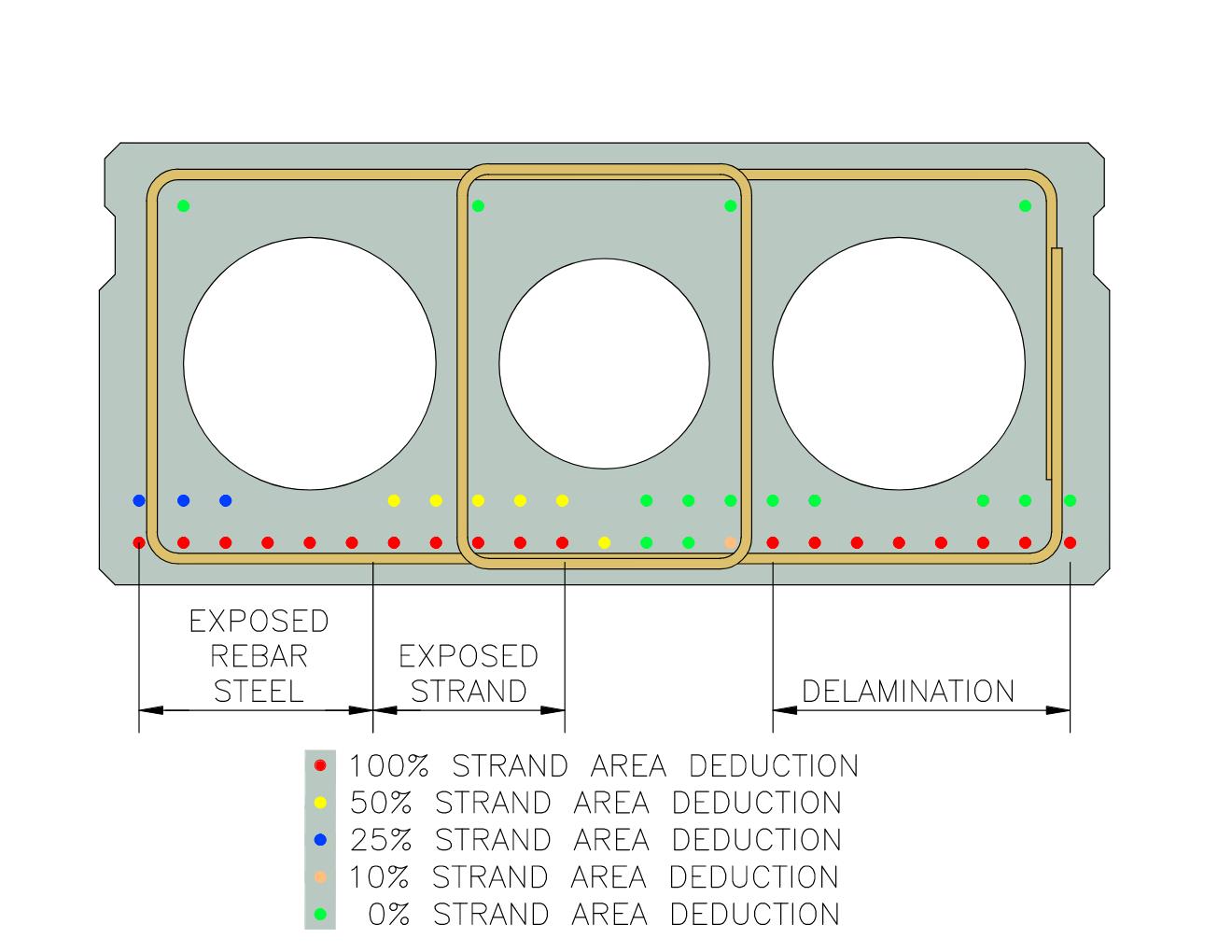


Figure 7.2.10-1: Example of Strand Losses

#### 7.3.1.2 Electronic Submissions. Submissions shall be made through MassDOT’s Bridge Inspections and Ratings SharePoint site. The site address starts as follows:

