PROJECT NO. C-06-13

LOAD AND RESISTANCE FACTOR RATING (LRFR) IN NYS

Volume II

Final Report

by

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16. Abstract

This report develops a Load and Resistance Factor Rating (NYS-LRFR) methodology for New York bridges. The methodology is applicable for the rating of existing bridges, the posting of under-strength bridges, and checking Permit trucks. The proposed LRFR methodology is calibrated to provide uniform reliability index values for all applications. The reliability calibration of live load factors is based on live load models developed using Truck Weigh-In-Motion (WIM) data collected from several representative New York sites. The live load models provide statistical projections of the maximum live load effects expected on New York bridges.

A new set of NYS Legal Trucks along with appropriate live load factors are proposed for performing Operating Level Ratings of existing bridges. Permit load factors are calibrated for unlimited crossings of divisible loads as well as single crossings and unlimited crossings of non-divisible loads. An equation is proposed for determining Posting weight limits for bridges with low Rating Factors as a function of the effective span length. It is proposed that different posting weights be imposed for single unit trucks and semi-trailer trucks.

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VOLUME II - PREFACE

This Volume II contains the appendices for the Final Report. The Volume consists of the following:

- Appendix I – Provides the proposed NYS-LRFR Guidelines for load rating and load posting of New York State bridges. The proposed guidelines are the result of the calibration of the live load factors and load posting procedures calibrated using reliability methods as described in Volume I. The primary author of this Manual is Mr. Bala Sivakumar.
- Appendix II – Provides a review of current NYSDOT procedures for load rating, load posting and permit load checking.
- Appendix III – Provides an overview of current National practice related to the load rating, load posting and permit load checking of highway bridges.
- Appendix IV – Presents a comparison between the ratings obtained using the proposed procedures to those from the current NYSDOT method for a representative sample of New York bridges. This comparison was prepared by Bala Sivakumar and the engineering staff of HTNB.

APPENDIX I

ENGINEERING INSTRUCTION MANUAL

Load Rating / Posting Guidelines for State-Owned Highway Bridges

To:		*New York State Department of Transportation* **ENGINEERING INSTRUCTION**	**EI** **00-000**
Title: LOAD RATING/POSTING GUIDELINES FOR STATE-OWNED HIGHWAY BRIDGES			
Distribution:	Approved:		

ADMINISTRATIVE INFORMATION:

- Effective Date: This Engineering Instruction (EI) is effective upon signature.
- Superseded Issuances: The Information transmitted by this EI supersedes Engineering Instructions 88-004, 88-005, 88-006, 94-004 and 05-034.
- Disposition of Issued Materials: The information included in this EI is intended to stand alone outside of any other document.

PURPOSE: To issue guidance for prioritizing and submitting load rating calculations, posting bridges for load restrictions, and documenting and reporting load tests on state-owned highway bridges.

TECHNICAL INFORMATION: The language used in this EI to describe personnel, entities and functions is in accordance with NYSDOT's present organizational structure, with the anticipated Program Support Division organizational entity in parenthesis where appropriate.

1) INTRODUCTION

Bridge load rating is the determination of the live load carrying capacity of a newly designed or existing bridge. Load ratings are typically determined by analytical methods based on information taken from bridge plans supplemented by information gathered from field inspections or field testing. This task is vital for several reasons, including (but not limited to) the following:

- To determine which structures have substandard load capacities that may require posting or other remedial action.
- To assist in the most effective use of available resources for rehabilitation or replacement.
- Mandated by the Code of Federal Regulations – Highways, Title 23. Chapter 1 – Federal Highway Administration (FHWA), DOT, Part 650 – Bridges, Structures and Hydraulics.
- Mandated by New York State Highway Law, §230, §231, §232, & §233. NY Code of Rules and Regulations, 17 (17NYCRR), Chapter V – Uniform Code of Bridge Inspection (UCBI).
- To assist in the overload permit review process.

The New York State regulations regarding bridge load ratings are part of the UCBI, which is contained in the current NYSDOT Bridge Inspection Manual.

The Federal Highway Administration (FHWA) requires that bridge load rating results be submitted to them annually. These results are used in conjunction with other bridge inventory and inspection information to determine the Federal Bridge Sufficiency Rating, which, in turn, is a factor used to determine the eligibility of a project for the Highway Bridge Replacement and Rehabilitation (HBRR) Program. Inaccurate bridge ratings may result in incorrect eligibility determinations under the program. This document provides guidance for prioritizing and submitting load rating calculations, posting bridges for load restrictions, and documenting and reporting load tests.

1.1 DEFINITIONS and TERMINOLOGY:

AASHTO - American Association of State Highway and Transportation Officials.

AASHTO MBE - AASHTO Manual for Bridge Evaluation.

BDMS - Bridge Data Management System.

Divisible Loads: Are overweight trucks which are issued permits to carry loads that can be broken down.

FHWA - Federal Highway Administration, U.S. Department of Transportation.

Internally Redundant - Supporting primary members made up of three or more elements that are mechanically fastened together so that if one should fail the other elements will be able to internally transfer the load and still support the main structure. An example would be a riveted girder.

Inventory Level Rating (LRFR) - Generally corresponds to the rating at the design level of reliability for new bridges in the *AASHTO LRFD Bridge Design Specifications*, but reflects the existing bridge and material conditions with regard to deterioration and loss of section.

Limit State - A condition beyond which the bridge or component ceases to satisfy the criteria for which it was designed.

Load Effect - The response (axial force, shear force, bending moment, etc.) in a member or an element due to externally applied loads.

Load Factor - A load multiplier accounting for the variability of loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads.

Load Path Redundant - A structure that has multiple paths between substructure units to distribute the load in the event of failure of one of the supporting members. Examples are steel multi-girder or prestressed concrete multi-girder bridge types. NYSDOT considers a structure to be load path redundant if it has four or more load paths.

Load Posting - Live load weight restriction placed on a structure, by the owner, when a bridge is incapable of carrying the maximum legal live load. Load postings are done after an analysis that accounts for the current condition of the structure.

Load Rating Engineer (LRE) - Engineer responsible for the accuracy and quality control of load rating data for a given bridge inventory in accordance with this EI, State and Federal requirements.

Load Rating Levels - Bridge load ratings in New York State are grouped into three distinct levels of accuracy, Level 1, Level 2, and Level 3. Load Rating Levels are discussed in detail in subsequent sections.

Load Rating Unit - Functional unit responsible for statewide implementation, operations, and quality assurance of the NYSDOT load rating program, including management of the Statewide load rating database.

National Bridge Inspection Standards (NBIS) - Federal regulations establishing requirements for inspection procedures, frequency of inspections, qualifications of personnel, inspection reports, and preparation and maintenance of bridge inventory records.

Nondivisible Loads – Are overweight trucks issued permits to carry loads that cannot be broken down.

Operating Level Rating (LRFR)—Maximum load level to which a structure may be subjected. Generally corresponds to the same reliability as that of the Operating Level Rating in past load rating practice.

Quality Assurance - The use of sampling to verify or measure the level of the entire bridge inspection and load rating program.

Quality Control - System that is intended to maintain the quality of a bridge inspection and load rating at or above a certain level.

R-Posting - A load restriction for a bridge, which based on design or condition, does not have the reserve capacity to accommodate most vehicles over legal loads but, can still safely carry legal loads. Vehicles operating pursuant to an overweight permit with structure use restrictions (known as "R" Permits) are not allowed to cross. Originally established for NYSDOT's divisible load permit program, R-Postings are also used to restrict other non-divisible overload permit classifications. These bridges are identified with signage stating "No Trucks with R Permits."

Reliability Index: β—A computed quantity defining the relative safety of a structural element or structure expressed as the number of standard deviations that the mean of the margin of safety falls on the safe side. A reliability index β=3.5 imply that, based on available statistical data, there is a 2.3×10^{-4} chance that the limit state being investigated will be exceeded. β=2.5 imply a 6.2×10^{-3} probability of exceedance, β=2.0 imply a 2.3×10^{-2} probability of exceedance and β=1.5 imply a 6.7×10^{-2} probability of exceedance.

Resistance Factor - A resistance multiplier accounting for the variability of material properties, structural dimensions, workmanship, and the uncertainty in the prediction of resistance.

Serviceability - A term that denotes restrictions on stress, deformation, and crack opening under regular service conditions.

Service Limit State - Limit state relating to stress deformation and cracking.

Specialized Hauling Vehicle (SHV)—Short wheelbase multi-axle trucks used in construction, waste management, bulk cargo and commodities hauling industries.

Strength Limit State - Safety limit state relating to strength and stability.

Substantial Structural Alteration - Any work that modifies the live load capacity, load distribution or load paths or structural behavior of the bridge (UCBI).

UCBI - Uniform Code of Bridge Inspection - NY Code of Rules and Regulations, 17, Chapter V.

2) LEVEL 1 LOAD RATING GUIDELINES

2.1 INTRODUCTION:

"A Level 1 rating refers to any fully documented analysis or capacity evaluation that is signed and certified by a professional engineer, licensed by the State of New York, as being complete and correct in its computation of bridge load capacity. Generally, a Level 1 analysis shall be in conformance with the analysis assumptions and provisions of the AASHTO Manual." – UCBI 165.8 (a) (1). Rating results from Level 1 calculations are used to determine need for member strengthening, load posting, or if a structure should be closed.

A complete Level 1 load rating will include analyses of the following items:

- All elements defined as "primary members" in the NYSDOT Bridge Inspection Manual, as well as all stringer-floorbeam, girder-floorbeam, and truss connections.
- Timber and metal bridge decks.
- Timber and metal pier elements.

It is not necessary to analyze concrete bridge decks, concrete and masonry substructure elements, or foundation elements unless there are unusual circumstances which, in the load rating engineer's judgment, will affect the load carrying capacity of the bridge. Secondary members subject to impact damage or deterioration shall also be investigated if the capacity of a primary member is affected.

Level 1 load ratings are required for all new and replacement bridges, and for all rehabilitation and repair designs involving a substantial structural alteration. Level 1 rating calculations shall be performed as part of the structural analysis process used for design and reflect the bridge as-built or as-rehabilitated construction and configuration. As an example, a new bridge design will account for a future wearing surface, but the Level 1 load rating does not include this future wearing surface as a dead load because it is not part of the as-built condition. This rule also applies to a Level 2 analysis which accounts for the current conditions of the structure.

Ratings shall be calculated following the guidelines contained in the latest edition of the AASHTO MBE and this document. This document provides guidance to load rating engineers for performing and submitting load rating calculations, posting bridges for load restrictions, and checking overweight permits using the Load and Resistance Factor Rating (LRFR) methodology. This document serves as a supplement to the AASHTO MBE and deals primarily with NYSDOT specific load rating requirements, interpretations, and policy decisions.

Load and Resistance Factor Rating is consistent with the LRFD Specifications in using a reliability-based limit states philosophy and extends the provisions of the LRFD Specifications to the areas of inspection, load rating, posting and permit rules, fatigue evaluation, and load testing of existing bridges. The LRFR methodology has been developed to provide uniform reliability in bridge load ratings, load postings and permit decisions. LRFR provisions allow for calibrating load factors based on statewide vehicle load data. This provides an opportunity to refine the LRFR process while meeting an acceptable index of structural reliability and provide results that are reasonably compatible with current NYSDOT procedures based on LFD/ASD rating methods. The LRFR procedures provide live load factors for load rating that have been calibrated using statewide vehicle load data to provide a uniform and acceptable level of reliability reasonably consistent with NYSDOT LFD/ASD rating practices.

2.2 ANALYSIS FREQUENCY:

Level 1 calculations eventually become outdated. Member deterioration, rehabilitation, redecking, and repaving of the wearing course are just a few of the occurrences that may force a reanalysis of the bridge. Therefore, the required frequency of Level 1 calculations can vary widely. A new bridge designed to current standards may not need another Level 1 for some time if it is maintained properly. However, for example, an old truss that is deteriorating steadily should be reanalyzed as conditions change every few years.

The Load Rating Engineer (LRE) or other qualified person should review any existing Level 1 data during or after each inspection to see if a reanalysis is needed. A new Level 1 analysis may be necessary if any of the following have occurred since the last Level 1 analysis was completed.

- The primary member condition rating on the inspection report has changed by more than one point, if the initial rating was 5 or lower.
- Dead load has changed significantly due to resurfacing or other nonstructural alterations.
- Section properties have changed due to rehabilitation, redecking, deterioration, or other alterations.

If Level 1 load ratings stored in NYSDOT's statewide database are invalid, these ratings shall be deleted from the database by the LRE or other designated qualified personnel.

The Priorities for Level 1 analysis may be set in the following order:

1. Bridges which appear to require R posting or load posting.
2. Bridges with primary member ratings less than 4 (using NYSDOT's 1-7 rating scale) that are not ratable by NYSDOT's standard load rating system.
3. Bridges that are ratable by NYSDOT's standard load rating system with primary member ratings less than 4.

2.3 LRFR RATING PROCESS FOR NYSDOT OWNED BRIDGES

Live loads to be used in the rating of bridges are selected based upon the purpose and intended use of the rating results. Live load models outlined below shall be evaluated for the Strength, Service and Fatigue limit states in accordance with Table 8:

1) **Design load Rating**: Design load rating is a first-level rating performed for all bridges using the HL-93 loading at the Inventory (Design) and Operating levels.
2) **Legal Load Rating**: Bridges that have an HL-93 Rating Factor < 1.0 at the Operating Level shall be load rated for the AASHTO posting load SU4 and Type 3S-2 to determine posting needs
3) **Permit load Rating**: Bridges that do not need load or "R" posting may be evaluated for Overload Permits. Bridges that have an HL-93 RF < 1.0 at the Operating level shall be evaluated for R-posting as specified in Section 6.2.

2.4 REPORTING LRFR RATINGS TO THE NBI

For all new load ratings based on the LRFR methodology, the load rating data shall be reported to the NBI as a Rating Factor, for items 63, 64, 65 and 66, using the HL-93 loadings.

2.5 TRUCK TRAFFIC CONDITIONS AT BRIDGE SITE

LRFR live load factors appropriate for use with legal loads and permit loads are defined based upon the Average Daily Truck Traffic (ADTT) available or estimated for the bridge site. FHWA requires an ADTT to be recorded on the Structural Inventory and Appraisal (SI&A) form for all bridges. In cases where site traffic conditions are unavailable from the bridge file, the NYSDOT Traffic Data Services should be contacted for current ADTT information.

2.6 SELECTION OF SURFACE ROUGHNESS RATING

LRFD dynamic load allowance of 33% reflects conservative conditions that may prevail under certain distressed approach and bridge deck conditions. For load rating of legal and permit vehicles for bridges with less severe approach and deck surface conditions, the dynamic load allowance (IM) may be decreased based on field observations in accordance with MBE Table C6A.4.4.3-1 (See Section 2.8.5). Inspection should carefully note these and other surface discontinuities in order to benefit from a reduced dynamic load allowance.

To ensure proper and consistent selection of dynamic load allowance values in all load ratings, NYSDOT should consider a new data item in the Bridge Inspection Forms for documenting the surface roughness of the bridge riding surface, with clear guidelines for inspectors on how to assign a rating for this item. Surface Roughness is defined as follows:

Table 1 Surface Roughness Rating

Surface Roughness Rating	Description
3 = Smooth	Smooth riding surface at approaches, bridge deck, and expansion joints
2 = Average	Minor surface deviations or depressions
1 = Poor	Significant deviations in riding surface at approaches, bridge deck, and expansion joints

2.7 LRFR LOAD RATING EQUATION AND FACTORS

The general rating equation in LRFR (MBE Eq. 6A.4.2.1-1) is given as:

$$RF = \frac{\phi_c \ \phi_s \ \phi \ R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_p)(P)}{(\gamma_L)(LL + IM)}$$

RF = Rating Factor

R_n = Nominal member resistance (as inspected)

ϕ_c = Condition Factor (Section 2.7.2)

ϕ_s = System Factor (Section 2.7.3)

ϕ = LRFD Resistance Factor

DC = Dead load effect due to structural components and attachments

DW= Dead load effect due to wearing surface and utilities

P = Permanent loads other than dead loads (secondary prestressing effects, etc.)

LL = Live load effect of the rating vehicle

IM = Dynamic load allowance (Section 2.8.5)

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

γ_L = Evaluation live load factor for the rating vehicle (Section 2.81., 2.8.2, 2.8.3)

Where, the following lower limit shall apply:

$$\phi_c \ \phi_s \geq 0.85$$

Additionally, for all non-strength limit states, φ =1.0, ϕ_c = 1.0, ϕ_s = 1.0

The NYSDOT LRFR methodology presented in this document is based on a recalibration of the live load factors performed to provide reliability levels consistent with those implied in NYSDOT load rating, posting and permitting practices based on load factor and working stress analyses that have been known to provide adequate levels of safety. A target reliability index β=2.0 was intentionally chosen to be slightly higher than that observed in current rating methods to account for the expected growth in truck volumes over time. This target reliability index was extracted based on current and past NYSDOT rating methods and truck loads and multiple presence frequencies observed from recent New York Weigh-In-Motion (WIM) data.

The Dead Load factors are the same as those provided in the AASHTO LRFD. The dead load factors in the AASHTO LRFD were calibrated to provide uniform levels of reliability at the design and inventory rating levels and should be maintained to ensure consistency between bridge load rating and design.

2.7.1 Resistance Factor ϕ

Resistance factor φ has the same value for new design and for load rating. Resistance factors, φ, shall be taken as specified in the LRFD Specifications for new construction. The resistance factors in the AASHTO LRFD were calibrated to provide uniform levels of reliability and should be maintained to ensure consistency between bridge load rating and design.
The nominal resistance used for load rating shall be the as-inspected member resistance accounting for measured section losses and deterioration.

2.7.2 Condition Factor ϕ_c

A reduction factor based on member condition as evaluated using the New York condition rating system that rates the condition on a scale of 1 through 7. The Condition Factor ϕ_c, is applied to the resistance of degraded members. The Condition Factor ϕ_c, does not account for section loss, but is used in addition to section loss. An increased reliability index is maintained for deteriorated and non-redundant bridges by using condition and system factors in the load rating equation.

Table 2 NYSDOT Condition Factor: ϕ_c .

Structural Condition of Member	Condition Rating	ϕ_c
Fair, satisfactory or good	$\geq$ 4	1.0
Poor	$\leq$ 3	0.95

2.7.3 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 factor member capacities reduced, and, accordingly, will have lower ratings. The aim of the system factor is to provide additional reserve capacity for bridges with primary members that are both internally and load path nonredundant. Subsystems that have redundant members should not be penalized if the overall system is non-redundant (i.e. multi stringer deck framing members on a two-girder or truss bridge). System Factor is used with all live load models.

Current NYSDOT policy is to use the system factors provided in Table MBE 6A.4.2.4-1 (Table 3) when load rating for Flexural and Axial Effects for steel members and non-segmental concrete members for Legal Load Ratings only. The system factor is set equal to 1.0 when checking shear. The load modifiers provided in LRFD shall be used for Design Load Inventory and Operating Ratings and for Permit Load Ratings (Annual Divisible, Non-Divisible and Special Hauling)

Table 3. MBE 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 or Bolted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
All Other Girder Bridges and Slab Bridges	1.00
Floorbeams with Spacing >12ft. and Non-Continuous Stringers	0.85
Redundant Stringer Subsystems Between Floorbeams	1.00

Definitions
Floorbeam: A horizontal flexural member located transversely to the bridge alignment.
Stringer : A longitudinal beam supporting the bridge deck.
Girder: A large flexural member, usually built-up, which is the main or primary support for the structure, and which usually receives load from floorbeams, stringers, or in some cases directly from the deck.

2.8 LRFR LIVE LOADS AND LOAD FACTORS

2.8.1 Design Load Rating

The design-load rating (or HL-93 rating) assesses the performance of existing bridges utilizing the LRFD HL-93 design loading and design standards with dimensions and properties for the bridge in its present as-inspected condition. It is a measure of the performance of existing bridges to new bridge design standards contained in the LRFD Specifications. The design-load rating produces Inventory and Operating level rating factors for the HL-93 loading. The evaluation live-load factors for the Strength I limit state shall be taken as given in MBE Table MBE 6A.4.3.2.2-1.

Table 4 MBE 6A.4.3.2.2-1 Load Factors for Design Load: γ_L

Evaluation Level	Load Factor
Inventory	1.75
Operating	1.35

The dynamic load allowance specified in the LRFD Specifications for new bridge design (LRFD Article 3.6.2) shall apply. For design load rating, regardless of the riding surface condition or the span length, always use 33% for the dynamic load allowance (IM).The results of the HL-93 rating are to be reported to the NBI as a Rating Factor.

2.8.2 Legal Load Rating

In LRFR, load rating for legal loads determines a single safe load capacity of a bridge. The previously existing distinction of Operating and Inventory level ratings is no longer maintained when load rating for legal loads.

The live load to be used in the NYSDOT LRFR rating for posting considerations shall be the governing loading from the legal loads given in Figure 1. For example, for simple spans less than 200 ft, the governing load effect from either the SU4 or the Type 3S2 loading shall be used in the load rating.

It is unnecessary to place more than one vehicle in a lane for spans up to 200 ft. because the LRFR live load factors provided have been modeled for this possibility (no lane load to be used). For negative moments and for span lengths greater than 200 ft., critical load effects shall be obtained by lane-type legal load models shown in Figure 1.

A reliability index β_{target}=2.0 was chosen for target during the recalibration of the live load factors for ratings. This target is slightly more conservative than the average reliability value implicit in New York State DOT Load Factor Rating procedures and loading projected from New York WIM data. The SU-4 vehicle and the AASHTO legal 3-S2 trucks for the NYSDOT LRFR ratings are recommended as these two trucks provide a good envelope of the live load effects by reducing the spread in the reliability index values for the range of spans considered.

The live load factors for multi-lane bridges were calibrated based on the weight histograms and probability of multiple presence of trucks assembled from New York WIM data to provide a uniform reliability index β=2.0 within a 5-year rating period. Bridges with higher ADTT have a higher probability of being loaded by heavy trucks and require higher live load factors.

The NYSDOT live load factors are higher than those of the AASHTO LRFR because of the heavier truck loadings observed in the State. The multi-lane live load factors were calibrated to also provide an envelope to multi-lane bridges loaded by a single lane of trucks. This implies that the single lane loading with the higher live load factor in Table 5 does not need to be checked for multi-lane bridges.

Higher live load factors are used for single lane bridges because of the higher probability of having one heavy truck in one lane bridges than having multiple heavy trucks in multi-lane bridges. The multiple presence factor included in the current AASHTO LRFD single-lane distribution factor does not sufficiently reflect the current truck load intensities in the State, requiring the adoption of higher live load factors for single lane bridges.

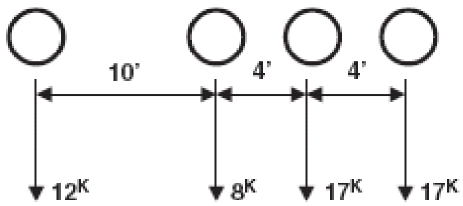
The rating live-load factors for legal loads for the Strength I limit state shall be taken as given in Table 5.

Table 5 NYSDOT Live-Load Factors, γ_L for Legal Loads

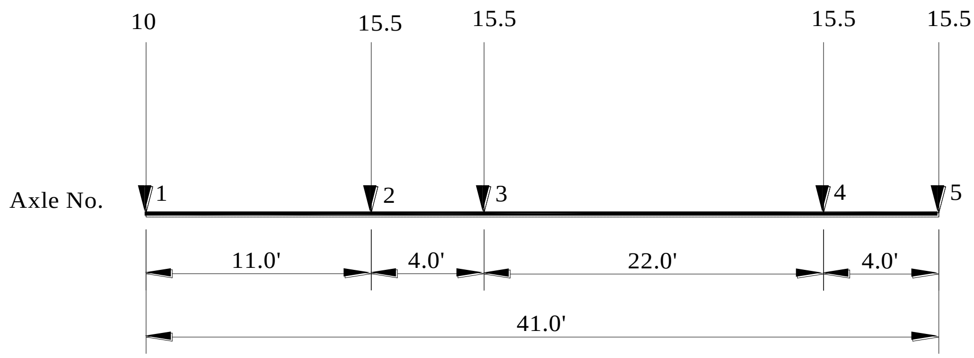
Traffic Volume (one direction)[1]	Load Factor for Multi-lane bridges (use LRFD load distribution factor for multi-lanes)	Load Factor for Single-lane bridges (use LRFD load distribution factor for a single lane without removing the multiple presence factor)[2]
ADTT $\geq$ 5000	1.95	2.65
ADTT=1000	1.85	2.50
ADTT $\leq$ 100	1.65	2.20

[1] Linear interpolation is permitted for other ADTT
[2] The AASHTO LRFD load distribution factor tables for single loaded lanes already includes a multiple presence factor MP=1.2. This factor must be included when the analysis employs other methods for determining the load on a bridge member. For instance, when the lever rule is used for live load distribution to longitudinal or transverse members, the 1.2 multiple presence factor shall be included in the distribution analysis.

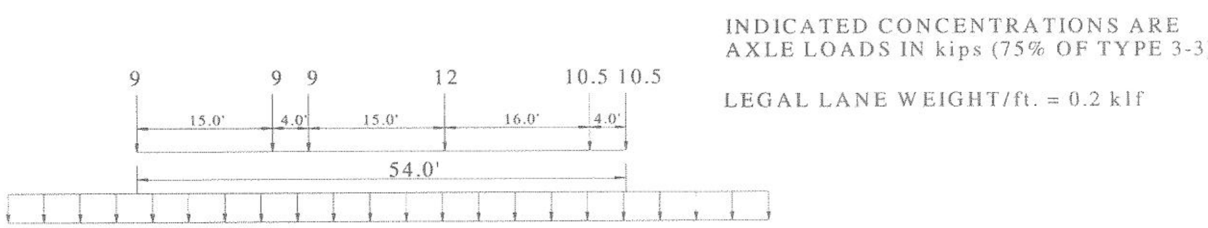
- SU4 Legal Load (27 tons)



- Type 3S2 Legal Load (36 tons)

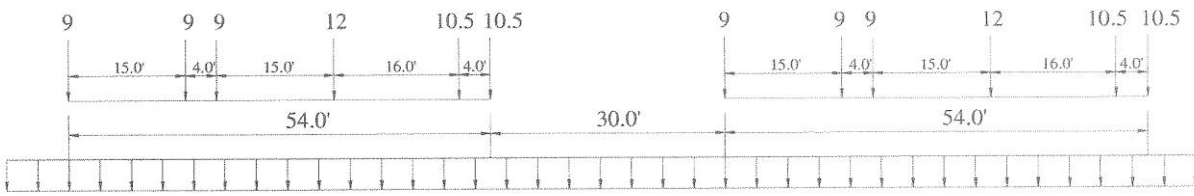


c) Lane-Type Legal Load Model—Apply for spans greater than 200 ft. and all load effects.



MBE APPENDIX A-6A.4, Figure A-6A.4-4

d) Lane-Type Legal Load Model—Apply for negative moment and interior reaction for all span lengths.



MBE APPENDIX A-6A.4, Figure A-6A.4-5

Figure 1 Legal Load Models for NYSDOT LRFR Ratings

2.8.3 Permit Load Rating

NYSDOT has a set of established procedures to allow the passage of vehicles that exceed the legally established weight limits. Special Hauling Permits and Divisible Load Overweight Permits are issued by NYSDOT to protect public safety and preserve the State's infrastructure. Special Hauling Permits are required to allow the movement on New York State highways of vehicles or loads that exceed the legal dimensions or weights specified in Section 385 of the New York State Vehicle and Traffic Law. If the permit application is for self propelled construction equipment or for vehicles with a gross weight of 140,000 lbs or greater, a structural review by the Office of Structures must be performed. Loads with gross weights that are 200,000 lbs or greater are classified as superloads and are subject to special requirements. .

New York State has several different Permit classifications depending on the permit loading type and number of trips allowed. For the purposes of LRFR evaluations the permit loading types have been grouped into two categories, those carrying divisible loads and those carrying non-divisible loads. Non-divisible load permits are assumed to be controlled so that the truck weights are known and taken to be to be equal to the permitted weight. Divisible load permits are allowed unlimited crossings over an unrestricted bridge for a year's period, with a probability of exceeding the permit weight limits. Permit configurations belonging to each permit type used as calibration trucks are shown in Figures 2 and 3.

In terms of trip categories, the permits in this report will be divided into single-crossing (single-trip) and unlimited crossing (multi-trip) permits. Single Trip Overweight Permit load analysis assumes only one permit load on the bridge, which allows the use of the single-lane distribution. As stated in Table 6, when using a single-lane LRFD distribution factor, the 1.2 multiple-presence factor should be divided out from the distribution factor equations. For single trip permit vehicles, it is important to note that the vehicle could traverse the bridge in any lane, making it necessary to investigate whether the interior or exterior girder controls the load rating.

For continuous spans one permit truck is applied.

A reliability index β_{target}=2.0 was chosen for target during the recalibration of the live load factors for permits. This target is slightly more conservative than the average reliability value implicit in current New York State DOT practice and loading projected from New York WIM data. Calibration studies demonstrate that using a live load factor γ_L=1.10 for non-divisible permit loads, where the vehicles operate at the permitted weight, will provide average reliability index values greater than the target β_{target}=2.0. For the cases of divisible loads where some data shows that Permit loads may exceed the Permit weight limits, the load factors have been increased accordingly.

For Multi-lane bridges, the Permit live load factors account for the probability of having a permit truck alongside a random overweight truck in the adjacent lane. These permit live load factors are lower than those for legal load rating reflecting the lower probability of having both trucks exceed the permit load limits as compared to the probability of having two random trucks exceed the legal truck weights.

Lower live load factors are used for non-divisible loads and special hauling permits because these trucks are less likely to exceed the authorized permit weight.

Similar divisible live load factors are used for single lane bridges to those of multi-lane bridges to envelope the reliability of continuous span single lane bridges.

The rating live-load factors for permits for the Strength II limit state shall be taken as given in Table 6.

Table 6 NYSDOT Permit Load Factors, γ_L

Permit Type	Frequency	Loading Condition	DF	ADTT* (one direction)	Permit Load Factor, γ_L
Annual Divisible Load	Unlimited trips	Multi-lane bridges Mix with traffic	Multi-lane	ADTT ≥ 5000	1.20
				ADTT=1000	1.15
				ADTT ≤ 100	1.10
Annual Divisible load	Unlimited trips	Single lane bridges	Single Lane DF after dividing out MP=1.2	ADTT ≥ 5000	1.20
				ADTT=1000	1.15
				ADTT ≤ 100	1.10
Non-divisible loads	Unlimited trips	Multi-lane bridges Mix with traffic	Multi-lane	All ADTT	1.10
Non-Divisible loads	Unlimited trips	Single lane bridges	Single Lane DF after dividing out MP=1.2	All ADTT	1.10
Special Hauling and Superloads	Single Crossing	Multi-lane bridges Mix with traffic	Single Lane DF after dividing out MP=1.2	All ADTT	1.10
Special Hauling and Superloads	Single Crossing	Single lane bridges	Single Lane DF after dividing out MP=1.2	All ADTT	1.10

*Linear interpolation is permitted for other ADTT

2.8.4 Use of HL-93 Ratings for Screening Bridges

The first level load rating in LRFR is the HL-93 design load check at the Inventory and Operating levels. This check can serve as an effective technique to identify bridges that can safely carry legal load ratings and/or permit loads, thus reducing the number of bridges needed further analysis for other load models.

Analysis of factored load effects (moment and shear) for NY divisible load permits, non-divisible load permits and legal loads used in the calibration (see Appendix A) were generated and compared to the factored HL93 loads at the Inventory and Operating levels for simple and continuous spans from 20 ft to 200 ft. The load effects were normalized by dividing by HL-93 load effects. Moment and shear ratios obtained are shown in the charts given in Appendix A. The results show the following rules regarding screening to be valid:

1. HL-93 rating at the Operating level (LF=1.35) is appropriate for screening multi-lane bridges for AASHTO and NY legal loads. That is multi-lane bridges that pass HL-93 rating (RF ≥ 1.0) at the Operating level will have adequate load capacity for legal loads.
2. HL-93 rating at the Inventory level (LF=1.75) is appropriate for screening single lane bridges for AASHTO and NY legal loads. That is single-lane bridges that pass HL-93 rating (RF ≥ 1.0) at the Inventory level will have adequate load capacity for legal loads.
3. HL-93 rating at the Operating level (LF=1.35) is appropriate for screening bridges for NY divisible load permits (LF = 1.2) similar to the NYP6 thru NYP 13 configurations shown in Fig 3. That is bridges that pass HL-93 rating (RF ≥ 1.0) at the Operating level will have adequate load capacity for the class of divisible load permits.
4. HL-93 rating at the Inventory level (LF=1.75) is appropriate for screening bridges for NY non-divisible load permits (LF=1.1) similar to the NYP1 thru NYP 5 configurations shown in Fig 2. That is bridges that pass HL-93 rating (RF ≥ 1.0) at the Inventory level will have adequate load capacity for the class of non-divisible load permits.

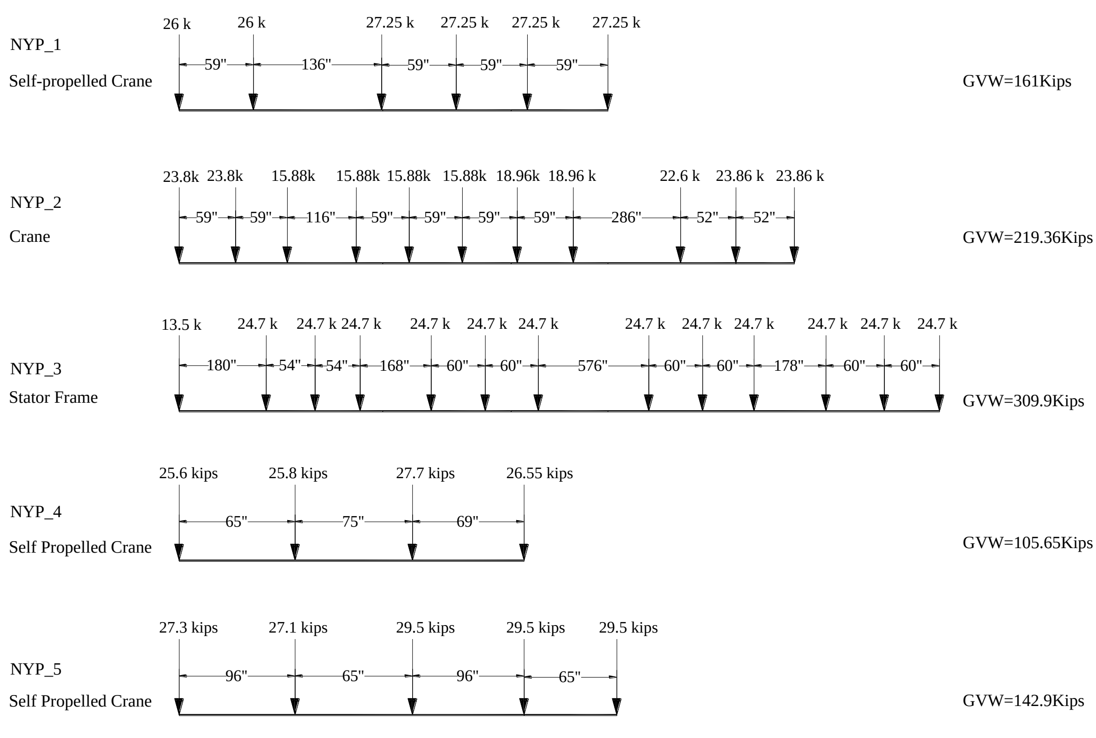


Fig. 2 - Calibration Trucks : NY Non-Divisible Load Permits

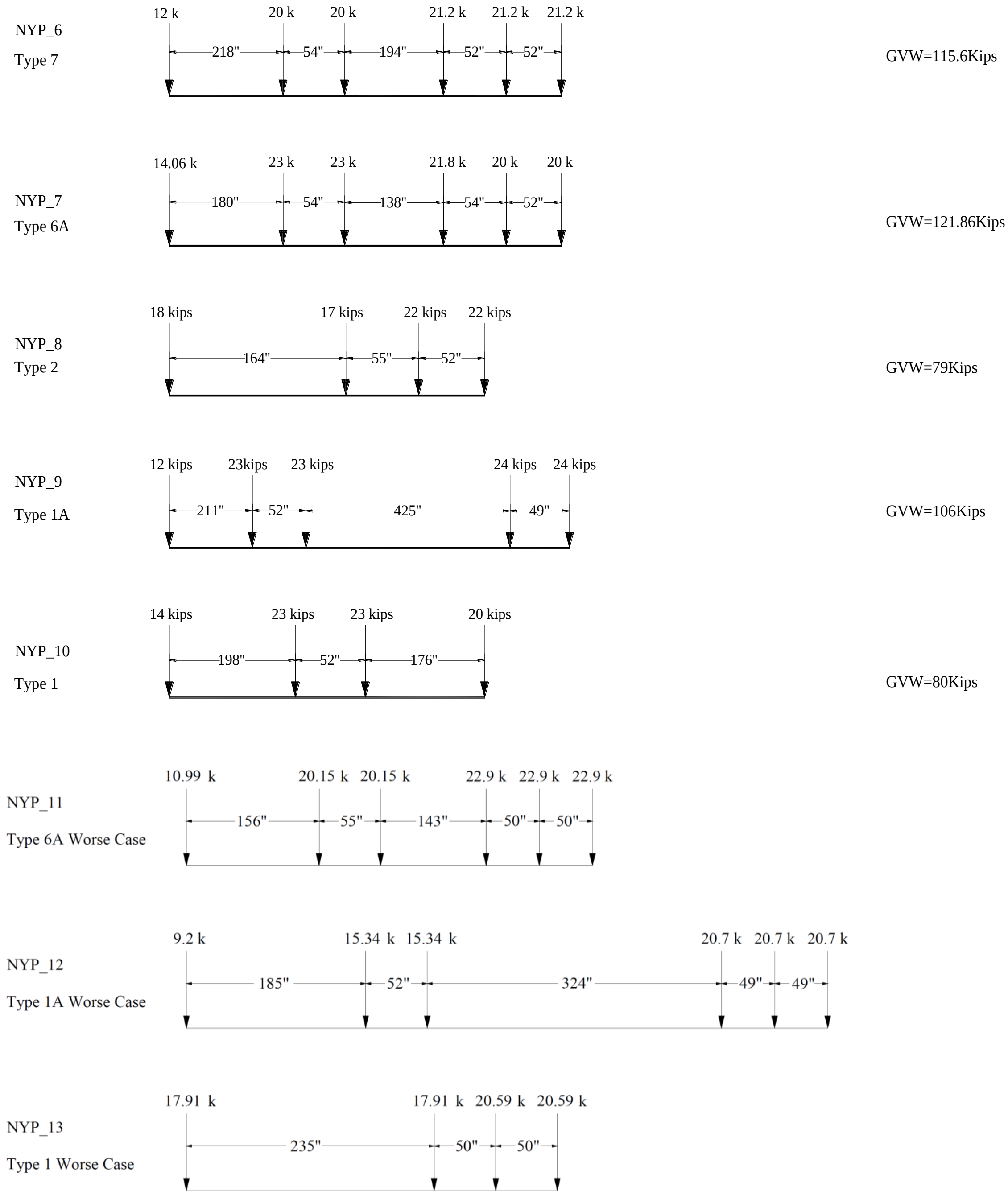


Fig. 3 - Calibration Trucks : NY Divisible Load Permits

2.8.5 Reduced Dynamic Load Allowance for Rating (Legal and Permit Loads)

For legal and permit vehicles rating, of longitudinal members having spans greater than 40 ft. with less severe approach and deck surface conditions, the Dynamic Load Allowance (IM) may be decreased from the LRFD design value of 33%, as given below in Table 7, for the Strength and Service limit states. Dynamic load allowance shall be applied to the state legal vehicles and not the lane loads. Selection of IM shall be in accordance with the requirements of Section 2.6 and the Surface Roughness rating noted in the inspection report. State or document what value of IM was used for the load rating in the Load Rating Summary Form. If the permit vehicle proceeds at a crawl speed, no more than 5 miles per hour, then the impact can be assumed to be 5%.

Table 7 Dynamic Load Allowance for Rating: IM.

Riding Surface Rating*	IM
3	10%
2	20%
1	33%

* Regardless of riding surface condition, always use 33% for spans 40 ft or less and for transverse members.

2.9 LRFR LIMIT STATES FOR EVALUATION

Live load models described in Section 2.8 shall be evaluated for the Strength, Service and Fatigue limit states in accordance with Table 8:

Table 8 LRFR Limit States

Bridge Type	Limit State	Design	Legal	Permits
		HL-93	SU4, Type 3-S2, Lane Loads	Divisible, Non-Divisible Special Hauling Superloads
Steel	Strength I	●	●	
	Strength II			●
	Service II	●	●	●
	Fatigue	●		
Reinforced Concrete	Strength I	●	●	
	Strength II			●
	Service I			●
Prestressed Concrete (non-segmental)	Strength I	●	●	
	Strength II			●
	Service III	●		
	Service I			●
Timber	Strength I	●	●	
	Strength II			●

2.9.1 Concrete Bridges

- For non-segmental prestressed concrete bridges, LRFR provides a limit state check for cracking of concrete (SERVICE III) by limiting concrete tensile stresses under service loads. SERVICE III check shall be performed during design load ratings. The allowable tensile stress in precompressed tensile zone for the Inventory level design load check shall be in KSI units. Service III need not be checked for design load Operating ratings.

- Service I and Service III limit states are mandatory for load rating of segmental concrete box girder bridges (MBE 6A.5.14).

- A new SERVICE I load combination for reinforced concrete components and prestressed concrete components has been introduced in LRFR to check for possible inelastic deformations in the reinforcing steel during heavy permit load crossings (MBE 6A.5.4.2.2.2). This check shall be applied to permit load checks and sets a limiting criterion of $0.9F_y$ in the extreme tension reinforcement. Limiting steel stress to $0.9F_y$ is intended to ensure that there is elastic behavior and that cracks that develop during the passage of overweight vehicles will close once the vehicle is removed. It also ensures that there is reserve ductility in the member.

2.9.2 Steel Bridges

- Steel structures shall satisfy the overload permanent deflection check under the SERVICE II load combination for design load and legal load ratings using load factors as given in Table 8. Maximum steel stress is limited to 95% and 80% of the yield stress for composite and non-composite compact girders respectively. Service II checks for permit loads are recommended but optional. During an overweight permit review the actual truck weight is available, so a 1.0 live load factor is specified.

- In situations where fatigue-prone details are present (category D or lower) a Fatigue limit state Rating Factor for infinite fatigue life may be computed for Level I load ratings. If directed by NYSDOT, bridge details that fail the infinite-life check can be subject to the more complex finite-life fatigue evaluation using evaluation procedures given in the AASHTO MBE (Section7).

2.10 DOCUMENTATION AND SUBMISSIONS:

All Level 1 calculations must be certified as accurate by a professional engineer currently licensed in New York State. They must be performed and checked according to standard structural engineering practice. If using a computer program, note the program name and version. Also, all input information must be documented. Both Allowable Stress and Load Factor are acceptable analysis methods but, Load and Resistance Factor is the preferred rating method. Load ratings may be submitted in English or metric units.

The attached flowchart at the end of this section (Fig 4) shows the proper work flow for the Level 1 calculations. When a new Level 1 analysis is done, a copy of all pertinent documentation should be kept in the responsible Region office

Each NYSDOT Region (or Program Support Center responsible for Regional load rating engineering services) shall provide new Level 1 summaries to the NYSDOT Load Rating Unit after completion. For each bridge, Level 1 data should be summarized in terms of structure rating units. A structure rating unit is defined as a single simple span or a continuous series of spans that are analyzed as a single structural unit. Thus, a bridge with three simple spans will have three rating units, but a bridge with four continuous spans will have only one rating unit.

Level 1 load rating documentation shall be incorporated into a comprehensive package to facilitate updating of the information and calculations in the future, as well as documenting the assumptions that were used. For new, replacement, or rehabilitation projects, the Level 1 load rating package shall be transmitted as part of the Plans Specifications and Estimate (PS&E).

The following information shall be included in the Level 1 Load Rating package. Additional information may be required as part of the scope of services.

- Cover sheet with BIN; feature carried/feature crossed; political unit and county; rating summary table; analysis method and controlling member; engineers responsible for Level 1 load rating calculations (done by, checked by), approving PE signature, license number, and date.

- Table of contents.

- Level 1 Load Rating Summary Sheets for each unique member type to include 'HL-93' inventory and operating ratings. Legal Load ratings shall also be included if the 'HL-93' operating rating is less 1.0.

- General Information Sheet:

  1) Bridge Identification Number (BIN)
  2) Date load rating performed:
  3) Political Unit:
  4) Feature carried:
  5) Feature crossed:
  6) Superstructure type
  7) Number of spans
  8) Skew:
  9) Total length:
  10) Out-to-out width:
  11) Bridge width curb-to-curb
  12) Number of actual travel lanes
  13) Number of lanes used in rating
  14) Type of deck
  15) Type of wearing surface
  16) Type of sidewalks
  17) Barrier or railing type
  18) Year built:
  19) Rehabilitation year(s)
  20) Design live load
  21) Existing posted load
  22) List of plans or sketches referenced should be provided for an existing structure
  23) Date of most recent inspection should be provided for an existing structure

- Drawings or sketches of Superstructure Framing Plan, Typical Cross Section and Girder Elevation. For new or rehabilitation designs, also include Moment and Shear Tables and Design Load Table.

- General description and comments affecting the Load Rating, such as structure condition, flags, posting history, etc.

- Assumptions and analysis methods

- Live load distribution method used (AASHTO Standard Bridge Specifications, lever rule, AASHTO Guide Specification, 3D analysis, etc.)

- Dead load distribution (tributary area, simple beam distribution, continuous transverse beam distribution, 3D analysis, etc.)

- Analysis method, assumptions and design criteria

- Analysis

    - Section properties: As-built and deteriorated section properties as applicable; composite section properties
    - Material properties and any assumptions.
    - Copy of any hand calculations.
    - Dead load effects, with distribution method stated. This may be taken from computer output, assuming it is easy to follow

    - All hand calculations for all dead loads or those needed for dead load inputs shall be included.
    - Dead load assumptions, such as the weight of barriers/railings, utility lines, etc., shall be included.

    - Live loads effects, with distribution method stated and impact factor calculation

        - All required hand calculations shall be included.
        - If alternative distribution factors are used, an explanation of why an alternative method was used and all necessary calculations shall be included

    - Member capacities for controlling section and limit state.
    - A listing of what software was utilized including version number.
    - Copy of software input where applicable.
    - At a minimum, a printout of the summarized output
    - Safe load and load posting calculations if applicable

- Rating Results: Tabulated by structural rating unit with controlling member for controlling unit with controlling limit state.

<u>Notes:</u>

All input sheets and calculation sheets shall show both the rater and checker.

All inspection reports, manuals, textbooks, and articles referenced as part of the load rating package shall be documented.

Typically, the substructure is not analyzed as part of a load rating; however there are cases where the substructure shall be analyzed, such as steel cap beams and steel columns. In these cases, those calculations shall be included in the load rating of the structure. At the LRE’s discretion, other substructure elements not normally included in a Level 1 may need to be analyzed on an existing structure. This may be necessary in cases of extreme concrete deterioration or other mitigating circumstances.

Note: As previously stated, All Level 1 calculations must be certified as accurate by a professional engineer currently licensed in New York State.