#### <https://massgov.sharepoint.com/sites/DOT-Highway-Bridge/InspRating/>

#### Since each consultant will have a folder with access restricted to that consultant, MassDOT, and FHWA, the remainder of the address will be unique to each consultant. Access to the Bridge Inspections and Ratings SharePoint will be established for a consultant group of staff by the Site Owner, currently the Bridge Load Ratings and Overloads Engineer. Requests for individual access will be rejected. Submissions shall be made by creating a .zip file of the entire contents of the submittal, and then dragging and dropping it into the Submissions folder under the consultant folder, then notifying the Site Owner and the State Bridge Inspection Engineer and/or the Project Manager for the project the submission is related to.

#### The report shall be submitted as an electronic file in PDF format. The first and last page of the PDF (“covers”) shall be color-coded as follows: RED, if the rating for any posting vehicle for any roadway element is 6 tons or less; YELLOW, if more than 6 tons but less than statutory; and GREEN for statutory or greater. The color shall be based on the values reported in the Summary of Bridge Rating. For example, if the actual lane locations or alternate load paths are used to provide improved numbers, and are reported in the Summary, then the cover color shall be based upon those rating values. The front cover shall be formatted in accordance with Figure 7.5.2-1.

The names of Facility Carried / Feature Intersected and the Memorial Name/Local Name must be exactly the same as those given on the SI&A with the following exceptions. The generic Feature and/or Facility Codes (i.e. WATER, HWY, RR, etc.) shall be omitted, but the Interstate (I-), US Route (US) and State Route (ST) code along with the route number, and direction if applicable (NB, SB, etc.), followed by the local street names (if any) in parentheses, shall be provided. The local street names shall be fully spelled out (e.g. N WSHNGTN ST on the SI&A shall be spelled out as North Washington Street). If the same stretch of road has several numbered routes associated with it, then all of the routes shall be provided separated by a slash (/) starting with the Interstate, then the US Route, then the State Route, then followed by the local street name (if any) in parentheses. The following are examples of the proper identification of the bridge with some common Facility Carried/Feature Intersected:

* *ST 19 (WALES ROAD) OVER MILL BROOK*
* *ST 20A (PLAINFIELD STREET) OVER I-91 NB*
* *US 202 (GRANBY ROAD) OVER ST 116 (NEWTON STREET)*
* *I-95/US 1/ST 3 OVER WEST STREET*
* *ST 31 (RESERVOIR STREET) OVER PROVIDENCE & WORCESTER RR*
* *WOLOMOLOPOAG STREET OVER AMTRAK/MBTA*

The files in the electronic submission shall be organized in the following four folders:

* COMPUTER INPUT FILES: all BrR and/or other MassDOT approved rating analysis software input files that were used to produce the rating
* CALCULATION FILES: the spreadsheets and other computer calculation aids that were used to develop the rating
* RATING REPORT: the Rating Report itself formatted as specified in Section 7.5
* BRIDGE PLANS: all plans of the bridge that were made available to the Rating Engineer for the preparation of the Rating Report

### 7.3.2 Report Distribution

Submissions that exceed MassDOT’s email file size limit, roughly 25 MB, shall only be made by creating a .zip file of the entire contents of the submittal, and then dragging and dropping it into the Submissions folder under the consultant folder on the Bridge Inspections and Ratings SharePoint site, then notifying the Site Owner and the State Bridge Inspection Engineer and/or the Project Manager for the project that the submission is related to.

Smaller submissions, roughly under 25MB, may be made by creating a .zip file of the entire contents or a portion of the submittal and emailing that .zip file directly to Bridge Load Ratings and Overloads Engineer, notifying the State Bridge Inspection Engineer and/or the Project Manager for the project that the submission is related to.

The preferred naming convention for submission of .zip files is as follows: BRIDGE#\_BIN Consultant.zip, where the BRIDGE# is without hyphens (i.e. B16256), BIN is the BIN, and Consultant is the consultant’s name.