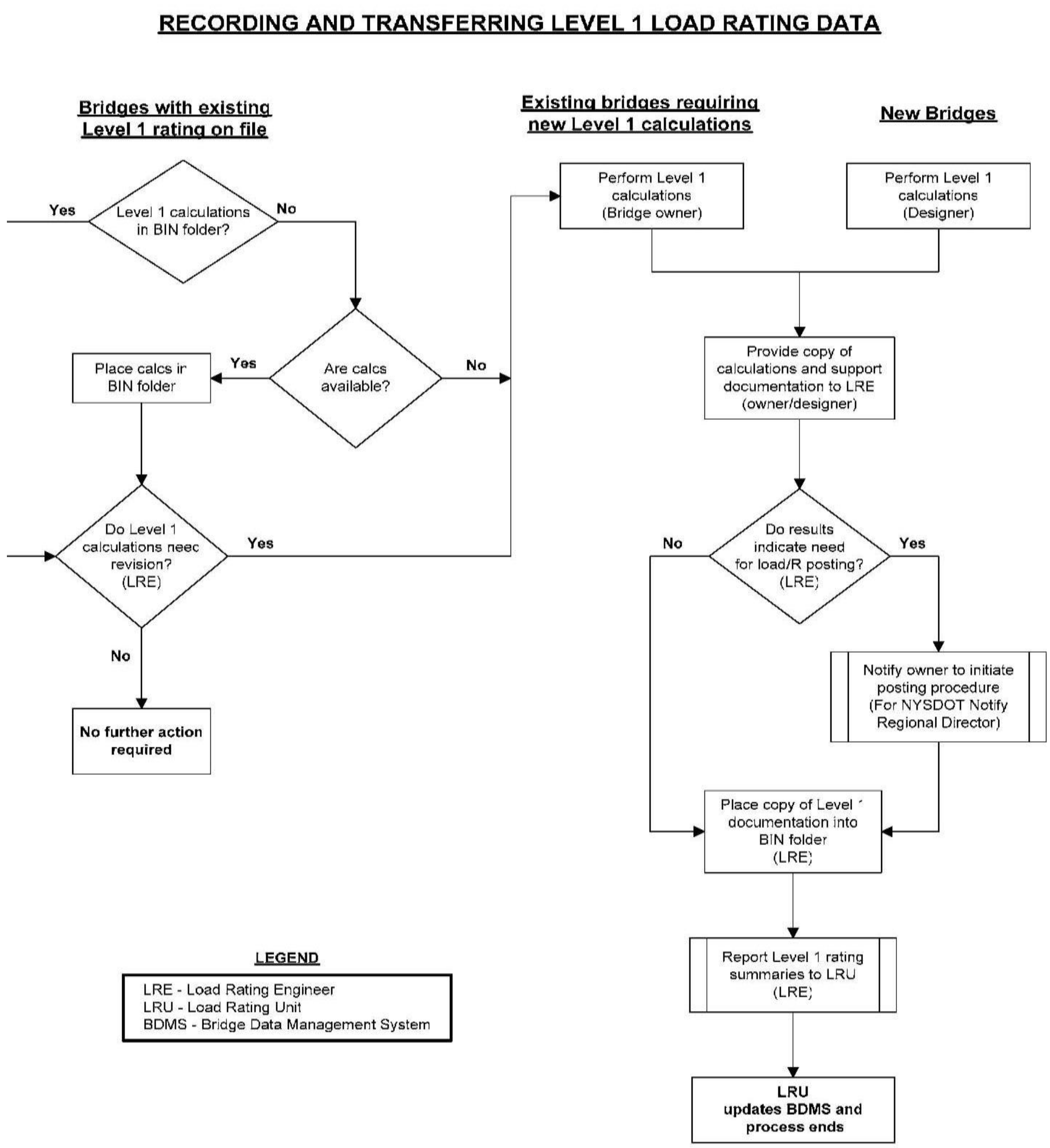


Fig 4. Level 1 Flowchart

3) LEVEL 2 LOAD RATING GUIDELINES

3.1 INTRODUCTION

Level 2 load ratings are computer generated analyses of bridges produced by NYSDOT using its current bridge load rating computer system - AASHTO Virtis. The Load Rating Engineer is responsible for collection and Quality Control of Level 2 data for their assigned bridge inventory. The Load Rating Unit is responsible for Quality Assurance of all load rating work and management of the statewide load rating database. Input data for Level 2 ratings is generally collected as part of the NYSDOT bridge inspection program. Level 2 load rating work that is performed by consultants as part of their general bridge inspection agreements for the NYSDOT shall conform to NYSDOT specifications and standards before it is submitted to NYSDOT.

3.2 ANALYSIS FREQUENCY

All bridges ratable by the current NYSDOT Level 2 Load Rating System shall be entered for analysis. As part of each Biennial bridge inspection, Level 2 load rating information shall be updated and the load ratings subsequently regenerated and submitted. An analysis shall be completed whether or not there has been any change to the input data. Specification changes, which are incorporated in each release of Virtis, may affect previous load rating results as well as new analysis modules that could analyze previously unratable structures. The Bridge Data Management System (BDMS) will also record an analysis date in the inventory database for processed ratings. By updating the analysis there will be a time stamp verification that the load rating for a particular structure was evaluated as part of its biennial inspection and is still valid.

Consultants performing a Level 2 load rating analysis shall submit their results to the respective LRE. The LRE shall be responsible for transferring this data into BDMS. The Load Rating Unit is responsible for all Level 2 Quality Assurance activities. This includes final approval of submitted Level 2 load ratings in BDMS.

3.3 ANALYSIS AND SUBMISSION PROCEDURE

The flowchart at the end of this section (Fig 5) outlines the updating, recording, and transferring of Level 2 load rating data.

The inspector shall verify in the field the information in the BIN folder needed for the Level 2 load rating analysis. This is the Level 2 field data and may include existing bridge plans that are marked up by the Inspector or spreadsheet forms prepared by the LRE. The Level 2 field data required to perform a Level 2 analysis is at the discretion of the LRE and may vary. If there are changes, the LRE or designated staff or consultant will update the information in the BIN folder with the new data. The LRE or consultant will regenerate the Level 2 Load Rating analysis with the current data and report the new results.

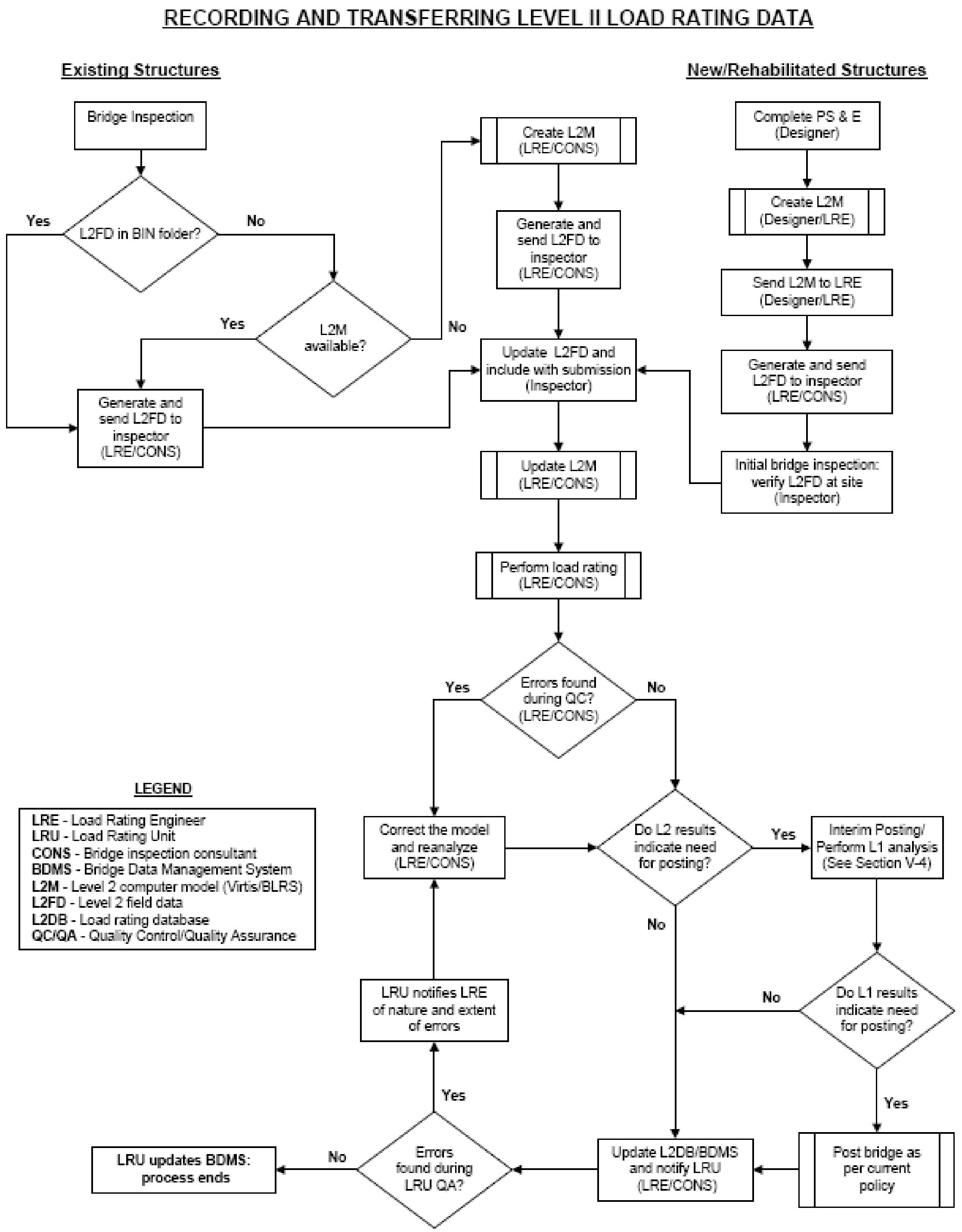


Fig 5. Level 2 Flowchart

4) LEVEL 3 LOAD RATING

When no Level 1 or Level 2 load rating exists, BDMS will generate a Level 3 load rating for the structure based on existing general inventory and inspection information such as design load, condition rating, existing posting values, etc. These ratings are not based on an analysis of the structure but on an estimate of the probable capacity of the bridge from the parameters mentioned.

These ratings are only to be used to report rating values to the FHWA when better information is not available. These ratings are not to be used for any type of structural evaluation or overload permit review

5) LOAD POSTING

5.1 LOAD POSTING REQUIREMENTS FOR BRIDGES

This section provides guidance for load posting of NYSDOT-owned highway bridges. Because of the varying nature of structural systems, materials, frequency of loadings, and other factors which may affect a load posting, no rigid set of rules can be adopted that would be appropriate in every case.

The Region initiating the posting or change in posting must immediately give written notification to the Regional (or assigned Program Support Center) Structural Engineering Unit Manager, who will update the inventory database to reflect the change. Copies of all documentation related to posting decisions, including calculations, inspection reports, load test reports, etc., will be kept in the state BIN folder or other permanent bridge file location.

The bridge owner is responsible for the decision to post a bridge and setting posting values. However, the following minimum standards must always be followed, according to Section 233 and 234 of the Highway Law, and the UCBI. Load posting signs shall conform to the standards for regulatory signs under the current NYSDOT (17NYCRR), Chapter V, a.k.a Manual of Uniform Traffic Control Devices (MUTCD).

NBIS regulations (23 CFR Part 650) require the rating of each bridge as to its safe loading capacity in accordance with the AASHTO MBE and the posting of the bridge in accordance with this document or in accordance with state law, when the maximum unrestricted legal loads or state routine permit loads exceed that allowed under the Operating rating. If a bridge is not capable of carrying statutory loads, it is posted for a lesser load limit.

Strength limit state is used for checking the ultimate capacity of structural members and is the primary limit state utilized by NYSDOT for determining posting needs. Service and fatigue limit states are utilized to limit stresses, deformations, and cracking under regular service conditions. In LRFR, Service and Fatigue limit state checks are optional in the sense that a posting or permit decision does not have to be dictated by the result. These serviceability checks provide valuable information for the engineer to use in the decision process.

A concrete bridge with unknown details need not be posted for restricted loading if it has been carrying normal unrestricted traffic and shows no distress. knowledge of the live load used in the original design, the current condition of the structure and live load history may be used to provide a basis for assigning a safe load capacity. Nondestructive proof load tests can be helpful in establishing the safe load capacity for such structures.

5.2 POSTING ANALYSIS

The goal of the LRFR methodology is to maintain target uniform reliabilities in all load ratings and load postings. Unlike past practice, it should be noted that in a reliability-based evaluation the relationship between posting values and rating factors is not proportional. For a posted bridge there is a greater probability of vehicles exceeding the posted limit compared to numbers exceeding the legal limit on an un-posted bridge.

A reliability calibration procedure has been performed to calibrate posting load levels for bridges with rating factor R.F. <1.0. The calibration is performed to ensure that posted bridges will still meet the target reliability level β_{target}=2.0 set during the calibration of the live load factors for rating. Because posting is normally used for bridges with low ADTT levels, the calculations were performed based on sites with ADTT=100. The calibration process involved conservative assumptions on the loading of posted bridges due to unavailability of WIM data at posted bridge sites.

According to the calibration results, two-lane bridges with low truck volumes should be posted if the rating analysis performed for the SU-4 single unit truck, the 3-S2 semi-trailer truck, or the Legal Lane Load lead to Rating Factors R.F.<1.0. The rating also envelopes the effects on multi-lane bridges loaded by a single line of trucks. Single-lane bridges should be posted based on a rating analysis using the single lane live load factors and maximum effects from the NYSDOT Legal Loads. The rating equation should also include the System Factor ϕ_s and the condition factor ϕ_c.

When for any Legal Load the RF is between 0.3 and 1.0, then the following equation should be used to establish the LRFR posting load for each posting vehicle type:

LRFR Safe Posting Load Equation:

$$\text{Safe Posting Load} = W\left[RF + 0.00375(L - 110)(1 - RF)\right]$$

Where W = Weight of Posting Vehicle
RF= Legal Load Rating Factor
L = Effective span length in feet as defined below

When the lane load model governs the load rating, W shall be taken as 80 Kips (40 tons). Table 9 shows the safe posting load using the equation for various rating factors and span lengths.

Table 9 Safe Posting Load

a) Posting weights in Tons for single unit trucks (W = 27 Tons)							
SPAN	R.F.=0.3	R.F.=0.4	R.F.=0.5	R.F.=0.6	R.F.=0.7	R.F.=0.8	R.F.=0.9
40 ft	3	7	10	14	17	20	24
100 ft	8	10	13	16	19	22	24
200 ft	15	17	18	20	22	24	25
b) Posting weights in Tons for semi-trailer trucks (W=36 Tons)							
SPAN	R.F.=0.3	R.F.=0.4	R.F.=0.5	R.F.=0.6	R.F.=0.7	R.F.=0.8	R.F.=0.9
40 ft	4	9	14	18	23	27	32
100 ft	10	14	18	21	25	29	33
200 ft	20	22	24	27	29	31	34

In a departure from current NYSDOT practice, two posting vehicles are specified, as using only one single unit truck for posting would be unnecessarily restrictive on the longer semi-trailer trucks. If only a single tonnage is to be used for posting, then the lowest safe load value shall be used. The higher posting loads for the longer spans are reflective of the higher reliability indices inherent in longer spans, when compared to the shorter spans. Posting is more restrictive on the shorter spans also due to their lower reliability indices. When the RF for any vehicle type falls below 0.3, then a recommendation should be made to not allow that particular vehicle type on the bridge. Other vehicle types with RF > 0.3 may continue to use the bridge.

Bridges that are determined not capable of carrying 3 tons shall be closed.

Definition of Effective Span Length

Member Type	Effective Span
1. Simple span stri8ngers or girders	Span length
2. Continuous stringers or girders	
a) Positive moment or shear	Span length
b) Negative moment	Average of adjacent span lengths
3. Floorbeams	
a) End floorbeam	Adjacent stringer or panel length
b) Intermediate floorbeam	Sum of two adjacent stringer spans or panel lengths
4. Trusses	
a) Chords and end posts	Total span length
b) Interior diagonals	Panel length + sum of panel lengths to far support
c) Vertical hangers or posts	Same as intermediate floorbeam
d) Vertical part of truss web	Same as interior diagonal

5.3 EXAMPLES

A single span rolled beam bridge with four stringers has a span length of 65 ft. Carries two lanes of traffic with an ADTT = 5000. There is significant deterioration and the primary member rating is 3. Dynamic load allowance = 20% (used only for legal load and permit load ratings). This bridge will be evaluated for three cases: 1) the as-built condition, 2) the as-inspected condition and 3) Condition that would require R-posting, to illustrate the use of the LRFR procedures.

M_{DC1} = 480.0 K-ft

M_{DC2} = 0.00 K-ft

φR_n = 2125.0 K-ft (As-built)

φR_n = 1738.9 K-ft (As-inspected)
Distributed two-lane live load moments:

M_{LL+I} (HL-93) = 952.6 K ft

M_{LL+I} (Type 3S2) = 531.2 K ft

M_{LL+I} (Type SU4) = 559.4 K ft

M_{LL+I} (Permit Type 6A) = 1028.8 K ft

M_{LL+I} (Permit Type 7) = 889.1 K ft

Case 1 -- As-Built Ratings

Condition Factor = 1.00
Load modifiers: importance factor η_I=1.0, ductility factor η_D=1.0, and redundancy factor η_R=1.0

HL-93 Ratings: Inv. Rating = 0.91 (LF =1.75)
Opr. Rating = 1.19 (LF =1.35)

As the HL-93 Operating Rating > 1.0 the bridge would have had adequate load capacity for NY legal loads (Article 2.8.4) and for NY divisible load permits (Article 2.8.4), as shown below:
System Factor = 1.00 (redundant system)

Type 3S2 Rating: RF = 1.47 (LF = 1.95)
Type SU4 Rating: RF = 1.40 (LF = 1.95)
Permit Type 6A Rating: RF = 1.24 (LF = 1.20)
Permit Type 7 Rating: RF = 1.43 (LF = 1.20)

In the as-built condition, the bridge would not have had to be load posted or R-posted (RF > 1.0). All load ratings are based on multi-lane distribution factors.

Case 2 --As-Inspected Ratings

Condition Factor = 0.95
Load modifiers: importance factor η_I=1.0, ductility factor η_D=1.0, and redundancy factor η_R=1.0

HL-93 Ratings: Inv. Rating = 0.63 (LF =1.75)
Opr. Rating = 0.82 (LF =1.35)

As the HL-93 Operating Rating < 1.0 the bridge may not have adequate load capacity for NY legal loads (Article 2.8.4) and for NY divisible load permits (Article 2.8.4). Perform legal load ratings:

System Factor = 1.00 (redundant system)

Type 3S2 Rating: RF = 1.02 (LF = 1.95)
Type SU4 Rating: RF = 0.96 (LF = 1.95)

As the legal load ratings are less than 1.0, the bridge will need to be load posted and permit loads should not be allowed on the bridge. All load ratings are based on multi-lane distribution factors.

Posting Analysis (Article 5.2)

$$\text{Safe Posting Load} = W[RF + 0.00375(L - 110)(1 - RF)]$$

Where W = Weight of Posting Vehicle
RF= Legal Load Rating Factor
L = Effective span length in
Governing Rating Factor R.F.=0.96
Posting load for Type 3S2:
L = 65 ft., W = 36 Tons, RF = 0.96
Safe Posting Load = 34.5 Tons
Posting load for Type SU4:
L = 65 ft., W = 27 Tons, RF = 0.96
Safe Posting Load = 25.7 Tons

Case 3 – R-Posting Analysis (Article 6.2)

To illustrate the R-Posting analysis, assume that the as-inspected resistance is R_n = 1785.0 K-ft
Condition Factor = 0.95
Load modifiers: importance factor η_I=1.0, ductility factor η_D=1.0, and redundancy factor η_R=1.0

HL-93 Ratings: Inv. Rating = 0.66 (LF =1.75)
Opr. Rating = 0.86 (LF =1.35)

As the HL-93 Operating Rating < 1.0 the bridge may not have adequate load capacity for NY legal loads (Article 2.8.4) and for NY divisible load permits (Article 2.8.4). Perform legal load ratings:

System Factor = 1.00 (redundant system)

Type 3S2 Rating:	RF = 1.06 (LF = 1.95)
Type SU4 Rating:	RF = 1.00 (LF = 1.95)

As the NY legal load ratings are ≥ 1.0, the bridge need not be load posted.
The bridge should then be evaluated for divisible loads to check if it has adequate reserve capacity for permits and if an R-Posting may be required.
For R-Posting, check either the governing divisible load for downstate bridges (Type 6A) or Type 7 for upstate bridges --- depending on where the bridge is located. If the bridge does not rate out for these trucks, an R-posted would be required. For the subject bridge the permit ratings are:
System Factor = 1.00 (redundant system)

Permit Type 6A Rating:	RF = 0.89	(LF = 1.20)
Permit Type 7 Rating:	RF = 1.03	(LF = 1.20)

The results indicate that only if the bridge was located downstate an R-Posting would be required, based on the rating factor (< 1.0) for Type 6A downstate permit.

6) CRITERIA FOR POSTING BRIDGES FOR R - PERMIT RESTRICTIONS

6.1 INTRODUCTION

The posting methodology also includes posting for divisible load restrictions, referred to as “R”-posting, as per NYS regulations for divisible permit loads. R-Postings are intended to keep most overloads from using bridges that, through design or deterioration, do not have the reserve capacity to accommodate most overload permit vehicles, but are still capable of safely carrying legal loads. These bridges have signage stating “No Trucks with R Permits.”

6.2 R-POSTING EVALUATION

Downstate bridges that do not have a Rating Factor ≥ 1.0 for the NYP 11 (Type 6A) Divisible Load permit and upstate bridges that do not have a Rating Factor ≥ 1.0 for the NYP 6 (Type 7) Divisible Load permit following the LRFR procedures shall be R - posted. NYP 11 and NYP 6 were shown to be the governing divisible permit load models with the highest moment and shear effects on a series of simple and continuous spans (See Appendix A). A simplified but more conservative approach would be to R-post bridges that have a RF ≤ 1.0 for HL-93 at the Operating Level. As seen in the graphs in Appendix A for simple and continuous spans, the factored load effects for HL-93 at the Operating level will envelope the factored load effects for the Divisible Load Permits with a 1.2 load factor. This is conservative for the longer spans, but could provide a convenient screening level for R-posting needs, as the HL-93 Operating ratings are routinely computed during LRFR evaluations and reported to the NBI.
The loading configurations and live load factors for NYP 11 and NYP 6 are as given below in Fig 6 and Table 10. Reduced Dynamic Load Allowance may be used as provided in Section 2.8.5.

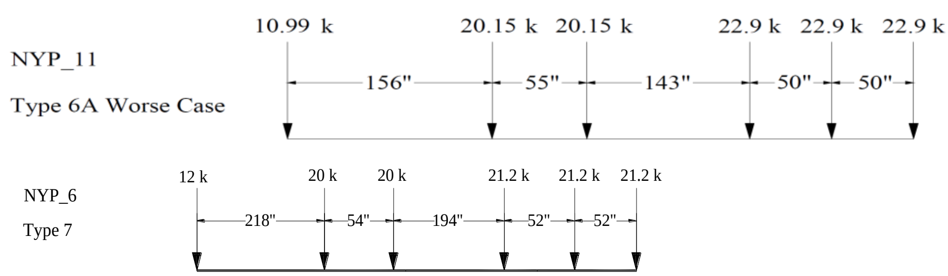


Fig 6 NYP 11 and NYP 6 -- Governing Divisible Load Permits for R-posting Evaluation

Table 10 NYP 11 Permit Load Factors, γ_L

Permit Type	Frequency	Loading Condition	DF	ADTT (one direction)	Permit Load Factor γ_L
Annual Divisible Load	Unlimited trips	Multi-lane bridges Mix with traffic	Multi-lane	ADTT 5000	1.20
				ADTT=1000	1.15
				ADTT 100	1.10
Annual Divisible load	Unlimited trips	Single lane bridges	Single Lane DF after dividing out MP=1.2	ADTT 5000	1.20
				ADTT=1000	1.15
				ADTT 100	1.10

Linear interpolation is permitted for other ADTT

7) FIELD LOAD TESTING

7.1 INTRODUCTION

The actual performance of most bridges is more favorable than conventional theory dictates. Safe load capacity for a structure can be determined from full scale non-destructive field load tests, which may be desirable to establish a higher safe load carrying capacity than calculated by analysis. Refer to the MBE Section 8 for information on conducting field load tests and using the results to establish a new or updated load rating.
There are many bridges for which common analytical methods are not adequate to determine a load rating. The following are some examples:

- Bridges that cannot be realistically modeled using routine analytical methods.
- Bridges with unavailable or incomplete plans and structural components that cannot be measured. Examples include (but are not limited to) steel beams encased in concrete and concrete structures with unknown reinforcement or prestressing.

For cases like these, alternate methods of load rating, such as a non-destructive load test, may need to be used to generate realistic load rating results.

Field load testing, also referred to as nondestructive load testing, is an experimental determination of a structure's load capacity by measuring the actual structural response to known loads. The measured response of the bridge under the field load test is then compared to the analytical predicted response. Load testing can be a useful part of a load rating calculation for a bridge that is difficult to load rate using conventional analytical methods. Load testing may also provide a more accurate and at times higher rating, which can be very helpful when the theoretical safe live load capacity is lower than desirable. Load testing is typically separated into two types; diagnostic and proof testing.

Diagnostic load testing involves measuring the load effects (such as moment, shear, axial force, stresses, and deflection caused by known loads, such as a specific vehicle or vehicles of known weight, axle loads, and spacings). The results of the load tests are then compared to those predicted using analytical calculations. The difference between the theoretical and measured load effects will then be reviewed and calibrated to the standard AASHTO rating vehicles. The results will then be used to establish the new load rating. Load testing typically involves measurements of load effects of several bridge members at critical locations.

Proof load testing involves loading the bridge with incremental loads until a targeted load level is safely reached. This level is then used to set the level of the new load rating. Loading should be done incrementally while the bridge is carefully monitored. The loading should be discontinued at any sign of distress or damage. Proof load testing requires careful preparation and experienced personnel. Care is required to avoid damage to the structure as well as to prevent injuries to personnel and to the public.

If done incorrectly, field load testing can lead to inaccurate load rating results. In addition, incorrect testing procedures can lead to permanent damage and even possible collapse of the bridge structure. Sound engineering judgment and analytical principles need to be taken into consideration before load testing is performed. See the AASHTO MBE Section 8 and references.

7.2 DOCUMENTATION OF RESULTS

Every test report must include certain information, regardless of test procedure. At a minimum, provide the following:

- Truck weights, axle spacing, and axle loadings.
- Exact location of truck(s) on the bridge for all strain or deflection measurements.
- Types of measuring instruments used (strain gauges, survey rods, etc.)
- Location of measuring instruments.
- Conversion calculations to legal load ratings.
- Reasons for increased capacity above the analytical predicted load rating.

The report shall be signed by the responsible professional engineer licensed by the State of New York, and filed with NYSDOT using the same procedures as for an in-depth Level 1 load rating. All load test documentation and results should be kept in the Region (or responsible Program Support Center) office. If used to generate a Level 1 load rating, the actual results of the load test are only a portion of the Level 1 documentation. In addition to the load testing documentation, the procedures in the preceding Level 1 guidelines shall be followed.

8) PERMIT LOAD RATING OF FLOORBEAMS

Load rating of floorbeams for permit loads shall be carried out by placing live loads in positions and combinations that maximize floorbeam load effects. The live load factors of Table 6 are applicable.

For a single crossing of a permit vehicle such as a Special Hauling Permit or a Superload Permit, the permit vehicle is placed in any one lane that produces the maximum load effect. When the one-lane loaded condition is evaluated using the permit load, it is not necessary to include the 1.2 multiple presence factor in the analysis. For the unescorted crossing on multi-lane bridges, live loads are placed in more than one lane when checking Special Hauling Permits or Superload Permits. The lanes other than the permit load lane shall be loaded with the SU4 truck with applicable reductions for multiple presence.

Divisible Load Permits on multi-lane bridges should be checked for the maximum loading condition with two permits side-by-side.

REFERENCES

1. *Load and Resistance Factor Rating Methodology in New York State*, C-06-13: Final Report The City College of New York, 2011.

2. ASHTO *Manual for Bridge Evaluation* First Edition, 2008, including all subsequent interim revisions.

3. AASHTO LRFD *Bridge Design Specifications*, 4th Edition (2007) including all subsequent interim revisions.

4. NCHRP Report 575; *Legal Truck Loads and AASHTO Legal Loads for Posting.*

5. NCHRP Report 454; *Calibration of Load Factors for LRFR Evaluation*

6. NYSDOT Research Report 163 "Highway Bridge Rating by Nondestructive Proof-Load Testing for Consistent Safety." NYSDOT Transportation Research and Development Bureau.

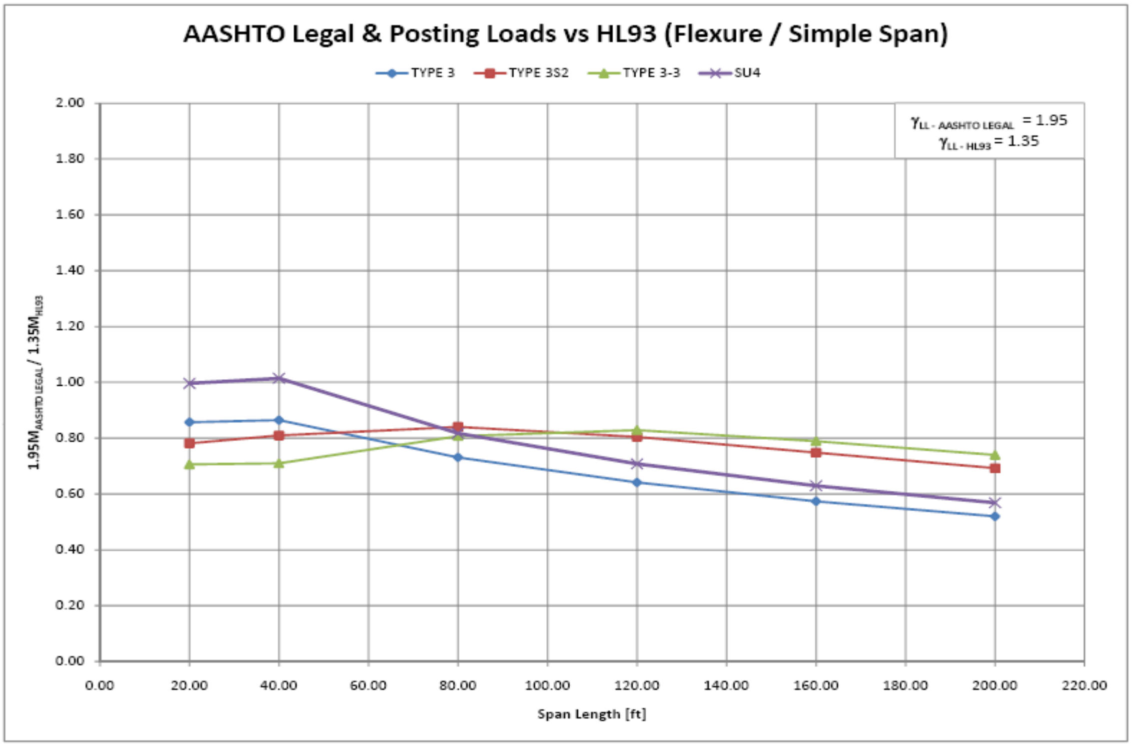
7. NYSDOT Research Report 153 "Proof Testing of Highway Bridges" NYSDOT Transportation Research and Development Bureau.

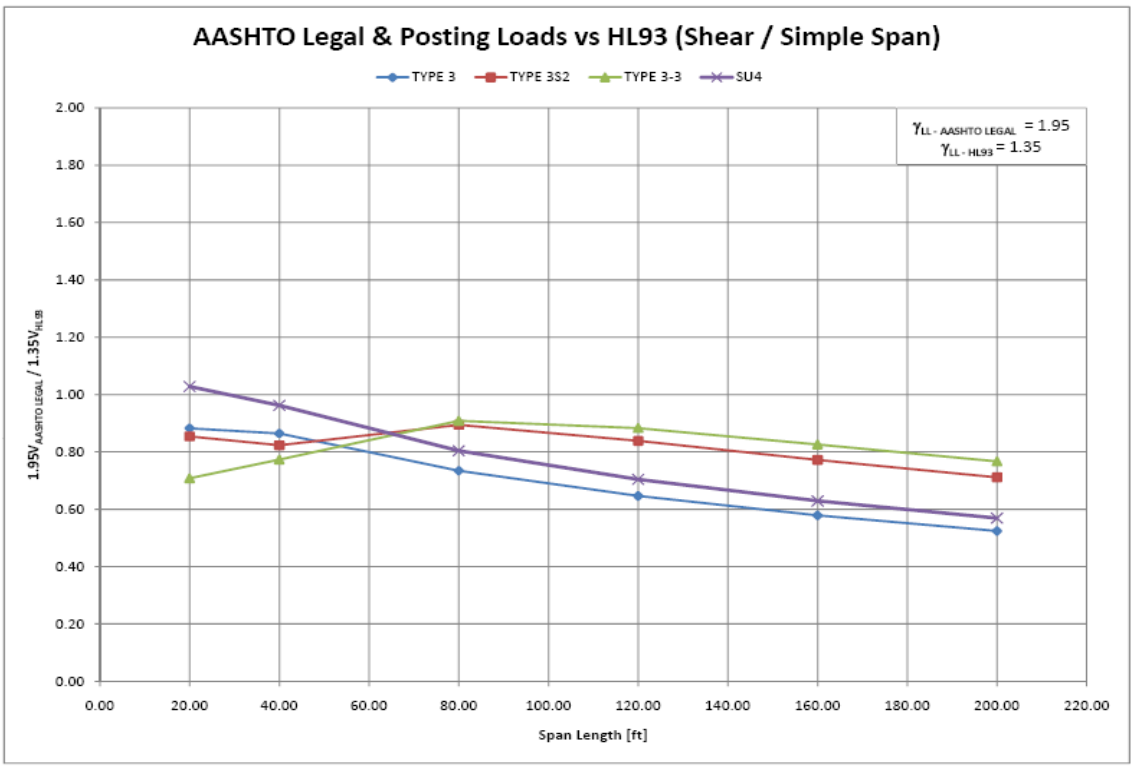
APPENDIX A

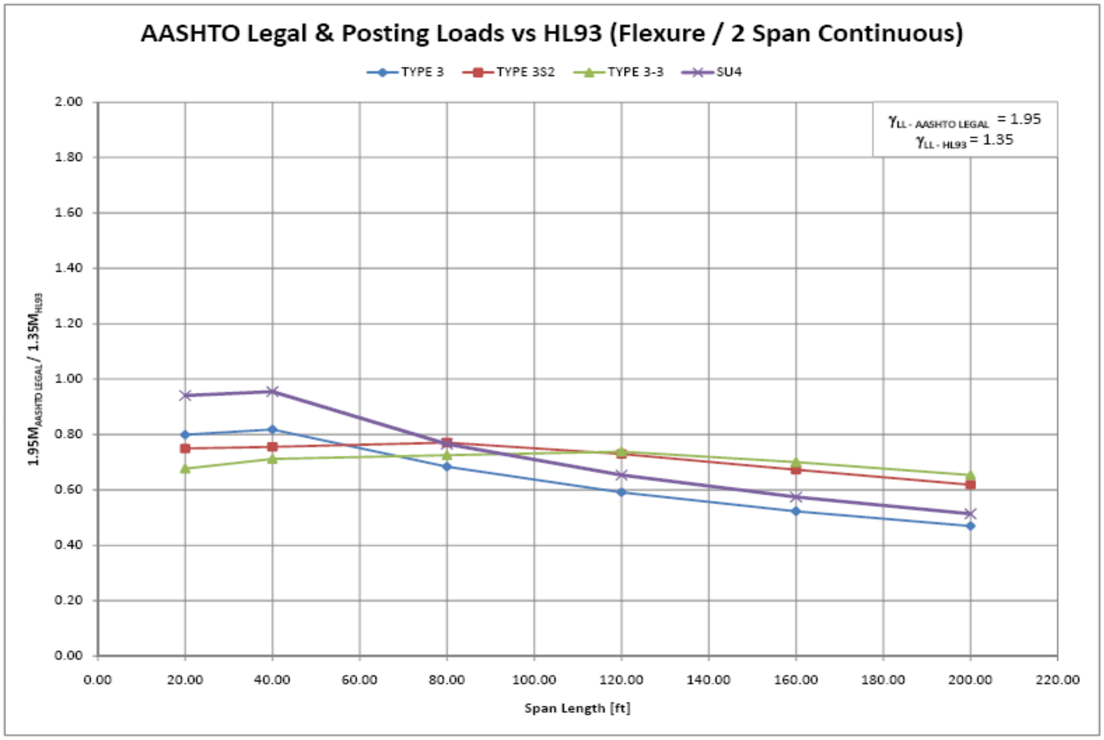
Moment & Shear Effects

For

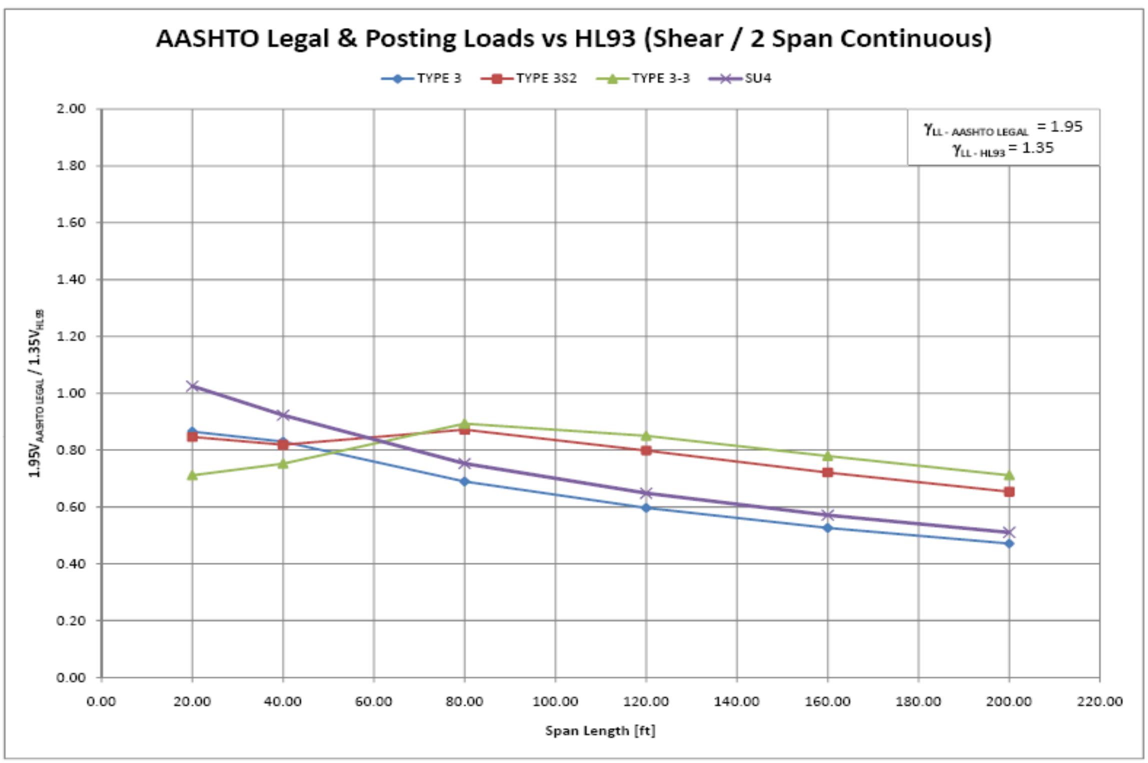
NYSDOT Legal Loads & Permit Loads


AASHTO Legal & Posting Loads vs HL93 (Flexure / Simple Span)
TYPE 3
TYPE 3S2
TYPE 3-3
SU4
$\gamma_{LL\text{ - AASHTO LEGAL}}$ = 1.95
$\gamma_{LL\text{ - HL93}}$ = 1.35
$1.95M_{AASHTO\ LEGAL} / 1.35M_{HL93}$
Span Length [ft]

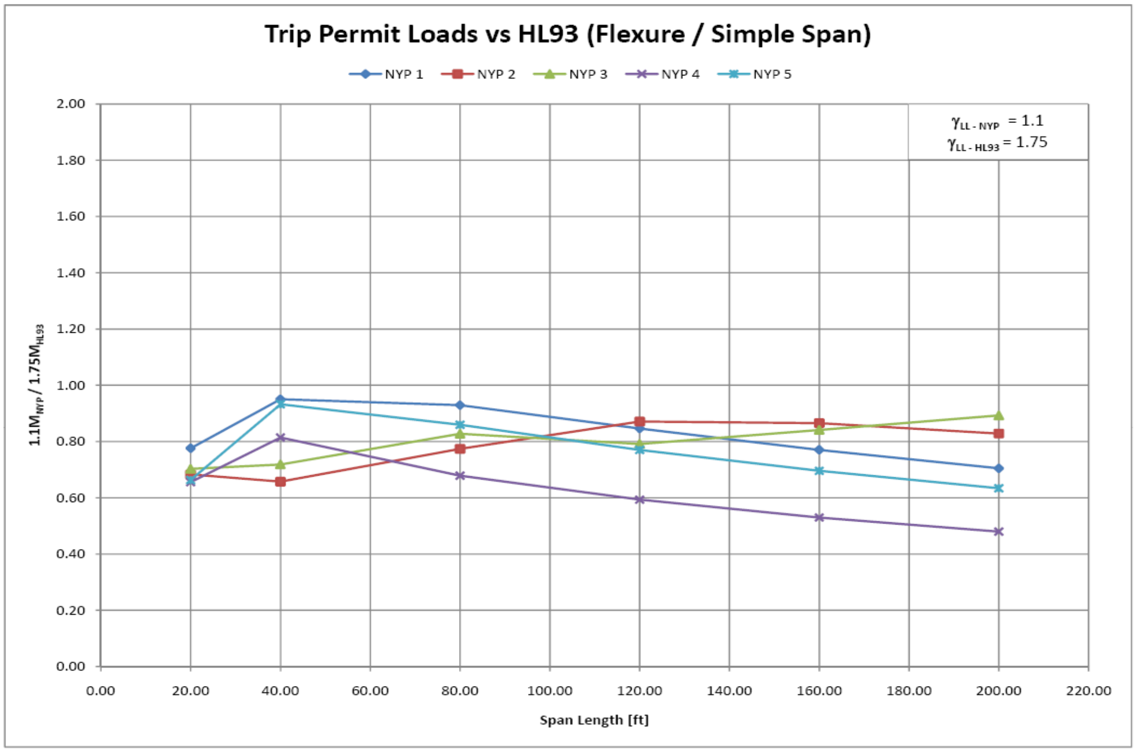

AASHTO Legal & Posting Loads vs HL93 (Shear / Simple Span)
TYPE 3
TYPE 3S2
TYPE 3-3
SU4
$\gamma_{LL\text{ - AASHTO LEGAL}}$ = 1.95
$\gamma_{LL\text{ - HL93}}$ = 1.35
$1.95V_{AASHTO\ LEGAL} / 1.35V_{HL93}$
Span Length [ft]

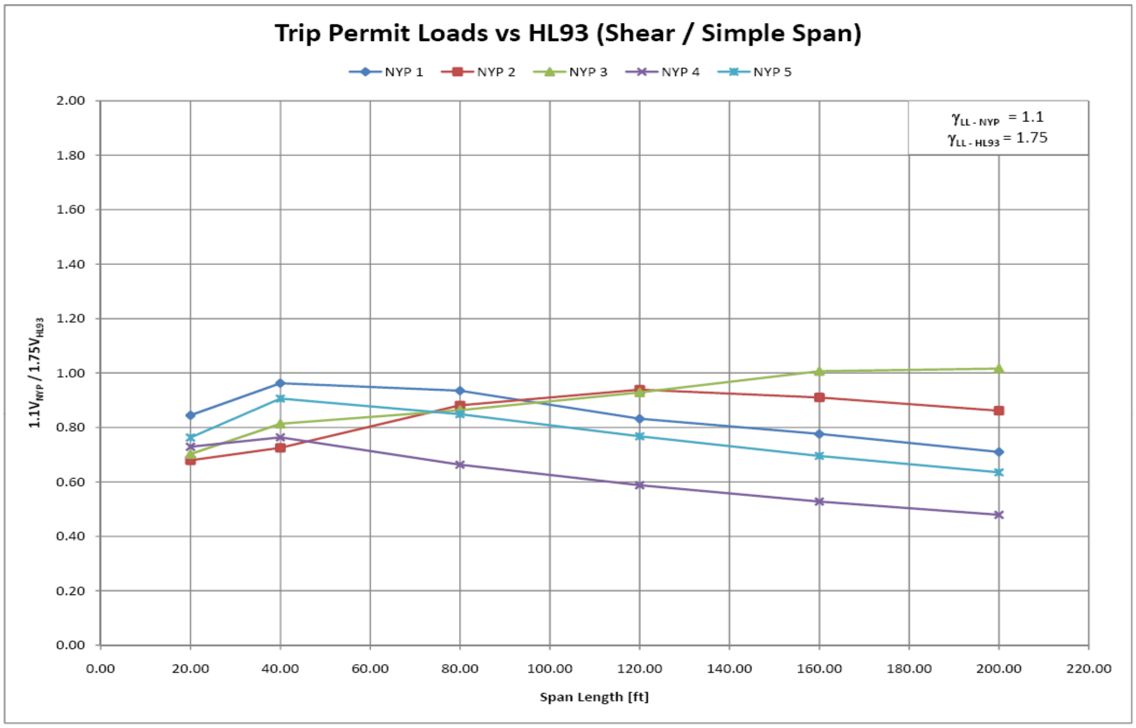


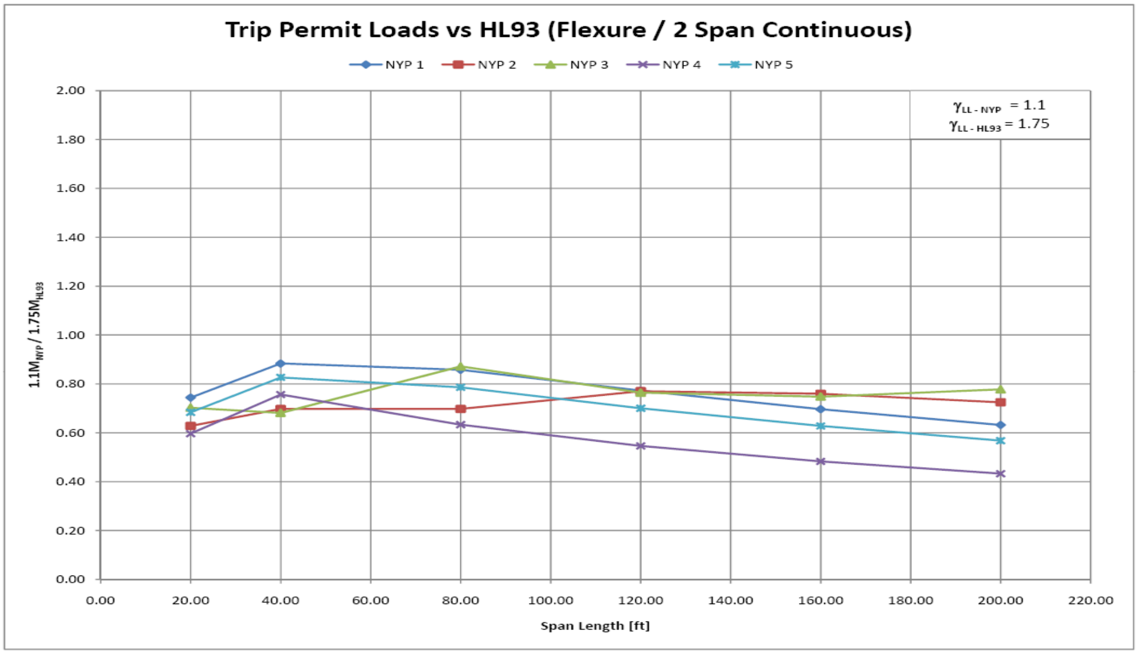
AASHTO Legal & Posting Loads vs HL93 (Flexure / 2 Span Continuous)
TYPE 3
TYPE 3S2
TYPE 3-3
SU4
γLL - AASHTO LEGAL = 1.95
γLL - HL93 = 1.35
1.95M AASHTO LEGAL / 1.35M HL93
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
220.00
Span Length [ft]



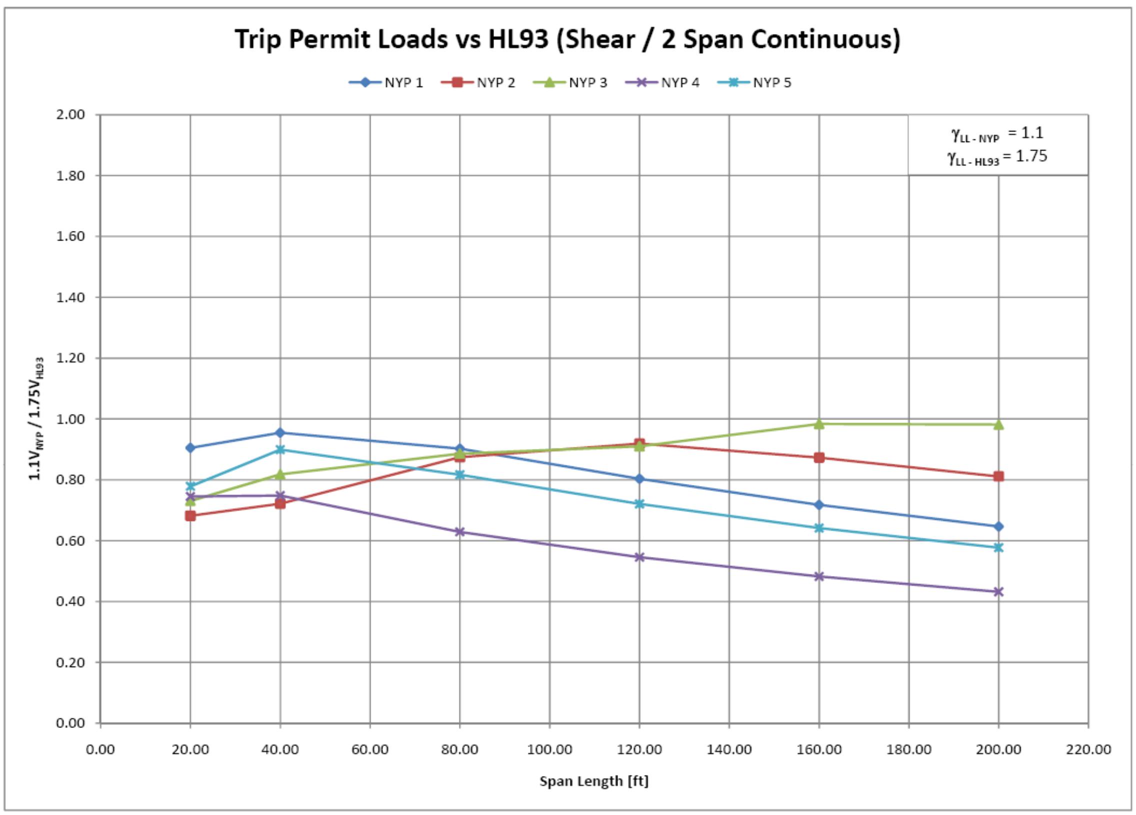
AASHTO Legal & Posting Loads vs HL93 (Shear / 2 Span Continuous)
TYPE 3
TYPE 3S2
TYPE 3-3
SU4
γLL - AASHTO LEGAL = 1.95
γLL - HL93 = 1.35
1.95V AASHTO LEGAL / 1.35V HL93
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
220.00
Span Length [ft]


Trip Permit Loads vs HL93 (Flexure / Simple Span)
NYP 1
NYP 2
NYP 3
NYP 4
NYP 5
γ_{LL-NYP} = 1.1
$\gamma_{LL-HL93}$ = 1.75
$1.1M_{NYP} / 1.75M_{HL93}$
Span Length [ft]
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
220.00

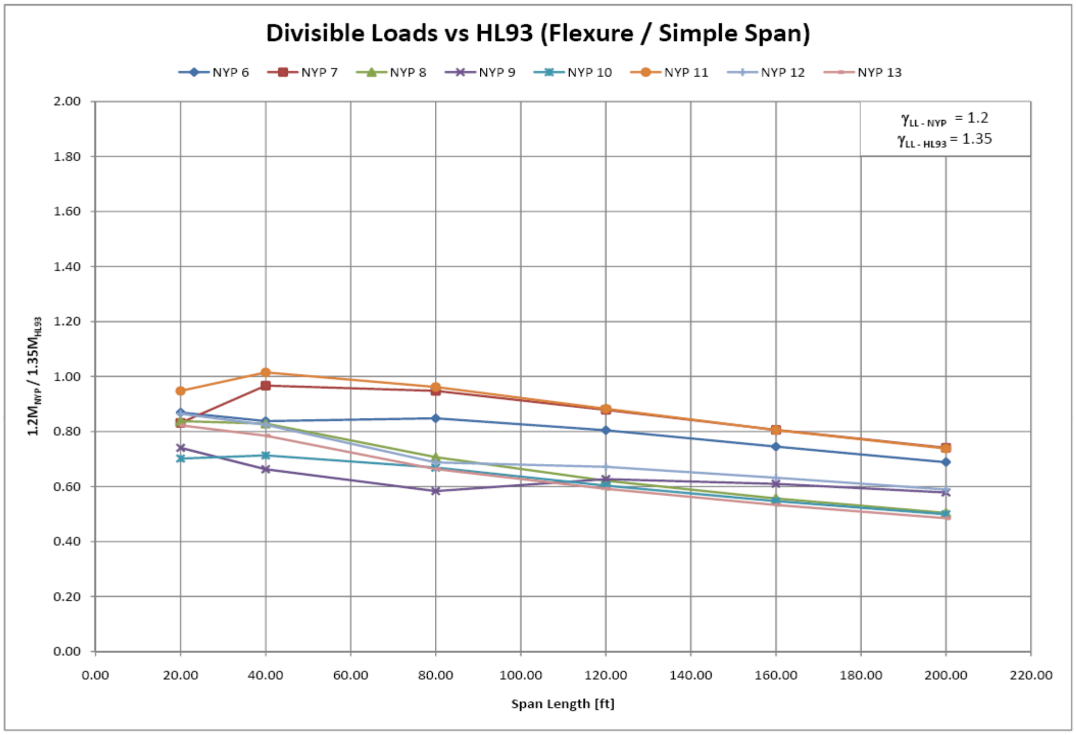

Trip Permit Loads vs HL93 (Shear / Simple Span)
NYP 1
NYP 2
NYP 3
NYP 4
NYP 5
γ_{LL-NYP} = 1.1
$\gamma_{LL-HL93}$ = 1.75
$1.1V_{NYP} / 1.75V_{HL93}$
Span Length [ft]
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
220.00



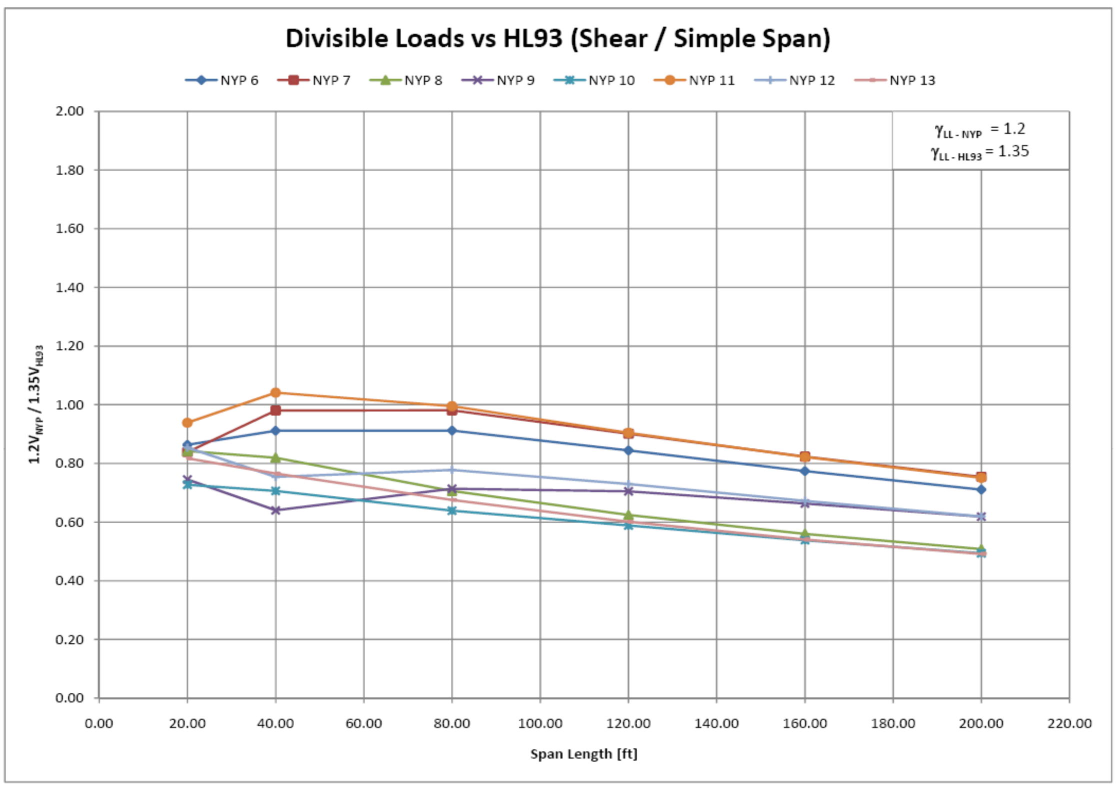
Trip Permit Loads vs HL93 (Flexure / 2 Span Continuous)
NYP 1
NYP 2
NYP 3
NYP 4
NYP 5
γ_{LL-NYP} = 1.1
$\gamma_{LL-HL93}$ = 1.75
1.1M_NYP / 1.75M_HL93
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
220.00
Span Length [ft]



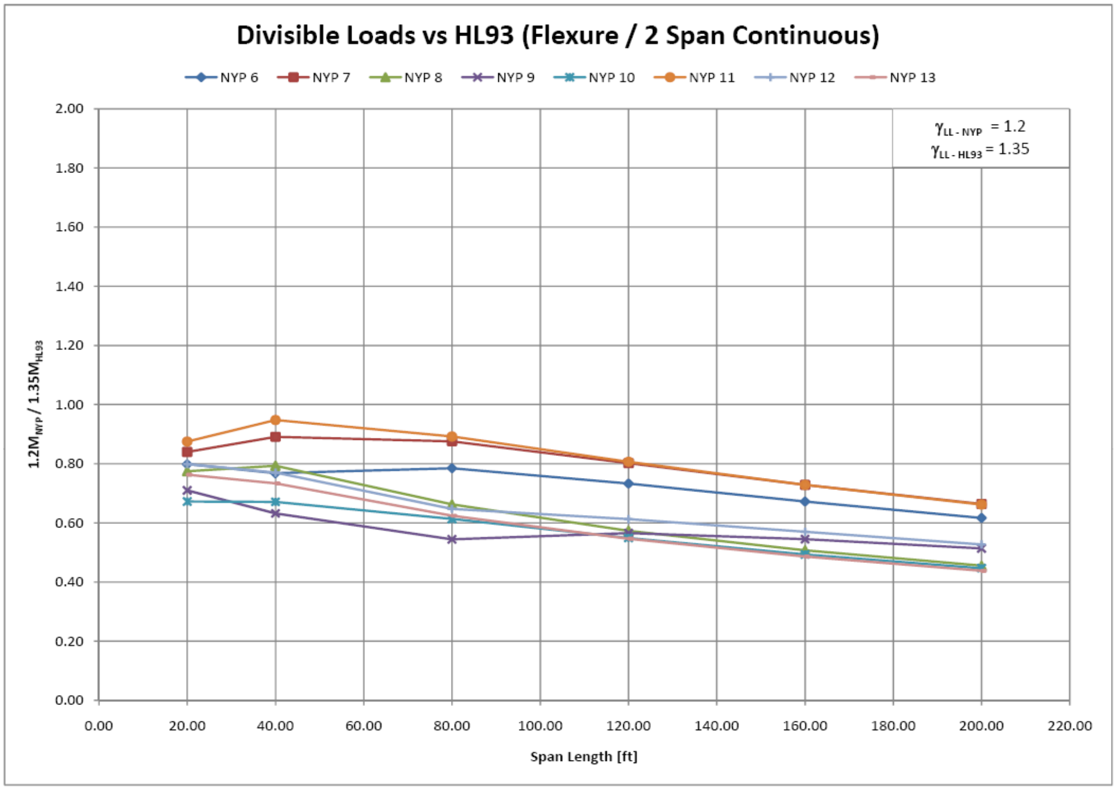
Trip Permit Loads vs HL93 (Shear / 2 Span Continuous)
NYP 1
NYP 2
NYP 3
NYP 4
NYP 5
γ_{LL-NYP} = 1.1
$\gamma_{LL-HL93}$ = 1.75
1.1V_NYP / 1.75V_HL93
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
220.00
Span Length [ft]



Divisible Loads vs HL93 (Flexure / Simple Span)
NYP 6
NYP 7
NYP 8
NYP 9
NYP 10
NYP 11
NYP 12
NYP 13
γLL - NYP = 1.2
γLL - HL93 = 1.35
1.2MNYP / 1.35MHL93
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
220.00
Span Length [ft]



Divisible Loads vs HL93 (Shear / Simple Span)
NYP 6
NYP 7
NYP 8
NYP 9
NYP 10
NYP 11
NYP 12
NYP 13
γLL - NYP = 1.2
γLL - HL93 = 1.35
1.2VNYP / 1.35VHL93
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
220.00
Span Length [ft]


Divisible Loads vs HL93 (Flexure / 2 Span Continuous)
NYP 6
NYP 7
NYP 8
NYP 9
NYP 10
NYP 11
NYP 12
NYP 13
γ_{LL-NYP} = 1.2
$\gamma_{LL-HL93}$ = 1.35
$1.2M_{NYP} / 1.35M_{HL93}$
Span Length [ft]

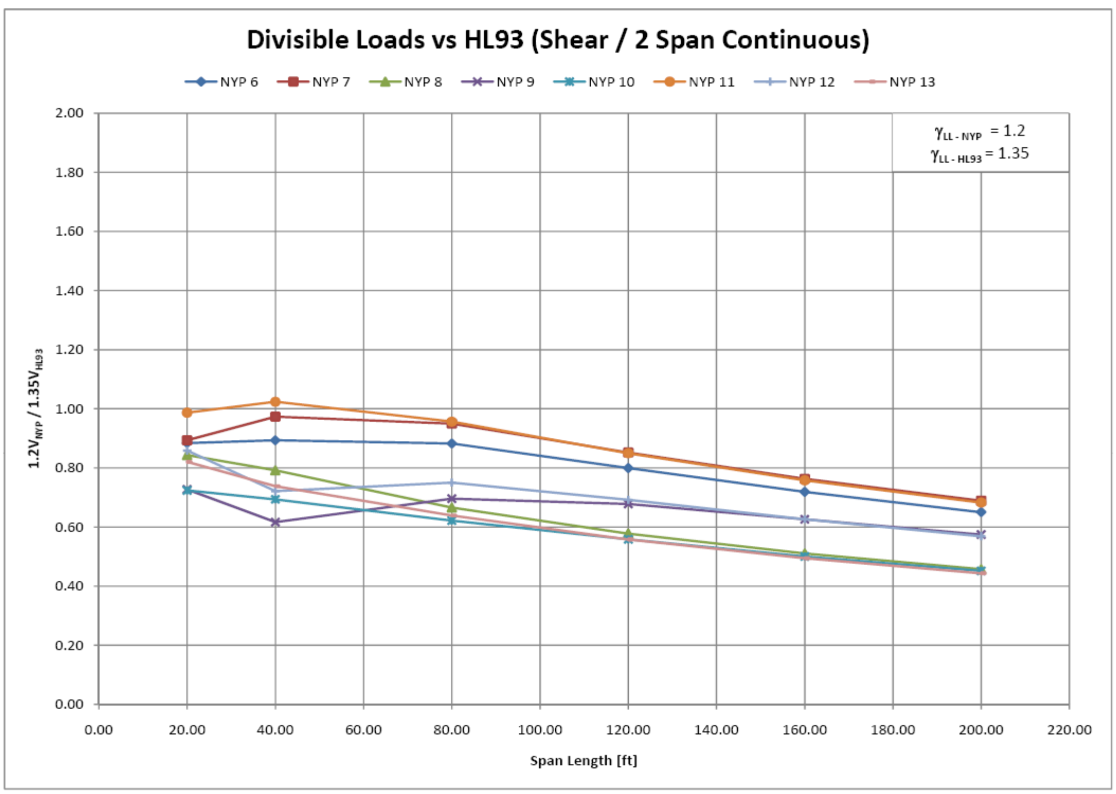

Divisible Loads vs HL93 (Shear / 2 Span Continuous)
NYP 6
NYP 7
NYP 8
NYP 9
NYP 10
NYP 11
NYP 12
NYP 13
γ_{LL-NYP} = 1.2
$\gamma_{LL-HL93}$ = 1.35
$1.2V_{NYP} / 1.35V_{HL93}$
Span Length [ft]

TABLE A.1 Moment and Shear Tables for Simple Span and 2-Span Continuous Cases (NY Legal Loads)

	LOADING	SPAN LENGTH	SIMPLE SPAN		2-SPAN CONTINUOUS	
			MOMENT	SHEAR	MOMENT	SHEAR
			kip-ft	kip	kip-ft	kip
NY LEGAL LOADS	TYPE 3S2	20 FT	126	30.4	102	32.1
		40 FT	324	38.8	260	42.4
		80 FT	973	55.3	773	59.4
		120 FT	1689	60.8	1354	64.5
		160 FT	2407	63.6	1942	66.7
		200 FT	3126	65.3	2533	67.9
	SU4	20 FT	160	36.6	128	38.9
		40 FT	406	45.3	328	47.8
		80 FT	946	49.7	769	51.3
		120 FT	1486	51.1	1213	52.3
		160 FT	2026	51.8	1658	52.8
		200 FT	2566	52.3	2103	53.0

*Moment and shear values given in the table do not include load factors, distribution factors or dynamic load allowance.

TABLE A.2 Moment and Shear Tables for Simple Span and 2-Span Continuous Cases (Non-Divisible Permits)

	LOADING	SPAN LENGTH	SIMPLE SPAN		2-SPAN CONTINUOUS	
			MOMENT	SHEAR	MOMENT	SHEAR
			kip-ft	kip	kip-ft	kip
NY NON-DIVISIBLE PERMITS	NYP_1	20 FT	286	69.1	232	78.9
		40 FT	874	104.2	699	113.6
		80 FT	2472	132.7	1979	141.4
		120 FT	4082	138.7	3294	149.0
		160 FT	5696	146.9	4617	152.4
		200 FT	7308	149.8	5942	154.4
	NYP_2	20 FT	252	55.5	196	59.4
		40 FT	604	78.5	551	85.8
		80 FT	2058	125.0	1609	136.9
		120 FT	4202	156.6	3282	170.5
		160 FT	6398	172.3	5037	185.5
		200 FT	8593	181.8	6820	193.7
	NYP_3	20 FT	259	57.4	219	63.7
		40 FT	661	88.0	539	97.2
		80 FT	2201	122.6	2010	138.8
		120 FT	3816	154.8	3257	168.8
		160 FT	6220	190.5	4958	209.0
		200 FT	9260	214.4	7315	234.3
	NYP_4	20 FT	242	59.6	186	65.0
		40 FT	748	82.6	598	89.0
		80 FT	1805	94.1	1460	98.5
		120 FT	2861	98.0	2329	101.2
		160 FT	3918	99.9	3200	102.4
		200 FT	4974	101.0	4071	103.1
	NYP_5	20 FT	245	62.4	213	67.9
		40 FT	857	98.0	653	106.9
		80 FT	2286	120.5	1813	127.9
		120 FT	3715	127.9	2986	133.7
		160 FT	5144	131.7	4163	136.3
		200 FT	6573	133.9	5341	137.8

*Moment and shear values given in the table do not include load factors, distribution factors or dynamic load allowance.

TABLE A.3a Moment and Shear Tables for Simple Span and 2-Span Continuous Cases (Divisible Permits)

	LOADING	SPAN LENGTH	SIMPLE SPAN		2-SPAN CONTINUOUS	
			MOMENT	SHEAR	MOMENT	SHEAR
			kip-ft	kip	kip-ft	kip
NY NON-DIVISIBLE PERMITS	NYP_6	20 FT	227	49.9	176	54.5
		40 FT	545	69.7	429	75.1
		80 FT	1595	91.5	1280	97.7
		120 FT	2744	99.6	2209	104.8
		160 FT	3896	103.6	3152	108.0
		200 FT	5050	106.0	4100	109.8
	NYP_7	20 FT	217	48.5	185	55.1
		40 FT	629	75.0	498	81.8
		80 FT	1782	98.4	1428	105.2
		120 FT	2997	106.3	2418	111.8
		160 FT	4214	110.2	3416	114.7
		200 FT	5432	112.5	4418	116.3
	NYP_8	20 FT	219	48.7	171	52.0
		40 FT	539	62.7	443	66.6
		80 FT	1329	70.8	1080	73.8
		120 FT	2119	73.6	1728	75.7
		160 FT	2909	74.9	2378	76.7
		200 FT	3699	75.7	3029	77.2
	NYP_9	20 FT	193	43.1	157	44.8
		40 FT	431	49.0	353	51.8
		80 FT	1097	71.6	888	77.1
		120 FT	2138	83.1	1705	88.9
		160 FT	3189	88.8	2555	94.1
		200 FT	4243	92.2	3415	97.0
	NYP_10	20 FT	183	42.1	148	44.6
		40 FT	464	54.0	375	58.3
		80 FT	1258	64.1	1001	68.8
		120 FT	2058	69.4	1655	73.2
		160 FT	2858	72.0	2313	75.2
		200 FT	3658	73.6	2972	76.3

*Moment and shear values given in the table do not include load factors, distribution factors or dynamic load allowance.

TABLE A.3b Moment and Shear Tables for Simple Span and 2-Span Continuous Cases (Divisible Permits)

	LOADING	SPAN LENGTH	SIMPLE SPAN		2-SPAN CONTINUOUS	
			MOMENT	SHEAR	MOMENT	SHEAR
			kip-ft	kip	kip-ft	kip
NY NON-DIVISIBLE PERMITS	NYP_11	20 FT	247	54.3	193	60.8
		40 FT	660	79.6	530	86.1
		80 FT	1809	99.9	1455	106.0
		120 FT	3010	106.6	2432	111.4
		160 FT	4211	110.0	3417	113.9
		200 FT	5412	112.0	4404	115.3
	NYP_12	20 FT	226	49.4	176	53.0
		40 FT	536	57.7	431	60.7
		80 FT	1294	78.1	1056	83.1
		120 FT	2290	86.0	1847	90.7
		160 FT	3303	90.0	2672	94.1
		200 FT	4319	92.4	3505	96.0
	NYP_13	20 FT	215	47.3	168	50.6
		40 FT	510	58.5	410	62.1
		80 FT	1248	67.8	1018	70.8
		120 FT	2017	70.8	1647	73.2
		160 FT	2787	72.4	2279	74.3
		200 FT	3557	73.3	2913	74.9

*Moment and shear values given in the table do not include load factors, distribution factors or dynamic load allowance.

APPENDIX II

REVIEW OF CURRENT NY STATE GUIDELINES

FOR

LOAD RATING, LOAD PERMITS AND POSTING

OF

EXISTING BRIDGES

Review of NY State Guidelines for Load Rating, Load Permits and Posting of Existing Bridges

1. Load Rating

Load rating is the determination of the live load carrying capacity of a newly designed or existing bridge. Load ratings are required for all new and replacement bridges, and for all rehabilitation and repair designs involving substantial structural alterations. These ratings are typically determined by analytical methods based on information taken from bridge plans supplemented by information gathered from field inspections or field-testing. For each bridge, the results are summarized in terms of structure-rating units. A structure-rating unit is defined as a single simple span or a continuous series of spans that are analyzed as a single structure. Thus, a bridge with three simple spans will have three rating units, but a bridge with four continuous spans will have only one rating unit. Rating results are used to determine needs for member strengthening, load posting, or if a structure should be closed. Load rating results are also used to assist in the overload permit review process and for bridge management applications.

Currently, there are three different load rating methods: Allowable Stress Rating (ASR), Load factor Rating (LFR), and Load and Resistance Factor Rating (LRFR). The New York State Department of Transportation (NYSDOT), like most other states, has traditionally used the ASR or the LFR. The American Association of Highway and Transportation Officials (AASHTO) has recently adopted the Load and Resistance Factor Rating (LRFR) method that was developed to be compatible with the AASHTO Load and Resistance Factor Design (LRFD) specifications. Currently, the Federal Highway Administration (FHWA) allows and encourages the use of LRFR for load rating new and existing bridges. But, more importantly, a Federal Highway Administration (FHWA) memorandum dated October 30, 2006 has required that the ratings of all bridges and replacement bridges designed by the LRFD Specifications after October 1, 2010, are to be computed and reported to the National Bridge Inventory (NBI) system using Rating Factors based on the LRFR method (FHWA Memorandum; 2006).

The LRFR developed by Lichtenstein Engineering Associates (Lichtenstein, 2001) was calibrated following a reliability-based procedure compatible with that adopted during the development of the AASHTO Load and Resistance Factor Design (LRFD) specifications (AASHTO; 2007, Moses; 2001). Although the ASR and LFR, which have been widely used for several decades, have proven their merit by successfully providing the traveling public with a safe bridge network, the LRFR offers more flexibility than either the ASR or the LFR by taking into consideration state-specific loading conditions.

According to New York State Guidelines as provided in the EI 05-034 document (2005), two ratings of bridges for AASHTO HS design vehicles are usually prepared: 1) An inventory rating, which corresponds to the long term capacity using the load safety factors normally used with the design equations; and 2) An operating rating, which describes the maximum permissible live load to which the structure may be subjected in the short term based on lower safety factors. Therefore, the operating rating always yields a higher load rating than an inventory rating. NYSDOT allows three different levels of rating: Level I ratings are in conformance with the provisions of the AASHTO and Uniform Code of Bridge Inspection (UCBI) Manuals; Level II ratings are computer-generated analyses of bridges using NYSDOT load rating computer programs; Level III ratings are based on estimates of the probable capacity of a bridge relying on existing general inventory and inspection information. The LRFR procedures are intended for Level I or Level II ratings done in conformance with the latest AASHTO evaluation manual.

Load rating results can be presented using Rating Factors (RF) in terms of the AASHTO HS, H or HL design vehicles' equivalent loading. In metric the HL loading is designated as MS. Another approach consists of presenting rating results in terms of the truck tonnage of specific configurations. This section of the report describes the NYSDOT load rating procedure and compares it to the LRFR method.

1.1 Rating Factor

Generally speaking, a structural unit's load rating is expressed in terms of a Rating Factor, RF. A RF value greater than or equal to 1.0 indicates satisfactory rating. Each structural member in the system should be rated for various load conditions (axial, moment and shear), and the lowest rating factor from all members should be taken as the overall rating for the structure. The rating factor is obtained from the rating equation given in Eq. (1)

$$R.F. = \frac{\phi R_n - \gamma_D D_n}{\gamma_L L_n} \qquad (1)$$

Where:

ϕ = resistance factor;

R_n = nominal component resistance;

γ_D = dead load factor;

D_n = nominal dead load effect;

γ_L = live load factor which depends on whether the structure is rated for inventory level or operating level.

L_n = nominal live loading effect.

The nominal live load effect, L_n, represents the effect on a structural member prescribed by a load model such as the AASHTO H20 or HS20 for ASR and LFR, the AASHTO HL-93 for LRFR, or some other legal vehicle model. These load models are described in Section 1.2. The load factors and resistance factors to be applied in the generic rating factor equation shown above will depend on the rating method used.

To find the fraction of the effect of a vehicle that will be carried by the most critical member of a multi-girder system, a load distribution factor represented by the factor, g, is provided by the different AASHTO manuals. In addition, a dynamic allowance factor or sometimes called an impact factor, I, is applied to account for the dynamic amplification due to the motion of the vehicle over the bridge. Thus, the nominal live load effect is obtained from:

$$L_n = \text{Effect of nominal vehicular load} \times \text{g} \times (1+\text{I}) \qquad (2)$$

A discussion on the load distribution and dynamic allowance factors is provided in Sections 1.3 and 1.4.

Equation (1) is valid for all three rating methods. The difference lies in the actual values used for the resistance and load factors and in the vehicular load models. For example, the ASR uses ϕ=0.55 for the Inventory rating and 0.75 for the Operating rating of steel beams in bending while all the dead and live load factors are set at 1.0. The LFR uses a dead load factor γ_D=1.3, a resistance factor ϕ=1.0 for the bending of steel beams, and a live load factor γ_L=2.17 for Inventory Rating and γ_L=1.30 for Operating Rating. For the LRFR, γ_D=1.25 is used for most dead loads and a live load factor γ_L=1.75 for Inventory Rating and γ_L=1.35 for Operating Rating.

The LRFR allows adjustments to the resistance factor by applying a condition factor, $\phi_{c,}$ that relates to the component condition as provided in the inspection report and a system factor, ϕ_s that relates to the redundancy of the bridge system. Thus, instead of ϕR_n, the final factored resistance to be used in Eq. (1) is given as $C=\phi_s\phi_c\phi\, R_n$. Typical values for ϕ_c and ϕ_s are provided in Tables 1 and 2. These factors are currently left as optional in the AASHTO LRFR Manual with the recommendation that they should be used in accordance with the load rating practices of the State.

In the LRFR, the live load factors for the legal vehicles are reduced from γ_L=1.80 for bridges with low truck traffic volumes where the Average Daily Truck traffic (ADTT) is less than 5000. Typical values for the live load factor for legal loads are provided in Table 3. Additional adjustments are also allowed in LRFR for the cases when statistical information on the site specific or state specific truck weights and truck volume are available from methods that would provide unbiased information on truck traffic and weights such as special Weigh-In-Motion systems.

Table 1. LRFR Condition Factor ϕ_c

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

Table 2. LRFR 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 Eye bar Members	0.90
Three-Girder Bridges with Girder Spacing≤6’	0.85
Four-Girder Bridges with Girder Spacing≤4’	0.85
All Other Girder Bridges and Slab Bridges	1.00
Floorbeams with Spacing>12’ and Non-Continuous Stringers	0.85
Redundant Stringer Subsystems between Floorbeams	1.00

For evaluating timber bridges, a constant value ϕ_s=1.0 is assigned for flexure and shear.

Table 3. LRFR Generalized Live-load Factors for Legal Loads, γ_L

Traffic Volume (one direction)	Load Factor
Unknown	1.80
ADTT>5000	1.80
ADTT=1000	1.65
ADTT< 100	1.40

1.2 Vehicular Load Models

The most commonly used vehicular load models for load rating include the H-20 and HS-20 AASHTO design vehicles which are used in conjunction with ASR and LFR, and the AASHTO HL-93 design load used with LRFR. In addition, the AASHTO legal trucks which have the configurations of typical trucks loaded to satisfy the legal weight requirements as prescribed in what is known as the bridge formula are used in all three rating methods. The configurations and the weights of the AASHTO HL-93, HS-20 and H-20 and trucks are provided in Figures 1 and 2. The configurations of the AASHTO legal trucks are provided in Figure 3. These trucks are the same AASHTO Legal loads provided in the AASHTO Manual for Condition Evaluation of Bridges (MCE) for ASR and LFR ratings suitable for states that comply with federal weight laws, including the Federal Bridge Formula B. The LRFR Manual also provides additional legal lane load models for spans greater than 200 ft and for rating of negative moment areas of continuous spans. Other truck configurations such as state-specific legal trucks are used when state legal loads exceed federal weight limits (exclusion vehicles) or permit trucks during the permit review process. The maximum moment effect, axial load, or shear load effect on a structural member is obtained by placing the appropriate loads on the most critical location and performing a structural analysis of each structure-rating unit of the bridge being rated.

According to the NYSDOT procedures, H operating ratings are used to determine load posting and R-posting requirements. Additional discussion on the NYSDOT posting procedures is provided in Section 3.

According to LRFR, there are three distinct load rating procedures: (1) design load rating, (2) legal load rating and (3) permit load rating. In LRFR, bridges that produce inventory ratings $RF \geq 1.0$ will have satisfactory load rating for all legal loads and no restrictive posting is required in states that allow the operation of exclusion vehicles. Load rating for the AASHTO legal loads is required only when a bridge fails the design load rating at the operating level i.e. when the operating rating factor with the HL-93 load model leads to a rating factor RF<1. Only bridges that pass the load rating for AASHTO legal loads should be evaluated for overweight permits. This is similar to the NYSDOT procedure that does not allow permit trucks on load posted bridges. The LRFR load rating, load posting, load-permitting process is performed in a sequential manner following the flow chart provided in Figure 4.

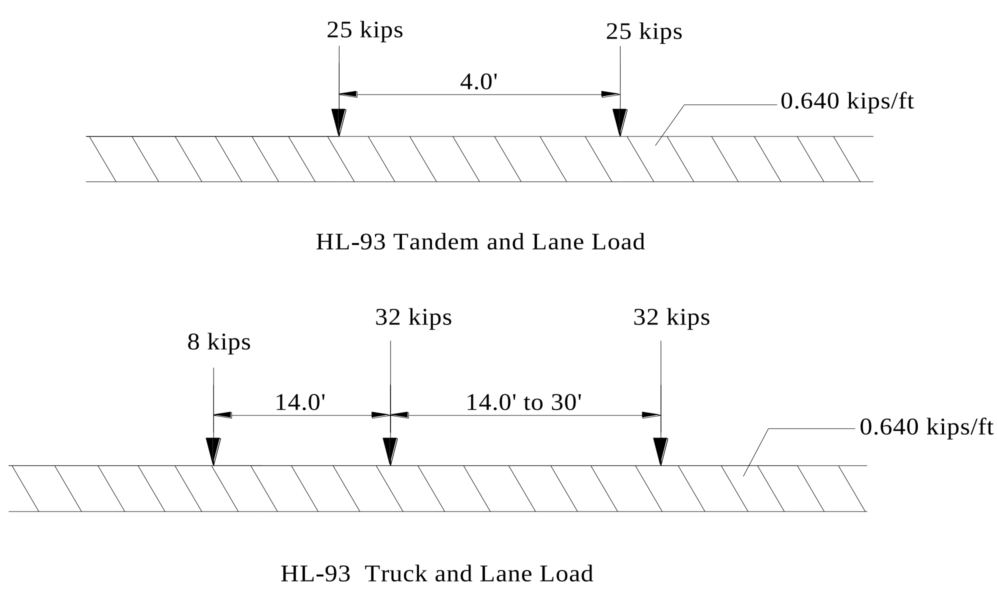


Figure 1. HL-93 Tandem and Truck plus Lane Loads
(Lane load model for negative moment and pier reaction not shown)

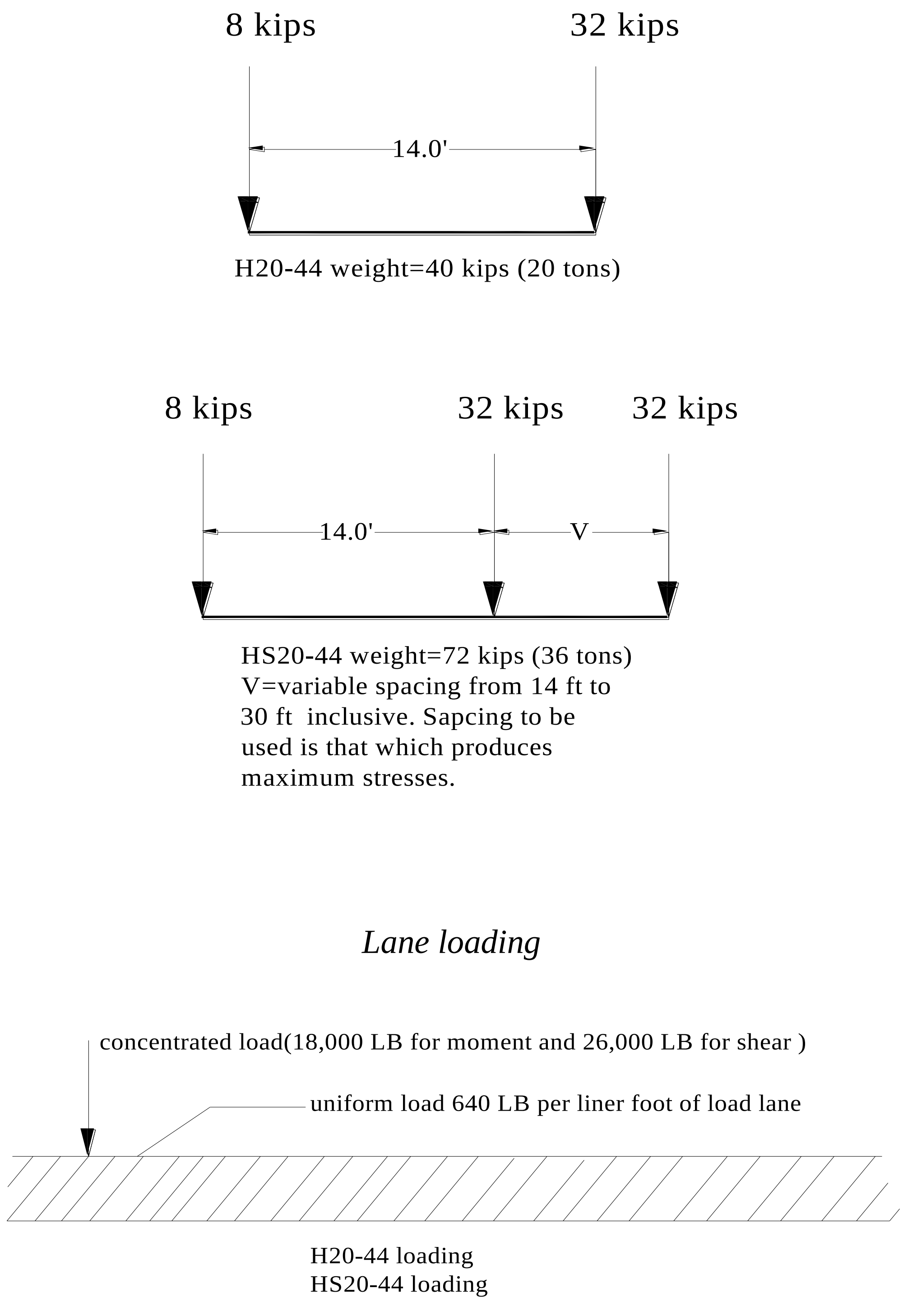


Figure 2. AASHTO H-20 and HS-20 Truck configurations

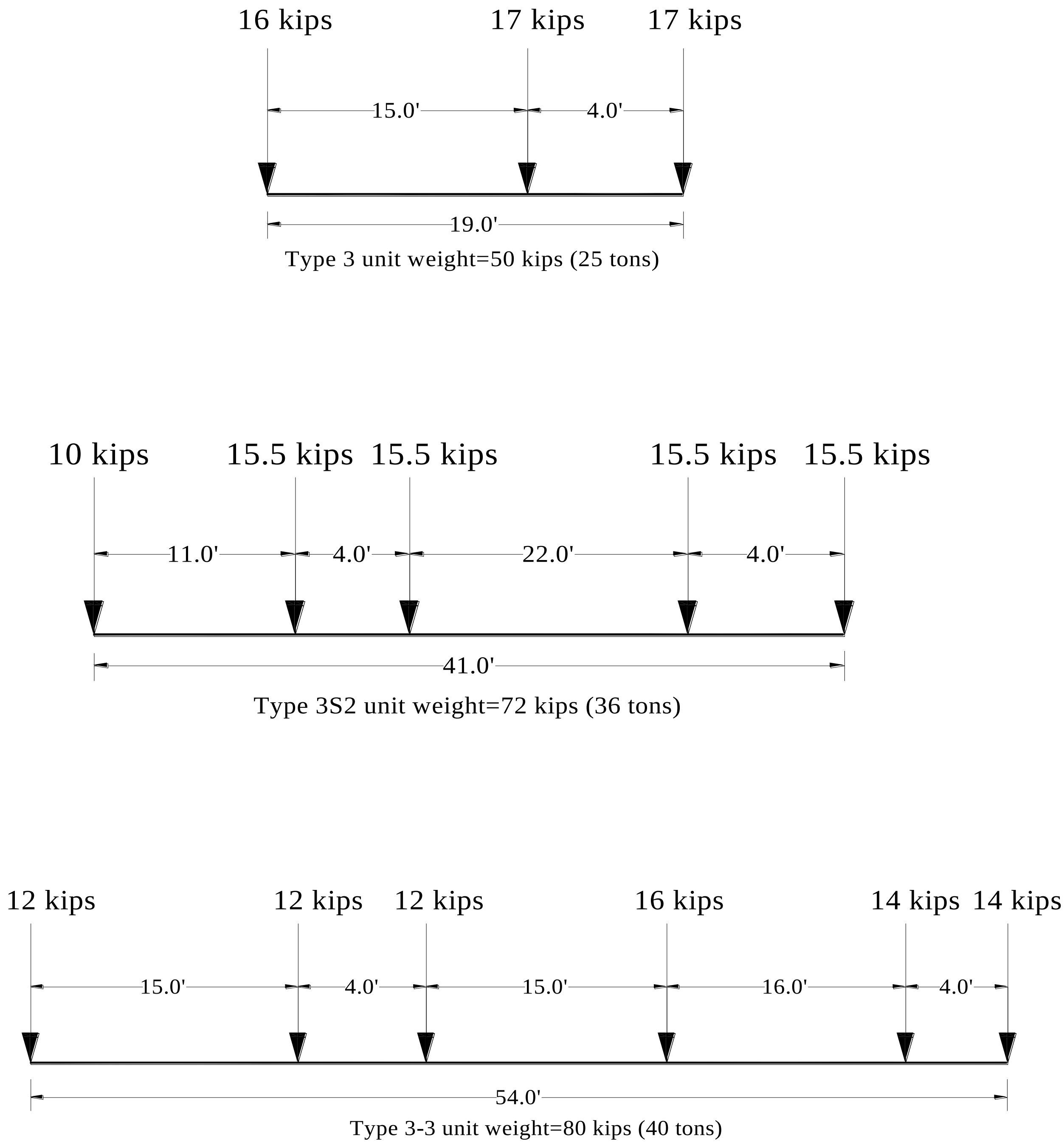


Figure 3. AASHTO Legal Truck Configurations

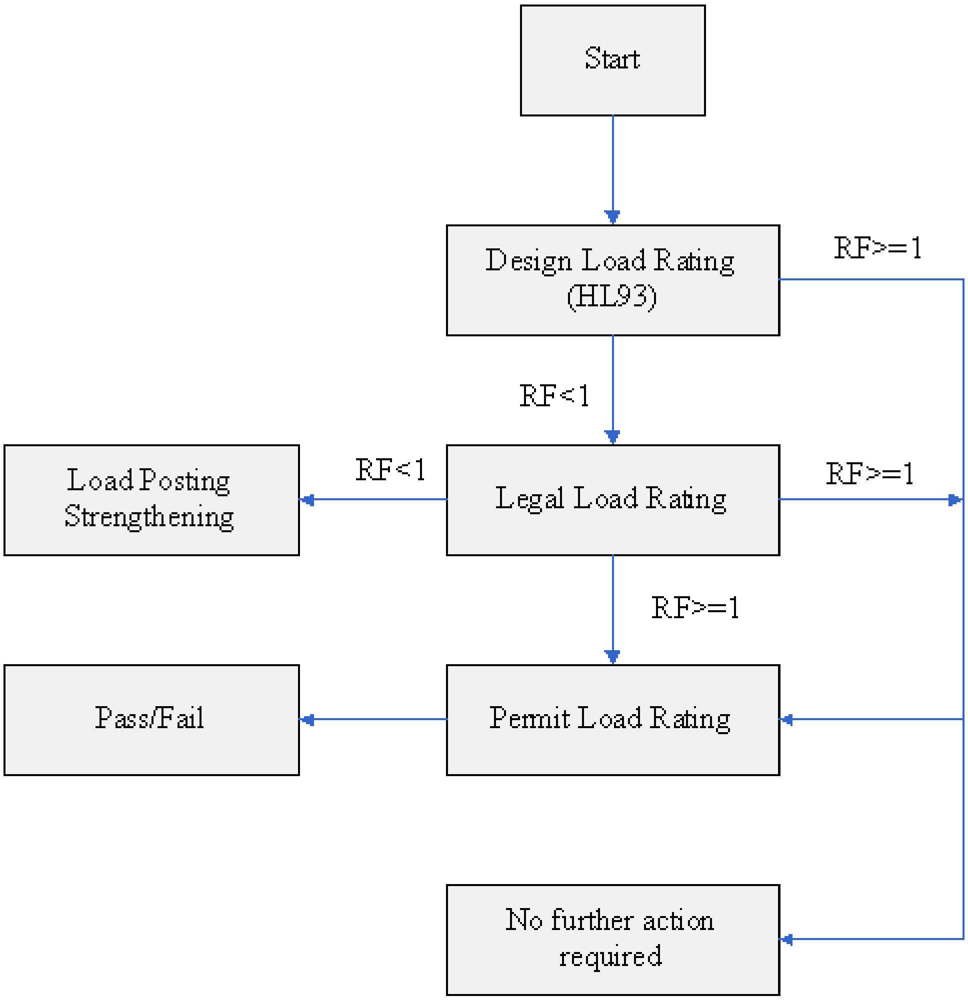
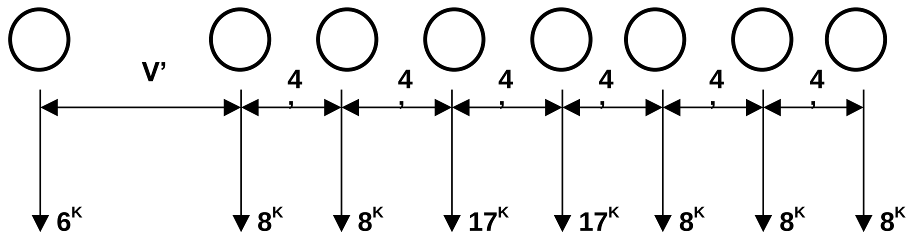


Fig. 4. Flow chart for LRFR load rating process

1.3 New AASHTO Legal Loads for Specialized Hauling Vehicles

The trucking industry has in recent years introduced new types of Specialized Hauling Vehicles (SHV) with closely-spaced multiple axles that make it possible for these short wheelbase trucks to carry a load of up to the 80,000 lbs maximum allowable and still meet the Bridge Formula. The three AASHTO legal loads do not properly represent these new axle configurations. For this reason, new rating and posting load models for SHVs were adopted by AASHTO in 2005. A new Notional Rating Load model NRL which is a screening level load shown in Figure 5 was developed to envelop the load effects on simple and continuous span bridges of the worst possible Formula B single unit truck configurations. Bridges that do not rate for the NRL loading should then be investigated to determine posting needs using the single unit posting loads SU4, SU5, SU6 and SU7 shown in Figure 6. Because different states impose different restrictions on the configurations and weights of multi-axle trucks, each state is required to only post for the vehicles that are allowed to operate in that State.



V = VARIABLE DRIVE AXLE SPACING — 6'0" TO 14'-0". SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM LOAD EFFECTS.

AXLES THAT DO NOT CONTRIBUTE TO THE MAXIMUM LOAD EFFECT UNDER CONSIDERATION SHALL BE NEGLECTED.

MAXIMUM GVW = 80 KIPS

Figure 5. AASHTO Notional Rating Load NRL for SHVs

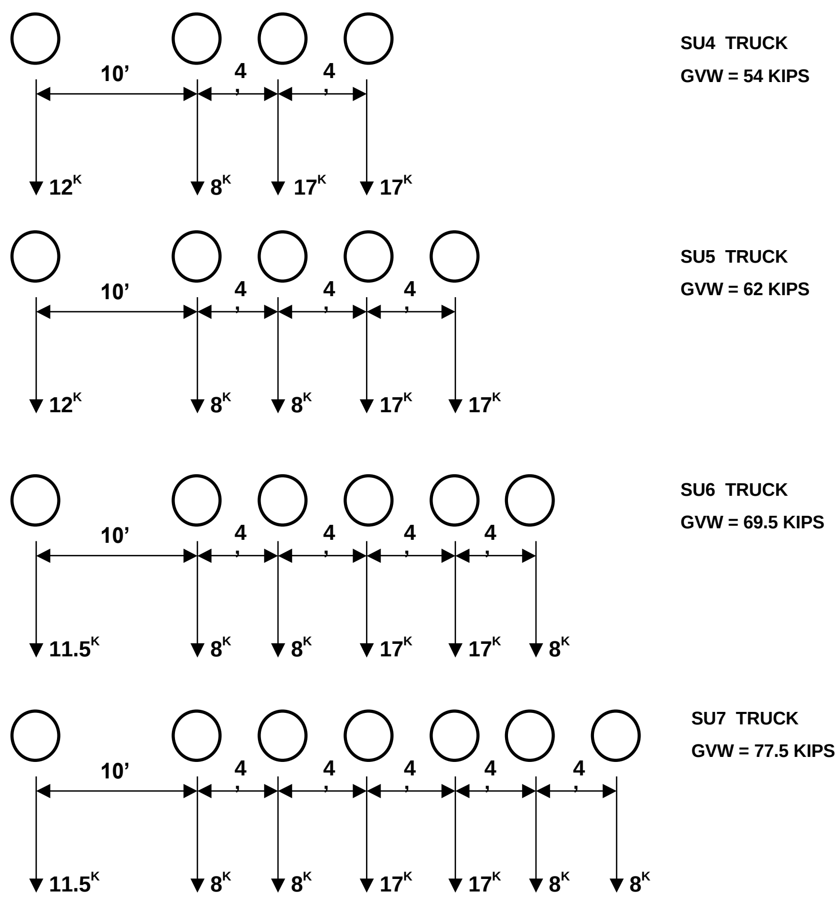


Figure 6. AASHTO Posting Loads for Single Unit SHVs

1.4 Impact factor

The effect of the live loads is increased to allow for dynamic, vibratory and impact effects. In ASR and LFR, the dynamic allowance or impact increment is expressed as a fraction of the static live load stress, and is determined by:

$$I = \frac{50}{125 + L} \qquad (3)$$

where, $I =$ impact fraction (maximum 30 percent); and $L =$ length in feet of the portion of the span that is loaded to produce the maximum stress in the member. (Note that the LRFD and LRFR use the notation IM instead of I for impact).

The AASHTO LRFD specifies a dynamic load allowance IM=15% for fatigue and fracture limit states, and IM =33%, which are only applied to the truck and tandem loads, for all other limit states. LRFR recommends the same IM values when rating for the HL-93 design loading but allows the optional use of reduced impact factors depending on the condition of the riding surface for legal load rating and for permit load rating. For example, an impact factor IM=10% is allowed for bridges with smooth riding surface at approaches, bridge decks and expansion joints. An impact factor IM =20% may be used for bridges with minor surface deviations or depressions. To ensure proper and consistent selection of dynamic load allowance values in all load ratings, the bridge inspection report should carefully note these surface discontinuities in order to benefit from a reduced dynamic load allowance.

1.*5* Load distribution factor

In ASR and LFR, the live load bending moment and shear force for each interior and exterior beam of a multi-beam system is determined by applying to the beam the fraction of a wheel load determined using the load distribution factor specified in the AASHTO standard specifications. For example, for interior stringers and beams of bridges having concrete decks on steel I-beam and pre-stressed concrete girders, the load distribution factor, g, is calculated based on the formula:

$$g = \frac{S}{5.5} \qquad (4)$$

where, S is the average distance between beams in feet. Other formulas are provided in the AASHTO Standard Specifications when S is larger than 14 feet, for exterior beams, and for other bridge types. The factor specified in the AASHTO Standard Specifications is applied to the moment or shear effects of vehicular wheel loads, that is, the g factor is applied to half the weight of the design load.