#### 7.4.1.1. All submitted hand calculations shall include either sketches or copies of the necessary sheets or details from the plans to support the calculations being prepared. All hand calculations shall include all details along with relevant notes and code references so that every step of the calculations can be easily followed, in a logical order, legible and prepared on 8 ½” x 11” sheets that are subsequently scanned for inclusion in the submission file.

#### 7.4.1.2 Calculations using spreadsheets and other computer calculation aids (e.g. Mathcad) shall be formatted and presented as hand calculations and formatted to allow for printing on 8½” x 11” sheets without scaling. These computer aided calculations shall be presented in a logical order along with relevant notes and code references so that every step of the calculations can be easily followed. Copies of the original calculation files shall be included with the submission as described in Subsection 7.3.1. For example, spreadsheets, and other similar formats, shall be appropriately documented with references and organized so that the calculations in them can be easily followed by an independent reviewer. Calculations shall be organized by name or in folders so that an independent reviewer can determine how each file is intended to be used. An index identifying each file by name with a brief explanation shall be provided. Naming of calculation files shall not include excessive description since file and folder character limitations can cause errors resulting in incomplete archiving of submissions.

#### 7.4.2.2 BrR shall be used to rate every primary load carrying element of the structure in order to determine the controlling live load capacity of the structure. The bridge shall be modeled as a Girder System, wherever possible. Links shall be used to define identical girders within a girder system. However, the following member types shall be modeled as described below:

1. When the structure is a concrete slab bridge it shall be modeled as a Girder Line;
2. When the exterior beam acts composite with a sidewalk or a safety curb, this particular member shall be modeled as a Girder Line and the remaining portion of the structure shall be modeled as a Girder System;
3. Each uniquely deteriorated member shall be modeled and not linked with other members within the Girder System unless the deterioration exhibited is nearly identical, in the Rating Engineer’s judgement, between linked members.

#### 7.4.2.3 The file naming convention for the BrR file shall be consistent with the following Massachusetts specific example of a Town Line bridge:

Bridge No. D-02-033=P-15-015, BIN = BG1, DANA-PRESCOTT, MAIN STREET / SWIFT RIVER shall be identified without any blank spaces using the following UPPER CASE characters:

Bridge ID (unlimited characters): D-02-033=P-15-015 (BG1)

NBI Structure ID (NBI Item 8, 15 characters): D02033BG1DOTNBI

Name (same as Bridge ID): D-02-033=P-15-015 (BG1)

Description (unlimited characters): 2 SPAN SIMPLE COMPOSITE

MULTIPLE STEEL STRINGER

Model developed by Consultant, for report dated: Date

*(Modify as required.)*

Where:

The first 13 characters (22 if town line bridge, as shown in the example) reflect the structure’s Bridge Number, including hyphens, equal sign, and parentheses, and the characters within the parentheses represent the structure’s BIN.

The Description shall include reference to the consultant that developed the model, and the date of the report associated with the submitted model file.For submission purposes, the file shall be exported with the extension .XML and named as follows:

D-02-033=P-15-015(BG1).XML

#### 7.4.2.8 All BrR files shall include a defined “BRIDGE ALTERNATIVES”. The “BRIDGE ALTERNATIVES” allows for Permit Route Analyses to be performed directly from BrR Bridge Explorer.

#### 7.4.2.9 If the submitted BrR file contains two (2) or more entries under “SUPERSTRUCTURE DEFINITIONS”, then a corresponding entry is required under “SUPERSTRUCTURES” (BRIDGE ALTERNATIVES/SUPERSTRUCTURES). A further entry the next level down in “SUPERSTRUCTURES ALTERNATIVES” (BRIDGE ALTERNATIVES / SUPERSTRUCTURES / SUPERSTRUCTURE ALTERNATIVES) is also required, and the dropdown box should point back to the related entry under “SUPERSTRUCTURE DEFINITIONS”.