Significant changes to the distribution of live loads to longitudinal members have been introduced in the LRFD Specifications. The traditional “S-over” formulas in the Standard specifications are easy to apply, but can be overly conservative in some parameter ranges while unconservative in others. For these reason, the LRFD introduced “exponential distribution” formulas which were derived to represent the girder distribution from refined analysis. The load distribution factors obtained from the LRFD formulas are believed to be accurate to within 5% of the results obtained through refined methods of analysis.

The AASHTO LRFR, following the LRFD approach, calculates the live load distribution factors based on the full vehicle and lane load effects, instead of the wheel load effects. For example, for bridges with a concrete deck on steel or prestressed concrete girders, the distribution factor for moments in interior beams is given as:

For one design lane loaded: $$g_{m1} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^{\ 3}}\right)^{0.1} \quad (5)$$

For two design lane loaded: $$g_{m2} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^{\ 3}}\right)^{0.1} \quad (6)$$

where, S=spacing of stringers (ft), L=span length (ft), t_s=depth of concrete slab (in), and K_g=longitudinal stiffness parameter of the stringer (in^4) given by:

$$K_g = n\left(I + Ae_g^{\ 2}\right) \quad (7)$$

where, n=ratio of modulus of elasticity between the beam material and concrete deck, I=moment of inertia of the beam (in^4), A=section area of the beam (in^2), and e_g=distance between the centroids of the beam and the deck (in).

The LRFD specifies different load distribution formulas for exterior and interior girders, for shear and moment, and for one-lane loaded and two-or-more-lanes loaded cases. For example, Equations 5 and 6 are used only for the moment distribution factors for interior girders of steel and prestressed concrete bridges. The LRFD specifies the ranges of applicability for each equation. If one or more of the parameters exceed the range of applicability, engineering judgment needs to be exercised before using these formulas or a refined analysis should be performed as the accuracy of the LRFD formulas have not been verified outside the specified ranges. The live-load distribution formulas can be applied to the AASHTO family of legal trucks, and permit vehicles whose overall widths is comparable to the width of the design truck.

In LRFD/LRFR the live load distribution for exterior beams shall be taken as the larger of the value obtained from three methods specified. They include, 1) The lever rule, 2) Distribution formulas, 3) Special Analysis. The Special Analysis provision that treats the bridge cross-section as a rigid body was added to the LRFD specifications because the original study that developed the distribution factor equations did not consider the effects of intermediate diaphragms.

It is important to recognize the influence of the changes in load distribution methods on load rating results when changing from the ASR and LFR methods to the LRFR/LRFD methods. The changes may result in beneficial effects on some ratings but lead to reduced ratings in other cases. Past ratings have shown that shear distribution analyses using LRFR/LRFD methods can result in higher load effects on members than with the traditional "S-over" formulas. This can however be offset by increased shear resistance in many cases when using the Modified Compression Field Theory (MCFT) for concrete structures introduced in the LRFD Specifications.

1.6 Rating example

To illustrate the differences between the load rating of a bridge using the LFR approach currently followed by NYSDOT and the LRFR approach, an example taken from the work prepared by Lichtenstein for NCHRP 12-46 project is presented (Lichtenstein, 2001).

In this example, the rating analysis is performed for a simple span composite steel multi-girder bridge shown in Fig. 7. The four-girder bridge has a span L=65 ft. Each section acts as a composite section for the live load and superimposed dead loads, while the permanent load acts on the non-composite section. The bridge's W33x130 rolled steel girders, spaced at 7' 4"center to center, are reinforced by 5/8 in x10 ½ in plates. The steel grade is A-36 producing a nominal steel yielding strength F_y=36 ksi while the composite concrete deck's strength is f'_c=3 ksi. The beams' conditions showed no deterioration (NBI item 59=7). The riding surface showed minor surface deviations and the Average Daily Truck traffic is ADTT=1000 trucks per day.

The section properties lead to a yielding moment M_y=2140.7 kip-ft and a plastic moment M_p=3011.7 kip-ft. The nominal flexural resistance is approximated to be $R_n = M_n$ =2877.8 kip ft.

The dead load effects are obtained as M_{DC1}=439 kip-ft for the weight of the deck and components and M_{DC2}=129 kip-ft for the superimposed dead loads producing a total dead load moment M_{DT}=568 kip-ft. Table 4 gives the maximum live load moments for the different vehicular loads described in Figures 1 through 3.

Table 4. Live load moments on 65-ft simple span from different vehicular loads.

	Loading	Maximum Moment
H-20 and HS-20 loading (LFR)	HS20-44	895 kip ft
	H20-44 truck	595 kip ft
	H20-44/ HS20-44 lane load	630 kip ft (governs for H-20)
Legal Vehicles (LFR & LRFR)	Type 3 legal load	660 kip ft
	Type 3S2 legal load	707 kip ft (governs)
	Type 3-3 legal load	654 kip ft
HL-93 loading (LRFR)	HL-93 truck load	890 kip-ft (governs)
	HL-93 tandem load	763 kip-ft
	HL-93 lane load	338 kip-ft

Load Factor Rating (LFR)

The impact factor is calculated as: $I = \frac{50}{125+L} = \frac{50}{125+65} = 0.263$

The LFR load distribution factor is obtained as: $g = \frac{S}{5.5} = \frac{7.33}{5.5} = 1.33$

The corresponding Rating Factors for the HS and H and Legal Truck loads are calculated based on the LFR method, which is the preferred method according to current NYSDOT procedures. The LFR specifies a resistance factor ϕ=1.0, a dead load factor γ_D=1.3 and a live load factor γ_L=2.17 for the inventory rating of the moment limit state. For operating ratings, a live load factor γ_L=1.3 is used. The rating factors for the HS, H and governing legal vehicle are calculated using Eqs. 1 and 2:

a. HS-20 ratings

$$\text{Inventory RF}=\frac{1.0*2877.8-1.3*568}{2.17*(1.263*1.33*895/2)}=1.31$$

$$\text{Operating RF}=\frac{1.0*2877.8-1.3*568}{1.3*(1.263*1.33*895/2)}=2.19$$

b. H-20 ratings

$$\text{Inventory RF}=\frac{1.0*2877.8-1.3*568}{2.17*(1.263*1.33*630/2)}=1.86$$

$$\text{Operating RF}=\frac{1.0*2877.8-1.3*568}{1.3*(1.263*1.33*630/2)}=3.10$$

c. Legal load ratings

$$\text{Inventory RF}=\frac{1.0*2877.8-1.3*568}{2.17*(1.263*1.33*707/2)}=1.66$$

$$\text{Operating RF}=\frac{1.0*2877.8-1.3*568}{1.3*(1.263*1.33*707/2)}=2.77$$

An H-20 operating rating of 3.10 indicates that the bridge has an equivalent H rating equal to H62 (62=20x3.1). An HS-20 operating rating of 2.19 indicates that the bridge has an equivalent HS rating equal to HS43 (43.8=20x2.19) or the rating can be said to be HS equivalent to 78 tons obtained by taking the weight of the HS20 vehicle in tons (36 tons) and multiplying it by the rating factor (2.19).

Load and Resistance Factor Rating (LRFR)

For LRFR, the dynamic allowance or impact factor is IM=33%.

The distribution factor for moment g_m is obtained for one lane and two lanes loaded using Eqs. 5, 6 and 7 as follows:

$$K_g = n\left(I + Ae_g{}^2\right)$$

$$n = \frac{E_{steel}}{E_{concrete}} = \frac{29000}{33000(0.145)^{1.5}\sqrt{3}} = \frac{29000}{3155.9} = 9.2$$

$$I = 8293\ in^4$$

$$A = 44.82\ in^2$$

$$e_g = 1/2(7.25) + 19.02 = 22.65\ in$$

$$K_g = (29000/3155.9)(8293 + 44.82*22.65^2) = 287493\ in^4$$

Distribution factor for one lane loaded:

$$g_{m1} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s{}^3}\right)^{0.1} = 0.06 + \left(\frac{7.33}{14}\right)^{0.4}\left(\frac{7.33}{65}\right)^{0.3}\left(\frac{287493}{12*65*7.25^3}\right)^{0.1}$$

$$= 0.46$$

Distribution factor for two or more lanes loaded:

$$g_{m2} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s{}^3}\right)^{0.1} = 0.075 + \left(\frac{7.33}{9.5}\right)^{0.6}\left(\frac{7.33}{65}\right)^{0.2}\left(\frac{287493}{12*65*7.25^3}\right)^{0.1}$$

$$= 0.626 > 0.46$$

Thus, the two-lane $g_m = 0.626$ governs.

The maximum live load moment effect is given as a combination of the design lane load moment = 338 kip-ft, and the design truck moment = 890 kip-ft. When the truck moment is multiplied by the dynamic allowance, the dynamic live load effect on one lane of traffic is obtained as $M_{LL+IMM} = 338 + 890 * 1.33 = 1521.7 \; kip \cdot ft$

a. HL-93 ratings

A dead load factor γ_D=1.25 is used for all components in Eq. (1).

$$\text{Inventory RF=}\frac{1.0 * 2877.8 - 1.25 * 568}{1.75 * 0.626 * 1521.7} = 1.30$$

$$\text{Operating RF=}\frac{1.0 * 2877.8 - 1.25 * 568}{1.35 * 0.626 * 1521.7} = 1.69$$

The inventory rating RF=1.30 for the LRFR is similar to the RF=1.31 obtained for the LFR with HS-20. On the other hand, the operating rating RF=1.69 obtained for the HL-93 load with LRFR is lower than the operating rating RF=2.19 obtained for the HS-20 loading in LFR. For inventory rating, the higher moment due to the HL-93 loading in LRFR compared to the HS-20 loading in LFR is offset by the lower LRFR live load factor leading to similar inventory ratings. The LRFR operating rating uses a slightly higher live load factor than the LFR operating live load factor leading to the lower RF operating ratings in LRFR (RF=1.69) compared to RF=2.19 for the LFR.

It should be noted that because the live load bases of the LFR and LRFR are different and because of the differences in the dead load factors as well as the impact allowance and the distribution factors, a direct comparison between the LFR and LRFR rating factors is not possible. A better comparison should be based on more uniform criteria such as the reliability index, β. The LRFR load factors for Inventory and Operating ratings were selected to provide uniform reliability indices of 3.5 and 2.5 respectively for typical bridge configurations. On the other hand, the LFR rating factors do not correlate well with the reliability index. In fact, Mertz (2005) found that some bridges with high LFR design load Rating Factors produced very low reliability index values.

The study by Mertz (2005) also showed that in LRFR the ratio of Operating Ratings to Inventory Ratings OPR/INR is about 1.30, compared to a ratio of 1.67 in LFR. The difference in these ratios is partially attributed to the fact that the LFR Operating rating denotes a maximum permissible live load for a structure suitable for one-time or limited crossings where the live load is reasonably well known and is associated with little uncertainty. Whereas the reliability-based calibration of the Operating Live Load factor in LRFR, assumes indefinite crossings as long as the bridge is properly maintained and regularly inspected.

b. Legal load ratings

The inventory design load rating produced rating factors greater than 1.0, therefore according to the LRFR, this bridge has adequate load capacity to carry all legal loads and need not be subject to legal load ratings. The load rating computations that follow have been done for illustrative purposes.

A reduced value for the impact I=20% (rather than the specified standard I=33%) is allowed based on field evaluation verifying that the approach and bridge riding surfaces have only minor surface deviations or depressions.

A reduced live load factor γ_L is allowed because the Average Daily Truck Traffic at this bridge is relatively low and a lower probability of multiple truck presence is expected. For ADTT=1000, the LRFR gives a generalized live load factor for legal load $\gamma_L = 1.60$

$$RF = \frac{1.0 * 2877.8 - 1.25 * 568}{1.6 * 0.626 * 1.2 * 707} = 2.55$$

The rating factor for legal load based on the LRFR, RF=2.55 is relatively close to the operating load factor rating RF=2.77 from the LFR. This is primarily due to the lower dynamic allowance (impact factor) used in this case along with the slightly lower dead load factor, which partially offset the higher live load factor while the truck load is the same. Note that the live load factor $\gamma_L = 1.60$ is used in this instance because of the relatively low truck traffic rate. Higher live load factors must be used for bridges with higher truck traffic.

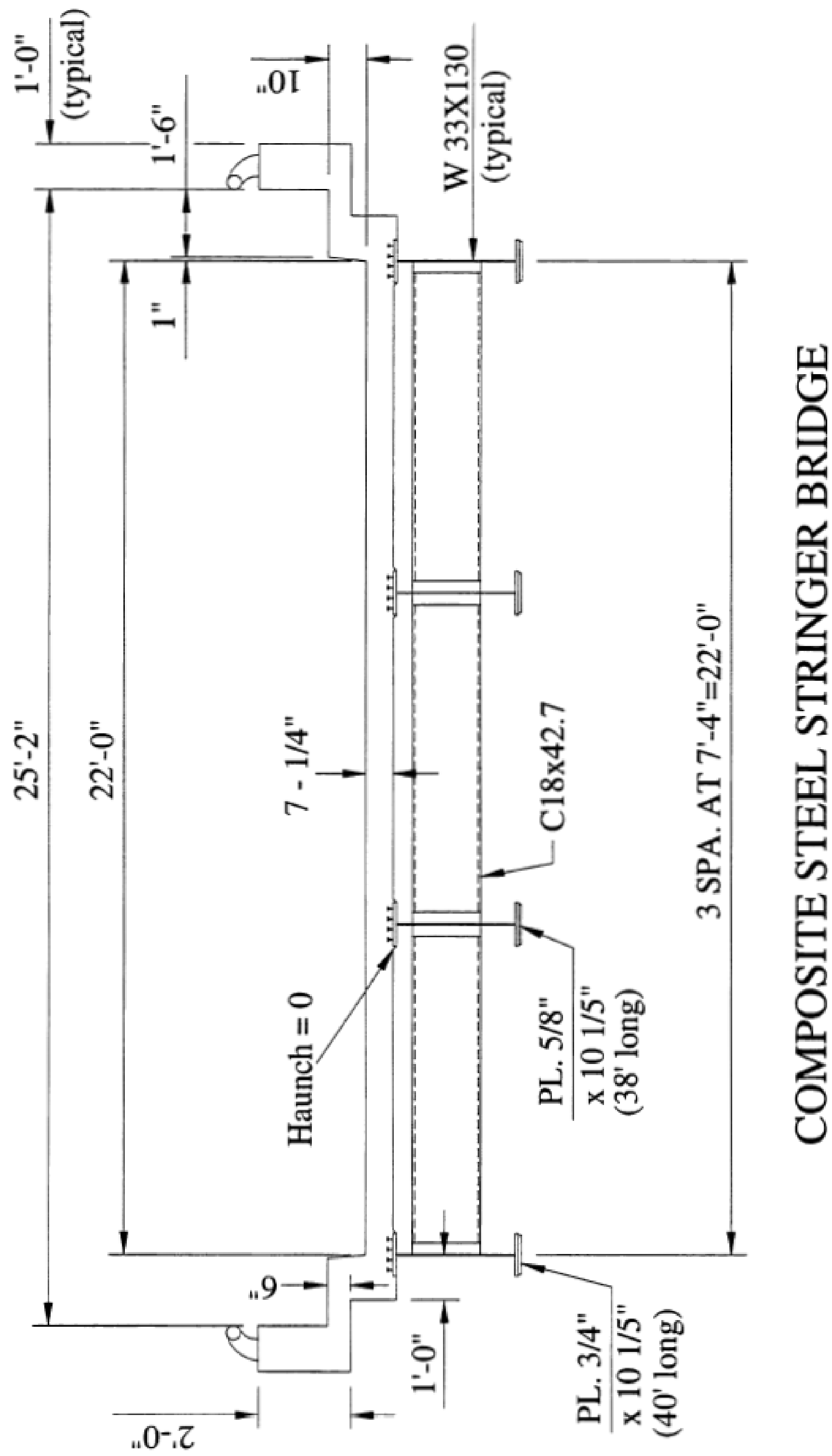


Figure 7. Example Bridge Cross Section

2. Permit Load Rating

According to NYSDOT regulations, a vehicle is considered overweight if it does not conform to the weight and axle spacing limits stipulated in Title III, Article 10, Section 385 of the New York State Vehicle and Traffic Law (NYS Government Documents 2005-2006). Legal weight vehicles are generally defined as vehicles with 3 or more axles weighing a total of 34,000 lbs plus 1000 lb per foot measured from the first to the last axle. If the vehicle's gross weight is less than 71,000 lbs, the higher value from the above stated limit or the limit imposed by the AASHTO Bridge Formula B will govern. Bridge Formula B is given as:

$$W = 500\left(\frac{L * N}{N-1} + 12N + 36\right) \tag{8}$$

where W equals overall gross weight on any group of two or more consecutive axles in pounds, L equals distance in feet from the center of the foremost axle to the center of the rearmost axle of any group of two or more consecutive axles, and N equals number of axles in the group under consideration. Two consecutive sets of tandem axles may carry a gross load of 34,000 lbs. For any vehicle or combination of vehicles having a total gross weight of 71,000 lbs or greater, Formula B shall apply to determine the maximum gross weight which is permitted. The total weight of legal vehicles should be less than 80,000 lbs and the maximum axle weight cannot exceed 22,400 lbs. (See section 385 of The New York State Vehicle and Traffic Law for additional restrictions)

NYSDOT has a set of established procedures to allow the passage of vehicles that exceed the legally established weight limits (Lagace; 2006). Special Hauling Permits and Divisible Load Overweight Permits are issued by NYSDOT to protect public safety and preserve the State's infrastructure by minimizing the risk of damage from large and overweight loads. Special Hauling Permits are required to allow the movement on New York State highways of vehicles or loads that exceed the legal dimensions or weights specified in Section 385 of the New York State Vehicle and Traffic Law. If the permit application is for self propelled construction equipment or for vehicles with a gross weight of 160,000 lbs or greater, a structural review by the Office of Structures must be performed. Loads with gross weights that are 200,000 lbs or greater are classified as superloads and are subject to special requirements. Under special circumstances, NYSDOT may require that the applicant for overweight permits have a Professional Engineer, who is licensed in the State of New York, perform the required structural analysis and prepare a report to the Department for review.

The bridge load rating factor, RF, is an important indicator of a bridge's capacity to safely carry vehicular load. Generally, standard vehicle loads, like the AASHTO HS, H, or legal loads, are used by NYSDOT in the calculation of the rating factor. Since permit vehicles have usually different configurations, the design and legal load factors cannot be directly used for permit reviews. Instead, a method is used to replace the permit vehicle by an equivalent vehicle having a standard truck configuration but an adjusted gross weight. Typically, an HS equivalent truck is used. Alternatively, a rating factor for the permit load is directly calculated and the corresponding RF is used to decide whether the permit load should be allowed to cross over a particular bridge. In the ASR and LFR methods there were no specific guidance provided as to how overweight permit loads should be evaluated. Review of permits was left up to the bridge owners without any accepted national standards for permit ratings. The lack of consistency in this area from state to state was an important reason for the more detailed treatment of permit load ratings in the LRFR method. Uniform permitting methods would encourage better compliance from the trucking companies that need to get permits in multiple states for moving a single load (Fu and Fu; 2006).

NYSDOT approach for permit load rating consists of loading the bridge with the permit vehicle and determining whether the bridge can sustain the applied load. This is done by comparing the permit vehicle load effect (PVLE) defined as the moment or shear caused by the permit vehicle compared to the effect of a standard design vehicle, which is usually taken to be the effect of the HS-20 design vehicle except for annual crane permits which are analyzed by determining the overstress above the H-20 design vehicle for multiple spans. The NYSDOT load permitting procedure depends on the permit types and overload vehicle types which are categorized below.

2.1 NYSDOT Permit types

The five different permit types currently used in New York State are listed as follows:

a. Trip Permit

A trip permit is issued for a limited time for a specific route only. Due to the low frequency and specific route of the overload, the allowable overloads for this type of permits are the highest. Speed restrictions may be required to reduce the impact and overall load effects. Also, restrictions may be imposed on the lateral position of the vehicle during crossing. For example, the permit vehicle may be required to cross over the center line of the bridge and might be escorted to restrict other trucks from travelling simultaneously over the bridge.

b. Annual Crane Permits

These are issued for either a 25 mile radius or as a "base county permit" and allow for movement on the State highways. Annual Cranes are not permitted to cross R-Restricted or Load Posted bridges.

c. Radius Permits

These are issued to allow vehicle movement on State highways within a certain air mile radius distance of a given location. The Office of Structures reviews 5 mile radius permits.

d. Blanket Permits

Blanket permits are issued to allow vehicle movement on State highways with a pavement width of 20 feet or more.

e. Emergency Blanket Permits

These are issued to allow emergency vehicle movements.

The Office of Structures of NYSDOT reviews permits for only categories a, b and c. The other categories are reviewed by the traffic safety units.

2.2 NYSDOT Overload Vehicle Types

The overload types reviewed by the Office of Structures of NYSDOT consist of:

a. Non-Divisible Loads:

These are comprised of single or combination vehicles whose axle weights and configurations exceed the legal limits. These include oil tankers, milk trucks, etc.

b. Self-Propelled construction equipment:

These include scrapers, cranes, loaders, and self-propelled drilling rigs.

c. Military tanks:

These should be moved on trailers whenever possible. If they are moving on tracks, only trip permits will be issued.

Based on the Codes, Rules and Regulations of the State of New York, Annual Crane Permits are analyzed using the H-20 overstress, which is the additional stress caused by the permit vehicle on the bridge above the total stress caused by an H-20 design vehicle. All other permits should be analyzed using the Permit Vehicle Load Effect (PVLE), which is the ratio, expressed in percentages, of live load moments and shears created by the Permit Vehicle over a range of span lengths, compared to the HS-20 design vehicle.

2.3 NYSDOT Permit Review Procedure

Depending on the PVLE and the permit type, overloads can be either: a) allowed on all non-posted bridges; b) allowed on bridges that are not load posted nor R posted; c) allowed for trip permits only, d) trip permits allowed with restrictions on speed and/or multiple presence; or e) not allowed.

Bridge safety for permit loads is checked by comparing the PVLE to the HS-20 operating rating. For example, assume that the moment effect of a permit vehicle on the 65-ft simple span studied earlier is equal to M_P=1495 kip-ft. This value is compared to the moment effect of the HS-20 design truck which is M_{HS20}=895 kip-ft. Thus, the permit truck has a PVLE =167% (1.67=1495/895). A PVLE of 1.67 is equivalent to and HS vehicle of 60 tons $\left(= 1.67 \frac{72{,}000 \ lbs}{2{,}000 \ lbs / ton} \right)$. This permit truck will be allowed to cross the 65-ft bridge described in Figure 7 which had been found to have an operating HS-20 rating RF=2.19 or an HS capacity of 79 tons $\left(= 2.19 \frac{72{,}000 \ lbs}{2{,}000 \ lbs / ton} \right)$.

The PVLE is calculated using a software program known as BIGTRUCK which takes as input the axle weights and spacing of the permit vehicle and calculates the PVLE for different span lengths. A Geographic Information System (GIS) that provides an electronic State Highway Map graphically indicates the location of bridges with travel restrictions. The GIS also links information on each restriction contained in the NYSDOT database to the graphical representation of the restriction on the map. The GIS is used to determine the routes that a permit truck can safely take. NYSDOT procedures do not allow the use of Level 3 Load ratings for permit review since these load ratings are only estimates that are not based on actual section properties. In addition, bridges where the inspection reports show primary member ratings of 3 or less are seriously deteriorated and are not allowed to be crossed by permit vehicles without special reviews. Similarly, bridges with deck ratings of 1 should not be crossed by permit overloads without the review of the Regional Office.

The NYSDOT permit review process is summarized in Table 5. Note that bridge operating ratings are based on all lanes being loaded. Most special hauling permit vehicles reviewed by the Office of Structures that have a PVLE below 150% of an HS20 AASHTO design vehicle, can be issued with the restriction that they not cross any R-Posted or Load Posted bridge. The 150% threshold combined with NYSDOT posting procedure ensures that the non-posted bridges are not overstressed.

Decisions for annual crane permits are made by determining the H20 overstress created by the specific crane. The overstress is calculated by BIGTRUCK. The program has pre-stored assumed Dead Load values for different simple span lengths. These are combined with the Live Load effects from an H20 truck to calculate total moments and shears for different simple span lengths. The total moments and shears for the H-20 are then compared to the Dead Load and Permit Truck Live Load moments and shears to determine the permit vehicles “overstress” above H20. An annual crane permit is recommended if the H-20 overstress is less than or equal to 50%. The annual cranes typically have the restriction that R-Posted and Load Posted bridges can not be crossed.

For Suffolk and Nassau Counties, a restricted annual crane permit is issued if the H-20 overstress is above 50%, but the PVLE is below 54 tons after the application of a multiple presence reduction factor given as 8ft divided by the Permit vehicle width without going less than 0.85.

The same conditions apply for Trip Permits. However, Trip Permits may be allowed if the PVLE is greater than 150%. In this case, the rating calculations are also adjusted by applying a multiple presence reduction factor The entire route is then checked to make sure the bridges along the route are sufficient to carry the permit vehicle in question. Speed restrictions may be required if the bridge load rating for trip permits is still exceeded despite the application of the multiple presence reduction factor to further reduce the PVLE by the value of the impact factor.

Although it uses a different approach for applying the multiple presence factors, the LRFR permit procedures are somewhat similar to those followed by the NYSDOT but more comprehensive in that all load effects on a bridge from permit loads are checked including very specific requirements for checking serviceability under permit load crossings.

Table 5. Permit review procedure used in New York State

Permit Type	Load effect (moment and shear) of Permit Vehicle without Multiple Presence Reduction Factor	Review Action Taken
All Permits (except Annual Crane Permits)	Less than or equal to 100% HS20	All permits recommended approved for bridges where the load posting is not exceeded.
5 Mile Radius Permits	Greater than 100% and less than or equal to 150% HS20	Radius permits recommended approved for bridges that are not “R” or load posted.
	Greater than 150% HS20	Radius Permit recommended disapproved, applicant should apply for a Trip Permit.
Annual Crane Permits	Less than or equal to 50% H20 overstress	Annual Crane Permits recommended approved for bridges that are not “R” or load posted.
	Greater than 50% H20 overstress	Annual Crane Permits recommended disapproved, applicant should apply for a Trip Permit (exceptions: Nassau and Suffolk counties).
Trip Permits	Greater than 100% and less than or equal to 150% HS20	Trip permit recommended approved for bridges that are not “R” or load posted.
	Greater than 150% HS20	Route is checked for full speed for all bridges (Moment and Shear Load Effects with Impact). Apply Multiple Presence Reduction Factor (8 foot/Vehicle width, min. 85). Speed Restrictions on individual bridges or routes may be required.

2.4 Permits for R-Posted Bridges

Many R-posted bridges are strength or condition deficient. Therefore, according to NYSDOT procedures, permits should typically not be issued if the load effect of a bridge is above 100% of an HS-20 unless there is no other viable alternative. In addition, Radius or Blanket permits may not be issued for R-posted bridges, unless the load effect of permit vehicle without reduction factor is less than or equal to 100% of an HS20 design vehicle. Assumptions made for permit review for R-posted bridges are more conservative than for normal bridges, because their operating ratings are relatively low. Procedures for load posting and R-posting NYSDOT bridges are discussed in Section 3.2.

2.5 LRFR Permit Load Rating

Introduction

The LRFR Manual provides procedures for checking overweight trucks that are analogous to load rating for legal loads except that load factors are selected based upon the permit type. The permit live load factors were derived to account for the possibility of simultaneous presence of non-permit heavy trucks on the bridge when the permit vehicle crosses the span. Thus, the load factors are higher for spans with higher ADTT and smaller for heavier permits. The calibration of permit load factors is also tied to the live load distribution analysis method with the one lane distribution used for heavy special permits.

The target reliability level for routine permit crossings is established as the same level as for legal loads, consistent with traditional AASHTO Operating ratings. For single and multiple-trip special permits that are allowed to mix with traffic (no restrictions on other traffic) the live load factors were explicitly derived to provide a higher level of reliability consistent with AASHTO Inventory ratings. The increased risk of structural damage and associated benefit/cost considerations leads to higher safety requirements for uncontrolled very heavy special permit vehicles than other classes of trucks. The live load factors for single trip escorted permits that are required to cross bridges with no other vehicles present have been calibrated to reliability levels consistent with traditional AASHTO Operating ratings. Target reliability at the Operating level is allowed because of the reduced consequences associated with allowing only the escorted permit vehicle alone to cross the bridge. An agency may also elect to check escorted permits at the higher design or Inventory level reliability by using an increased live load factor as noted in the LRFR Manual commentary.

According to the LRFR, and as shown in the flow chart of Figure 4, permit load rating should be used only if the bridge has a rating factor greater than or equal to 1.0 when evaluated for the AASHTO HL-93 loads or the legal loads. The LRFR recognizes two permit types:

a) Routine (or Annual) Permits.

These are valid for unlimited trips for a period not to exceed one year. These permits should not exceed 150 kips.

b) Special (or Limited Crossing) Permits.

These are usually valid for a single trip or a very limited number of trips. They may require the use of escorts to restrict other traffic from the bridge being crossed; the crossing along a certain line on the bridge; and/or reduced speed to minimize the dynamic effects. Special Permits are allowed for bridges having a rating factor RF>1.0 for the legal loads or the AASHTO HL-93 design load.

For spans up to 200-ft in length, only the effects of the permit vehicle are considered in a lane. An additional lane load of 0.2kip/ft should be added to the permit load for spans longer than 200-ft or when checking negative moments of continuous spans. The permit live load factor, γ_L, used in conjunction with Eq. (1) varies between 1.10 and 1.85 depending on the permit type; frequency of permit crossings; whether the permit vehicle will be escorted or whether other vehicles may cross the bridge simultaneously; the average daily (ADTT) truck traffic on the bridge; and the gross weight of the permit vehicle. Table 6 shows the suggested LRFR live load factors for permit loads.

For routine permits, the distribution factor for two or more lanes should be used. For special permits, the one-lane distribution factor of the AASHTO LRFD is used after removing the built-in multiple presence factor by diving the tabulated LRFD girder distribution factor by MP=1.2. The dynamic factor specified for the general load rating procedure is used unless the vehicle speed is restricted to less than 10Mph, in which case the dynamic amplification factor is eliminated.

Table 6. LRFR Permit Load Factors γ_L

Permit Type	Frequency	Loading condition	DF[a]	ADTT (one direction)	Load factor by Permit Weight[b]	
					Up to 100 kips	≥150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.80	1.30
				=1000	1.60	1.20
				<100	1.40	1.10
					All weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.50	
				=1000	1.40	
				<100	1.35	
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.85	
				=1000	1.75	
				<100	1.55	

Notes

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

[b] For routine permits between 100 kips and 150 kips interpolate the load factor by weight and ADTT value.

Use of Standard Permit Vehicles in LRFR Evaluations

The use of equivalent load ratings or load effects to check overload permits is widely used to achieve speed and efficiency in permit reviews. For example, NYSDOT compares the permit load effects to the HS-20 load effects to decide on load permits. Such a comparison though simple and easy to apply can be vastly improved to take into considerations the specific conditions that pertain to permit crossings as compared to general traffic conditions. For example, a comparison of the permit load effects to HS-20 effects implicitly assumes that the multiple presence scenarios are the same for all live load crossings. LRFR approach draws a significant distinction between permit load crossings and other service loads with regard to multiple presence assumptions and these are reflected in the live load factors. The LRFR also accounts for the fact that the overload probabilities for permit loads and non-permit loads are different. The LRFR also considers the differences in the live load distribution analysis for permit loads and non-permit loads. Specially calibrated live load factors are provided in LRFR for permits based on different multiple presence scenarios and truck overload probabilities. A better approach that would allow for a comparative analysis for permit evaluations can be adopted by defining a suite of standard permit vehicles and include them in the live loads library for use in LRFR. The standard permit vehicles would represent classes of overweight trucks most frequently used to carry loads requiring annual permits or single trip permits. These standard permit vehicles can be selected by reviewing past permit applications received by NYSDOT and by comparing the load effects induced by the various truck configurations to extract a small number of representative vehicles as New York Standard Permits. For most subsequent permit load investigations, the results of the standard permit vehicles will provide a sound basis for screening the load for bridge safety without the need for a reanalysis. LRFR permit procedures are more comprehensive in that all load effects on a bridge from permit loads, including moment, shear, axial are checked at the strength and service limit states.

2.6 Permit Review Example

Based on the LFR and LRFR methods, a direct approach can be used to check whether permit loads can be allowed over a bridge by verifying that the permit's load effects produce ratings RF>1.0. The direct load rating for permit loads correlates the permit vehicle with the structural capacity of the bridge and does not require the use of the design vehicles.

For example, a Trip Permit applicant with a gross vehicular weight of 220 kip shown in Figure 8 produces a maximum moment effect M_{LL}=2127.9 kip-ft on the 65-ft simple span bridge described in Figure 7 and used in the load rating example. For the 65-ft bridge this permit load produces the following Operating Rating Factor:

LFR Permit Rating

In LFR method, RF= $\dfrac{1.0*2877.8-1.30*568}{1.30*(1.263*1.33*2127.9/2)}=0.92<1.0$

A RF=0.92 indicates that the crossing of this permit truck over the 65-ft bridge described earlier is not safe for general permits but may be allowed for a trip permit.

Alternatively, when the PVLE method is used, the moment effect of the permit which is obtained as 2127.9 kip-ft is compared to the HS-20 moment effect of 895 kip-ft to yield a PVLE=238% or an equivalent 86 tons. Since the PVLE is greater than 150% of the HS20 moment effect, all the bridges on the trip permit route are checked for full speed. Per Table 5, the Multiple Presence Reduction Factor (8 foot/Vehicle width, min. 85) is then applied to the PVLE of 238%. This reduction factor can vary from 1.0 to 0.85 depending on the permit vehicle width. For illustrative purposes it is assumed the reduction factor is equal to 0.9, resulting in a PVLE of 77 Tons. As a result all bridges on the route are checked against a PVLE of 77 tons. The load rating calculations executed earlier for the HS-20 loading produced an operating rating factor RF=2.19 or an equivalent 79 tons for the given 65-ft bridge. With this in mind, crossing of this permit vehicle over the 65-ft bridge described in Figure 7 would be allowed as a Trip Permit. It is noted that per Table 5, this vehicle would not be allowed to obtain a 5 Mile Radius Permit if the vehicle operated within a 5 mile radius of the bridge.

As noted in Table 5, an additional reduction can be applied to the PVLE when a Speed Restriction is specified for any number of bridges along the route. This Speed Restriction reduces the impact factor, which in turn reduces the PVLE, for the permit vehicle.

LRFR Permit Rating

According to the LRFR manual, the load rating for special permits allows the use of the one lane distribution factor after dividing the tabulated value by the multiple presence factor MP=1.2. This will produce a distribution factor for the moment on an interior girder:

$$g_m = g_{m1} = 0.46/1.2 = 0.383$$

Since the inspection revealed that bridge's surface has minor deviations a dynamic amplification factor $IM = 20\%$ can be used. Assuming that the special permit will cross the bridge on a single trip and can mix with other vehicles and given that the ADTT at this site is equal to 1000 trucks per day, a live load factor allowed γ_L=1.40 can be used. Thus, the LRFR permit rating factor is:

RF= $\dfrac{1.0*2877.8-1.25*568}{1.40*(0.383*1.20*2127.9)}=1.58>1.0$

This means that according to the LRFR, this permit vehicle would be allowed to travel unescorted over this bridge on a single trip with a mix of other vehicles.

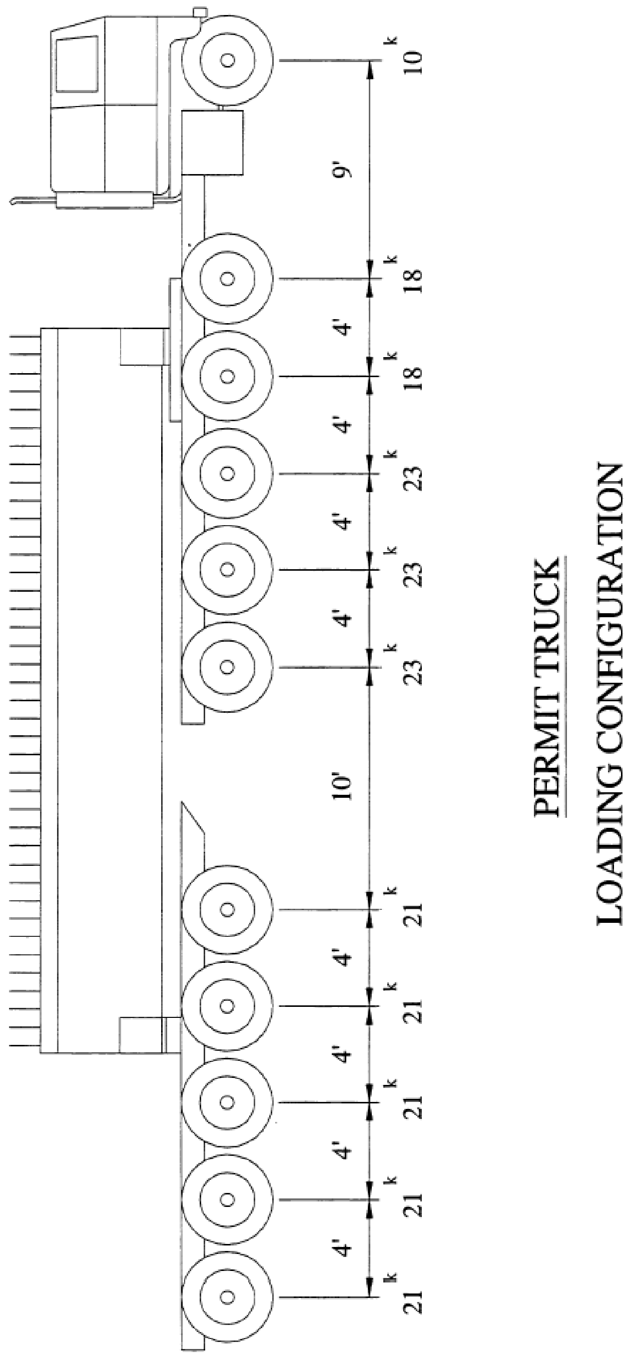


Figure 8. Permit Load Configuration

3. Bridge Load Posting Guidelines

When the computed rating factor, RF, for a bridge component is less than 1.0, the bridge may need to be load posted or closed. According to NYSDOT procedures, decisions on load postings are based on load ratings obtained from the standard AASHTO H and HS vehicles or the Legal Trucks. Both inventory and operating ratings must be calculated. NYSDOT assigns two kinds of postings: a) load posting, and b) R-posting.

3.1 NYSDOT Load-Posting Procedure

Load posting is a vehicle weight restriction placed on a structure by the owner when a bridge does not have enough capacity for carrying the maximum legal live load. According to NYSDOT guidelines, the RF from any type of vehicle can be converted into equivalent H inventory and operating ratings, which in turn are used to determine the Safe Load Capacity (SLC).

The SLC is a load rating value that corresponds to an acceptable stress level from actual traffic loads. Load posting is required if the SLC for a given span is less than the H equivalent rating of a legal load. The maximum Legal Truck load effect will produce different equivalent H rating values depending on the span length. Table 7 adopted from NYSDOT EI 05-034 gives the equivalent H rating for the maximum AASHTO Legal Truck load effect as a function of the effective span length. The effective span length is the length of the live load influence line for the member action (moment or shear) that the member's rating is based on. The effective span length is obtained following the guidelines provided in Table 8.

Determination of H Equivalent Rating of Legal Loads

As shown in Figure 3, there are three types of AASHTO legal loads classified as Type 3, Type 3S2, and Type 3-3. The maximum moment for a bridge for these three types of legal loads will be designated as M_{Legal}. Similarly, the maximum moment for the same bridge under the effect of the AASHTO H-20 truck or lane loading will be obtained as M_{H20}. The H equivalent legal load is equal to $\frac{M_{Legal}}{M_{H20}} W_{H20}$, where W_{H20} =20 tons corresponds to the H-20 truck gross weight.

As an example, given a bridge with a simple span length of 95 feet, the maximum moment under the Type 3, Type 3S2, and Type 3-3 Legal Trucks are respectively 1035, 1242 and 1243 kip-ft. Hence, the maximum Legal truck moment effect M_{Legal} is equal to 1243 kip-ft. The maximum moment for the AASHTO H-20 truck load (W_{H20}=20 tons) is 894 kip-ft, and the maximum moment for the AASHTO lane loading is 1149.5 kip-ft. Hence, the maximum H-20 moment is M_{H20}=1149.5 kip-ft. Therefore, the H equivalent legal load for this span is equal to $\frac{M_{Legal}}{M_{H20}} W_{H20} = 21.6$. Indicating that the equivalent H rating of the Legal trucks for this span is H21.6. The slightly higher H23 value shown in Table 7 reflects slight variations in the effects of legal trucks that comply with the NYSDOT legal load limits but are slightly different than any of the 3 AASHTO Legal trucks.

If a bridge's H Safe Load Capacity is lower than the values in Table 7, Load posting of the bridge may be necessary. As an example, the H Safe Load Capacity of the 65-ft bridge analyzed above yielded an operating rating equivalent to H-62. This rating is way above the value of H-25 of Table 7 for the span indicating that posting is not required. For additional safety, the H Safe Load Capacity will be further reduced under certain conditions as will be explained in the next section.

Table 7. H-loading Equivalent to Legal Loads.

Effective Span Length-ft	H Equivalent of Legal Load
Up to 12[*]	H16
13-19[*]	H18
20-34	H22
35-45	H23
46-53	H24
54-75	H25
76-90	H24
91-105	H23
106-120	H22
121-140	H21
Over 140	H20

* Generally applies to stringers and floorbeams only

Table 8. Effective Span Length for Different Bridge Configurations

Member Type		Effective Span
Simple span stringers or girders		Span length
Continuous stringers or girders	Positive moment and shear	Span length
	Negative moment	Average of adjacent span lengths
Floorbeams	End floorbeam	Adjacent stringer span or panel length
	Intermediate floorbeam	Sum of two adjacent stringer spans or panel lengths
Trusses	Chords and end posts	Total span length
	Interior diagonals	Panel length+sum of panel lengths to far support
	Vertical hangers or posts	Same as intermediate floorbeam
	Vertical part of truss web	Same as interior diagonals

Determination of Safe Load Capacity (SLC) and Posting Values

According to NYSDOT procedures, the calculated H Operating Rating (HOR) of the controlling member, whether determined using Allowable Stress (ASR) or Load Factor (LFR) rating methods should be modified to produce the Safe Load Capacity (SLC) based on the field inspection report, member type, bridge redundancy, and the type of structure to which the rated member belongs. The NYSDOT modifications are provided in Table 9. In a sense, these NYSDOT modifications have similar objectives to the approach followed in LRFR that consists of applying a component condition factor ϕ_c and a system redundancy factor ϕ_s.

It is noted that operating rating results from both the ASR and LFR methods will be generally comparable for the same bridge. However, inventory ratings for both methods can differ greatly. The H inventory rating (HIN) for the LFR method is directly proportional to the operating rating (HIN=0.6HOR). Whereas, the inventory rating for ASR can fluctuate independently of the operating rating for different bridge types and for bridges with different dead load to live load ratios. Basing the SLC calculations on the inventory rating could significantly penalize a bridge with a low inventory but high operating rating.

As provided in Table 9, the SLC may be allowed up to the operating rating for load path redundant members in good condition and floor systems where it can be demonstrated that there is capacity above that calculated by the Load Rating Specification assumptions. The added capacity is normally attributed to excess roadway widths in comparison to the actual number of travel lanes and/or sufficient redundant members. A posting decision can be based on the operating rating if it can be shown that there is at least 125% of equivalent legal load capacity available due to excess roadway width or redundancy.

The SLC may be used directly as the posting value. However, this may be over-conservative for some span lengths, since an H-type vehicle is not a legal weight and has the spacing configuration of a two-axle truck. To account for the different configurations of legal loads, Table 10 may be used to convert the SLC, which is based on the H vehicle, into a posting value. It should be noted that NYS Vehicle and Traffic law requires load posting to be single tonnage values, and Table 10 reflects this requirement

Table 9. Safe Load Capacity (SLC) Determination Guidelines

<table>
<tr><th>Bridge Type & Characteristics</th><th>Primary Member Rating</th><th>SLC</th></tr>
<tr><td rowspan="2">1. Steel primary members that are both internally and load path nonredundant:
• Two and three member welded plate girder bridges or rolled beams that have partial-length welded cover plates or other fatigue category D,E,or E’ details.
• Truss members with pinned eye bars or threaded rods.
• Welded truss members and truss members with welded connections.
• Floorbeams spaced at more than 12’that have timber or steel grating decks.
• Pin and hanger connections.
• Floorbeam hanger connections.
2. All primary members with extensive section loss that significantly affect the load rating of the structure.</td><td>≤3</td><td>0.60HOR*</td></tr>
<tr><td>≥4</td><td>0.70HOR</td></tr>
<tr><td rowspan="2">3. All load path redundant steel members including welded girders, riveted girders and rolled stringers.
4. Rolled or welded truss members with riveted or bolted connections.
5. Rolled two girder bridges without fatigue category D,E, or E’welds.
6. All internally redundant members (excluding floorbeams described in #1) regardless of load path redundancy including: Riveted truss members; Riveted through or deck main girders.
7. Floor system members:
• All floor system stringers.
• All steel floorbeams with concrete decks regardless of spacing.
• All steel floorbeams spaced 12 or less regardless of deck type.
8. All concrete beam or slab members.</td><td>≤3</td><td>0.80HOR</td></tr>
<tr><td>≥4</td><td>0.85HOR</td></tr>
<tr><td>9. Load path redundant members and floor system members where it can be demonstrated that there is capacity above computed by the normal load rating assumptions. This added capacity may be demonstrated by a greater roadway width than is required by the actual number of traffic lanes and also, excess redundant members.
10. Box or H shaped compression chords of trusses with adequate lateral support and no signs of lateral movement.</td><td colspan="2">Up to HOR</td></tr>
</table>

* Decimal values resulting from these guidelines should be truncated to the nearest ton. For instance, a calculated result of 12.71 tons should be truncated to 12 tons.

Table 10. Maximum Posting Values in Tons (Based on H type truck)

	SLC														
Eff. span (feet)	3-9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
<=12ft		12	15	16	18	20	22									
13-19		10	12	14	15	16	18	20	22							
20-34		10	12	12	14	15	16	16	18	18	20	22	22			
35-45		10	11	12	13	14	15	16	18	18	20	20	22	22		
46-53		10	11	12	13	14	15	16	17	18	19	20	22	22	24	
54-64	No Change	10	11	12	13	14	15	16	17	18	19	20	22	22	24	25
65-75		10	12	12	14	15	16	16	18	20	20	22	22	24	25	25
76-90		10	12	14	15	16	18	18	20	20	22	24	25	25	25	
91-105		10	12	15	16	16	18	20	22	22	24	25	25	28		
106-120		12	14	15	18	18	20	22	22	25	25	28	28			
121-140		12	16	18	20	20	22	25	25	28	28	30				
Over 140		12	16	18	20	20	22	25	25	28	30					

3.2 NYSDOT R-Posting Procedure

R-posting is a load restriction for a bridge, which does not have the reserve capacity to accommodate most vehicles over legal loads, but can still safely carry legal loads. R-posting requirements were originally established under the NYS divisible load permit law to restrict certain classes of divisible load permit vehicles from bridges with low overload capacities. NYSDOT criteria used to determine R-posting, which are different for upstate versus downstate bridges, are listed as follows:

a. Low Operating Ratings

For Upstate bridges an operating rating threshold equal to H-29 is used. For Downstate bridges the cutoff operating rating is H-33. For example, the 65-ft bridge analyzed above yielded an operating rating H-62. Therefore, this bridge does not need to be R-posted.

Downstate bridges are those located in Region 8 (including Dutchess, Putnam, Orange, Rockland and Westchester Counties) and Region 10 (including Nassau and Suffolk Counties). NYSDOT does not have permitting responsibilities in New York City and therefore New York City bridges are not included.

The H-29 and H-33 were developed using multipresence reduction factors, which are based on the unlikelihood of two overload permit vehicles being situated at the most critical location of a bridge simultaneously. In a sense, this implicit approach to account for multi-presence is similar to the explicit approach adopted in LRFR.

b. Design Load Below H-20

If the bridge's design load is below H20 and no level 1 or level 2 load ratings are available, then R-posting may be required.

c. Bridge Width

If the bridge width (curb-to-curb) is below 24 feet Upstate or below 28 feet Downstate, then the bridge may need to be R-posted. This is due to the fact that the bridge was likely only designed for a single land, and would not have additional reserve capacity. However, if a level 1 or level 2 load ratings exist, the rating should be used to determine the overload capacity of the structure and the width criterion may not be considered.

d. Primary Bridge Member Condition

If a bridge's primary member condition rating is below 4, which indicates serious deterioration, the bridge should be reviewed for possible R-posted.

e. Deck Condition

If the bridge has a structural deck with condition rating of 1, the bridge would likely need to be R-posted.

f. Regional Prerogative

Regional prerogative may be used where circumstances warrant restricting overload vehicles from crossing a structure for reasons other than those listed above.

3.3 LRFR Load Posting Procedure

According to the LRFR procedure, when the maximum legal load under State law exceeds the safe load capacity of a bridge, restrictive load posting may be required, using either a single weight-limit sign or a three-vehicle combination sign. The three AASHTO legal loads used in the load rating are also appropriate for posting purposes.

The aim of the LRFR methodology is to maintain target uniform reliabilities, even for bridges subject to load posting. In a reliability-based evaluation the relationship between posting values and rating factors is not proportional. The LRFR Manual provides guidance to the users on how to translate rating factors less than 1.0 into posting values that maintain the criteria of uniform reliability as discussed below. This is easily achieved through a posting equation / graph given in the LRFR Manual that presents posting weights for different vehicle types as a function of LRFR rating factors.

Load posting is needed when the rating factor (RF) calculated for each legal truck (AASHTO vehicle) is less than 1.0. When for any legal truck the RF is between 0.3 and 1.0, then the following equation should be used to establish the safe posting load for that vehicle type:

$$\text{Safe Posting Load} = \frac{W}{0.7}[(RF) - 0.3] \qquad (9)$$

where:

RF = legal load rating factor
W = weight of rating vehicle

When the RF for any vehicle type falls below 0.3, then that vehicle type should not be allowed on the span. When RF falls below 0.3 for all three AASHTO legal trucks, then the span should be considered for closure. If the safe posting load is less than 3 Tons then the bridge shall be closed to all traffic.

When the RF is governed by the lane load shown in Figure 9, then the value of W in Equation 8 shall be taken as 40 Tons. When States use their own legal loads which are different from the AASHTO legal loads, Equation 8 may be used for load posting, but the gross weight of the State's legal vehicle shall be substituted for W in the posting equation.

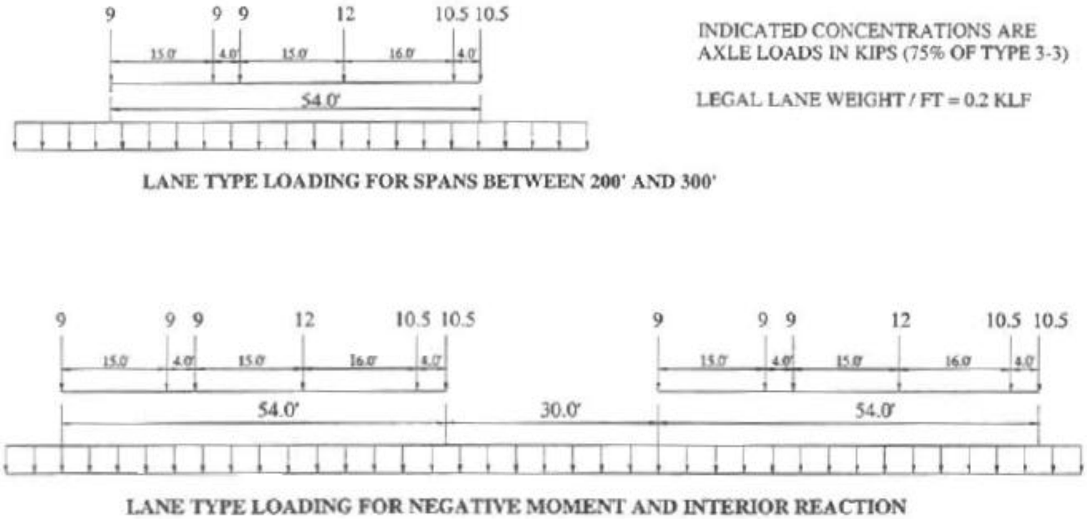


Figure 9. Lane type legal load models

Unlike LFR, LRFR provides only one safe load capacity for legal loads. One important difference between LFR and LRFR is that LRFR considers the truck traffic volume at a bridge site in the load rating analysis. A second difference between the LRFR ratings and the LFR ratings is the inclusion of condition and system resistance factors. These factors address two important concerns in rating older bridges. The condition factor represents the change in member resistance variability, which increases with deterioration, and the system factor has been calibrated to address system failure (instead of member failure). In the LFR method site traffic exposure, bridge redundancy and condition rating were considered when setting a posting weight limit but were not included in the load rating process. In LFR, the bridge owners use State specific procedures to pick a posting level between Inventory and Operating levels. NYSDOT load posting procedures given in Table 9 employ a similar process for setting a posting load. Thus, LRFR provides more consistent assessments of the safe load capacity for posting through a more systematic and scientific consideration of these important variables that have a significant effect on bridge safety. The NYSDOT posting guidelines can be easily reformulated to fit within the LRFR methodology as many of the considerations for posting are already a part of the LRFR ratings.

4. Implementation of Weigh-In-Motion (WIM) Data in LRFR

4.1 Adjustment of Live Load Factors Based on WIM Data

The main purpose of the LRFR code is to account in a rational manner for the uncertainties associated with determining the load carrying capacity of new and existing bridges as well as the uncertainties associated with estimating the loads to be applied. The LRFR specifications were calibrated to provide uniform reliability levels represented by a target reliability index β=3.5 for inventory rating and β=2.5 for operating rating. The former target value was selected in order for the LRFR to remain consistent with the LRFD specifications, while the latter value is equal to the upper range of reliability indices for a sample of bridges that satisfy the Allowable Stress Operating ratings. The target reliability levels were obtained from the reliability analysis of a set of typical bridge configurations assuming that the live loads expected on these bridges are similar to those collected in the mid and late 1970's at a site in Ontario Canada. The generic live load model resulting from the Ontario data is assumed to be valid for all U.S. sites.

Although the calibration of the LRFR followed rational and technically sound methods, some bridge agencies have voiced concerns that certain LRFR procedures and load factors calibrated for national use may not be entirely compatible with their particular procedures and operational needs. Research studies conducted by Lichtenstein (LCE; 2001) have shown that some of the differences between the load ratings obtained from LRFR and those from traditional procedures are due to the fact that the LRFR design load rating is based on the heavier AASHTO LRFD HL-93 design loading rather than the AASHTO standard HS-20 or H-20 trucks which are the most widely used criteria for current traditional ratings. The same study also showed that in many cases, the adoption of the LRFD distribution factor formulas contributed significantly to the observed differences in the rating factors between the LRFR and LFR. This was especially the case with shear ratings of thin web concrete members and ratings of certain exterior girders. However, the research studies have also found that in some instances and even after accounting for the effects of the different design truck weights and configurations or when rating based on the AASHTO Legal trucks, the LRFR procedures may still lead to more conservative operating ratings than the traditional ASR or LFR. This is attributed to the fact that the reliability index value of β=2.5 used for calibrating the operating load rating in LRFR is generally higher than the reliability index value implicit in traditional procedures.

A comparative study of LRFR and LFR ratings was performed by Mertz (2005) under NCHRP Project 20-07 (Task 122) using 74 example bridges obtained from NYSDOT and WYDOT. Only flexural strength ratings were made using the BRASS™ programs. The results of this investigation showed that, in general, LRFR rating factors are equal to or greater than LFR ratings factors except for reinforced-concrete slab bridges. This limited study also suggests that LRFR is technically sound with the LRFR rating factors in good correlation with the reliability index. In other words, LRFR rating factors lower than one demonstrated relatively high probability of failure or low reliability index values. On the other hand, LFR ratings did not correlate as well with the probability of failure. In fact, many bridges with LFR rating factors above one demonstrated unacceptably high probability of failure values.

The current HL-93 load model and the calibration of the AASHTO LRFD Specifications are based on the heaviest 20% of trucks, which were measured in 1975 from a single site in Ontario over a two-week period. At that time, it was thought that the Canadian traffic was sufficiently representative of US traffic. These measured truck configurations and weights consisted primarily of five-axle semi trailer trucks. To ensure consistency with the LRFD Specifications, the AASHTO LRFR Manual was calibrated using the same (1975) Ontario truck weight data (Moses; 2001). Therefore, current AASHTO specified live load models and load factors, that are based on past Canadian traffic data, may not represent modern and future traffic conditions in New York State

Recognizing the limitations of the generic truck weight data used during the calibration process, the LRFR specifications provide sufficient flexibility and allow state agencies to adjust the LRFR load factors based on their individual conditions and site-specific or state-specific information. Hence, more refined load factors appropriate for a specific state or site may be estimated if more detailed traffic and load data are available for the state or bridge site. Specifically, truck load data collected through Weigh-In-Motion (WIM) measurements recorded over a period of one year period can be used to obtain state-specific or site-specific live load factors γ_L which would be taken as:

For two or more lanes:
$$\gamma_L = 1.80\left[\frac{2W^* + t_{ADTT}\,1.41\sigma^*}{240}\right] > 1.30 \qquad (10)$$

For one lane:
$$\gamma_L = 1.80\left[\frac{W^* + t_{ADTT}\,\sigma^*}{120}\right] > 1.30$$

where W*=mean truck weight for the top 20% of the weight sample of trucks, σ* =standard deviation of the top 20% of the truck weight sample, t_{ADTT}=fractile value for the maximum expected loading event given as shown in Table 12.

Table 12. t_{ADTT} Fractile Value for the Maximum Expected Loading

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

Fortunately, NYSDOT has established several Weigh-In-Motion (WIM) sites that are collecting continuous long-term data on truck weights that can be used to adjust the live load factors using the LRFR proposed approach.

4.2 Reliability-Based Recalibration of LRFR

While the LRFR provides specific instructions as to how to adjust the live load factors based on state-specific or site-specific WIM data, this adjustment is based on the same statistical analysis procedure originally used in the development of the specifications. By using the same statistical analysis, the target reliability index values used in the original development of the LRFR are maintained and the new live load factor will only reflect the differences in the truck weight data.

By maintaining the same reliability levels as those used in the development of the current LRFR, it is assumed that the target reliability index values used to calibrate the LRFR are satisfactory and are consistent with current procedures. This may not be always true given that the reliability index target for the LRFR operating rating calibration was extracted from the upper range of the reliability indices for a sample of bridges that satisfy the Allowable Stress operating ratings exactly with RF=1.0. Therefore, to ensure the compatibility of any future NYSDOT LRFR with current NYSDOT load rating procedures, it would be necessary to study the reliability levels implicit in current NYSDOT procedures and use these as the target values that the NYSDOT LRFR should match. The reliability analysis follows a clear set of steps as outlined by Moses (2001); Nowak (1999); Ghosn and Moses (1986).

A critical input to the reliability analysis is the statistical representation of the live load expected to cross over New York State highway bridges. A current study under investigation for NCHRP12-76 by the same research team (Sivakumar, Ghosn & Moses; 2006) is proposing a methodology and a set of protocols for using available WIM data for developing live load models for bridge design and evaluation. The protocols include statistical projection methods to obtain the maximum expected live load effect for different return periods. These methods are particularly applicable for use in determining the live load models necessary for calibrating new LRFR factors and adjusting the load rating equations to represent the live loads observed on New York State bridges. As an example, Figure 10 shows the cumulative distribution histogram for the maximum moment on a 60-ft simple span bridge obtained from the WIM data collected at NY State Site 9121 located on I-81 NB.

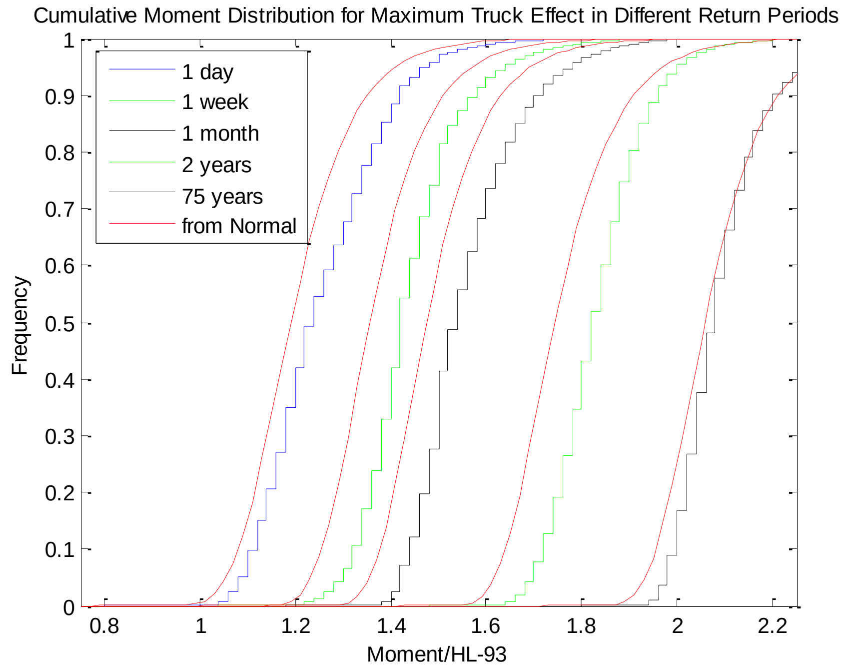


Figure 10. Cumulative distribution of maximum load effect of side-by-side trucks for different return periods (New York I-81 site)

Figure 10 gives a plot of the cumulative distribution function for different return periods for the maximum live load moment effect expected from two side-by-side trucks. Two methods were used in the calculations. The first method applies a simulation to add the load effects of trucks in each traffic lane based on the number of side-by-side events and assembles these into a cumulative histogram for each loading event and then projects this cumulative histogram for the different return periods. These results are shown in the step-type curves of Figure 10. Another approach assumes that the tail end of the load effect histogram approaches that of a Normal distribution function and the projection of the results for different projection periods is then obtained from a closed form expression of the extreme value distribution function. These results are depicted in the continuous red curves in Figure 10. The results demonstrate that the two approaches yield essentially similar results.

The results of Figure 10 show that the maximum expected moment effect on the 60-ft simple span bridge will be equal to 1.44 times the expected moment effect of the HL-93 load for a one-week return period. For a two-year return period the expected moment effect is 1.80 times the HL-93 moment effect, while for a 75-year period the expected moment is 2.08 times the HL-93 effect. The standard deviation in the estimated maximum moment is equal to 0.10 leading to a COV for the 75-year return period of 4.8% (=0.10/2.08). These values will be used as input for the reliability analysis of typical bridge configurations that are assumed to have truck traffic characteristics similar to those of the I-81 sites. Similar calculations can be easily applied to any set of WIM data collected for a sample of representative NY State sites. It is interesting to note that the values obtained from this I-81 site produce much higher expected moments than those obtained by Nowak (1999) for the generic data subsequently used in the calibration of the LRFR (Moses; 2001). Review of more statewide data from several WIM sites will need to be performed to obtain representative statewide traffic statistics. Such differences however emphasize the need for a review of the LRFR calibration to ensure consistency with currently observed live loads in New York State as well as compatibility with the safety levels implied in current NYSDOT rating and bridge evaluation procedures rather than relying on the previously used generic data or the target reliability levels implied in the LRFR.

5. Conclusion

The NYSDOT load rating, load permitting and load posting procedures primarily follow the classical LFR and ASR methods which are based on checking the safety of bridges under the effect of the AASHTO HS-20 or H-20 design vehicles and the AASHTO legal trucks for load rating and load posting and checking the safety of bridges under the effect of permit vehicles for the permit review process. The LRFR follows a similar approach but uses the HL-93 design vehicular loading and also specifies different live load factors than either the LFR or ASR. Additionally, LRFR provides specific load rating procedures for legal loads and permit loads that maintain uniform reliability goals in load ratings, postings and permit review processes (NYSDOT EI 05-034; 2005, AASHTO, 2003).

The NYSDOT approach and the LRFR have several similar special features. An important special feature common to both methods is that they both account for the field inspection reports of member condition rating to modify the criteria for load posting. Also, both the NYSDOT approach and the LRFR allow for the consideration of redundancy before making a final decision on load posting. Although, the NYSDOT procedure considers multiple presence and the reduced chances of simultaneous heavy truck presence on a bridge during the permit review process, the approach taken by LRFR is more direct as it provides an explicit approach for taking the probability of multiple presence into consideration by changing the live load factors based on the type of permit issued and the average daily truck traffic at the bridge site. Furthermore, the LRFR provides an approach to adjust the live load factors based on truck weight statistics and truck volume collected using unbiased Weigh-In-Motion systems. The adjustments of the live load factors can be based on site-specific WIM data or statewide data. These redundancy and member condition factors are currently left as optional in the AASHTO LRFR Manual with the recommendation that they should be used in accordance with the load rating practices for the State. There is need to study the appropriate use of these factors for NYSDOT bridge load ratings in order to remain consistent with current NYSDOT procedures. (Lichtenstein, 2001; NYSDOT EI 05-034; 2005)

The examples presented in this report and previous studies have shown that the LRFR may yield different load rating results than the traditional LFR or ASR methods. This is explained by the fact that the calibration of the LRFR live load factors followed a conservative approach so that bridges designed using the LRFD method would yield rating factors RF=1.0 for inventory rating. In fact, Nowak (1999) has shown that on the average the LFD yielded a reliability index β=3.5 for a typical set of bridge configurations and he used that value as the target for the calibration of the LRFD code. However, the LRFD calibration study performed by Nowak (1999) has shown that to match the target reliability index β=3.5, it would be sufficient to use a live load factor γ_L lower than the γ_L=1.75 adopted in the LRFD specifications. Thus, generally speaking the LRFD is on the average more conservative than the LFD and consequently the Inventory Ratings of LRFR would be more conservative than those of the LFR. Also, the reliability index target value used during the calibration of the LRFR live load factors for operating rating was actually equal to the upper range of reliability indices for a sample of bridges that satisfy the Allowable Stress Operating ratings. An additional source for the observed differences is the fact that the LRFR philosophy is based on providing uniform reliability levels for a typical range of multi-girder bridge span lengths and configurations while previous studies have shown that the reliability levels from the LFR and ASR methods are not necessarily uniform. (Nowak, 1999; Moses; 2001; Lichtenstein, 2001)

Historically, the NYSDOT procedures have performed very well by providing the travelling public with a safe and economically sustainable highway bridge network. Therefore, it is important that the future adoption of LRFR criteria in New York State leads to maintaining similar levels of safety consistently for all bridge types and configurations. Since the calibration of the current LRFR live load factors was based on truck traffic data collected over 30 years ago in Ontario Canada which may not be representative of the truck weights and traffic conditions currently encountered in New York State, and since the objective of the LRFR calibration was to conservatively match the safety levels corresponding to the upper ranges of the safety levels of ASR rated bridges, it is then necessary to review the LRFR live load calibration procedure and adjust the current LRFR live load factors so that New York State bridges evaluated using an adjusted LRFR will yield acceptable levels of structural safety based on the truck traffic currently observed or projected for the State of New York (Moses; 2001, Ghosn and Moses; 1986, Nowak; 1999).

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APPENDIX III

REVIEW OF NATIONAL PRACTICE FOR LOAD RATING, LOAD PERMITS AND POSTING OF BRIDGES

Review of National Practice for Load Rating, Load Permits and Posting of Bridges

Bridge load rating, load permitting, and bridge posting processes consist of: a) determining the safe load carrying capacity of highway bridges; b) determining if specific legal or overweight vehicles can safely cross rated bridges within a state or a region or bridges along a specific route; and c) determining if a bridge, which is unable to carry general truck traffic, needs to be restricted from use by heavy vehicles and the level of truck weight limits that should be imposed. Decisions on load rating, permitting and bridge posting are made in conjunction with the bridge inspection process and form the important tasks that bridge owners undertake to ensure the safety of the bridge infrastructure system and its users.

There currently are three rating methods in use in the U.S. These methods are: a) Load Factor Rating (LFR); b) Allowable Stress Rating (ASR); and c) Load and Resistance Factor Rating (LRFR). Each of the methods consists of two levels of ratings which are known as Operating Rating (OR) and Inventory Rating (IR). The LFR method which has been the most favored by the NYSDOT for load rating, over-load permitting and bridge posting is known to be similar to that practiced by most states. In keeping with the Federal Highway Administration (FHWA) recommendation, the LRFR method is often used for rating bridges designed with the Load and Resistance Factor Design (LRFD) method. Differences between different states may be due to the use of different legal truck configurations and how the methods are applied. Although the LFR method is currently most widely used, some states still use ASR with its distinct safety factors and approach for evaluating member capacities for final decisions on load capacity evaluation or bridge posting. It has been observed that for operating level ratings, the differences between the LFR and ASR ratings are minimal and are not usually expected to lead to large variations.

The implementation of the newly adopted AASHTO Load and Resistance Factor Rating (LRFR) procedures in the U.S. is still in its early stages but a growing number of states have been making progress towards its implementation. Currently, all new and existing bridges are allowed by the Federal Highway Administration (FHWA) to be load rated by the LRFR methodology. But, more importantly, a FHWA memorandum dated October 30, 2006 requires that the ratings of all bridges and replacement bridges designed by the Load and Resistance Factor Design (LRFD) Specifications after October 1, 2010 be computed and reported to the National Bridge Inventory (NBI) system using Rating Factors based on the LRFR method (FHWA Memorandum; 2006).

This report reviews the current state of practice in the US related to load rating, permit issuance and bridge load posting. The report is divided into three sections addressing 1) Load rating practices; 2) Load posting practices and 3) Over-load permitting. Following the general review of current methodologies, more specific information is provided in each section for a few states. The states that appear to be making the most progress toward implementing the LRFR procedures are Florida, Oregon, Hawaii, Michigan and Wisconsin.

1. Load Rating

1.1 Introduction

The current AASHTO Manual for Condition Evaluation of Bridges (MCEB) provides the most widely used guidelines for bridge load rating methodology. MCEB recommendations are based on the classical ASR and LFR load rating methods. Normally, load rating is based on the AASHTO standard HS-20 (figure 1) or H-20 vehicle loads (figure 2) or the standard AASHTO legal loads (figure 3). Although most bridge agencies follow AASHTO guidelines, several agencies have developed their own legal load configurations to better reflect the actual legal load limits within their jurisdictions. These may be different than the Federal limits due to grandfather clause exemptions. The differences in the legal truck configurations along with the use of different rating methods and differences in implementing the MCEB specifications have led to some lack of uniformity in bridge rating processes. [Fu and Fu; 2006]

Recently and in response to the required shift to the Load and Resistance Factor Design (LRFD) bridge design specifications, AASHTO has approved the adoption of the Load and Resistance Factor Rating (LRFR) guide specifications as presented in the AASHTO Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges (2003). The LRFR specifications were calibrated based on structural reliability concepts to meet uniform levels of structural safety as expressed in terms of the reliability index, β. The current version of the AASHTO LRFR was calibrated so that bridges designed using the LRFD will produce Inventory Rating Factors (R.F.) equal to 1.0 when using the LRFD HL-93 design live load (figure 4). Although the original intent of the LRFD calibration was to provide a reliability index β=3.50, subsequent conservative adjustments to the LRFD load factors have led to increasing the average reliability index when using the HL-93 load model to a value close to β=3.9 [Nowak, 1999, Kulicki et al 2007]. For the HL-93 and for the AASHTO Legal Loads, the LRFR was calibrated to produce a reliability index β=2.5 for Operating Rating [Moses, 2001]. These target LRFR reliability levels were selected to be close to those obtained for the MCEB rating results for a range of typical bridge configurations. A limited number of studies comparing the ratings obtained from different methods have led some bridge agencies to express their concern that in certain cases, the LRFR produces more conservative results than the MCEB leading to what is believed to be unnecessary bridge replacements or postings. For example, the rating of five prestressed concrete girder bridges in New Mexico, showed that the LRFR method generally yielded lower ratings for flexure than the LFR with the longer spans showing the larger deviations. The LRFR shear strength ratings also showed generally lower values than LFR. The discrepancies were found to be primarily due to the different live load models and the differences in the shear resistance models used by LFR and LRFR [Brandy & Jauregui; 2005]. On the other hand, Mertz (2005) observed that the LRFR results are better correlated to the reliability index, β, and consequently to the probability of failure than the LFR. He noted that in certain cases, bridges with high LFR ratings were still associated with high probabilities of failure, while the correlation between LRFR ratings and the reliability index were much more consistent.

The following parts of this section provide a comparison between currently used load rating procedures as seen in Section 1.2. Section 1.3 reviews the current state of practice pertaining to the load rating of highway bridges. The information on the National practice was primarily assembled from the results of two surveys of state agencies conducted as part of NCHRP Synthesis Report 359 [Fu and Fu; 2006] and NCHRP Report 575 [Sivakumar et al; 2007]. Subsequently, Section 1.4 presents a summary of recommendations and progress made by several state agencies in implementing LRFR methodologies.

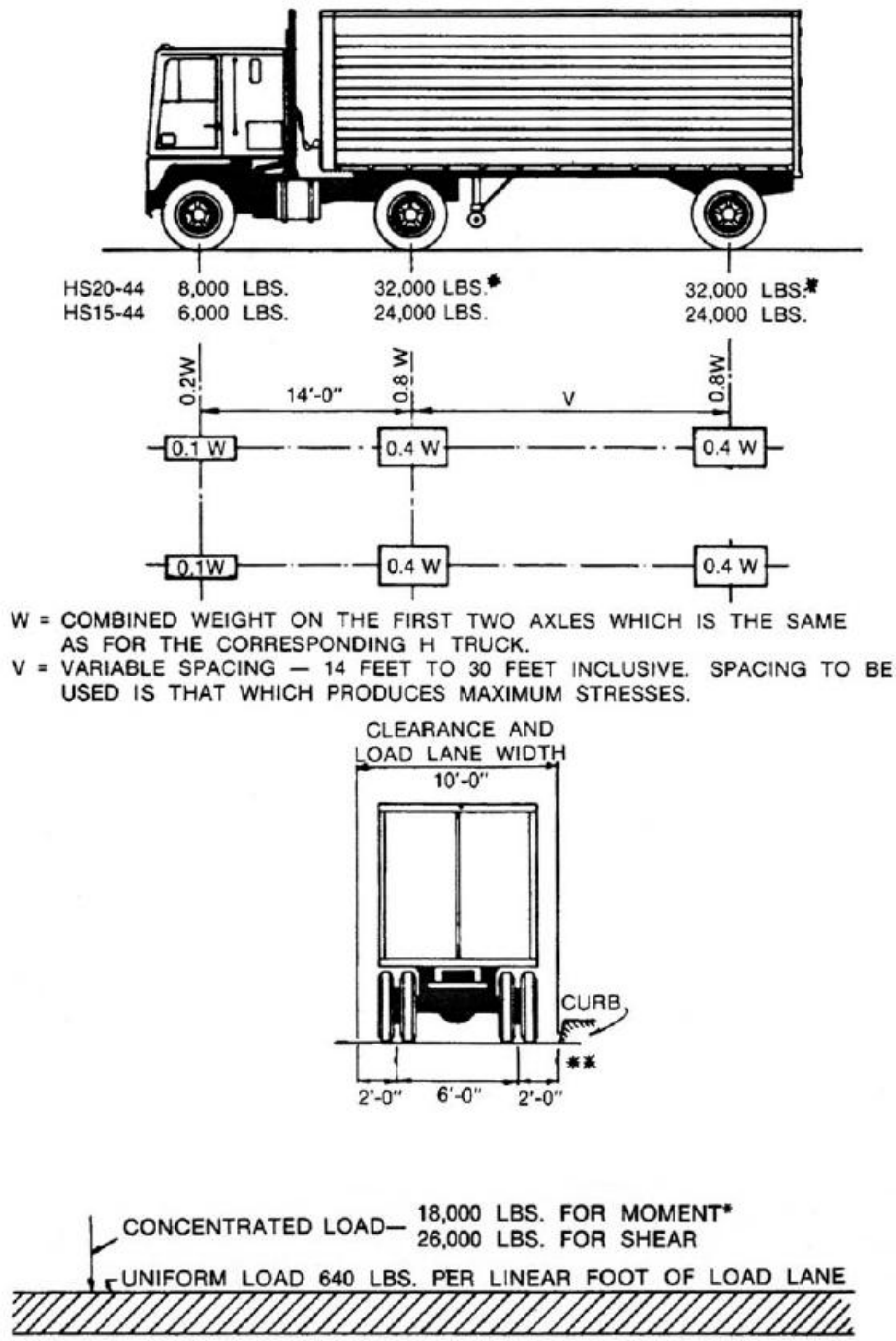


Figure 1. AASHTO Standard HS design trucks

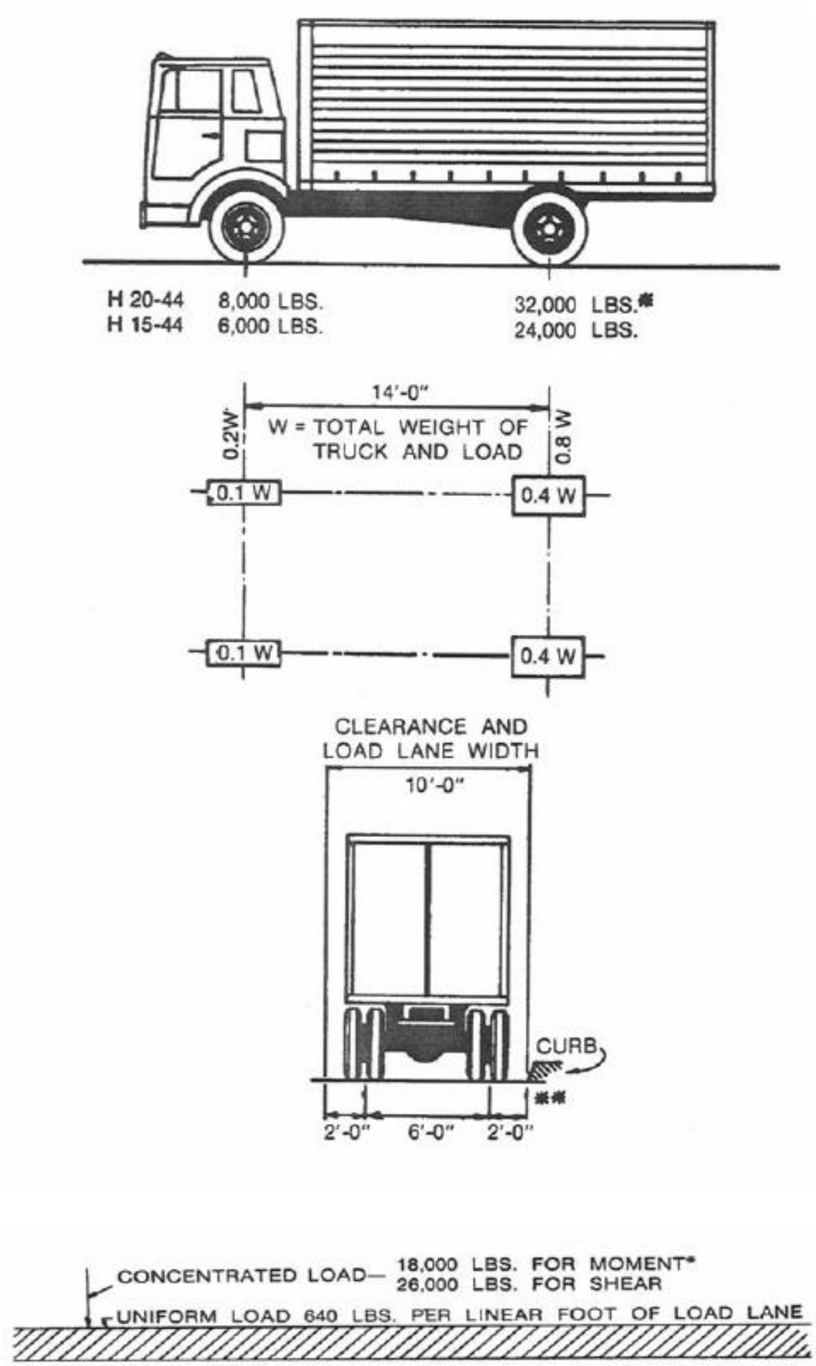
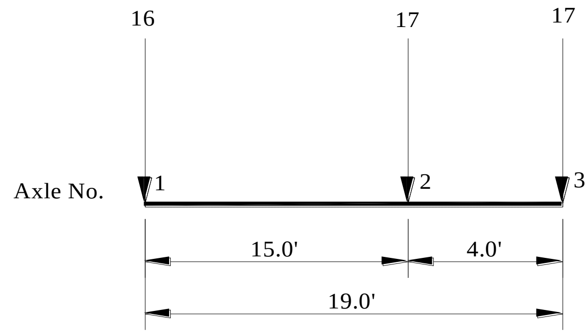
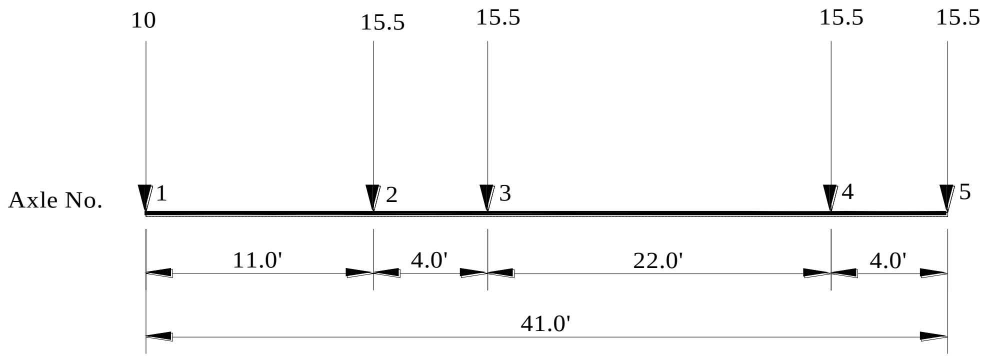


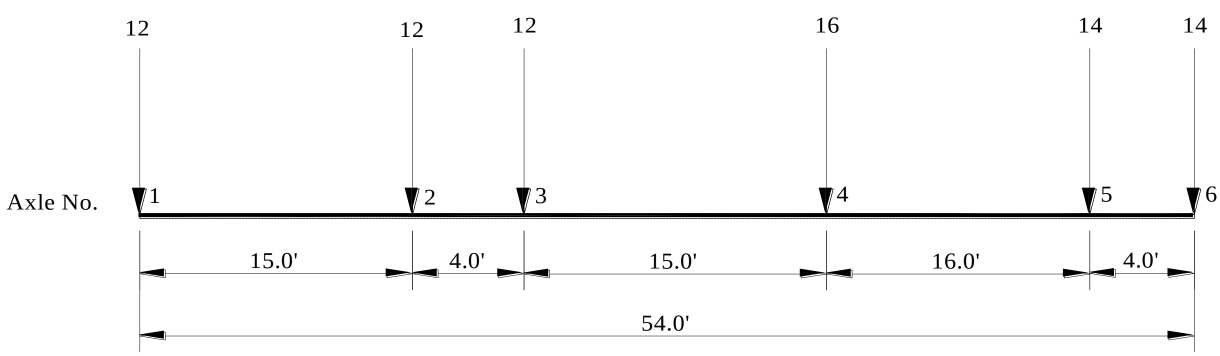
Figure 2 AASHTO Standard H design trucks



Type 3 unit weight=50 kips (25 tons)

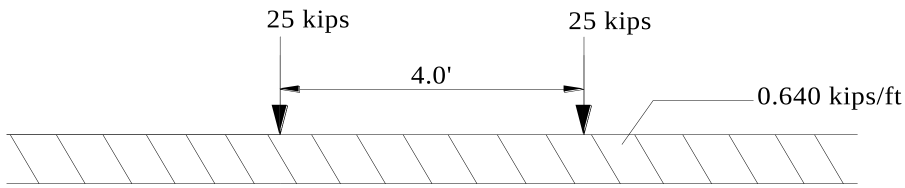


Type 3S2 unit weight=72 kips (36 tons)

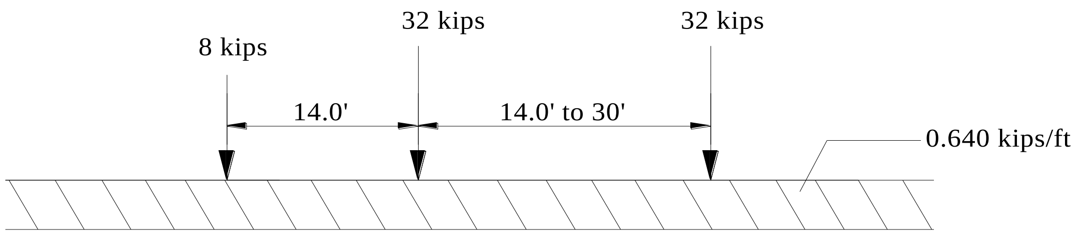


Type 3-3 unit weight=80 kips (40 tons)

Figure 3. AASHTO Legal Truck Configurations

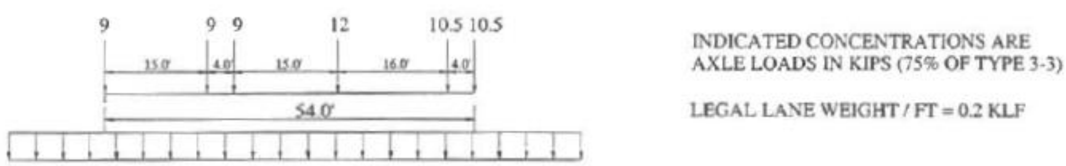


HL-93 Tandem and Lane Load

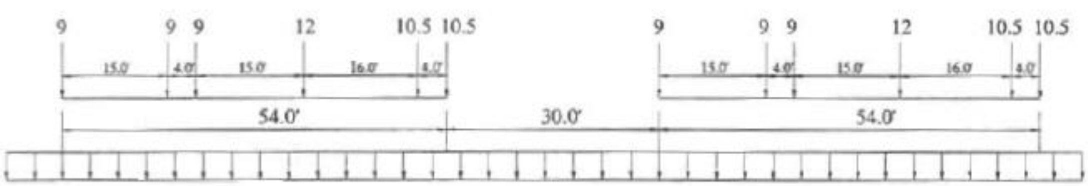


HL-93 Truck and Lane Load

Figure 4. HL-93 Tandem and Truck plus Lane Loads



LANE TYPE LOADING FOR SPANS BETWEEN 200' AND 300'



LANE TYPE LOADING FOR NEGATIVE MOMENT AND INTERIOR REACTION

Figure 5. Lane type legal load models

1.2 Load Rating Procedures

In general, a structural unit's load rating is expressed in terms of a Rating Factor, R.F., which is obtained from the rating equation expressed as:

$$R.F. = \frac{\phi R_n - \gamma_D D_n}{\gamma_L L_n} \qquad (1)$$

ϕ is the resistance factor;

R_n is the nominal unit's resistance;

γ_D is the dead load factor;

D_n is the nominal dead load effect on the unit;

γ_L is the live load factor.

L_n is the nominal live load effect on the unit including the dynamic amplification

When the rating factor, RF obtained from Eq. (1) is greater than or equal to 1.0 for the effects of a standard vehicular live load, the bridge is said to be able to carry that standard load. The results are sometimes reported in terms of the design vehicle weight designation times RF for example H25 implies that the rating factor calculated for the H20 vehicle is RF=1.25. The results of the load rating can also be reported in Tons whereby the tonnage is obtained by taking the weight of the standard vehicular load used to obtain the live load effect L_n multiplied by RF.

The Load rating process is usually executed at two levels: 1) Inventory Rating and 2) Operating Rating. Inventory Rating is defined as the live load which can safely use the structure over an extended period of time corresponding to the design life of the bridge. Various interpretations have been provided for the Operating Rating: in LFR and ASR, the Operating Rating corresponds to the maximum one-time load that can be applied on the bridge. On the other hand, the LRFR Operating Rating live load factor was calibrated based on the maximum load level that the bridge is expected to carry over a five-year period [Moses, 2001]. A lower return period is used for Operating Rating as compared to the 75-year design period associated with the LRFD and LRFR Inventory ratings along with for a lower reliability index than that of the Inventory Rating to reflect the fact that the bridge will be regularly inspected and there is more confidence in the safety assessment. With either interpretation of the Operating Rating, the same bridge will yield a higher Operating Rating than the Inventory Rating.

The dead and live load factors as well as the resistance factors used in Equation (1) differ depending on which rating method is used. Different live load factors are specified depending on whether Inventory or Operating ratings are sought. As an example, the LFR Specifications require a load factor γ_L=1.3 for Operating rating and γ_L=2.17 for Inventory rating (the dead load factor γ_D being set at 1.3 for both rating levels). When the ASR method is used, the live load and dead load factors are both set at $\gamma_D = \gamma_L$=1.0 and the resistance factor (or overall safety factor) ϕ is set at 0.55 for Inventory and 0.75 for Operating ratings of steel bridge members in bending. For all these cases, the basic design vehicle used as required by FHWA is the AASHTO HS-20 (see Figure 1), even though many states have used the HS-25 for design load which is the HS-20 load increased by a factor of 25% to reflect increases in truck traffic weights over the last few decades. Some states also use the H-20 vehicle (see Figure 2) which has a different configuration than the HS-20 truck and has a total weight of 20 tons as compared to the HS-20 truck's weight of 36 tons.

The LRFR uses the HL-93 design load (Figure 4) as a basis for rating and applies a live load factor γ_L=1.75 for Inventory Rating and γ_L=1.35 for Operating Rating with a dead load factor γ_D=1.25 for the dead weights and γ_D=1.50 for the wearing surface. The rating of the AASHTO legal vehicles of Figure 3 and using figure 5 for longer spans and continuous spans is based on the live load factors of Table 3.a.

The AASHTO legal trucks were developed in the 1970's to provide a representative set of loads that model the effects of trucks that follow the Federal Bridge Formula (FBF) limits on short, medium and long span bridges. But recently, a significant number of Short Hauling Vehicles (SHV) with closely-spaced multiple axles have been increasingly used by the trucking industry. Therefore, a set of SHV legal loads (shown in Figure 6) were recently adopted by AASHTO [Sivakumar et al, 2007]. According to the AASHTO LRFR these SHV's should be rated using the live load factors of Table 3.b

The LRFR allows adjustments to the resistance factor by applying a condition factor, $\phi_{c,}$ that relates to the component condition as provided in the inspection report and a system factor, ϕ_s that relates to the redundancy of the bridge system. Thus, instead of ϕR_n, the final factored resistance to be used in Eq. (1) is given as $C=\phi_s\phi_c\phi\, R_n$. Typical values for ϕ_c and ϕ_s are provided in Tables 1 and 2. These factors are currently left as optional in the AASHTO LRFR Manual with the recommendation that they should be used in accordance with the load rating practices of the State.

In the LRFR, the live load factors for the AASHTO legal vehicles are reduced from a maximum γ_L=1.80 for bridges with Average Daily Truck traffic (ADTT) greater than 5000 down to a γ_L=1.40 for bridges with low truck traffic volumes where the Average Daily Truck traffic (ADTT) is less than 100. Typical values for the live load factor for legal and SHV loads are provided in Table 3.

Table 1. LRFR Condition Factor ϕ_c

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

Table 2. LRFR 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 Eye bar Members	0.90
Three-Girder Bridges with Girder Spacing $\leq$6ft	0.85
Four-Girder Bridges with Girder Spacing $\leq$4ft	0.95
All Other Girder Bridges and Slab Bridges	1.00
Floorbeams with Spacing>12' and Non-Continuous Stringers	0.85
Redundant Stringer Subsystems between Floorbeams	1.00

For evaluating timber bridges, a constant value ϕ_s=1.0 is assigned for flexure and shear.

Table 3.a LRFR Generalized Live-load Factors for Legal Loads, γ_L

Traffic Volume (one direction)	Load Factor
Unknown	1.80
ADTT>5000	1.80
ADTT=1000	1.65
ADTT< 100	1.40

Table 3.b LRFR Generalized Live-load Factors for SHV Loads, γ_L

Traffic Volume (one direction)	Load Factor
Unknown	1.60
ADTT>5000	1.60
ADTT=1000	1.40
ADTT< 100	1.15

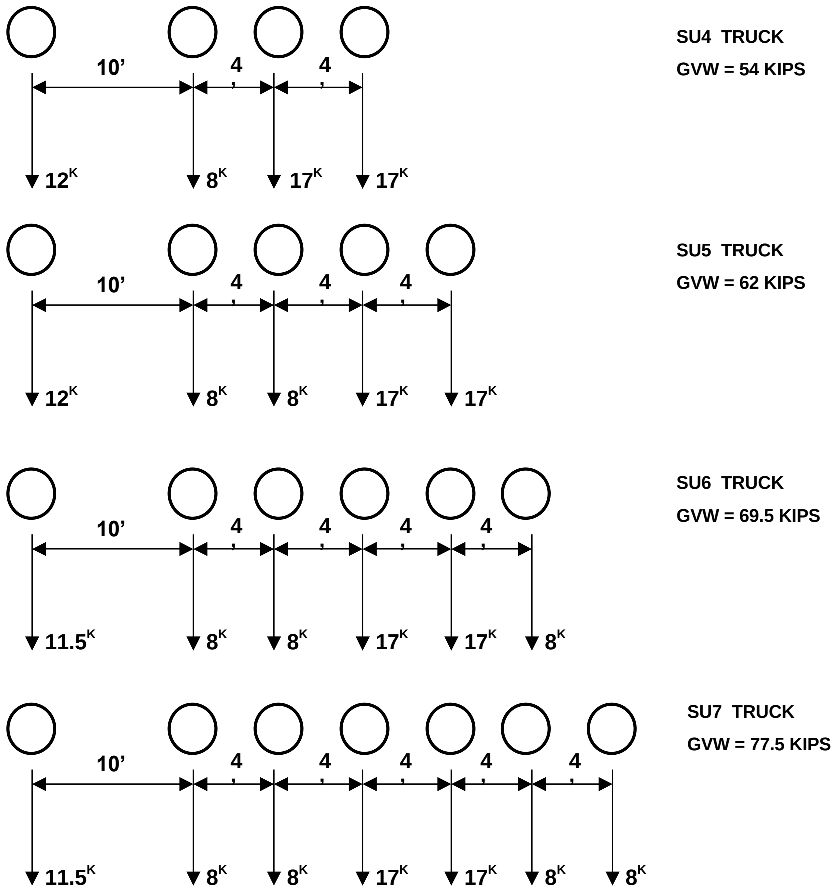


Figure 6. AASHTO Short Hauling Vehicles (SHV)

Additionally, the AASHTO LRFR manual allows for the adjustment of the live load factors based on state specific Weigh-In-Motion (WIM) data that would account for the differences between the truck weight spectra observed in a particular state as compared to the generic set of truck weight data that was used during the calibration of the LRFD and LRFR load factors. Hence, more refined live load factors appropriate for a specific state or site may be used when more detailed traffic and load data are available for the state or bridge site. Specifically, truck load data collected through Weigh-In-Motion (WIM) measurements recorded over a period of one year period can be used to obtain state-specific or site-specific live load factors γ_L which would be taken as:

For two or more lanes:

$$\gamma_L = 1.80\left[\frac{2W^* + t_{ADTT}1.41\sigma^*}{240}\right] > 1.30 \qquad (2)$$

For one lane:

$$\gamma_L = 1.80\left[\frac{W^* + t_{ADTT}\sigma^*}{120}\right] > 1.30$$

where W*=mean truck weight for the top 20% of the weight sample of trucks, σ*=standard deviation of the top 20% of the truck weight sample, t_{ADTT}=fractile value for the maximum expected loading event given as shown in Table 4.

Table 4. t_{ADTT} Fractile Value for the Maximum Expected Loading

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

Currently, the FHWA requires the Inventory Rating and Operating Rating to be reported as Rating Factors (RF) rather than in tonnage using either the Load Factor Rating (LFR) method with HS-20 loading, or Load and Resistance Factor Rating (LRFR) method with HL-93 loading. But, effective the year 2010, ratings of all bridges and replacement bridges designed by the LRFD Specifications are to be computed and reported using LRFR. Even though there is no mandate, FHWA also strongly encourages the rating of all existing bridges by the LRFR methodology. Previously, the LFR method using MS loading (which is the HS metric equivalent) was the FHWA standard for computing Inventory and Operating Ratings. The FHWA uses the Inventory Rating Factors along with other inventory and geometric data reported to the NBI to compute the Sufficiency rating for a bridge. This sufficiency rating is used to help determine if a bridge should be considered structurally deficient and to determine whether the bridge will be eligible for replacement or rehabilitation funding.

1.3 National Load Rating Practices

Rating Methods

Consistently with current FHWA requirements, the most commonly used method for bridge rating is the LFR method. In the 2005 survey conducted by Fu and Fu (2006), 42 out of the 44 responding state agencies reported that they use the LFR method. Twenty three state bridge authorities, including New York State, use both the ASR and LFR. Although Massachusetts reported exclusive use of ASR, most other states use the ASR for special cases such as timber and masonry bridges or steel truss bridges. Such states include North Carolina, Ohio, South Dakota and Tennessee. Connecticut also uses the ASR for mildly reinforced concrete structures. Alabama uses the ASR for serviceability checks when the LFR yields ratings close to 1.0 or if the ASR "yields better results". North Dakota has used the ASR in the past but is slowly converting to the LFR. Kentucky reports the use of the ASR for bridges designed with the Allowable Stress

Design (ASD) method and the LFR for bridges designed using the Load Factor Design (LFD) method.

In the NCHRP 359 survey performed in 2005 (Fu and Fu; 2006) only Pennsylvania reported using the LRFR. Since then, Florida, Wisconsin, Oregon, Hawaii and Michigan have started implementing LRFR procedures or are in the planning phase of LRFR implementation. The PennDOT approach, initiated before the adoption of the LRFR by AASHTO as an optional method for load rating, consisted of modifying the then existing LRFD Specifications to also obtain load ratings. The other five states are adopting modified versions of the AASHTO LRFR specifications as discussed at the end of this section.

The two main reasons given by some states for their reluctance to use the LRFR is a concern expressed by some agencies that in certain cases, the LRFR gives lower ratings than the ASR or LFR methods, and the complexity of the LRFR procedure. For example, North Carolina stated its objection to using the LRFR unless absolutely required [Fu and Fu; 2006].