In this manner, each entry under the “SUPERSTRUCTURE DEFINITIONS” will be assigned an (E)(C), Existing and Current status.

All (E) (C) “SUPERSTRUCTURE DEFINITIONS” shall be included in the “BRIDGE ALTERNATIVES” under a single Bridge, “SUPERSTRUCTURES” so that all spans and member definitions are run concurrently for Permit Route Analysis.

#### 7.4.2.10A All BrR files shall be capable of running the HL-93 design live load and all Rating Vehicles used in the rating analysis.

#### 

#### 7.4.2.10B All BrR files shall be capable of running all Rating Vehicles used in the rating analysis.

### 7.5.1 Preparation and Format

The entire Rating Report shall be prepared as an electronic file in PDF format. The PDF file pages shall be sized as 8½” x 11” sheets. The font shall be Times New Romans with a minimum size 11. The PDF file shall also have a front and back cover that shall be color coded as follows: RED, if the rating for any posting vehicle is 6 tons or less; YELLOW, if the rating for any posting vehicle is more than 6 tons but less than statutory; and GREEN if the rating for any posting vehicle is statutory or greater. All pages that require a P.E. stamp shall be scanned after the stamp is affixed, signed and dated or generated electronically with an authentication method that links the stamp to the associated Registered Professional Engineer.

The entire PDF file of the Rating Report shall be bookmarked so that the reader can navigate to each individual section directly without having to scroll through the entire file. The Appendices containing calculations or computer output shall be further bookmarked to match the index of the calculations or by each computer output (e.g., Beam #1, etc.) so that the reader can navigate to a particular calculation or output of interest.

The Facility Carried / Feature Intersected and Memorial Name/Local Name listed on the Rating Report cover shall be as described in Paragraph 7.3.1.1.

Supplemental Rating Reports may be required for a variety of reasons. The sections required to be included in a Supplemental Rating are indicated in the following with (SUPPLEMENTAL). Other sections required may be considered and discussed on a case-by-case basis with the Ratings and Overloads Unit.

### 7.5.2 Report Organization

The Rating Report PDF file shall consist of the following sections, organized in the following order:

1. **REPORT COVER (SUPPLEMENTAL)**
   1. P.E. Stamp with date and signature of the Rating Engineer shall be placed here.
   2. Color coded background and formatted as discussed in Subsection 7.5.1 and as shown in Figure 7.5.2-1
   3. The date of the latest Inspections used to develop the rating shall be noted on the front cover. List each type of Inspection and the date of the Inspection as shown in Figures 7.5.2-1(1) and 7.5.2-1(2). Note if the Inspection was unapproved at the time of use by adding DRAFT following the date.
2. **INDEX (SUPPLEMENTAL)**
   1. Index of sections outlined with page numbers.
3. **SUMMARY OF BRIDGE RATING (SUPPLEMENTAL)**
   1. Tabular listing of the controlling rating values from the Breakdown of Bridge Rating (see below). Item 64 shall not be lower than Item 66.
   2. P.E. Stamp with date and signature of the Rating Engineer shall be placed here.
   3. Formatted as shown in Figure 7.5.2-2A or 7.5.2-4B for all structures.
4. **BREAKDOWN OF BRIDGE RATING (SUPPLEMENTAL)**
   1. Tabular listing of controlling rating locations and rating values for all bridge elements that must be rated. These ratings shall be summarized from those developed by rating at all points of interest as described in Subsection 7.2.2. For example, if flexure controls a beam element, report the controlling locations for flexure, not every point of interest, cover plate transition, splice, repair location, and support. All ratings below statutory shall have the text highlighted with the appropriate color. The controlling rating cells shall be shaded solid with the appropriate color, Green (red=0, green=176, blue=80), Yellow (red=255, green=255, blue=0), Red (red=255, green=0, blue=0), and the text shall be bold. For legibility, the font color for red and green shading and highlighting shall be white. All cells in the Breakdown shall be filled in. Elements that do not require a rating shall be noted with a dash.
   2. When alternative ratings using actual lane locations are provided, then these ratings shall be performed at the same points of interest and placed underneath the original ratings at each row. The cells are to be shaded and formatted as described above. These rating values shall be the controlling ratings for these members.
   3. Formatted as shown in Figures 7.5.2-3A(1) through 7.5.2-3A(3), and 7.5.2-6A, or 7.5.2-5B(1) through 7.5.2-5B(3), and 7.5.2-6B.
   4. Formatted as shown in Figure 7.5.2-7 when alternative rating factors using actual lane locations are provided.
   5. For Supplemental Rating Reports, only the revised and/or added Breakdown values are required assuming no changes to the values previously reported are required. If changes to the values previously reported are required, only provide the revised values.
5. **LOCATION MAP (SUPPLEMENTAL)**
   1. The location map shall be a street map in color and provide sufficient landmarks and adjacent highway information to allow the user to find the bridge in the field without additional information. Satellite or aerial photographs and topography maps are not acceptable substitutes.
6. **DESCRIPTION OF BRIDGE (SUPPLEMENTAL)**
   1. Formatted as shown in Figure 7.5.2-8. It shall be noted that the Modifications to Superstructure and Substructure sections are reserved for structural changes. Revisions to railings, wearing surface, etc. shall be covered elsewhere in the Description of Bridge.
7. **RATING ANALYSIS ASSUMPTIONS AND CRITERIA (SUPPLEMENTAL)** 
   1. Description of all methods, assumptions, allowable stresses, and strengths used to determine the rating of the structure, including computer programs, with version or release numbers utilized.
   2. Statement of the applicability of the substructure and/or deck to the rating.
   3. For Supplemental Rating Reports, only the revised assumptions and criteria are required.
8. **EVALUATION OF RATING AND RECOMMENDATIONS (SUPPLEMENTAL)**

Summary of controlling elements of the structure and recommendations to either improve or maintain the condition of the structure as described in Subsection 7.2.11.

Comparison of rating to previous rating as described in Paragraph 7.2.11.1 and as shown in Figure 7.5.2-9.

For Supplemental Rating Reports, only the revised evaluations and recommendations are required.