LFR and LRFR are based on different load rating philosophies with different safety goals so direct comparisons of results are difficult to make. Some of the differences between the rating values of LRFR and LFR can be attributed to the different live load basis (HL-93 versus HS-20), the different load distribution factors used in the LRFR, the different dynamic amplification factors and also due to the different methods for evaluating the strength capacities of some members such as the shear capacity of prestressed concrete members. Also, the LRFR load factors were calibrated with the goal of producing uniform reliability levels expressed in terms of an average reliability index values β=3.5 for Inventory rating or to be consistent with the LRFD designs when using the HL-93 design load for rating. For operating ratings, the LRFR was calibrated to provide a reliability index, β=2.5. However, the work by Nowak (1999) and the subsequent review by Kulicki et al (2007) show that the target reliability index of β=3.5 has been generally exceeded when using the live load factor γ_L=1.75 in conjunction with the HL-93 live load. Also, the reliability index, β=2.5 used to calibrate the LRFR operating rating live load factor of γ_L=1.35 with the HL-93 live load has been found to correspond to the higher end of a range of bridge configurations that produce rating factors RF=1.0 with the LFR Operating Rating criteria. In fact, Moses (2001) reports that earlier studies have shown that the Operating Rating using the MCEB criteria would lead to an average reliability index β in the range of 2.3 for ASR and an average $\beta \approx 2.5$ for LFR when the analysis was performed for the maximum loading in a bridge's design life [Moses and Verma, 1987].

The pre-set target reliability β=3.5 was selected as the calibration criterion for the LRFD (and consequently the Inventory level LRFR) because bridges designed to satisfy the LFD design criteria were found to produce an average β=3.5. However, in LFD large deviations from this average value have been observed for different bridge span lengths. The fact that the LRFD produced uniformly an average β value higher than 3.5 indicates that LRFR Inventory Ratings may be on the average higher than those of the LFR although some bridges may produce lower LRFR ratings.

The reliability index would provide a common basis for comparison since it is difficult to make a direct comparison between the LFR and LRFR inventory ratings given the differences in the HL-93 and HS-20 vehicles as well as the differences in the dynamic amplification and the load distribution factors. Nevertheless, Mertz (2005) states: "An LRFR design-load rating factor of 0.63 could be simplistically considered equivalent to an LFR rating factor of 1.00 for longer span bridges". His statement was based on a very rough comparison of LRFR and LFR vehicle gross weights performed using a typical truck configuration with a gross weight of 57 tons which is found to produce a similar load effect as the HL-93 load. It should be noted that such comparisons are only provided for information purposes and that a direct one on one comparison should not be made between the LFR and LRFR tonnage weights because of the above stated differences.

Analyzing a set of typical New York State and Wyoming bridges, Mertz (2005) found that the ratio of LRFR design load (HL-93) Inventory Rating over LFR design load (HS-20) Inventory Rating is on the average equal to 1.07. Large variations were observed depending on the bridge type and within each bridge type category. For example, for reinforced concrete slab bridges, the ratio of LRFR Inventory Rating to LFR ratings was 0.80 while for prestressed concrete slab bridges, the ratio was 1.31. When comparing the ratios of the LRFR Operating Ratings to the LFR Operating Ratings Mertz (2005) found that the average ratio was 0.84. A comparison between the Rating Factors can only be done on an average basis because the LRFR was calibrated to achieve a uniform reliability levels whereas the LFR is known to produce different reliability levels for different bridge configurations and span lengths. Furthermore, as noted by Moses (2001), the LRFR Operating and Inventory Ratings with HL-93 design loading were deliberately calibrated to be on the conservative side of the LFR ratings and the recommendation was to use such ratings for screening purposes while the dterminat8ion if a bridge has adequate capacity to safely carry legal loads would be made based on the LRFR Legal Load ratings.

On the other hand, a recent study by FHWA of a small sample of bridges found that using the LRFR instead of LFR would result in an average change on a national level in the areas of deficient bridges of about 1.7%. The maximum and minimum values range between 0.1% to 5.8% for individual states. Although the FHWA study confirms that LRFR inventory ratings are generally more conservative than LFR, considering the relatively small magnitude of the changes in deficient bridge areas, the FHWA does not anticipate any significant financial or policy impacts associated with adopting the LRFR.

Basic Load Models

In most cases, the basis of the ASR and LFR ratings is the HS-20 vehicle as this is the loading required for use by the FHWA for the NBI files. However, in order to develop a comfort level with the HS-20 load rating results in comparison with previous practice, many states also rate their bridges for the AASHTO H-20 vehicle. For example, in addition to New York, Georgia, Indiana, Kansas, New Hampshire, and West Virginia reported using HS and H loadings for load rating according to the survey performed by Sivakumar et al (2007). New Jersey, South Carolina, Texas, Wisconsin and Wyoming Departments of Transportation use the truck loads but not the lane loads for rating. Puerto Rico rates its bridges using the HS-30 truck and lane loads.

The LRFR uses the HL-93 design load as the basic configuration. While the origins of the HS-20 and H-20 vehicles date back to the 1930's and 1940's, the HL-93 live load model was developed in the 1990's as a notional model to represent the effect of the shearing forces and moments produced by a group of vehicles routinely permitted on highways of various states under "grandfather" exclusions to weight laws. The vehicles were based on a 1990 study conducted by the Transportation Research Board, which identified twenty-two representative configurations of vehicles allowed by states as exceptions to weight laws. The smallest and largest of which were a three-axle 48 Kip single truck and an eleven axle 149 Kip trailer truck (Kulicki; 1994).

In addition to the rating of bridges for the standard HS, H or HL-93 loadings, most states also rate their bridges based on what are known as the AASHTO "Legal Trucks". The AASHTO Legal Trucks were developed to represent actual truck configurations given that the AASHTO HS-20, H-20, as well as the HL-93 standard vehicles and associated lane loads are only nominal loads that provide load effects similar to those that would be expected from actual trucks and truck combinations. On the other hand, the AASHTO Legal Trucks are representative vehicles with the configurations of typical trucks on U.S. highways having axle weights that meet the Federal weight limits through what is known as the Federal Bridge Formula (FBF). Because of grand father clause exemptions, many states allow on their highways some trucks with total or axle weights that exceed the limits set by FBF. For this reason many states actually use their own Legal Trucks instead of the AASHTO Legal Trucks. In the survey conducted by Sivakumar (2007) only 11 of the 45 states that responded reported that they use the AASHTO legal trucks exclusively. These states are AZ, CA, IN, KS, MA, NE, NV, OR, SC, WV, WY. The following twenty-three states reported that they exclusively use their own state's legal loads: AL, AK, AR, DE, FL, GA, ID, IL, KY, MI, MN, MO, NH, NY, NC, ND, OH, PA, SD, TN, TX, VA, WI. The remaining eleven states use a combination of AASHTO and state legal loads. The latter set consists of CO, CT, HI, IA, LA, MS, NJ, NM, OK, RI, and WA.

1.4 Implementation of LRFR

A number of states are making progress toward implementing their state-specific versions of the AASHTO LRFR for the routine load rating of highway bridges. A summary of the procedures that are being implemented or recommended is provided in this section.

Florida LRFR

In the past couple of years the Florida Department of Transportation decided to begin applying the LRFR method by stipulating that the load rating of new and rehabilitated bridges must be performed in accordance with the LRFR method. Accordingly, an LRFR Inventory Rating of 1.0 (or 1.25 for short span bridges such as cast-in-place and precast flat slab bridges) was required for allowing the widening of a bridge or the addition of lanes to the bridge. The loadings for Florida LRFR rating are the design vehicles or the FL120 Permit vehicle (see Figure 7). If the calculated LRFR Rating Factor is found to be less than 1.0, more advanced methods of analysis may be allowed for determining the safety of the rehabilitated or widened bridge.

Florida State LRFR Legal Trucks are the SU4, C5 and ST5 defined in Figure 8. The rating analysis assumes that the SU4, C5 and ST5 trucks are in each loaded lane and does not allow for mixing trucks. The AASHTO LRFR live load factor γ_L=1.75 is maintained for the Inventory Rating for the design live load. Also, the Operating Rating uses the AASHTO-specified Live load factor γ_L=1.35. However, no modifications to γ_L are allowed when using the Legal vehicles for any ADTT. Similarly, no modifications of the live load factor are allowed based on state or site-specific data. The use of the γ_L=1.35 for all situations involving Legal Trucks is justified based on the fact that the LRFD HL-93 live load model envelopes FDOT legal loads and the live load factor of 1.35 for the design-load operating rating yields a reliability index consistent with traditional operating ratings (FLODOT Exceptions to LRFR; 2008).

The Florida LRFR allows the application of system factors, ϕ_s, to account for bridge redundancy and system capacity as compared to individual member capacity. The set of system factors provided by the Florida LRFR are significantly different than the optional values proposed by the AASHTO LRFR. The Florida LRFR specifies different factors for general systems, post-tensioned concrete beams as a function of the number of tendons per web, number of girders, span type and number of hinges to form a mechanism. For steel girder bridges the system factor is given as a function of the number of girders, the span type and the number of hinges to form a mechanism. For concrete box girder bridges the system factor is given as a function of the bridge type, span type, number of hinges to form a mechanism and the number of tendons per web. The FLODOT system factors values are shown in Table 5.a, 5.b, 5.c, 5.d. On the other hand, the Florida LRFR prefers the use of field measured member deterioration and thus the condition factor ϕ_c would be used only in the absence of measurements.

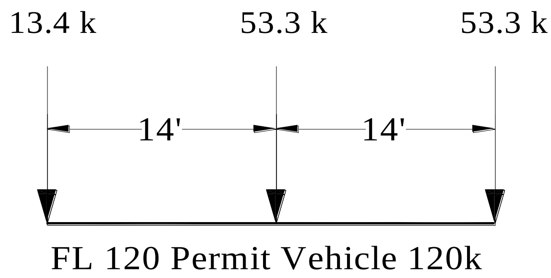
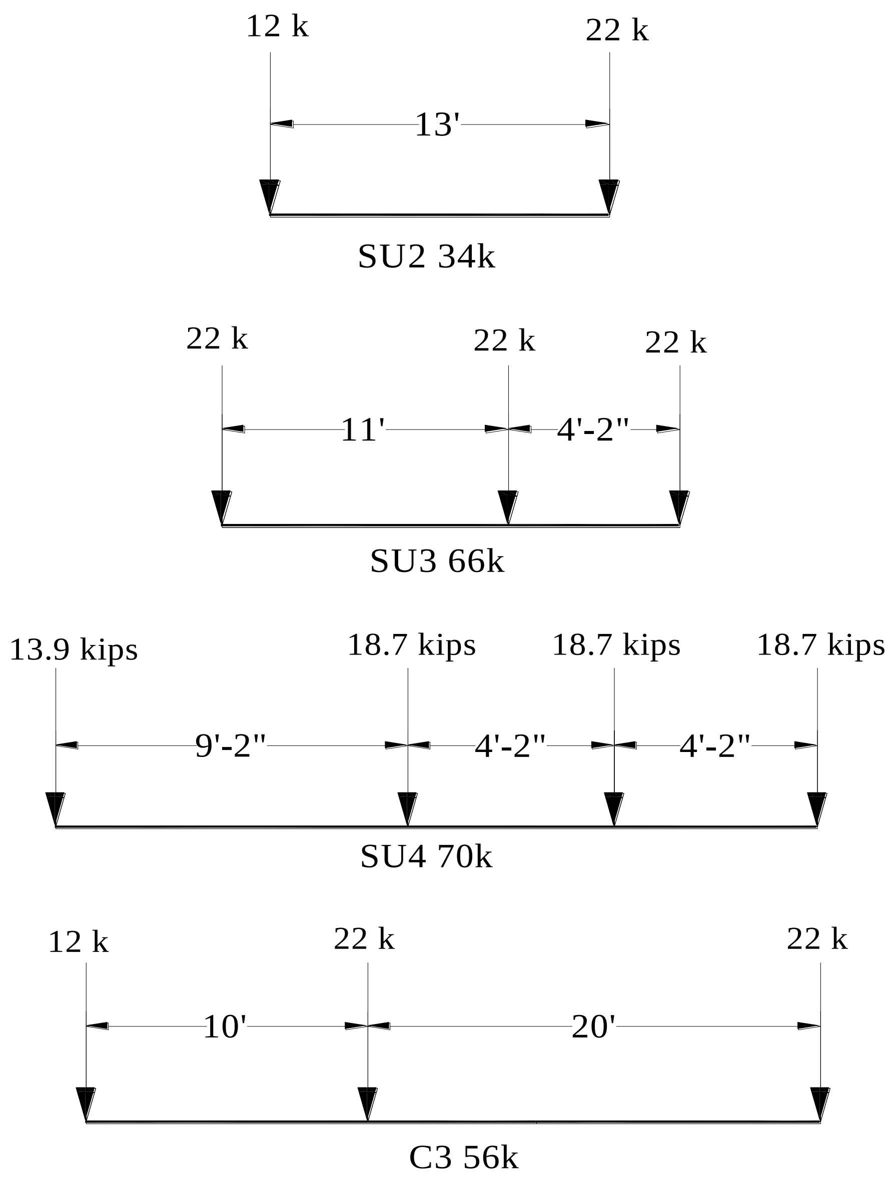


Figure 7. Florida FL 120 vehicle



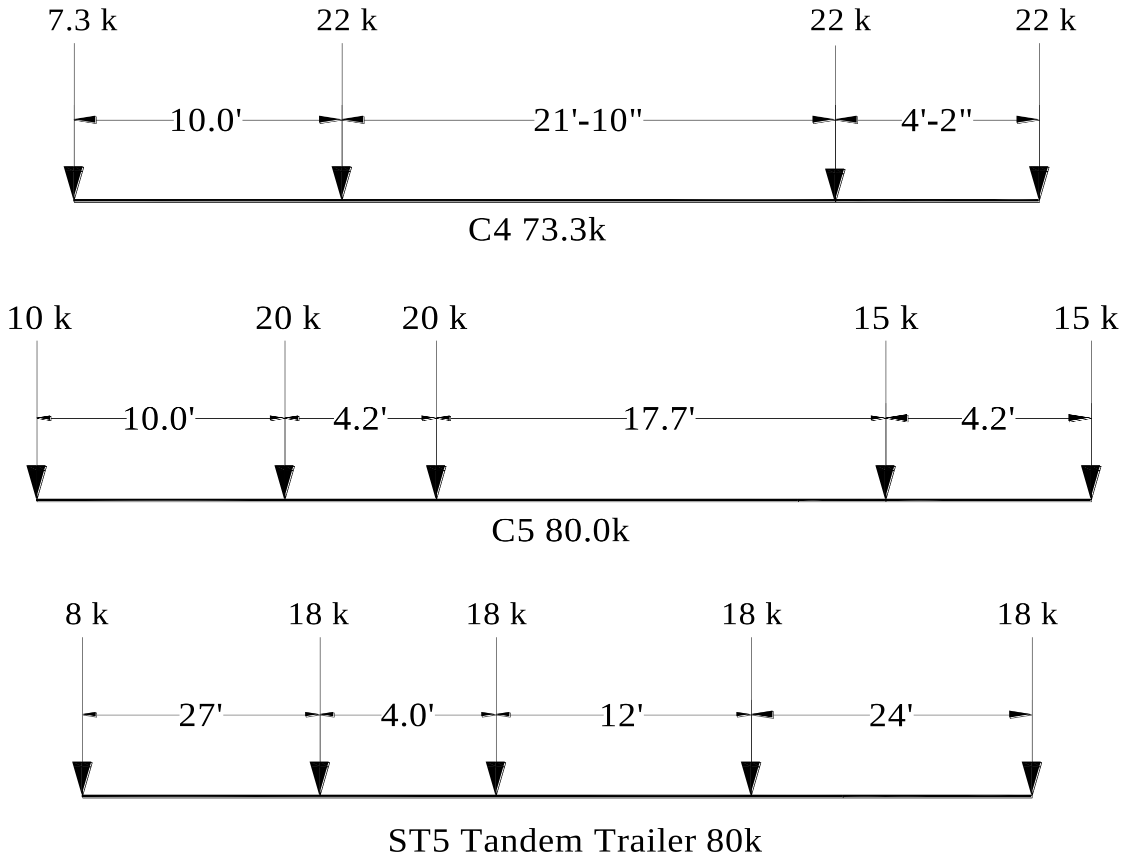


Figure 8. Florida Legal Vehicles.

Table 5.a. FDOT General System Factors (ϕ_s)

Superstructure Type	System Factors (ϕ_s)
Welded Members in Two Truss/Arch Bridges	0.85
Riveted Members in Two Truss/Arch Bridges	0.90
Multiple Eye bar Members in Truss Bridges	0.90
Floor beams with Spacing >12 feet and Non-continuous Stringers and deck	0.85
Floor beams with Spacing >12 feet and Non-continuous Stringers but with continuous deck	0.90
Redundant Stringer subsystems between Floor beams	1.00
All beams in non-spliced concrete girder bridges	1.00
Steel Straddle Bents	0.85

Table 5.b. FDOT System Factors (ϕ_s) for Post-Tensioned Concrete Beams

Number of Girders in Cross Section	Span Type	Number of Hinges Required for Mechanism	System Factors (ϕ_s) Number of Tendons per Web			
			1	2	3	4
2	Interior	3	0.85	0.90	0.95	1.00
	End	2	0.85	0.85	0.90	0.95
	Simple	1	0.85	0.85	0.85	0.90
3 or 4	Interior	3	1.00	1.05	1.10	1.15
	End	2	0.95	1.00	1.05	1.10
	Simple	1	0.90	0.95	1.00	1.05
5 or more	Interior	3	1.05	1.10	1.15	1.20
	End	2	1.00	1.05	1.10	1.15
	Simple	1	0.95	1.00	1.05	1.10

Table 5.c FDOT System Factors (ϕ_s) for Steel Girder Bridges

Number of Girders in Cross Section	Span Type	Number of Hinges Required for Mechanism	System Factors (ϕ_s)
2	Interior	3	0.85
	End	2	0.85
	Simple	1	0.85
3 or 4	Interior	3	1.00
	End	2	0.95
	Simple	1	0.90
5 or more	Interior	3	1.05
	End	2	1.00
	Simple	1	0.95
The tabularized values above may be increased by 0.10 for spans containing more than three evenly spaced intermediate diaphragms in addition to the diaphragms at the end of each span. These values may be increased by 0.05 for riveted members.			

Table 5.d. FDOT System Factors (ϕ_s) for Concrete Box Girder Bridges

Bridge Type	Span Type	Number of Hinges to Failure	System Factors (ϕ_s)			
			No. of Tendons per Web			
			1/web	2/web	3/web	4/web
Precast Balanced Cantilever Type A Joints	Interior Span	3	0.90	1.05	1.15	1.20
		2	0.85	1.00	1.10	1.15
	End or Hinge Span Statically Determinate	1	n/a	0.90	1.00	1.10
Precast Span-by-SpanType A Joints	Interior Span	3	n/a	1.00	1.10	1.20
		2	n/a	0.95	1.05	1.15
	End or Hinge Span Statically Determinate	1	n/a	n/a	1.00	1.10
Precast Span-by-SpanType B Joints	Interior Span	3	n/a	1.00	1.10	1.20
		2	n/a	0.95	1.05	1.15
	End or Hinge Span Statically Determinate	1	n/a	n/a	1.00	1.10
Cast-in-Place Balanced Cantilever	Interior Span	3	0.90	1.05	1.15	1.20
		2	0.85	1.00	1.10	1.15
	End or Hinge Span Statically Determinate	1	n/a	0.90	1.00	1.10

Wisconsin LRFR

In July 2007, the Wisconsin Department of Transportation (WisDOT) issued revisions to its Bridge Rating procedures. Accordingly, WisDOT requires that all bridge structures designed utilizing the Load and Resistance Factor Design (LRFD) specifications be rated using the LRFR method. The HL-93 design load inventory rating is performed using a live load factor γ_L=1.75. Also, the HL-93 design load operating rating is performed using a live load factor γ_L=1.35. However, the live load factors are set at γ_L=1.80 for the AASHTO Legal vehicles and state specific vehicles, while γ_L=1.60 is specified for the specialized hauling vehicles SU4, SU5, SU6 and SU7 depicted in Figure 6.

The Wisconsin DOT uses the same system factor ϕ_s specified in the AASSHTO LRFR except for the four-girder bridges with spacing less than 4.0 ft when the value is raised to ϕ_s =0.95. The condition factor ϕ_c is set at 1.0 for all cases.

Oregon LRFR

In 2006, the Oregon Department of Transportation (ODOT) also moved to adopt the LRFR. The ODOT LRFR manual was published in 2007 and recently updated in June 2008. The procedures stated in this document were developed based on the AASHTO LRFR Manual to provide a methodology that will result in consistent and reproducible Load Rating inputs and deliverables.

The manual essentially follows the AASHTO LRFR with live load factors γ_L=1.75 for the HL-93 design load Inventory Rating and γ_L=1.35 for the HL-93 design load Operating Rating. However, the ODOT went one step further to become the first state to adjust the LRFR live load factors based on actual truck weights as recorded at representative Weigh-In-Motion (WIM) sites within the State. Different tables are provided for state-owned and local agency bridges. These load factors however do not affect the general rating based on the design trucks but rather apply to the rating based on the Legal Trucks. Oregon legal loads include Type 3 legal truck, Type 3S2 legal truck and Type 3-3 legal truck (Figure 9). Note that the Oregon Legal type 3S2 vehicle is different (heavier) than the AAHTO 3S2 legal vehicle of Figure 3.

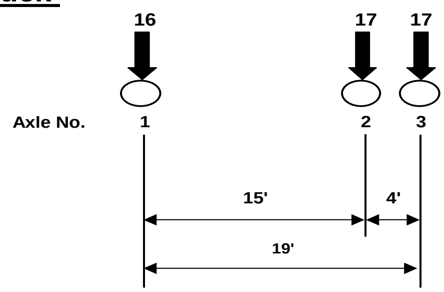
The same condition factors ϕ_c specified in the AASHTO LRFR are used. Additionally, the ODOT LRFR Manual provides specific instructions on how to select the appropriate factor based on condition ratings and cracking condition. The ODOT LRFR system factor, ϕ_s, table is a modified version of the table provided in the AASHTO LRFR Manual and gives values for the factor as a function of the number of girders and girder spacing of multi-girder bridges as well as values for cross beams as a function of the number of supporting columns.

OREGON LEGAL LOADS - Load Rating Tier-2

Indicated concentrated loads are axle loads in kips

TYPE 3 Legal Truck

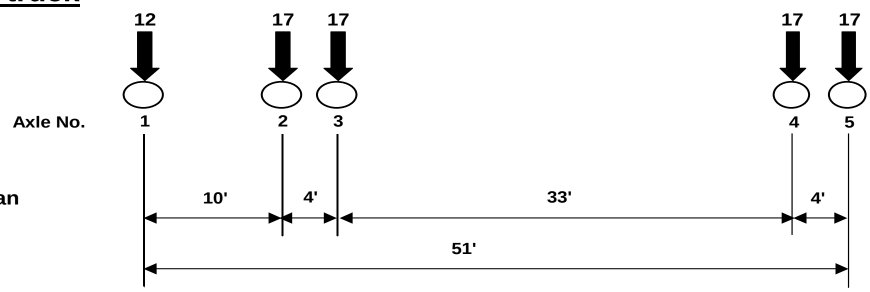
3 Axle Vehicle
Gross Weight = 50 k



TYPE 3S2 Legal truck

5 Axle Vehicle
Gross Weight = 80 k

Note:
This truck is greater than the standard AASHTO Type 3S2, which has Gross Weight = 72 k



TYPE 3-3 Legal Truck

6 Axle Vehicle
Gross Weight = 80 k

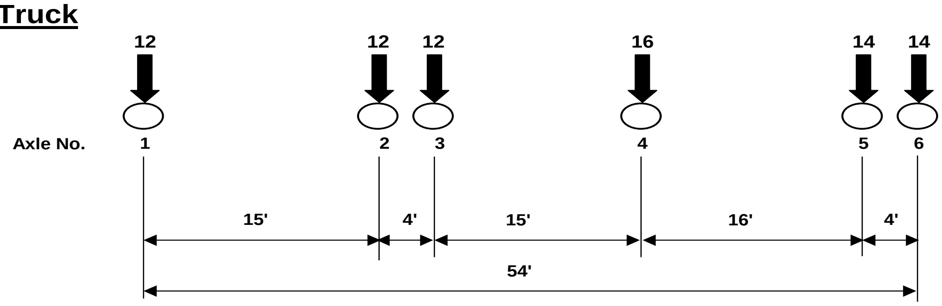


Figure 9. Oregon Legal Truck Loads

Michigan LRFR

A report issued in April 2008 provided an analysis of Michigan Weigh-in-Motion data that led to the development of customized live load factors which have been recommended by the Michigan DOT research team for future adoption in the planned Michigan LRFR Manual [Curtis & Till, 29008]. The report noted the importance of modifying the current AASHTO LRFR which if left unmodified would prevent a group of Michigan legal loads from driving across efficiently designed new structures. Accordingly, the report recommends the use of newly calibrated live load factors applicable for Michigan Designated Legal Vehicles for different ADTT. The calibration was based on Weigh-in-Motion truck data as well as a probability of side-side-events of 1/30. The 1/30 side-by-side probability which was found to be compatible with observations made on Michigan highways is lower than the 1/15 probability used during the calibration of the AASHTO LRFR. The Michigan study also recommended loading configurations for different bridge spans including spans between 200-ft and 400-ft in length.

The report also recommends the use of an HL-93 modified loading to replace the 25 kip tandem axle truck with a single 60-kip axle and adds a 1.2 factor to the lane and maximum of the truck and or axle loading.

The Michigan DOT uses an extensive list of Michigan Legal Vehicles shown in Figure 10. The recommended values of the live load factors for each legal truck type are provided in Table 6. These recommended live load factors along with the effects of the corresponding Michigan Legal vehicles would replace the AASHTO LRFR Legal Load Ratings for making decisions regarding the structural safety and the load carrying capacity of Michigan bridges. Other load factors have also been proposed for Permit Overloads for trucks having similar configurations as those described in Figure 10 but carry heavier weights.

Table 6. Michigan Designated Legal Vehicle Load Factors

Michigan Designated Legal Vehicle Load Factors for Strength Limit States				
Vehicle Number	GVW (kips)	5000 ADTT γ_{LL}	1000 ADTT γ_{LL}	100 ADTT γ_{LL}
1	33.4	1.80	1.60	1.40
2	47.4	1.80	1.60	1.40
3	54.4	1.80	1.60	1.40
4	67.4	1.80	1.60	1.40
5	84.0	1.75	1.60	1.40
6	101.4	1.54	1.51	1.40
7	119.4	1.39	1.36	1.31
8	91.4	1.65	1.60	1.40
9	51.4	1.80	1.60	1.40
10	65.4	1.80	1.60	1.40
11	83.4	1.76	1.60	1.40
12	117.4	1.41	1.37	1.32
13	125.4	1.35	1.32	1.27
14	132.4	1.31	1.28	1.23
15	143.3	1.25	1.22	1.18
16	138.4	1.28	1.25	1.20
17	151.4	1.21	1.19	1.14
18	154.0	1.20	1.18	1.13
19	117.4	1.41	1.37	1.32
20	87.4	1.71	1.60	1.40
21	151.4	1.21	1.19	1.14
22	161.4	1.17	1.15	1.11
23	154.0	1.20	1.18	1.13
24	122.0	1.37	1.34	1.29
25	164.0	1.16	1.14	1.10
26	50.0	1.80	1.60	1.40
27	72.0	1.80	1.60	1.40
28	80.0	1.80	1.60	1.40

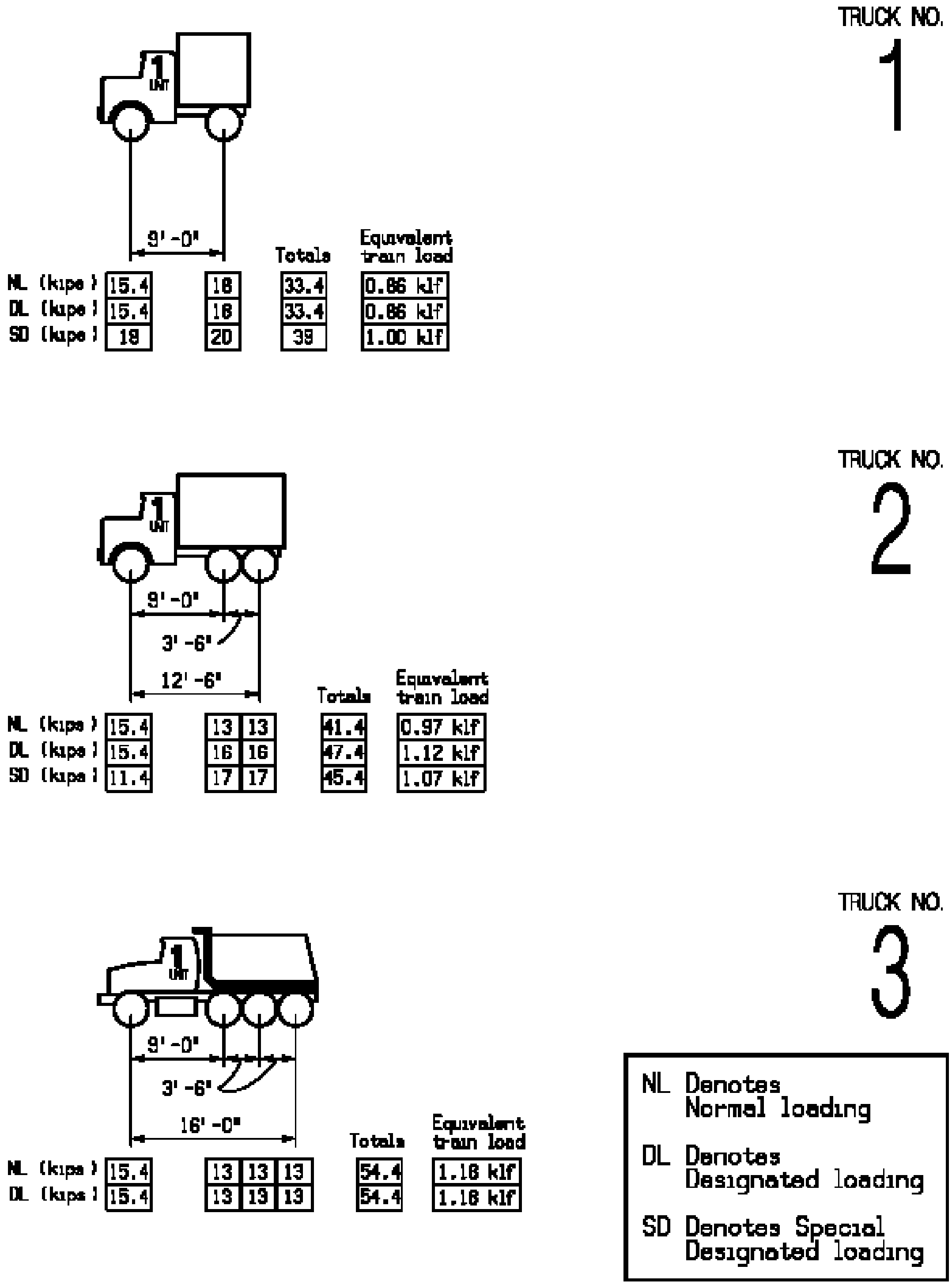


Figure 10. Michigan Legal Vehicles

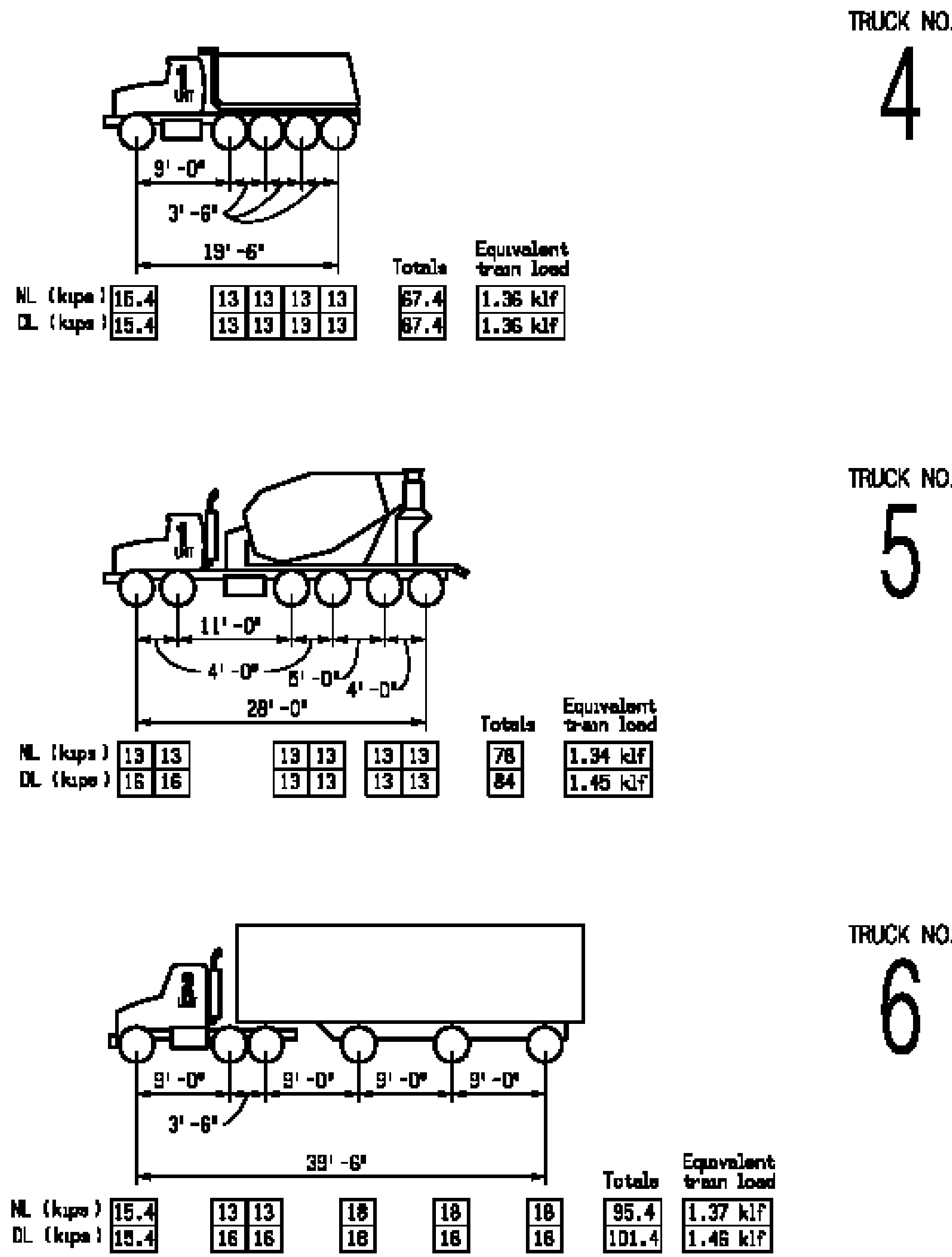


Figure 10. Michigan Legal Vehicles

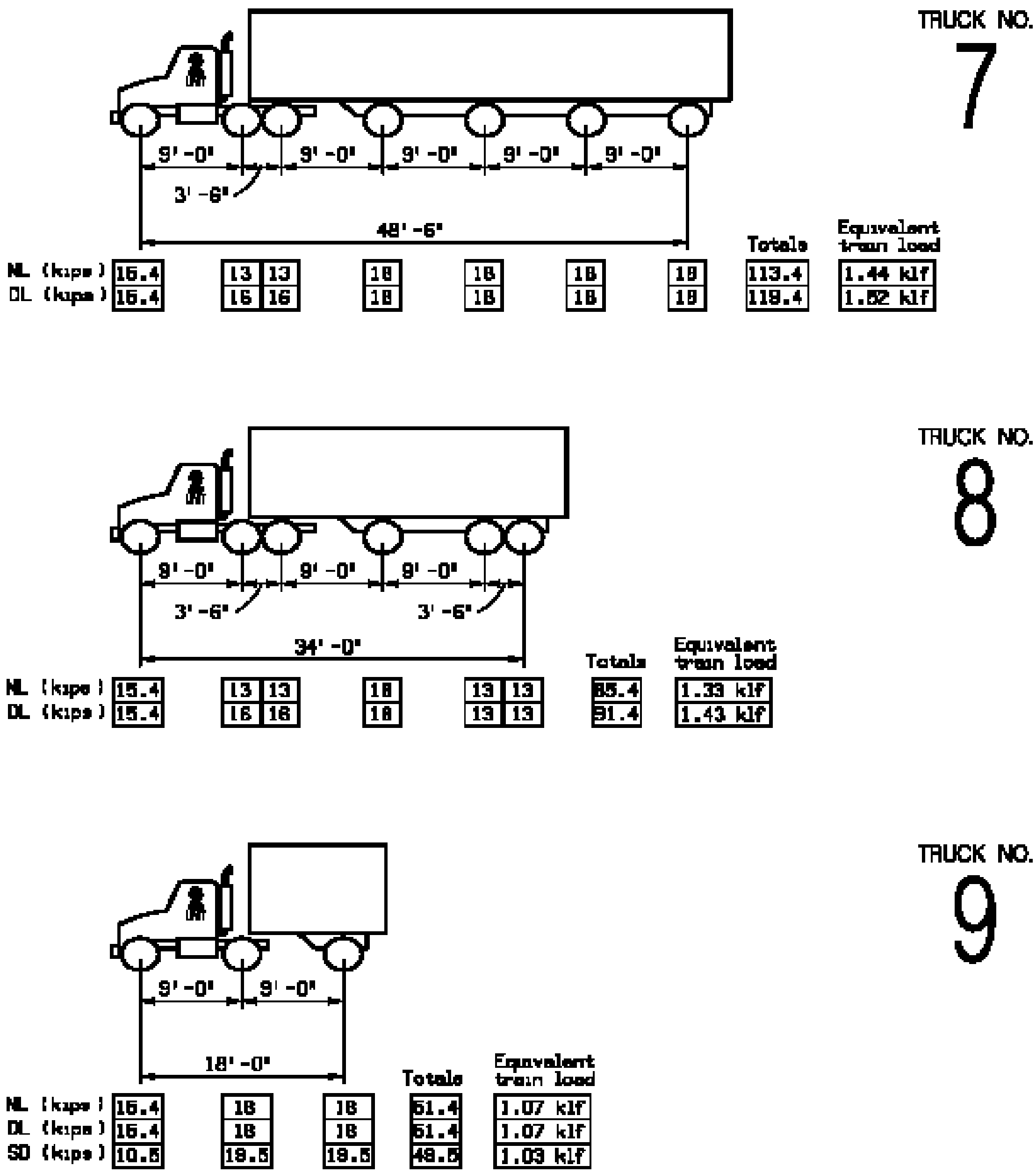


Figure 10. Michigan Legal Vehicles

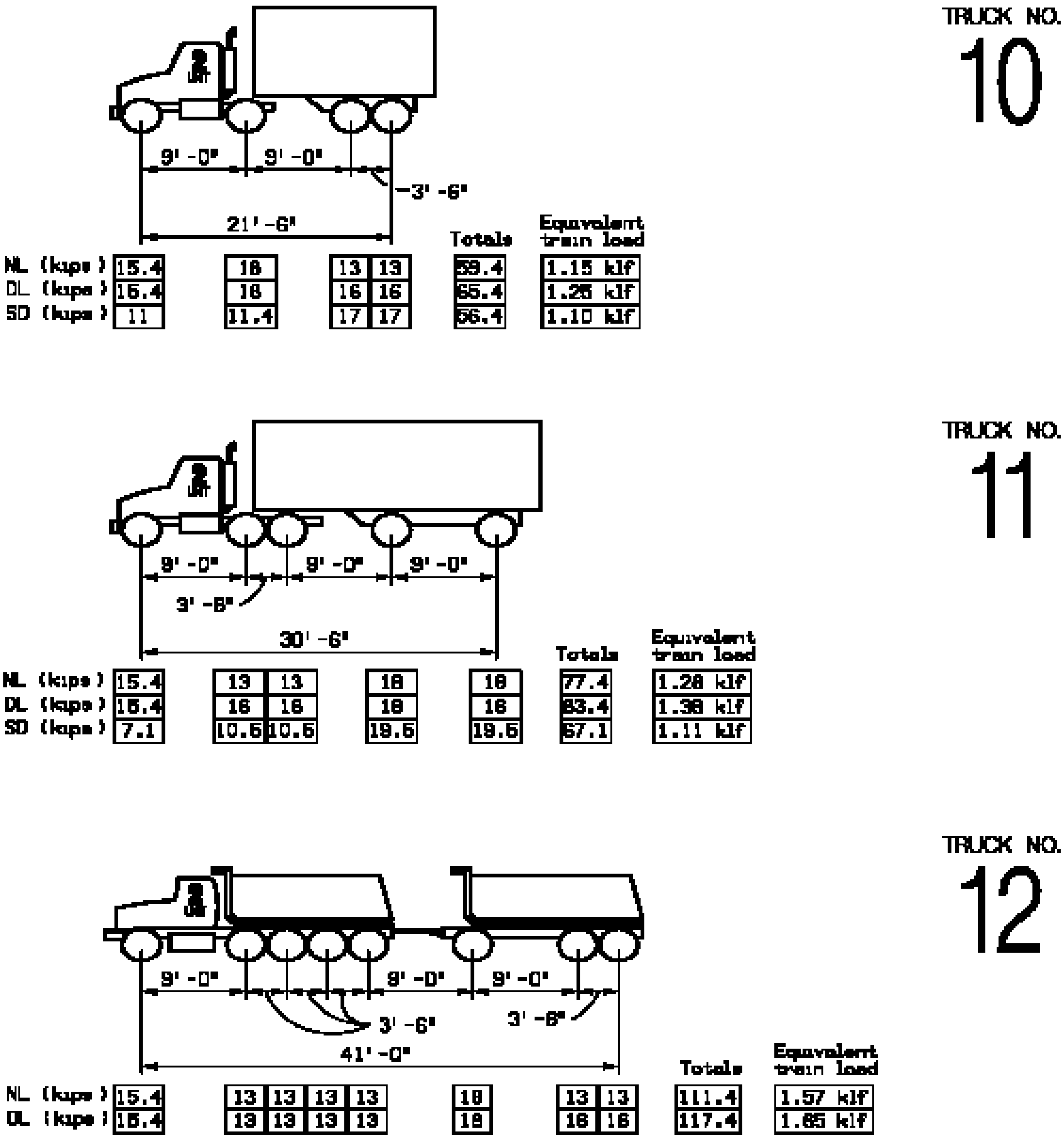


Figure 10. Michigan Legal Vehicles

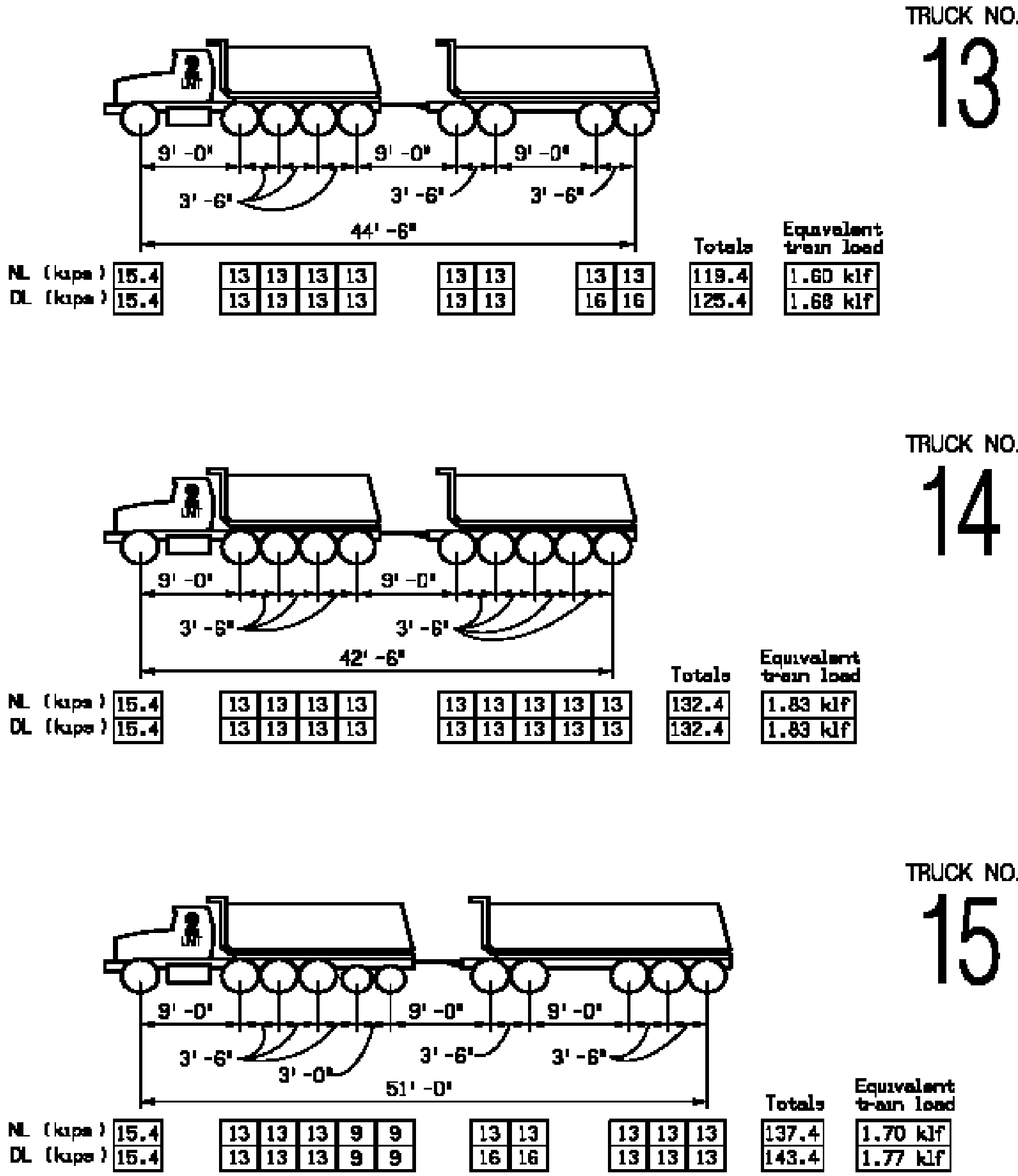


Figure 10. Michigan Legal Vehicles

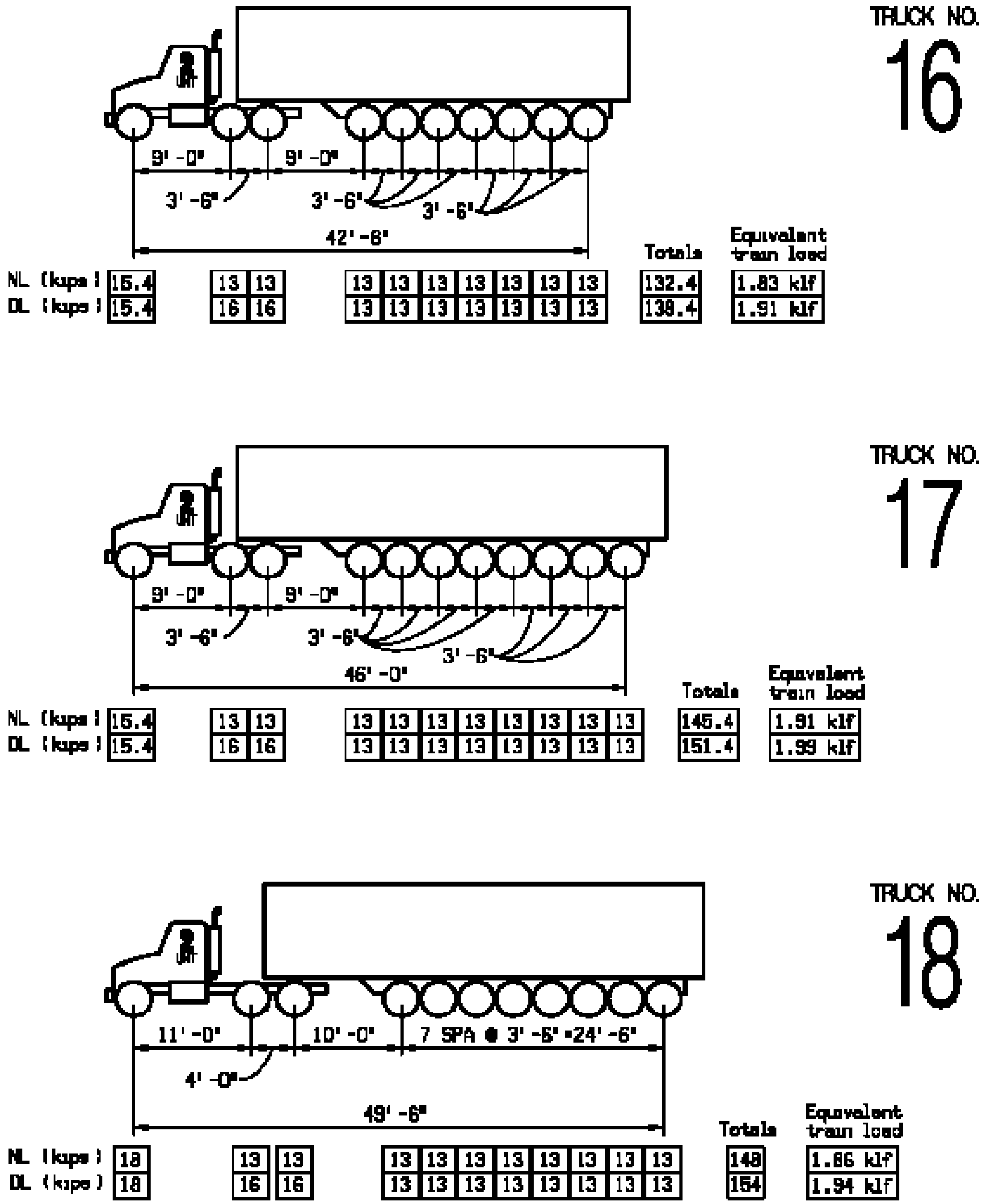


Figure 10. Michigan Legal Vehicles

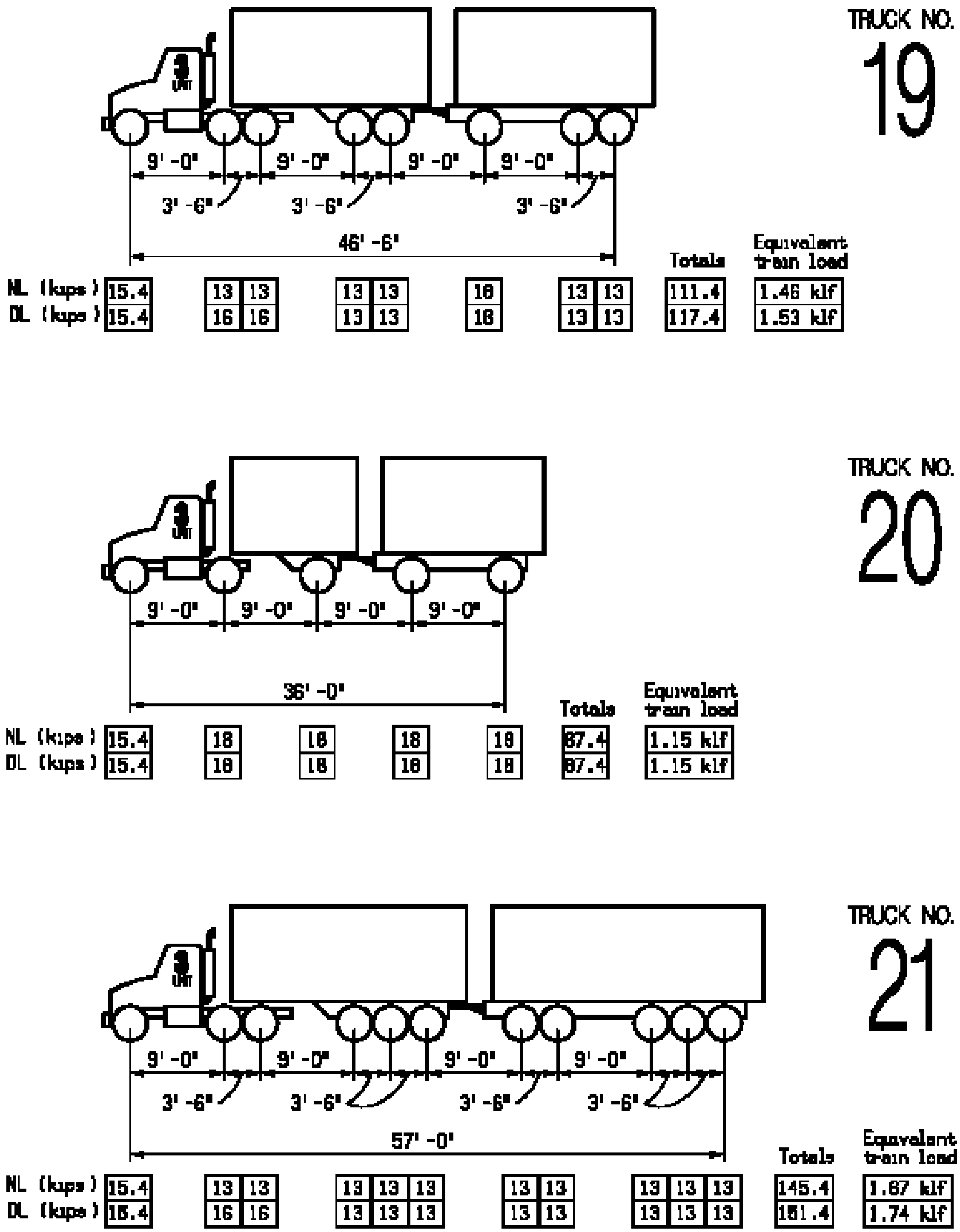


Figure 10. Michigan Legal Vehicles

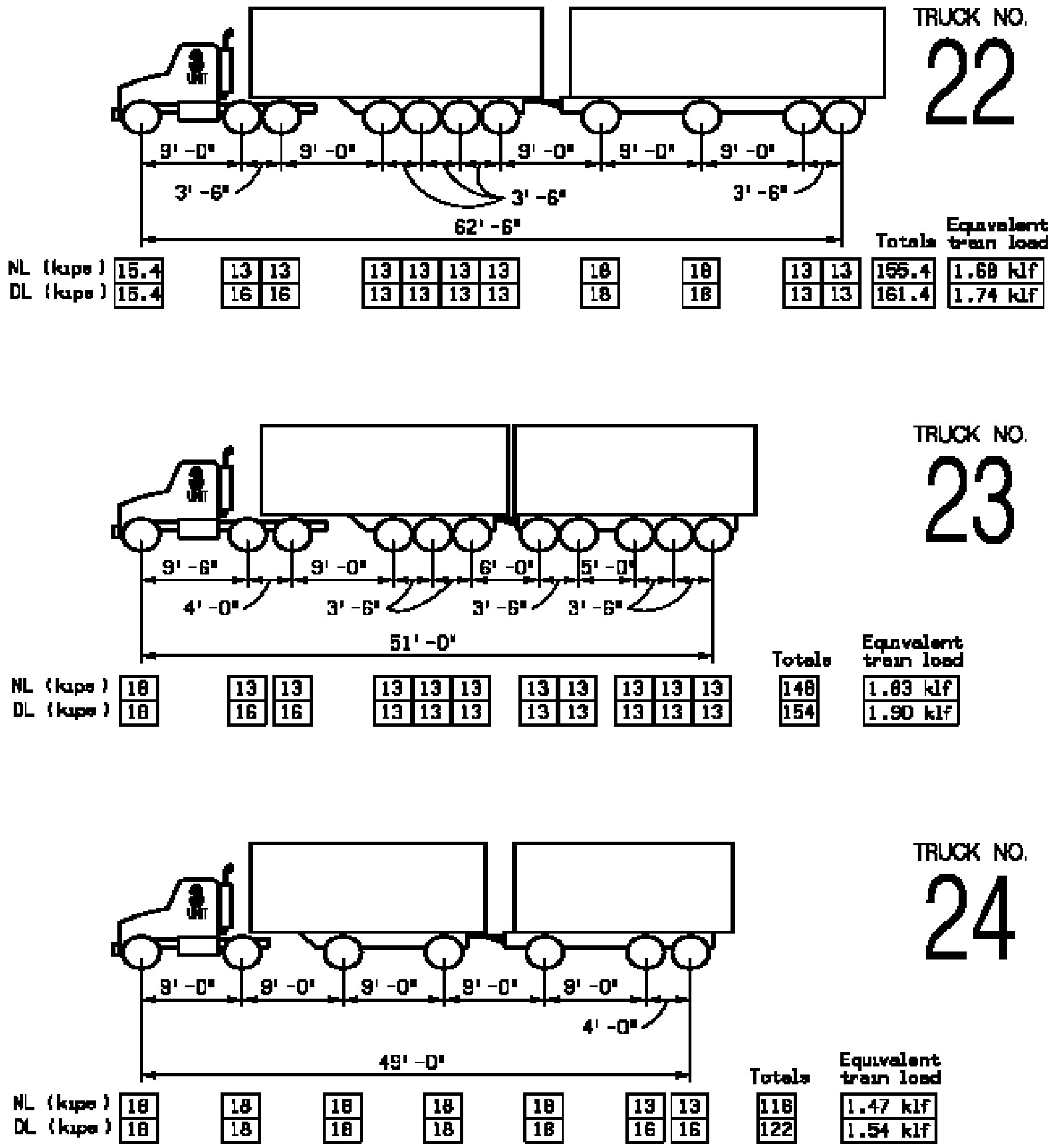


Figure 10. Michigan Legal Vehicles

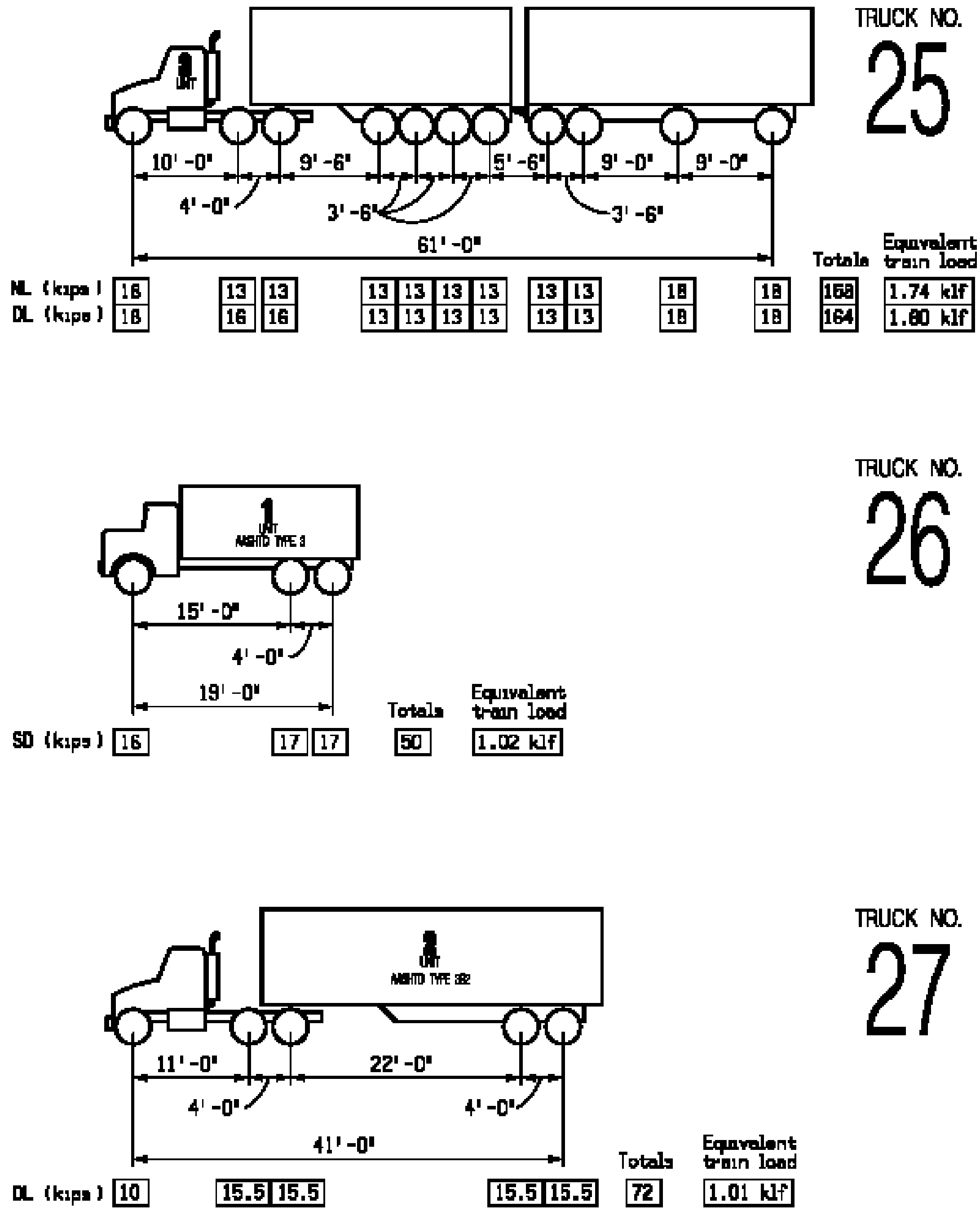


Figure 10. Michigan Legal Vehicles

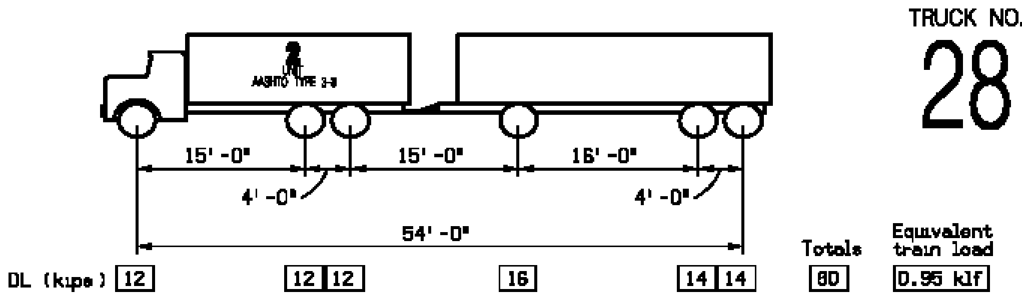


Figure 10. Michigan Legal Vehicles

Hawaii LRFR

A report issued in March 2008 provided a set of implementation guidelines for the LRFR rating of Hawaii DOT Bridges [Sivakumar, 2008]. The report requires that all existing bridges that have not been previously load rated be load rated using LRFR at the time of the next inspection. The document recommends the use of the standard AASHTO LRFR live load factors in combination with the HL-93 design load and the AASHTO Legal loads. The document also recommends the use of the Notional Rating Load and specialized Hauling Vehicles SU4, SU5, SU6 and SU7 (see Figure 6) that have been recently introduced into the AASHTO LRFR along with the corresponding live load factors that are provided as a function of the ADTT. Table 3.b provides the live load factors that are to be used with the NRL and the specialized short hauling vehicles.

The application of the standard AASHTO LRFR condition factor ϕ_c and system factor ϕ_s are also recommended [Sivakumar, 2008].

2. Load Posting

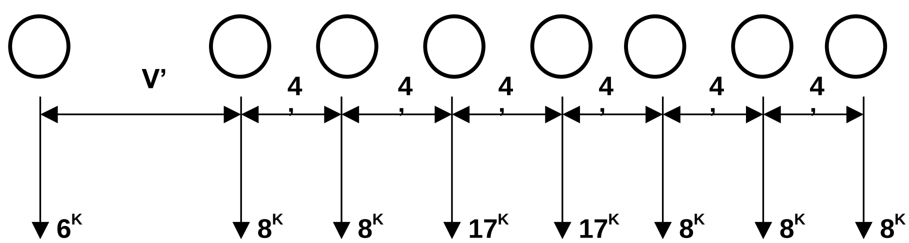
2.1 Load Posting Requirements

Statutory laws govern the maximum weight of vehicles legally allowed on highways and bridges without special overload permits. However, when a bridge is found to be structurally inadequate, weight limit restrictions must be imposed. Load posting is the process followed to determine the required truck weight limits. The load posting process involves a consideration of safety, compliance with state weight regulations, economy, and the public interest. Therefore, a considerable use of engineering judgment needs to be exercised when determining load posting criteria.

The federal government became involved in weight-limit posting in 1968 with the creation of the National Bridge Inspection Standards (NBIS) (Imbsen, 1984). The NBIS requires that every bridge be rated for its safe load-carrying capacity in accordance with the current *AASHTO Manual for Condition Evaluation of Bridges* (MCEB). MCEB section 6.7.2 specifies the HS-20 truck or lane load as the rating live load to be used in the basic load rating equation. The Inventory and Operating ratings for HS-20 are reported to the FHWA for inclusion in the National Bridge Inventory and the Structure Inventory & Appraisal sheet. Bridges that do not pass the HS-20 ratings with an Inventory Rating factor of 1.0 or higher must be subjected to a posting analysis to determine the need for weight-limit posting. According to the NBIS: "If it is determined under this rating procedure that the maximum legal load under State law exceeds the load permitted under the Operating Rating, the bridge must be posted in conformity with the AASHTO Manual or in accordance with State law". According to the AASHTO LRFR Guide Manual, the HL-93 design load performs the same screening and reporting function as that of the HS-20 vehicle for the MCEB. The LRFR recommends that bridges with LRFR design load Inventory Ratings $RF \geq 1.0$ need not be load posted for AASHTO Legal Loads and State Legal Loads within the LRFD Exclusion limits. Bridges with LRFR design load Operating Rating factors $RF \geq 1.0$ do not require restrictive load posting for AASHTO Legal Loads and State Legal Loads having only minor variations from the AASHTO Legal Loads. Although the HS-20 or the HL-93 design load set the trigger for initiating the load posting process, the determination of the actual weight limits is usually an involved process which depends on the AASHTO Legal Loads or the State Legal Loads. The HL-93 design load is used as a trigger for LRFR load posting because the HL-93 model was developed to provide an envelope to the effects of legal vehicles. It is only when the HL-93 inventory rating falls below 1.0 that a more specific analysis of the AASHTO Legal or other state legal loads would be necessary.

The AASHTO Legal Trucks were developed in the 1970's to provide a representative set of loads that model the effects of trucks that follow the Federal Bridge Formula (FBF) limits on short, medium and long span bridges. But recently, a significant number of Short Hauling Vehicles (SHV) with closely-spaced multiple axles have been increasingly used by the trucking industry. Even when these trucks meet the FBF limits, their load effects may in certain cases exceed those of the AASHTO Legal Trucks. In addition, due to grandfather exemptions, many states allow some trucks (usually SHV) to exceed the FBF limits. These trucks are normally free to operate unrestricted except as limited by bridge postings. Thus, bridge posting for these vehicles seems to be the only safeguard that some states may have to protect their bridges from overstress. For these reasons, many states have adopted a variety of short multi-axle vehicles as state legal and posting loads. Tables 7 and 8 describe posting loads that conform to the FBF and those that do not conform to the FBF as adopted by different states (Sivakumar et al; 2007).

To provide more consistent ratings between the states, new rating and posting load models for SHVs were adopted by AASHTO in 2005. A new Notional Rating Load model NRL which is a screening level load shown in Figure 11 was developed to envelope the load effects on simple and continuous span bridges of the worst possible Formula B single unit truck configurations. In that sense the NRL plays the same role as the HL-93 and only bridges that do not rate for the NRL loading need to be investigated to determine posting requirements using the recommended single unit posting loads SU4, SU5, SU6 and SU7 shown in Figure 6. Because different states impose different restrictions on the configurations and weights of multi-axle trucks, each state is required to only post for the vehicles that are allowed to operate in that State.



V = VARIABLE DRIVE AXLE SPACING — 6'0" TO 14'-0". SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM LOAD EFFECTS.

AXLES THAT DO NOT CONTRIBUTE TO THE MAXIMUM LOAD EFFECT UNDER CONSIDERATION SHALL BE NEGLECTED.

MAXIMUM GVW = 80 KIPS

Figure 11. AASHTO Notional Rating Load NRL for SHVs

Table 7.a Posting Loads that conform to FBF

FORMULA B CHECKS FOR SINGLE-UNIT STATE POSTING LOADS THAT DIFFER FROM AASHTO TRUCKS							
DOT	**Truck Designation**	**Total Axle Spacing L (ft.)**	**No of Axles N**	**GVW (Kips)**	**FBF Gross Weight Limit (Kips)**	**Satisfies FBF Gross Weight Limit?**	**Satisfies FBF Axle Weight Limit?**
Alabama	Tandem Axle	19	3	59.0	50.3	No	No
	Tri-Axle	19	4	75.0	54.7	No	No
	Concrete Truck	18	3	66.0	49.5	No	No
Arkansas	T3	12	3	45.0	45.0	Yes	Yes
	T4	16	4	62.0	52.7	No	No
	T3S2	24	5	80.0	63.0	No	Yes
Connecticut	Construction Vehicle	18.2	4	76.5	54.1	No	No
Delaware	DE 2	10	2	40.0	40.0	Yes	Yes
	DE 3 Inter-State	16.83	3	54.0	48.6	No	No
	DE 3	16.83	3	70.0	48.6	No	No
	DE 4	17	4	74.0	52.9	No	No
Florida	SU2	13	2	34.0	43.0	Yes	No
	SU3	15.17	3	66.0	47.4	No	No
	SU4	17.51	4	70.0	53.7	No	No
	C3	30	3	56.0	58.5	Yes	No
Georgia	H20-MOD	14	2	43.0	44.0	Yes	No
	Type 3	19	3	66.0	50.3	No	No
Idaho	Type 3	14	3	54.0	46.5	No	No
Illinois	Type 3	16	3	44.0	48.0	Yes	Yes
	Type 3-S1	28	4	58.5	60.8	Yes	Yes
	Type 3-S2	30	5	72.0	66.8	No	Yes
Kentucky	Type 1	14	2	40.0	44.0	Yes	No
	Type 2	16	3	56.7	48.0	No	No
	Type 3	20	4	73.5	55.3	No	No
	Type 4	34	5	80.0	69.3	No	No
Michigan	No 1	9	2	33.4	39.0	Yes	Yes
	No.2	12.5	3	41.4	45.4	Yes	Yes
	No.3	16	4	54.4	52.7	No	Yes
	No.4	19.5	5	67.4	60.2	No	No
	No.5	28	6	78.0	70.8	No	No
	No.9	18	3	51.4	49.5	No	Yes
	No. 10	21.5	4	59.4	56.3	No	Yes
	No.11	30.5	5	77.4	67.1	No	Yes
Minnesota	Type 3	16	3	48.0	46.5	No	Yes
Mississippi	Concrete Truck	16	3	60.0	48.0	No	No
	HS-Short	30	5	80.0	66.8	No	No

Table 7.b Posting Loads that conform to FBF

DOT	Truck Designation	Total Axle Spacing L (ft.)	No of Axles N	GVW (Kips)	FBF Gross Weight Limit (Kips)	Satisfies FBF Gross Weight Limit?	Satisfies FBF Axle Weight Limit?
New Hampshire	Two Axle Truck	14	2	33.4	44.0	Yes	No
	Three Axle Truck	16	3	55.0	48.0	No	No
	Four Axle Truck	18	4	60.0	54.0	No	No
North Carolina (Interstate Traffic)	SH	14	2	25.0	44.0	Yes	No
	S3A	13	3	45.5	45.8	Yes	No
	S3C	15	3	43.0	47.3	Yes	No
	S4A	17	4	53.5	53.3	Yes	No
	S5A	21	5	61.0	61.1	Yes	No
	S6A	25	6	69.0	69.0	Yes	No
	S7A	34	7	80.0	79.8	Yes	No
	S7B	29	7	77.0	76.9	Yes	No
	T4A	22	4	56.5	56.7	Yes	No
	T5B	26	5	64.0	64.3	Yes	No
	T6A	30	6	72.0	72.0	Yes	No
	T7A	34	7	80.0	79.8	Yes	No
	T7B	34	7	80.0	79.8	Yes	No
North Carolina (Except Interstate Traffic)	SH	14	2	25.0	44.0	Yes	No
	S3A	13	3	50.1	45.8	No	No
	S3C	15	3	43.0	47.3	No	No
	S4A	17	4	58.9	53.3	No	No
	S5A	21	5	67.1	61.1	No	No
	S6A	25	6	75.9	69.0	No	No
	S7A	34	7	80.0	79.8	No	No
	S7B	29	7	80.0	76.9	No	No
	T4A	22	4	62.2	56.7	No	No
	T5B	26	5	70.4	64.3	No	No
	T6A	30	6	79.2	72.0	No	No
	T7A	34	7	80.0	79.8	No	No
	T7B	34	7	80.0	79.8	No	No
Ohio	2F1	10	2	30.0	40.0	Yes	Yes
	3F1	14	3	46.0	46.5	Yes	Yes
	4F1	18	4	52.0	54.0	Yes	Yes
Pennsylvania	ML80	18	4	75.5	54.0	No	No
	TK527	34	7	80.0	80.0	Yes	No
South Dakota	Type 3	16	3	48.0	48.0	Yes	Yes
Tennessee	TN4	19.17	4	74.0	54.8	No	No
Texas	Single Delivery Truck	17	2	38.0	47.0	Yes	No
	Concrete Truck	20	3	69.0	51.0	No	No
Virginia	Single Unit Truck	24	3	54.0	51.8	No	Yes

Table 8. Posting load that do not conform to FBF

State Posting Loads that Exceed Formula B Gross Weight Limits						
DOT	**Truck Designation**	**No of Axles**	**Total Spacing**	**Truck Weight (Kips)**	**FBF Limit for Gross Wt (K)**	**Excess Over FBF Limit (K)**
Alabama	Tandem Axle	3	19.00	59.00	50.30	8.70
	Concrete Truck	3	18.00	66.00	49.50	16.50
Delaware	DE 3 Interstate	3	16.83	54.00	48.60	5.40
	DE 3	3	16.83	70.00	48.60	21.40
Florida	SU3	3	15.17	66.00	47.40	18.60
Georgia	Type 3	3	19.00	66.00	50.30	15.70
Idaho	Type3	3	14.00	54.00	46.50	7.50
Kentucky	Type 2	3	16.00	56.70	48.00	8.70
Michigan	No. 9	3	18.00	51.40	49.50	1.90
Mississippi	Concrete Truck	3	16.00	60.00	48.00	12.00
New Hampshire	Three-Axle Truck	3	16.00	55.00	48.00	7.00
Texas	Concrete Truck	3	14.00	69.00	51.00	18.00
Virginia	Single-Unit Truck	3	24.00	54.00	51.80	2.20
Alabama	Tri-Axle	4	19.00	75.00	54.70	20.30
Arkansas	T4	4	18.00	62.00	52.70	9.30
Connecticut	Construction Vehicle	4	18.20	76.50	54.10	22.40
Delaware	DE 4	4	17.00	73.00	52.90	20.10
Florida	SU4	4	18.34	70.00	53.70	16.30
Kentucky	Type 3	4	20.00	73.50	55.30	18.20
Michigan	No. 3	4	16.00	54.40	52.70	1.70
	No. 10	4	21.50	59.40	56.30	3.10
New Hampshire	Four-Axle Truck	4	18.00	60.00	54.00	6.00
North Carolina	S4A	4	17.00	58.85	53.30	5.55
Pennsylvania	ML80	4	18.00	75.48	54.00	21.48
Tennessee	TN4	4	19.17	74.00	54.80	19.20
Arkansas	T3S2	5	24.00	80.00	63.00	17.00
Illinois	3-S2	5	30.00	72.00	66.80	5.20
Kentucky	Type 4	5	34.00	80.00	69.30	10.70
Michigan	No. 4	5	19.50	67.40	60.20	7.20
	No. 11	5	30.50	77.40	67.10	10.30
Mississippi	HS-Short	5	30.00	80.00	66.80	13.20
North Carolina	S5A	5	21.00	67.10	61.10	6.00
Michigan	Concrete Truck No. 5	6	28.00	78.00	70.80	7.20
North Carolina	S6A	6	25.00	75.90	69.00	6.90

2.2 Load Posting Practices

General Procedure

A bridge that does not pass the operating design load rating with a rating factor R.F. of 1.0 or higher is subjected to an analysis to determine if an appropriate posting load is required. Posting loads give the weight limits for typical truck configurations. The FHWA regulations leave the determination of the actual load limits to the individual state. Although different notional and actual truck configurations can be used for executing the load rating analysis, the actual load posting must be based on state weight regulations and use truck configurations which are representative of the truck population within a state.

Federal Regulations 23 CFR Part 650 NBIS 650.313 (c) requires to "rate each bridge as to its safe loading capacity in accordance with the AASHTO manual. Post or restrict the bridge in accordance with the AASHTO Manual or in accordance with state law, when the maximum unrestricted legal loads or State routine permit loads exceed that allowed under the operating rating or equivalent rating factor."

The statement given above demonstrates that the NBIS provides limited guidance on evaluating and posting weight limits on bridges. Therefore, considerable engineering judgment is required to fill the gaps. In addition to the variations in the configurations of the posting loads used, posting regulations vary widely among agencies that currently follow the MCEB. These differences include the criteria for initiating a posting action, the methodology for setting the allowable truck weight limit and the techniques for how the limits should be represented on highway signage. This leads to differences in posting criteria in different jurisdictions that reflect different rating and evaluation philosophies, different jurisdictional needs, different bridge inventories, and different traffic conditions. For instance, the bridge posting level may be set at the Operating level, Inventory level, or somewhere in between, depending upon factors such as: bridge type, condition rating, redundancy, fatigue sensitive details, ADTT, inspection frequency and enforcement.

In order to provide some uniform guidance, AASHTO LRFR Section 6.8.3 recommends that the safe posting load be calculated using the formula:

$$\text{Safe Posting Load}=\frac{W}{0.7}[(RF)-0.3] \qquad (3)$$

Where,
RF = legal load rating factor
W = weight of rating vehicle

When the RF is governed by the lane load, then the value of W in Equation 1 shall be taken as 40 Tons. When States use their own legal loads which are different from the AASHTO legal loads, Equation (3) may be used for posting load, but the gross weight of the State's legal vehicle shall be substituted in the posting equation. When the RF for any vehicle type falls below 0.3, then that vehicle type should not be allowed on the span. When RF falls below 0.3 for all three AASHTO legal trucks, then the span should be considered for closure. When the safe load capacity falls below 3 Tons, the bridge should be closed.

Additional Criteria

There is considerable leeway in the way bridges are currently posted as most of the factors used to determine the weight limits are selected by the bridge owner based on established procedures and past experiences. The recently adopted LRFR procedure provides only a single load rating for use on load posting through a more systematic consideration of these factors in the rating process and therefore encourages more uniform posting practices. For example, the LRFR provides a table of live load factors to be used with Legal loads based on the Average Daily Truck Traffic (ADTT) or based on ADTT and truck weight statistics obtained from weigh-in-motion (WIM) data. Other factors that can be directly considered using the LRFR procedures include the bridge condition, bridge configuration and redundancy as well as riding surface conditions.

Most states following the traditional AASHTO MCEB guidelines post bridges for the weights of the posting vehicles that produce legal load Operating Ratings less than 1.0. Table 9 provides a list of posting criteria adopted by some of the states that responded to the survey conducted by Sivakumar in 2006. [Sivakumar et al, 2007].

For example, the New York State Department of Transportation (NYSDOT) guidelines set the load posting based on H 20 equivalent ratings. These can then be used to determine the Safe Load Capacity (SLC). The SLC is a load rating value that corresponds to an acceptable stress level from actual traffic loads. Load posting is required if the SLC for a given span is less than the H equivalent rating of a legal load. The NYSDOT provides a table that gives the equivalent H rating for the maximum AASHTO Legal Truck load effect as a function of the effective span length. [NYSDOT Engineering Instruction EI 05-034]. In addition, the calculated H Operating Rating (HOR) of the controlling member, whether determined using Allowable Stress rating (ASR) or Load Factor rating (LFR) analysis should be modified to produce the Safe Load Capacity (SLC) based on the field inspection report and the type of structure to which the rated member belongs. One example is for deteriorated load path non-redundant members, the NYSDOT lowers calculated H Operating Rating (HOR) of the controlling member to get the Safe Load Capacity. This approach is very similar to that used in LRFR, which uses the system factor, ϕ_s, that considers the redundancy and non-redundancy of structures and the condition factor ϕ_c, which differentiates between members in good or deteriorated conditions. Besides load posting bridges, NYSDOT also designates some bridges as R-posted. R-posting is a load restriction for a bridge, which does not have the reserve capacity to accommodate most vehicles over legal loads, but can still safely carry legal loads.

As shown in Table 9, States that also use more stringent criteria for fracture critical bridges include California and West Virginia as well as Delaware which follows the guidelines described in the flow chart of Figure 12. Other states that report using bridge deterioration as a criterion for determining the posting loads include Delaware, Kansas, Puerto Rico, and Washington State. States that use traffic intensity to determine the posting level include Delaware which uses the Average Daily Truck Traffic (ADTT) and Missouri and West Virginia which use the Average Daily Traffic (ADT).

Determination of Posting Levels

Most states use the operating rating to determine whether load posting is required, although some states provide some allowance. For example, Colorado gives a 5% allowance while Washington state gives a 10% allowance. Other states such as Minnesota may under certain conditions post bridges even when their ratings are slightly higher than one (e.g. 1.0<RF<1.1). Posting levels generally vary between Inventory Stress levels (e.g. Georgia, Nevada), Operating stress levels (California, Iowa, Nebraska, New Jersey, North Carolina) or somewhere in between (e.g. Delaware, Kansas, West Virginia, Wyoming). For example, Delaware uses four different levels of posting varying between inventory stress to operating stress levels depending on bridge condition rating, redundancy, fatigue sensitive details, ADTT, level of truck weight enforcement, and detour length. The flow chart used by Delaware for Load Posting is provided in Figure 12.

Posting Loads

Posting loads are in general a subset of the state or federal legal loads used for implementing bridge weight restrictions. Many load models, actual or notional, may be utilized for load rating but load posting when represented by truck symbols with associated weight limits is based on trucks representative of actual track traffic. Both the MCBE and LRFR Manuals use the three AASHTO trucks, Type3, Type 3S2 and Type 3-3 as the basic posting loads. Recently, AASHTO has added four single unit posting loads SU4, SU5, SU6 and SU7 (Figure 6) to better represent the effects of Short Hauling Vehicles.

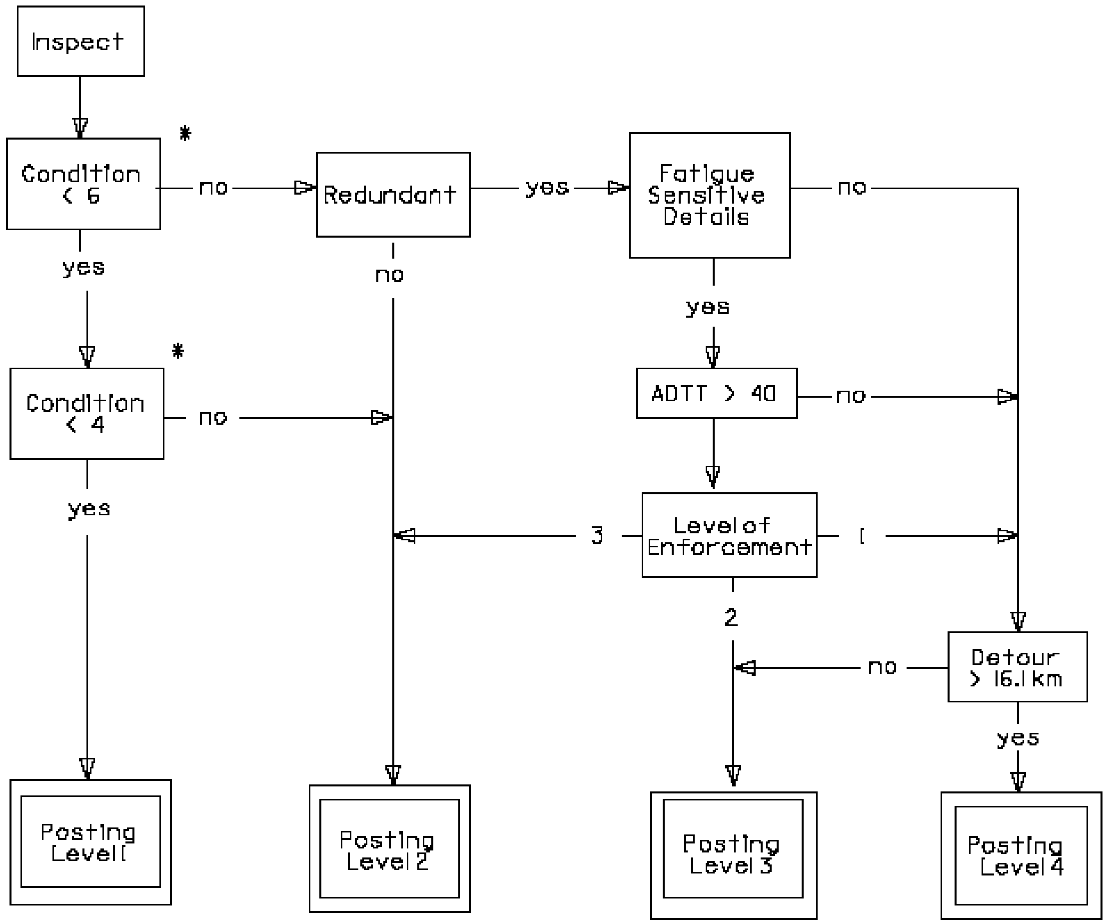
States however have considerable leeway in selecting the posting vehicles that best represent the types of trucks encountered on their highway system. For example, Oregon State and Colorado use variations on the AASHTO legal loads, their Legal trucks are slightly different from those in the LRFR Manual. Oregon's type 3S2 vehicle is heavier than the 3S2 vehicle in the LRFR Manual. In Colorado, the posting rating is computed using the Posting Vehicles shown in Figures 13 or Figure 14. For mainline Interstate routes, or Interstate access ramps, the Posting Vehicles shown in Figure 13, are used. For all other routes, including Interstate business routes, the posting Vehicles shown in Figure 14 are used. Similar trucks are used by most other states.

A study conducted by Asantey and Bartlett (2005) analyzed the impact of posted load limits on highway bridge reliability and the importance of enforcing these limits. The study observed that in the case of perfect compliance with the posted load level, the reliability of the bridges can be significantly enhanced. However, violation rates as low as 2.5% would result in a significant decrease in reliability.

Table 9. A Sample of State Posting Criteria. [Sivakumar et al, 2007]

California Department of Transportation	Usually redundant concrete structures in good condition will not be posted even if calculations indicate otherwise.
Colorado Department of Transportation	Structures with posting rating factor less than 0.95 require posting signs.
Georgia Department of Transportation	A bridge requires posting when the legal weights exceed the bridge operating rating. We generally post at the Inventory rating.
Iowa Department of Transportation	If truck reaction is within one ton of the operating capacity we do not post for that truck. If the 4S3 or the 4S2 trucks need to be restricted we post for the maximum of 40 tons. If any of the other legal trucks need to be restricted we include a triple axle limit to cover 4S3 and 4S2 trucks.
Kansas Department of Transportation	If the bridge is in good condition , only weak, we post as the load rating approaches the operating stress level, we generally post midway between inventory and operating, rounding down to an even 5 tons. If the bridge is in poor condition, the need to post and posting limits become judgment based.
Massachusetts Department of Transportation	In general, bridges are posted at the inventory or inventory +10% allowable stress level. However, higher allowable stress levels are occasionally used at the discretion of the Bridge Engineer to determine the posting level.
Minnesota Department of Transportation	Sometimes for 1.0<RF<1.1 the bridge is posted at the legal limit (24-40-40).
Missouri Department of Transportation	The HS20 truck posting rating is compared to the legal limit of 23T. If rating is less than 23T then bridge is posted. The 3S2 truck posting rating is compared to the legal limit of 40T. If rating is less than 40T then bridge is posted. In commercial zones, if MO5 operating rating is less than 70T then bridge is posted.
Nevada Department of Transportation	When Operating rating falls below HS 20, bridge is posted at inventory rating total tonnage.
Oklahoma Department of Transportation	Posted if : H rating less than 23 tons. HS rating less than 36 tons. 3-3 rating less than 45 tons.
Puerto Rico Highway and	We use the operating rating for the state legal

Transportation Authority	trucks to establish the posting weight limit. The structural condition of the bridge is also taken into account in order to determine the final weight limit.
Washington State Department of Transportation	If we do not see any significant deterioration in the structural elements of a bridge with RF< 1.0, then we monitor it. If rating factor for any of the legal loads is less than 0.9, then we post the bridge. Posting weight would be the rating factor of the vehicle multiplied by its weight.
West Virginia Department of Transportation	Fracture critical bridges are posted at inventory stress if the ratings are less than H 20 or HS30 (in tons) at Inventory stress. Some low ADT bridges on 65,000 pound routes are not posted unless the ratings are below H 15 at Inventory stress. Non-fracture critical may require posting if the ratings are less than H 20 or HS 30(in tons) Truck or Lane at operating stress. Many non-fracture critical bridges are posted at some level between inventory and operating.
Wisconsin Department of Transportation	Bridges which can not carry the max. weight by statue HS 20. Bridges with a permit rating of 120,000lbs or less are posted to keep off Annual Permit Vehicles.



Posting Level 1 - Inventory Stress
Posting Level 2 - Inventory Stress + 1/3 (Operating - Inventory)
Posting Level 3 - Inventory Stress + 2/3 (Operating - Inventory)
Posting Level 4 - Operating Stress

* Condition Rating From NBI Items 59, 60 and 62.

Level of Enforcement	Degree of Enforcement of Load Limit
1	Vigorous enforcement of weight limit (Interstate, US13 and US113)
2	Moderate enforcement of weight limit (Delaware and US Routes except for those in Level 1)
3	Minimal enforcement of weight limit (usually local roads)

Figure 12 – Delaware Load Posting Flow Chart

Colorado Posting Trucks

Vehicles used to determine Posting Ratings of bridges subject to Colora
Loadings.

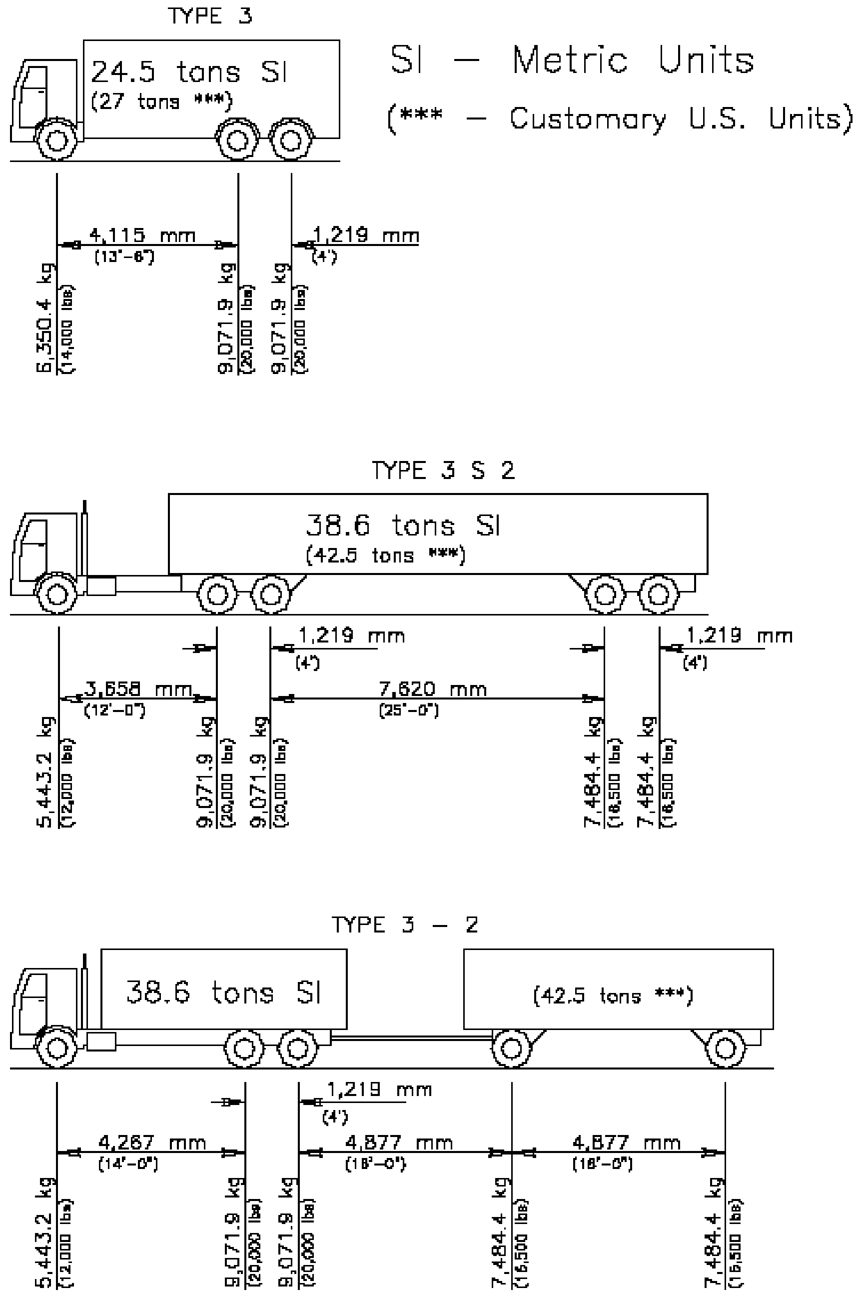


Figure 13- Colorado State Posting Trucks

Interstate Posting Trucks

Vehicles used to determine Posting Ratings of bridges subject to Interst
Loadings.

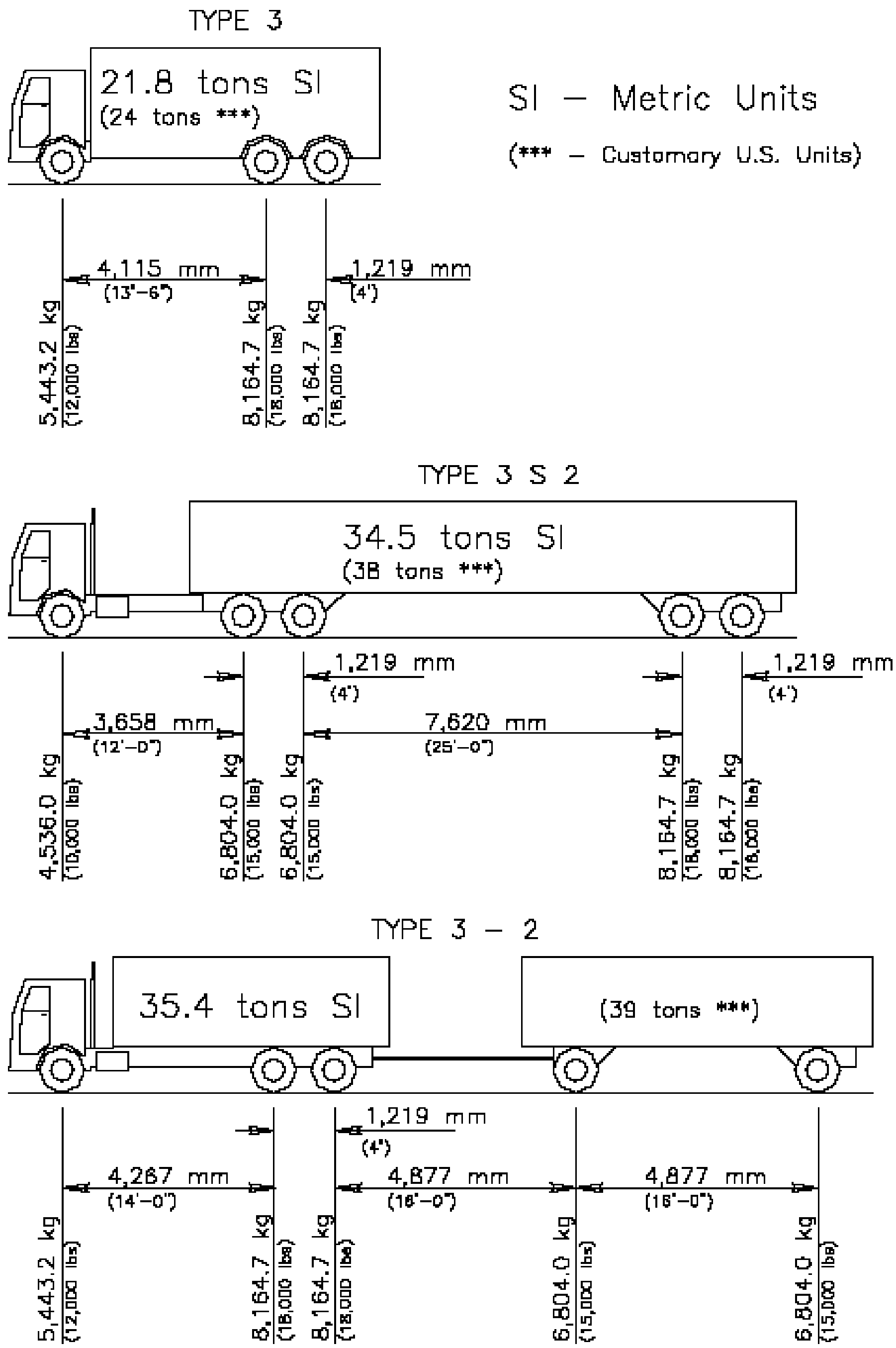


Figure 14- Interstate Posting Trucks used by Colorado State

Analysis Methods

For multi-girder bridges, the rating analysis usually uses the AASHTO tabulated load distribution factors to determine the fraction of the total load effect that is being carried by an individual member. Actually, the load distribution factors provide engineering approximations of the stress distribution in a structure. The conservative assumptions implicit in these tabulated factors could imply that the actual load carrying capacity of a bridge may be much larger than that calculated [Puckett; 2001]. Recent studies have shown the limitations of the AASHTO load distribution factors and proposed alternative formulas for different bridge categories [Puckett et al; 2005, Shahawy and Huang; 2001]. In many cases, refined structural analysis programs or load testing may be used for load rating especially when initial analyses may require the posting of a bridge that is not showing signs of distress. Analysis methods used by different states include the girder line method, the grillage analysis method, and the finite element method.