1. **AVAILABLE PLANS, INSPECTION REPORTS, AND REFERENCES (SUPPLEMENTAL)**
   1. Listing of all plans, latest inspection report(s) used and their sources that were made available to the Rating Engineer for the purpose of preparing the Rating Report.
   2. Identify unique references that were used to develop the rating that an independent reviewer may not be aware of, i.e. studies or reports, and textbooks included in Appendix F. For a Design Rating, identify the specific version and date of the MassDOT Bridge Manual that was used for the design and rating.
   3. For Supplemental Rating Reports, identify the available plans, inspection reports, and references which were used to develop the supplemented (original) rating.
2. **LOADINGS USED FOR BRIDGE RATING (SUPPLEMENTAL)**
   1. Standard diagrams of vehicles used in the rating showing axle weights and spacing as shown in Figures 7.5.2-10A,7.5.2-10B, 7.5.2-11, 7.5.2-12, and/or 7.5.2-13A as applicable.
   2. For Supplemental Rating Reports, only provide the new or revised loadings used to develop the supplement.
3. **APPENDIX A - INSPECTION REPORTS (SUPPLEMENTAL)**
   1. Inspection Reports including structure inventory and appraisal (SI&A), structures inspection field report and field notes. The first page shall be the latest SI&A sheet. Inspection Reports must be the latest available Routine, Routine & Special Member and Underwater at the time the Rating Report is submitted and shall include color reproductions of all inspection report photos. The National Bridge Element Inspection (PONTIS) pages shall not be included.
   2. For Supplemental Rating Reports, only provide the Inspections required to develop the supplement.
4. **APPENDIX B – PHOTOS**
   1. An abundant number of color photographs of the structure, each no smaller than 3” by 5” on the page, including both elevation views, views of both approaches, framing views (if it varies, one of each type) and sufficient critical member photos shall be provided to adequately display the current condition of the structure. An index of all photos shall precede the photos. Photos of deficiencies where findings are different from those noted in the Inspection Reports shall be included.
5. **APPENDIX C - COMPUTATIONS (SUPPLEMENTAL)**
   1. The Standard Statement of Concurrence of the Independent Reviewer (see Subsection 7.4.3) on a separate page.
   2. Tabular summary of all non-composite dead loads, composite dead loads, and live load distribution factors, etc., per beam.
   3. Plan, framing plan, and bridge cross sections, as well as unique details and elements, as appropriate to identify all members that have been rated and included in Breakdown of Rating tables.
   4. All hand calculations and computer aided calculations prepared as specified in Subsection 7.4.1 along with an index.
   5. For Supplemental Rating Reports, only provide the new or revised calculations used to develop the supplement.
6. **APPENDIX D - COMPUTER INPUT AND OUTPUT (SUPPLEMENTAL)**
   1. Copies of all input and output summary pages, including software generated sketches of the bridge/framing plan, bridge cross section (including sidewalks, railings, barriers, and/or medians), member elevations and cross sections from computer programs used in rating the structure, and the controlling ratings that are included in the Breakdown of Bridge Rating, controlling values highlighted.
   2. A summary page of all rating factors and rating values for each structure’s particular elements shall be created and placed in front of each output of each particular element.
   3. For Supplemental Rating Reports, only provide the new or revised input and output used to develop the supplement.
7. **APPENDIX F – MISCELLANEOUS (SUPPLEMENTAL)**
   1. Copies of material testing results and other miscellaneous reports/data that were used in the preparation of the Rating Report. Copies of unique reference or textbook pages that were used by the Rating Engineer in addition to AASHTO, including but not limited to those from old textbooks, codes, manuals, catalog cuts (i.e. custom rails, light standards, etc.), and design tables from manufacturers (i.e. Acrow panels, custom beam shapes, etc.).
   2. For Supplemental Rating Reports, only provide the new or revised miscellaneous information used to develop the supplement.
8. **CHECKLIST (separate from Report)**
   1. The Bridge Load Rating Report Checklist shall be submitted in a separate PDF file consisting of 8½”x11” pages.

**BRIDGE RATING**

Prepared For

**MASSACHUSETTS DEPARTMENT OF TRANSPORTATION**

**HIGHWAY DIVISION**

**DANA-PRESCOTT**

**MAIN STREET**

**OVER**

**SWIFT RIVER**

**BRIDGE NO. D-02-033=P-15-015(BG1)**

**STRUCTURE NO. D02033-BG1-DOT-NBI**

**SHODDY MILL BRIDGE**

DATE OF ROUTINE INSPECTION

DATE OF SPECIAL MEMBER, DAMAGE, OR OTHER *(if applicable)*

DATE OF RATING

PREPARED BY

DRAFT - DATE

Consultant Logo

Consultant Name

Consultant Address

Figure 7.5.2-1(1): Draft Report Cover

**BRIDGE RATING**

Prepared For

**MASSACHUSETTS DEPARTMENT OF TRANSPORTATION**

**HIGHWAY DIVISION**

**DANA-PRESCOTT**

**MAIN STREET**

**OVER**

**SWIFT RIVER**

**BRIDGE NO. D-02-033=P-15-015(BG1)**

**STRUCTURE NO. D02033-BG1-DOT-NBI**

**SHODDY MILL BRIDGE**

DATE OF ROUTINE INSPECTION

DATE OF SPECIAL MEMBER, DAMAGE, OR OTHER *(if applicable)*

DATE OF RATING

PREPARED BY

P.E. Stamp with Signature and Date

Consultant Logo

Consultant Name

Consultant Address

Figure 7.5.2-1(2): Report Cover