BRASS and Virtis are programs that are widely used for load rating and load posting processes using the girder analysis method. BRASS and Virtis, which uses the BRASS LRFR engine, have added LRFR capabilities in their recent releases. Various versions of BRASS are used by CO, IL, KS, OH, AZ, DE, NV, OR and PR, while Virtis is used by AZ, CO, ID, IL, IA, MA, MN, MT, NY and TN. Some states also use BARS (e.g. CO, FL, ID, IL, IN, IA, MO, OH, SC, SD, TN, VA States). Computer software programs used for finite element analysis and grillage analysis include BRUFEM (used by AL, FL States), MDX (used by CT, FL, MN States), GT-STRUDL (used by AZ, CT States), and STAAD (used by CT, MA, TX, VA States). [Fu and FU, 2006]

Posting Signage Practices

The AASHTO MCEB Manual requires that the standards maintained in the *Manual on Uniform Traffic Control Devices (MUTCD)* be followed when placing regulatory signs on posted bridges. The current issue of MUTCD recommends five standard signs that can be used for bridge posting (Figure 15). Signs may specify a single truck weight limit or axle weight limit, or weight limits tied to truck types. The recommended sign, labeled as R12-5, with truck symbols does not always give precise definitions of the axle configurations and may have to be modified slightly to conform to local regulatory statutes.

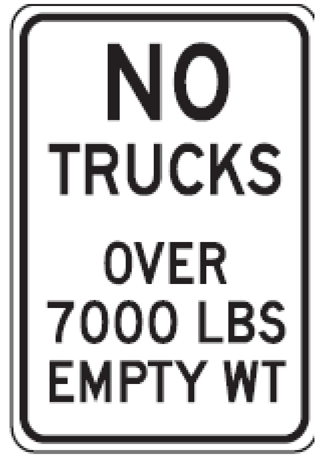
The survey conducted by Sivakumar et al (2007) as part of NCHRP 12-63 shows that 27 agencies (AL, AK, AR, CA, CO, CT, FL, GA, ID, IN, IA, KS, KY, MA, MN, MS, MO, NE, NJ, NM, OK, OR, PR, RI, TN, VA, WA, WY) use truck symbols on posting signs. Eleven agencies did not use truck symbols (AZ, HI, IL, NV, NH, NY, NC, OH, SC, TX, WI) while two agencies (VA and WV) use both types. Representative examples of posting signs used by various states are given in Figs. 16 through 21. Most states seem to prefer the R12-5 sign with the truck symbols or some similar modified version (shown on Fig. 17, 19, and 20). The next most popular sign is R12-1 that limits the gross weights of vehicles or a modified version that shows weight limits for single-unit trucks and vehicle combinations (Fig. 16, 18). Alaska uses axle loads for posting for maximum loads that a deck can carry with R12-2 signs (Fig. 21). Stringers and floorbeams are also evaluated for axle loads along with the three posting vehicles. North Carolina uses a large number of rating and posting loads but the policy is to post for no more than two weight limits per bridge. These postings are for single vehicle and truck-tractor semi-trailer (Fig. 16). Georgia uses both R12-1 and R12-5 signs (Figure 17). Similarly, in Florida the weight limit shown on the posting sign represents the gross vehicular weight in tons for a maximum of three truck types: Single unit trucks, combination trucks with a single trailer, and combination trucks with two trailers or a single unit truck with one trailer. Alabama gives a whole range of truck configurations (Figure 19) while Iowa uses a combination of truck weight limits per type and axle weights (Figure 20).

One of the drawbacks in having many unusual truck configurations as posting vehicles is that they could cause confusion to the truckers even if represented pictorially on posting signs. It may be evident that as these unusual state legal truck configurations proliferate, the need to have exact truck representations as posting vehicles becomes somewhat less important as it would not be possible to show most of them on posting signs. What's more important is that the legal load models utilized in the posting analysis adequately envelope the load effects induced by these unusual trucks. Envelope vehicles and safe load capacities (determined by vehicle class or type) may allow simplification of the posting issue. [Sivakumar et al, 2007]

The survey by Sivakumar et al (2007) indicates that most states post some or all of their bridges for a single gross tonnage for all truck types. The states that did not use a single gross tonnage at all include AR, GA, ID, NJ, NM, OH, TX, WA, and WY. The responses to the survey show that in most cases the single gross tonnage is used when the posting is quite low as determined by the lowest tonnage rating vehicle or the H truck. This simplifies the posting and is conservative for these low rated bridges. Single tonnage posting appears to be more common on locally owned bridges than on State owned bridges. A larger inventory of low-rated bridges is on the local system. States use various lower limits (in Tons) of load capacities for triggering a single tonnage posting. For example, California's uses single tonnage posting when the posting is below 10 T. Other state limits are Illinois 15T, Kansas 12T, Massachusetts 6T, New Hampshire 10T, Oregon 5T and Rhode Island 10T. This practice is also used by Minnesota and Colorado for posting very short simple span bridges, which are controlled by single axle or axle groups rather than a whole truck.

R12-1

R12-2



R12-3

R12-4

R12-5

Figure 15 - Typical Posting Signs

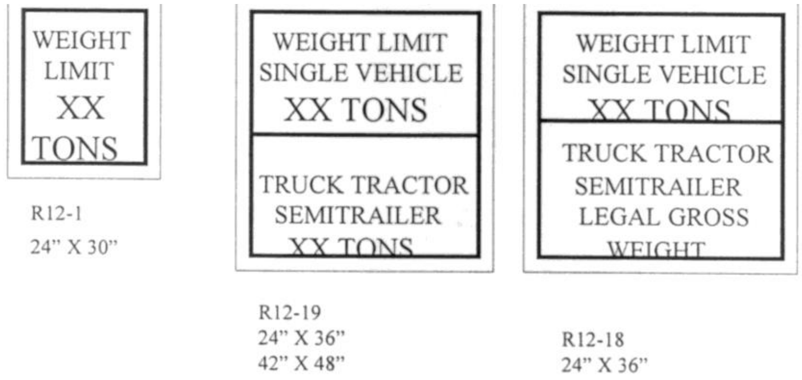
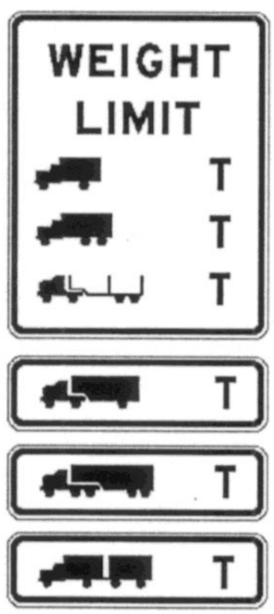


Figure 16 - North Carolina Posting Signs

R12-1
GROSS WEIGHT LIMIT SIGN
(Type A)



MODIFIED R12-5
WEIGHT LIMIT SYMBOL SIGN
(Type B)

	Gross	Tare
H-TRUCK	21	5
TYPE 3	33	7
TIMBER	~~36~~ 37	10
HS-TRUCK	30	10
3S2	40	15
PIGGYBACK	40	20

Figure 17. Georgia Posting Signs

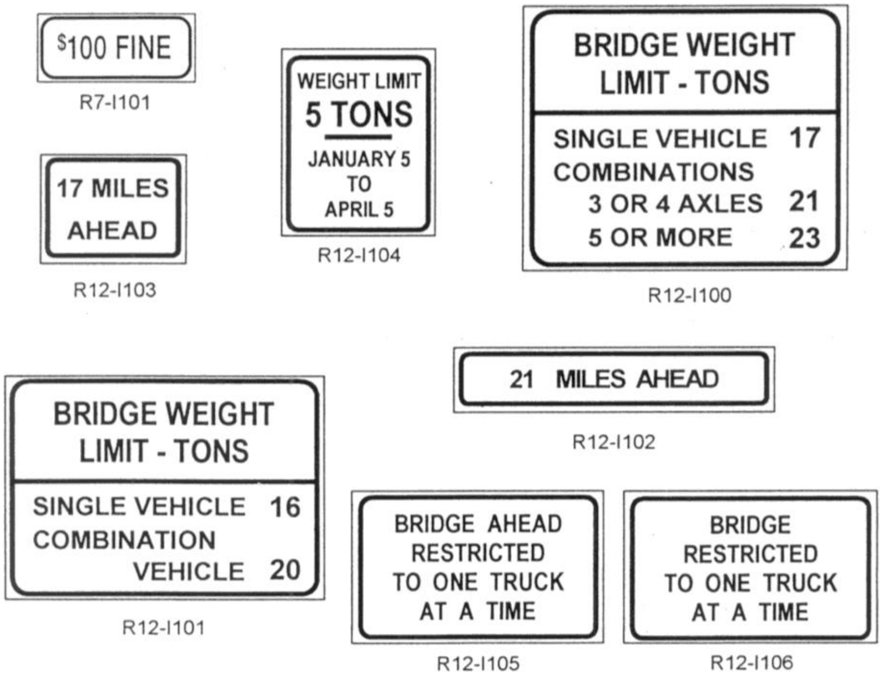


Figure 18. Illinois Posting Signs

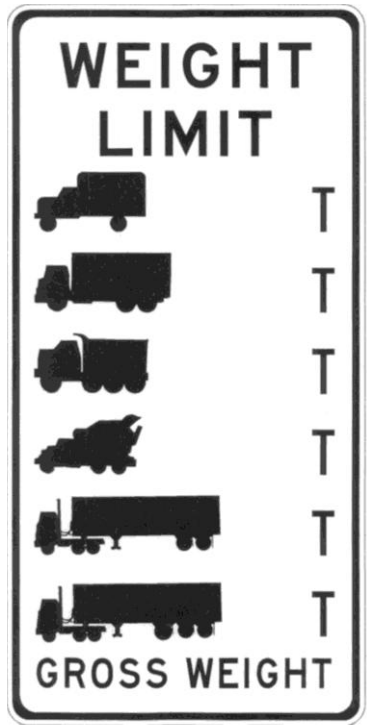
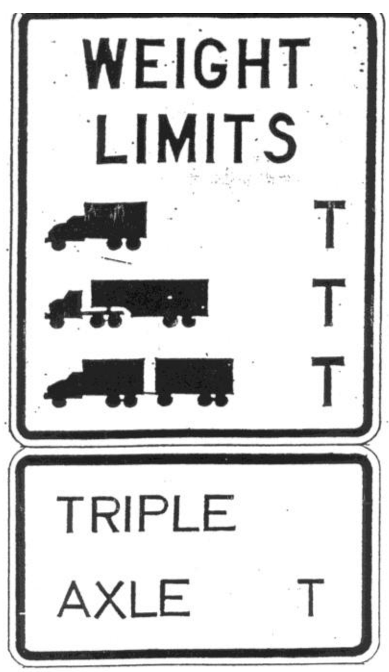
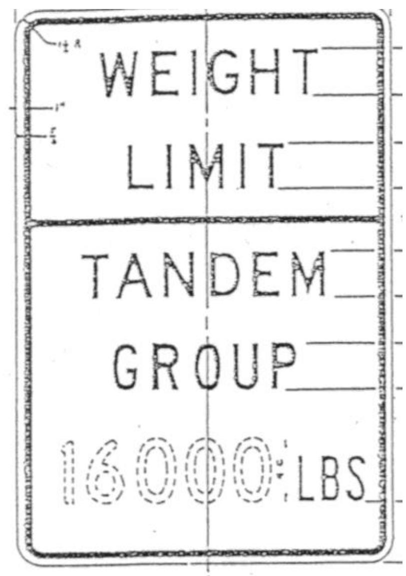


Figure 19. Alabama Posting Sign



NOTE: Triple Axle sign is optional for non-interstate highways.

Figure 20. Iowa Posting Sign



R12-5A

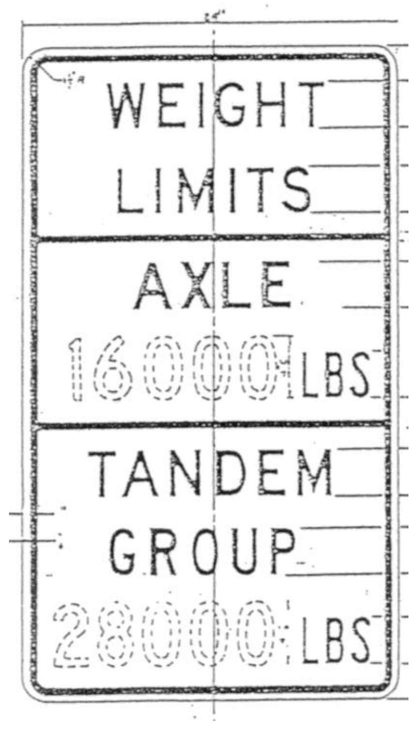


Figure 21 Alaska Posting Sign Showing Axle Weight Limits

Load Testing

Load testing is often used to verify and improve on the results of analytical analyses particularly for critical bridges that initial analysis may require to be load posted. Load testing is usually recommended in the following cases:

(1) When analytical results lead to posting a bridge that is otherwise showing no sign of distress (e.g. Chajes, Mertz & Commender; 1997).
(2) For bridges that are difficult to analyze (see for example, Cai. Shahawy and El-Saad; 1999 or Huang; 2004).
(3) For bridges for which plans are not available (see for example, Boothby, T.E. and Craig, R.; 1997 or Shenton and Huang; 2005) or for the cases where deterioration or other changes may have altered the behavior of the bridge (Fu et al, 1997, Chajes et al, 2000 or Phares et al, 2005).
(4) When calibrating analytical models for simplified or advanced analysis models including the consideration of load distribution, end fixities or composite action (see for example, Chajes et al 1997, Schenk et al, 1999, or Yost, Schulz and Commander; 2005)

Section 8 of the LRFR Manual entitled "Non-Destructive Load Testing" covers the incorporation of field load testing results into the load rating process. The procedures in Section 8 cover diagnostic and proof load tests and provide a systematic method for using the benefits of load testing when the changes in behavior can be understood and explained. The recommendations are based on several studies that developed approaches for load rating of bridges using field measurements (see for example Ghosn, et al, 1986) and methods to apply the results of diagnostic and proof load testing [Fu and Tang; 1992, and Lichtenstein, Bakht and Moses; 1998]. Proposals have also been made to use strain measurements at specific locations of a bridge to obtain field-based ratings of bridges (see for example Bhattacharya, B. et al; 2005 and 2007). However, such methods can only perform the evaluation at specific locations of a bridge and the results cannot be generalized to other locations of the same bridges or for other bridges.

The survey conducted by Klaiber, Wipf, and Russo (2004) for NCHRP synthesis study 327, found that 18% of the responding states use load testing for evaluating existing bridges while 59% of the states use computer modeling techniques to supplement the use of the traditional AASHTO MCEB methods. Load testing is used primarily in situations where a structure could not be rated otherwise or for lack of confidence in the analytical results. Examples given include the load testing of stone arch bridges, covered timber bridges and steel trusses when the rating is unavailable analytically or is suspect.

The results of the survey by Klaiber et al (2004) confirm the results of a previous survey of State DOTs done in 1997 as part of NCHRP 12-46 (Lichtenstein; 2001). This 2001 report distinguishes between two types of load tests: proof load testing and diagnostic testing. As explained by Cai and Shahawy (2003), the difference between these two types is the load level and the strategy used to interpret the results. For proof loading, the load is increased incrementally up to a target live load value and the rating established is a lower bound of the true loading capacity of the bridge. A diagnostic load test is used to compare actual bridge response to analytical results. Information obtained from this test is used to validate the analytical assumptions and determine whether the analytically predicted capacity is accurate.

Thirteen of the 32 surveyed states by NCHRP 12-46 utilize load testing as a means of evaluating bridges. Eleven states have used diagnostic testing while six have used proof load tests. Fourteen states reported using test results to reevaluate load ratings. Thirteen states used the test results to reevaluate the need for posting. Seven states used the tests to review overload capacity and permits. Seven states used tests to evaluate bridges with unknown reinforcement. Three states used tests to evaluate fatigue prone details. Six states used test results to determine impact factor. Ten states used tests results to determine live load distribution and ten states used tests to assess bridge strength capacity. However, neither the NCHRP 327 Synthesis nor the NCHRP 12-46 report provides specific information on which states were surveyed and categorized.

2.3 Implementation of LRFR for Load Posting

Florida LRFR

The Florida LRFR [FDOT exceptions to LRFR, 2008] uses the design-load operating rating as a first step in determining the need for load posting. Bridges that produce design load operating rating RF<1.0 must be rated for the SU4, C5 and ST5 Florida DOT Legal loads. These trucks are placed in each loaded lane without mixing the trucks. For negative moment loading and for long spans the AASHTO LRFR loading is applied. However, the live load factor is specified as γ_L=1.35 for all traffic volumes.

Oregon LRFR

The Oregon LRFR recalibrated the live load factors for the Oregon DOT legal loads based on the WIM data collected within the state. Two sets of factors are specified depending on the ADTT as shown in Table 10. One set is for state-owned bridges and the other set is for local agency bridges. The same live load factors are valid for the regular legal loads and the SHV trucks. The SHV ratings are only used for informational load rating and not for load restriction purposes. Note that the 3S-2 Oregon DOT legal vehicle is different than the AASHTO 3S-2 as shown in Figure 9. [ODOT LRFR Manual, 2008].

Table 10. Oregon DOT Legal Load factors

Traffic Volume (one direction)	γ_L for State owned bridges	Traffic Volume (one direction)	γ_L for Local agency bridges
Unknown	1.40	Unknown	1.80
ADTT $\geq$ 5000	1.40	ADTT $\geq$ 5000	1.80
ADTT=1500	1.35	ADTT=1000	1.65
ADTT $\leq$ 500	1.30	ADTT $\leq$ 100	1.40

Wisconsin LRFR

The Wisconsin LRFR specifies a load factor γ_L=1.80 for all the AASHTO Legal vehicles including lane loads for all ADTT. A live load factor γ_L=1.60 is specified for the four Specialized Hauling Vehicles SU4, SU5, SU6 and SU7.

According to the WisDOT Bridge Manual (2007), a bridge should be capable of carrying a minimum gross live load weight of thee tons at the Inventory level. Bridges not capable of carrying a gross live load of three tons at the Operating levels must be closed. The decision on closing or posting must consider traffic volume, the character of traffic, the likelihood of overweight vehicles and the enforceability of weight posting. Multiple lane distribution factors are used for posting analysis when the bridge width is 18ft or larger. Single lane distribution factors are used for bridge widths less than 18 ft. To calculate the capacity in tons for a posting vehicle having RF<1.0, the gross vehicle weight is multiplied by RF. For State Trunk Highway Bridges, the structure is posted with a Wisconsin Standard Permit Vehicle (Wis-250 shown in Figure 22) rating of 120 kips or less utilizing a single lane distribution factor. When lane loads govern the load rating, the equivalent weight for use in calculating the safe load capacity for the bridge is taken as 40 tons. The posted weight limit in tons is for the lowest restricted weight limit of the standard posting vehicles.

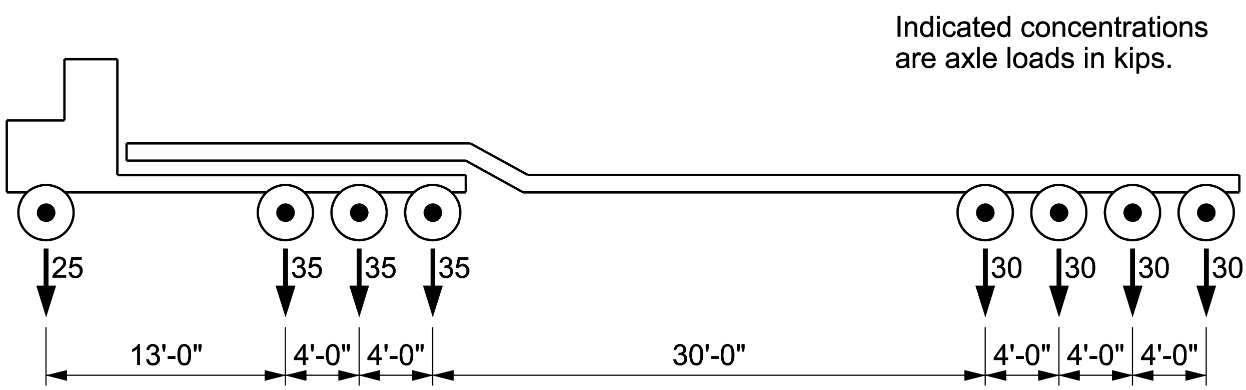


Figure 22. Wisconsin DOT Wis-250 Permit Truck

Michigan LRFR

The recommendations made for the future implementation of the Michigan LRFR are based on the fact that the Michigan Legal loads produce load effects considerably higher than those of the Exclusion Vehicles with a ratio as high as 1.34 for certain truck configurations. The calibration of live load factors for Michigan Legal Loads is thus based on the AASHTO LRFR recommendation that state legal loads significantly heavier than the AASHTO Legal Loads should be load rated using the load factors specified for routine permits. Accordingly all trucks greater than 100-kips are labeled as Legal-Heavy vehicles using the live load factors of Table 5. [Curtis and Till, 2008]

Hawaii LRFR

The Hawaii LRFR bases all posting decisions on the results of field inspections and LRFR load rating. A bridge that cannot carry the maximum weight for the legal vehicles must be posted. The bridge is posted for only one tonnage capacity for the governing vehicle. The Hawaii LRFR posting load is calculated using Eq. 3. The live load factors for the AASHTO Legal loads, lane loads and SHV loads are the same as those specified in the AASHTO LRFR. [Sivakumar, 2008]

3. Overweight Load Permitting

3.1 General Considerations

Load permitting is the process followed by bridge owners to allow the passage on highways and bridges of vehicles that exceed the established legal weight limits. Different types of permits are issued, including single-trip permits, multi-trip permits, annual permits, and radius permits.

The MCEB is vague on the issue of permit evaluation. It leaves it up to the states to decide which types of permits are to be evaluated and which methods are used for the evaluation process. In practice, most states determine whether a permit truck load rating analysis would be required based on the magnitude of the overload.. Single trip permits for heavy loads may have certain conditions and restrictions imposed to reduce the load effect. Such restrictions include: 1) the use of escorts to restrict all other traffic from the bridge being crossed, 2) requiring the permit vehicle to be in a certain position on the bridge to reduce the loading on critical components 3) requiring crossing at crawl speed to reduce the dynamic load allowance.

According to the LRFR, and as shown in the flow chart provided in the AASHTO LRFR Guide Manual (2003), permit load rating should be used only if the bridge has an operating rating factor greater than or equal to 1.0 when evaluated for the AASHTO HL-93 design loads or the legal loads. The LRFR recognizes two permit types:

c) Routine (or Annual) Permits.

These are valid for unlimited trips for a period not to exceed one year. These permits should not exceed 150 kips.

d) Special (or Limited Crossing) Permits.

These are usually valid for a single trip or a very limited number of trips. They may require the use of escorts to restrict other traffic from the bridge being crossed; the crossing along a certain position on the bridge; and/or reduced speed to minimize the dynamic effects. Special Permits are allowed for bridges having a rating factor RF>1.0 for the legal loads or the AASHTO HL-93 design load.

For spans up to 200-ft in length, only the effects of the permit vehicle are considered in a lane. An additional lane load of 0.2kip/ft should be added to the permit load for spans longer than 200-ft or when checking negative moments of continuous spans. The permit live load factor, γ_L, used in conjunction with Eq. (1) varies between 1.10 and 1.85 depending on the permit type; frequency of permit crossings; whether the permit vehicle will be escorted or whether other vehicles may cross the bridge simultaneously; the average daily (ADTT) truck traffic on the bridge; and the gross weight of the permit vehicle. Table 11 shows the LRFR live load factors for permit loads.

For routine permits, the distribution factor for two or more lanes should be used. For special permits, the one-lane distribution factor of the AASHTO LRFD is used after removing the built-in multiple presence factor by dividing the tabulated LRFD girder distribution factor by MP=1.2. The dynamic factor specified for the general load rating procedure is used unless the vehicle speed is restricted to less than 10 Mph, in which case the dynamic amplification factor is eliminated.

The LRFR procedures provide a reliability-based approach for permit review with specially calibrated load factors. This approach avoids using the same conservative design load factors for permit checking where the level of uncertainty is far different from that assumed in new designs and random truck loading. The LRFR Manual provides procedures for checking overweight trucks that are analogous to load rating for legal loads except that the live load factors are selected based upon the permit type. The permit live load factors were derived to account for the possibility of simultaneous presence of non-permit heavy trucks on the bridge when the permit vehicle crosses the span. Thus, the load factors are higher for spans with higher ADTT and smaller for heavier permits. The calibration of permit load factors is also tied to the live load distribution analysis method with the one lane distribution used for heavy special permits. [Moses, 2001]

The target reliability level for routine permit crossings is established as the same level as for legal loads, consistent with traditional AASHTO Operating Ratings. For single and multiple-trip special permits that are allowed to mix with traffic (no restrictions on other traffic) the live load factors were explicitly derived to provide a higher level of reliability consistent with AASHTO Inventory Ratings. The increased risk of structural damage and associated benefit/cost considerations lead to higher safety requirements for uncontrolled very heavy special permit vehicles than for other classes of trucks. The live load factors for single trip escorted permits that are required to cross bridges with no other vehicles present have been calibrated to reliability levels consistent with traditional AASHTO Operating Ratings. Target reliability at the Operating level is allowed because of the reduced consequences associated with allowing only the escorted permit vehicle alone to cross the bridge [Moses, 2001]. An agency may also elect to check escorted permits at the higher design or Inventory level reliability by using an increased live load factor as noted in the LRFR Manual commentary.

Table 11. LRFR Permit Load Factors γ_L

Permit Type	Frequency	Loading condition	DF[a]	ADTT (one direction)	Load factor by Permit Weight[b]	
					Up to 100 kips	≥150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.80	1.30
				=1000	1.60	1.20
				<100	1.40	1.10
				All weights		
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.50	
				=1000	1.40	
				<100	1.35	
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.85	
				=1000	1.75	
				<100	1.55	

Notes

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

[b] For routine permits between 100 kips and 150 kips interpolate the load factor by weight and ADTT value.

3.2 Current Practice

Permit Types

Most states have not yet adopted the LRFR is their permit review process leading to large differences in the approaches they follow [Fu and Fu, 2006]. Different states divide their permits into different categories, but most commonly, permits are separated into single trip permits and annual (or blanket) multi-trip permits. For example, New York State divides its permits into two main categories: a) Divisible Load Permits and b) Special hauling. Special hauling permits are in turn divided into Trip Permits or Yearly Permits. The Yearly Permits include: Annual Crane Permits; 5, 25 and 100-mile Radius Permits; Blanket Permits; and Emergency Blanket Permits. In Florida, there are two kinds of permits: 1) a trip-basis permit, which is issued to cover a trucker's move from point of departure to destination only on one particular trip; 2) Blanket permits, which are issued to operators who need permits for a specific period of time, not to exceed 12 months. Similarly, Oregon and Minnesota define two types of permits identified as Continuous Trip Permits (CTP) and Single Trip Permit (STP) in Oregon, and Single Trip permit and multi-trip permit in Minnesota. Ohio separates its permits into several permit types, such as: Single Trip, Round Trip, 90 day Continuing permit, 365 day Continuing permit, Construction Equipment permit, Farm Equipment permit, Manufactured Building permit, Steel Coil permit, Toledo Port Area permit, Delta Steel Complex permit, Marina permit, Emergency permit, and Ohio Turnpike permits.

Permit Weight Criteria

Permits are required for trucks that exceed the state legal limits. Most legal limits conform to the Federal bridge Formula (FBF) although in some instances, grand father exemptions are applied. For example, according to NYSDOT regulations, a vehicle is considered overweight if it does not conform to the weight and axle spacing limits stipulated in Title III, Article 10, Section 385 of the New York State Vehicle and Traffic Law. Legal weight vehicles are generally defined as vehicles with 3 or more axles weighing a total of 34,000 lbs plus 1000 lb per ft measured from the first to the last axle. The total weight of legal vehicles should be less than 80,000 lbs and the maximum axle weight cannot exceed 22,400 lbs. If the vehicle's gross weight is less than 71,000 lbs, the higher value from the above stated limit or the limit imposed by the AASHTO Bridge Formula B will govern. The Federal bridge Formula (FBF) gives the limits tabulated as shown in Table 12. Similarly, in Delaware, a permit is required if the gross weight of vehicle and load or vehicle combination and load exceeds the limits for which the vehicle combination are licensed or exceeds the limits imposed by statute as shown in Table 13.

Table 12. FBF Maximum Gross Weights

Bridge Formula Maximum Gross Weights For The Interstate System

Distance in feet between the extremes of any group of 2 or more consecutive axles	Maximum Load in Pounds Carried on Any Group of 2 or More Consecutive Axles					
	2 axles	3 axles	4 axles	5 axles	6 axles	7 axles
4	34,000					
5	34,000					
6	34,000					
7	34,000					
8 and less	34,000	34,000				
more than 8	38,000	42,000				
9	39,000	42,500				
10	40,000	43,500				
11		44,000				
12		45,000	50,000			
13		45,500	50,500			
14		46,500	51,500			
15		47,000	52,000			
16		48,000	52,500	58,000		
17		48,500	53,500	58,500		
18		49,500	54,000	59,000		
19		50,000	54,500	60,000		
20		51,000	55,500	60,500	66,000	
21		51,500	56,000	61,000	66,500	
22		52,500	56,500	61,500	67,000	
23		53,000	57,500	62,500	68,000	
24		54,000	58,000	63,000	68,500	74,000
25			58,500	63,500	69,000	74,500
26			59,500	64,000	69,500	75,000
27			60,000	65,000	70,000	75,500
28			60,500	65,500	71,500	76,500
29			61,500	66,000	71,500	77,000
30			62,000	66,500	72,000	77,500
31			62,500	67,500	72,500	78,000
32			63,500	68,000	73,000	78,500
33			64,000	68,500	74,000	79,000
34			64,500	69,000	74,500	80,000
35			65,500	70,000	75,000	
36			66,000	70,500	75,500	
37			66,500	71,000	76,000	
38			67,500	71,500	77,000	
39			68,000	72,500	77,500	
40			68,500	73,000	78,000	
41			69,500	73,500	78,500	
42			70,000	74,000	79,000	
43			70,500	75,000	80,000	
44			71,500	75,500		
45			72,000	76,000		
46			72,500	76,500		
47			73,500	77,500		
48			74,000	78,000		
49				78,500		
50				79,000		
51				80,000		

Table 13- Delaware Maximum Gross Weights

ALL ROADS EXCEPT INTERSTATE	THE INTERSTATE SYSTEM
20,000 lbs. per single axle;	20,000 lbs. per single axle
34,000 lbs. per tandem axle;	34,000 lbs. per tandem axle
2 axle vehicle: 40,000 lbs. maximum;	2 axle vehicle: 40,000 lbs. maximum
3 axle vehicle: 65,000*lbs. maximum;	3 axle vehicle: 54,000 lbs. maximum
4 axle vehicle: 73,280 lbs. maximum;	4 axle vehicle: 74,000 lbs. maximum
5 axle vehicle: 80,000 lbs. maximum	5 axle vehicle: 80,000 lbs. Maximum or the Bridge Formula, whichever is less

* 3 axle vehicle registered for 65,000 lbs. may purchase an annual permit ($100) to carry up to 70,000 lbs.

Although all trucks that exceed the weight limits usually require a permit, a bridge rating analysis for the permit load is not required unless some other criteria are exceeded. For example, in New York State all special hauling permits reviewed by the Office of Structures are recommended approved for bridges where the load posting is not exceeded if the load effect of the permit vehicle is less than or equal to 100% of the effect of the HS-20 vehicle. A trip permit is generally approved for bridges that are not “R” or load posted when the load effect is greater than 100% and less than or equal to 150% of HS-20. Otherwise the bridges on the route must be individually evaluated. Other criteria apply for crane permits or for 5-mile radius permits. Other states that use similar screening methods by comparing the load effect of the permit to that of the design vehicle include AZ, AR, CA, CO, GA, HI, MT, NJ, NC, ND, OK, PA, PR, RI, VT, VA and WI. [Fu and Fu, 2006]

Many states use the Gross Vehicle Weight (GVW) or a combination of weight and spacing to screen the loads for permitting. For example, Delaware and Illinois require bridge evaluations for permit trucks with weights over 120,000 lbs. For Iowa, the trigger is set for truck weights over 156,000 lbs while Massachusetts uses 130,000 lbs as a threshold. For New Mexico, the limit is 140,000 lb, for Minnesota it is 145,000 lbs and for New Jersey, Tennessee and Vermont it is 150,000 lbs. States such as South Carolina and Ohio use a comparison between the trucks to previously issued permits in order to decide whether an analysis is required. Other states such as South Dakota and Nebraska perform the analysis using an automated permit analysis system for each bridge on a permit route. [Fu and Fu, 2006]

Dimension limits

In addition to weight limits, some states require permits when a vehicle exceeds dimension limit criteria. For instance, in Florida, except for certain vehicles exempted by law, any vehicle which exceeds the following size will not be allowed to move without a permit:

a. Maximum WIDTH of vehicle or vehicle combination and load exceeds 102" or exceeds 96" on less than 12' wide travel lane.

b. Maximum HEIGHT of vehicle or vehicle combination and load exceeds 13'6" or 14" for automobile transporters.
c. Maximum LENGTH of single-unit vehicle exceeds 40'; trailer of combination unit exceeds 48'; 53' trailer with a kingpin distance which exceeds 41', measured from the center of the rear axle, or group of axles, to the center of the kingpin of the fifth wheel connection; front overhang of vehicle extends more than 3' beyond the front wheels or front bumper if so equipped.

The Delaware dimension limits are listed as follows:

a. For single vehicles, the length limit is set at 40 feet including front and rear bumpers, except for buses, which are allowed with an overall length of up to 45 feet. Maximum overhang limit is set at 6 feet beyond the rear of the vehicle.
b. The overall width of any vehicle or combination of vehicles should not exceed 102 inches.
c. The maximum height of any vehicle or combination of vehicles can not exceed 13 feet 6 inches.

However, these vehicle dimension restrictions are related to operational safety rather than structural safety and no structural analysis is required for issuing the permits.

Permit Load Rating Method, Rating Level and Limit States

The most commonly used method for bridge permit analysis is the LFR method. In the 2005 survey conducted by Fu and Fu (2006), 42 out of the 44 responding state agencies reported that they use the LFR method. Several states, including New York, use the ASR for special cases such as timber and masonry bridges or steel truss bridges.

All the states that responded to the 2005 survey reported using the operating rating level for permit review. The bridge authorities feel comfortable with using the operating rating due to the limited number of heavy permit loads. [Fu and Fu, 2006]

While most states check the safety of the bridge under permit loading for moments and shears, several states such as Alabama, Arkansas, Colorado, Delaware, Georgia, Idaho, Indiana, Iowa, Massachusetts Minnesota, Missouri, New Mexico, Ohio, Oklahoma, Pennsylvania, Rhode Island, South Dakota, Tennessee, Texas, Virginia and West Virginia also check the serviceability. A few states including Nebraska, North Dakota and Wisconsin check only the moments.

All the states reported that they may restrict the permit vehicle's speed to reduce the dynamic impact allowance if necessary.

Permit Load Placement

States differ on how to place the load on a bridge during permit evaluation and how to determine the associated load distribution factor. For example, several states (such as AZ, AR, CO, DE, IN, IA, NV, NM, ND, OK, PR, SD, TX) load only one lane with the permit vehicle. Whereas some states load other lanes also with the same vehicle (e.g. GA, MN, MO, NY, OH, PA, WV), a few states load the bridge with other vehicles (e.g. CA, FL). A number of states (such as AL, IL and WY) consider different loading options such as permits in both lanes, permit alone, or permit plus other vehicles depending on the permit type. On the other hand, most states would consider loading only one lane with the permit when special controls are applied. Such controls include restricting the position of the permit truck on a bridge and/or escorting the permit to ensure that no other vehicles cross the bridge when the permit is on.

Application of Field Measurements

Bridge monitoring studies have demonstrated that actual measured stresses are generally much lower than those projected from analysis and that most bridge types exhibited good load distribution during heavy truck crossings. Numerous studies have been performed to identify the main reasons for the lower stresses observed in field measurements as compared to analytical calculations. The main factors were found to be the significant contributions of secondary members (including diaphragms curbs, railings and barriers) as well as bearing restraint forces that produce support fixities, and compressive deck slab membrane action. Also, composite action was observed in bridges with members designed as non-composite. Many of these factors affected the load distribution even when the load effects were restricted to the linear elastic range. Some examples of such studies include the research performed by Barker (1995) Schenk et al (1999), Aktan et al (1995), Cai et al (1999), Hag-Elsafi and Kunin (2004), Culmo, DeWolf and Delgrego (2004), Phares et al (2005). For example, Culmo et al (2004) observed that for a typical continuous composite steel bridge under heavy permit vehicles exhibited stresses less than one half the estimated stresses. Similarly, Aktan et al (1995) showed that the Operating Rating Factors obtained based on field data for three steel-stringer bridges may actually exceed by about 2.5 to 4 times those obtained using traditional analysis methods. Culmo et al (2004) also noted that a highway bridge subject to heavy vehicular loads can be analyzed to obtain conservative results using a conventional line girder analysis using the AASHTO Guide Specification for Distribution of Loads for Highway Bridges when the vehicles are composed of singlewide trailers. For doublewide trucks, the authors recommend to model the truck as two side-by-side trucks using the multilane live load distribution factors. Potisuk and Higgins (2007) found that the AASHTO load distribution and the impact factors were conservative compared to the values measured on two reinforced concrete bridges. Hwang et al (2004) also found that the LRFD load distribution factors were conservative for the positive bending moments of bridges with skews of 60^o but that for negative bending the LRFD load factors are accurate but not conservative. A research team in Iowa performed diagnostic load tests on seven typical bridge structures: three steel-girder bridges with concrete decks, two concrete slab bridges, and two steel-girder bridges with timber decks, in addition, to another steel-girder bridge with a concrete deck which was tested in an earlier study. The tests were performed by attaching strain transducers on the bridges at critical locations to measure strains resulting from truck loading positioned at various locations on the bridge. Based on experimental and analytical results, it was determined that the load ratings, in general, were greater than the ratings obtained using the codified LFD Method. Wipf et al (2003). Chajes et al (1999) report that the use of values from load testing of bridges have allowed the Delware DOT to remove load restrictions for a bridge on a major truck route and allowed for the passage of heavy vehicular permit loads on bridges on major truck routes thus eliminating extensive detours. Barker et al (1999) report that analytical capacity rating procedures tend to underestimate the true stiffness and overestimate the response of bridges and that in most cases bridges exhibit capacities higher than analytical load capacity rating predictions. Similarly, Nowak and Eom (2003) carried out field tests on 17 selected short and medium span steel girder bridges using 11-axle truck loads having a total weight of 667 kN. They found that in all the cases, the measured girder distribution factors are well below AASHTO Code specified values whether using the LRFD or the standard specifications. Such observations led some researchers to propose reliability based methods for overload permit checking as proposed by Ghosn and Moses (1987) or Fu and Moses (1991), or Casas (2000).

Modifications to LFR for Permit Loads

Based on the above listed studies and several other studies available in the vast literature on load testing of bridges, the current AASHTO load distribution factors are found to be conservative and may unnecessarily restrict the passage of permit vehicles which may not necessarily be damaging to rated bridges. The differences may be even more pronounced when dealing with permit trucks with non-standard axle widths and configurations. Although load testing may provide accurate estimates of the actual load distribution factors, such tests may be too costly to implement on a routine basis. Therefore, permit load rating engineers have explored different methods to change the AASHTO standard lane distribution factors to accommodate specific loading conditions or to accommodate permit vehicles with nonstandard axle gage widths and spacing configurations. For example, researchers such as Goodrich and Puckett (2000) or McLelland (2003) developed simplified procedures to estimate the load distribution factors for nonstandard vehicles which are used by some states such as Alabama and Kansas. Some states such as Arkansas Georgia, Hawaii, Mississippi, New Jersey, Oklahoma simply use the one lane or multi-lane standard AASHTO distribution factors. Others such as New York and Nebraska modify the AASHTO load distribution factor. For example, New York applies a multiple lane reduction factor based on permit truck width. Other states such as Alabama, Arizona, California, Illinois, Kentucky, Minnesota, Montana Nevada, New Mexico, North Carolina, South Carolina, Texas, West Virginia, Wisconsin and Wyoming may adjust the distribution factors when necessary. The methods used to adjust the load distribution factors may consist of performing an analysis using the lever rule or a transverse girder analysis (e.g. Alabama, Arizona, Montana, South Carolina, and Tennessee). Other states have used specially-derived graphs and charts, interpolation methods or other rules of thumbs to increase the load distribution factor by a certain percentage depending on the vehicle’s width, or number of tires in each axle (e.g.

California, Ohio, South Dakota, New Mexico, Washington State, Wisconsin, Wyoming, and Minnesota).

3.3 Implementation of LRFR

Florida LRFR

The Florida LRFR requires that the LRFD one lane distribution factor be used for single permits and the multiple lane distribution factor be used for routine or annual permits. This approach is similar to that proposed in the AASHTO LRFR. In Florida LRFR all the lanes are loaded by permits and mixed traffic calculations are not performed. For service load calculations for multiple lanes, a multiple presence factor equal to 0.9 is used. The FL-120 permit truck is used for screening bridges designed after Jan. 1 2005. Only bridges showing RF>1.0 for both strength and serviceability under the FL-120 treated as a routine annual permit can be permit-load rated. When performing a refined analysis, the permit vehicle is combined with the same permit vehicle in the adjoining lanes. For spans over 200 feet, a uniform lane load of 0.20 kip/ft should be applied in the same lanes as, but beyond the footprint of, the permit vehicles.

Florida revised the AASHTO LRFR live load factors for all permit types to γ_L=1.35 except for the escorted single trip live load factor which is kept at γ_L=1.15.

Oregon LRFR

Oregon revised the AASHTO LRFR permit live load factors based on the WIM data collected at representative sites within the state. The resulting live load factors for the different permit types and vehicle configurations given in Figure 23 are provided in Table 14. A significant decrease in the load factors is observed for all ADTT categories. The existing AASHTO LRFR factors however are maintained for local agency bridges.

Because ODOT's Motor Carrier Transportation Division (MCTD) issues Single Trip Permits in such large numbers on a routine basis without a specific structural review, they are treated the same as "Routine or Annual" permits. ODOT load rates all bridges for all of the following CTP and STP permits: CTP-2A, CTP-2B, CTP-3, STP-3, STP-4A, STP-4B, STP-4C, STP-4D, STP-4E and STP-5BW. Each load model represents a specific class of permit that operates in large numbers within Oregon. These load models have been added to the BRASS LRFR loads library and analyzed along with other state legal loads on all bridges. A two-lane distribution analysis is used as it is likely that two such permits could be side-by-side. ODOT has calibrated live load factors for each of these permit types. ODOT does not use a pre-screening approach for permits. For future permits that may deviate slightly from these standard permits, the results could be scaled using the permit configuration closest to the new permit.

Table 14.a (Adaptation of upper portion of LRFR Table 6-6) for ODOT Routine Permits

Permit Type	Frequency	Loading Condition	DF[a]	Permit Vehicle	Liveload Factor γ_L by ADTT[b] (one direction)[c]			
					Unknown	≥ 5000	= 1500	≤ 500
Continuous Trip (Annual)	Unlimited Crossings	Mix w/traffic (other vehicles may be on the bridge)	2 or more lanes	CTP-2A	1.35	1.35	1.35	1.25
				CTP-2B	1.35	1.35	1.35	1.25
				CTP-3	1.45	1.45	1.40	1.30
Single Trip	Route-Specific Limited Crossings	Mix w/traffic (other vehicles may be on the bridge)	2 or more lanes	STP-3	1.25	1.25	1.20	1.10
				STP-4A	1.40	1.40	1.35	1.25
				STP-4B	1.00	1.00	1.00	1.00
				STP-4C	1.10	1.10	1.05	1.00
				STP-4D	1.05	1.05	1.05	1.00
				STP-4E	1.00	1.00	1.00	1.00
				STP-5BW	1.00	1.00	1.00	1.00

Use Table 6-6B whenever one-lane Distribution Factors are used. Note: ODOT assumes the multiple-lane loading to always control over the single-lane loading, so this table would only be used in (1) the exceptional case where single-lane loading is shown to govern over multiple-lane loading, or (2) in the "Super-Load" case where the loading is known to be single-lane.

Table 14.b (Adaptation of lower portion of LRFR Table 6-6) for ODOT "Super-load" Permits

Permit Type	Frequency	Loading Condition	DF[a]	Permit Vehicle	Liveload Factor γ_L by ADTT[b] (one direction)[c]			
					Unknown	≥ 5000	= 1000	≤ 100
Special or Limited Crossings (Super-Loads)	Single-Trip	Escorted w/no other vehicles on the bridge	One Lane	Specific	1.15	1.15	1.15	1.15
	Single-Trip	Mix w/traffic (other vehicles may be on the bridge)	One Lane	Specific	1.50	1.50	1.40	1.35

Notes: (Tables 6-6A & 6-6B)

[a] DF = LRFD Liveload Distribution Factor. To mitigate the effects of the Oregon-specific Liveload Factor calibration, ODOT has decided, for state-owned bridges, when a one-lane Distribution Factor controls for an exterior girder, the built-in Multiple Presence Factor for one lane (1.2) should *not* be divided out of the Distribution Factor (this approach is conservative). However, for escorted super-load permit reviews (done only by ODOT personnel), where the national LRFR Liveload Factor of 1.15 applies and a one-lane Distribution Factor controls, ODOT *will* divide out the Multiple Presence Factor to be consistent with the national LRFR code. These adjustments will be accomplished in the coding of the Summary Spreadsheet

[b] Interpolate the Liveload Factor by ADTT values. Liveload Factors from this table should not be used when advanced methods of analysis are employed.

[c] If there are two directions of traffic, use only half of the structure ADTT (use one direction) to determine the Liveload Factors.

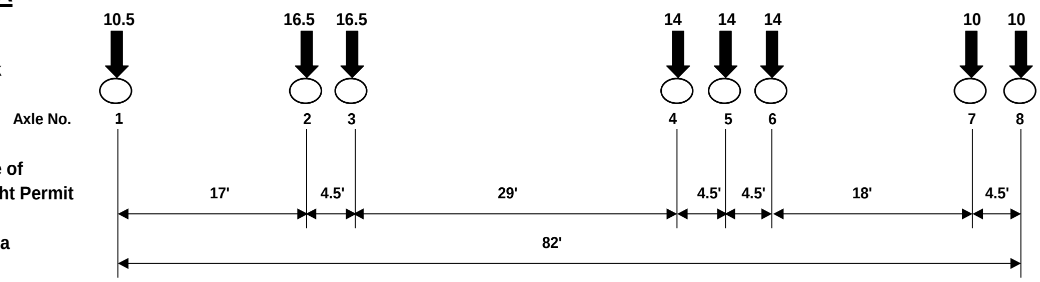
OREGON CONTINUOUS TRIP PERMIT (CTP) LOADS - Load Rating Tier-2

Indicated concentrated loads are axle loads in kips

Type OR-CTP-2A

8 Axle Vehicle
Gross Weight = 105.5 k

Representative Sample of
Annual Extended Weight Permit
Weight Table 2
MCTD refers to this as a
"Canadian Mule Train"
(This load was not used in Tier-1)

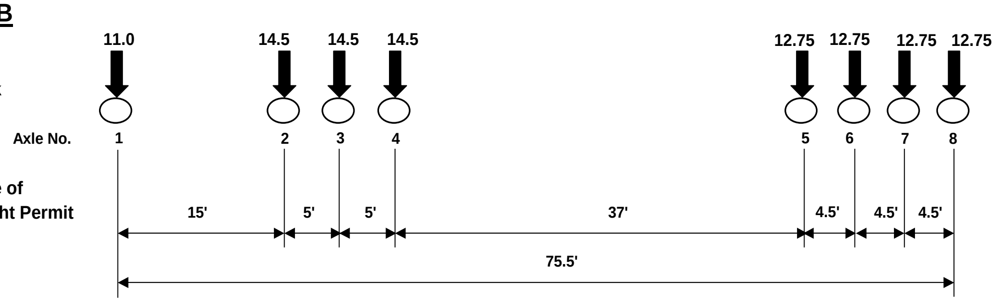


Type OR-CTP-2B

8 Axle Vehicle
Gross Weight = 105.5 k

Representative Sample of
Annual Extended Weight Permit
Weight Table 2
Maximum 4-axle group

(This load was not used in Tier-1)



Type OR-CTP-3

5 Axle Vehicle
Gross Weight = 98 k

Representative Sample of
Annual Heavy Haul Permit
Weight Table 3

(Similar to "Permit-1" in Tier-1)

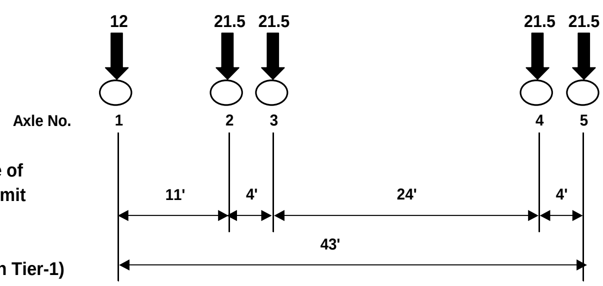


Figure 23.a Oregon Continuous trip permit trucks

OREGON SINGLE-TRIP PERMIT (STP) LOADS - Load Rating Tier-2

Revised May 12, 2006

Indicated concentrated loads are axle loads in kips

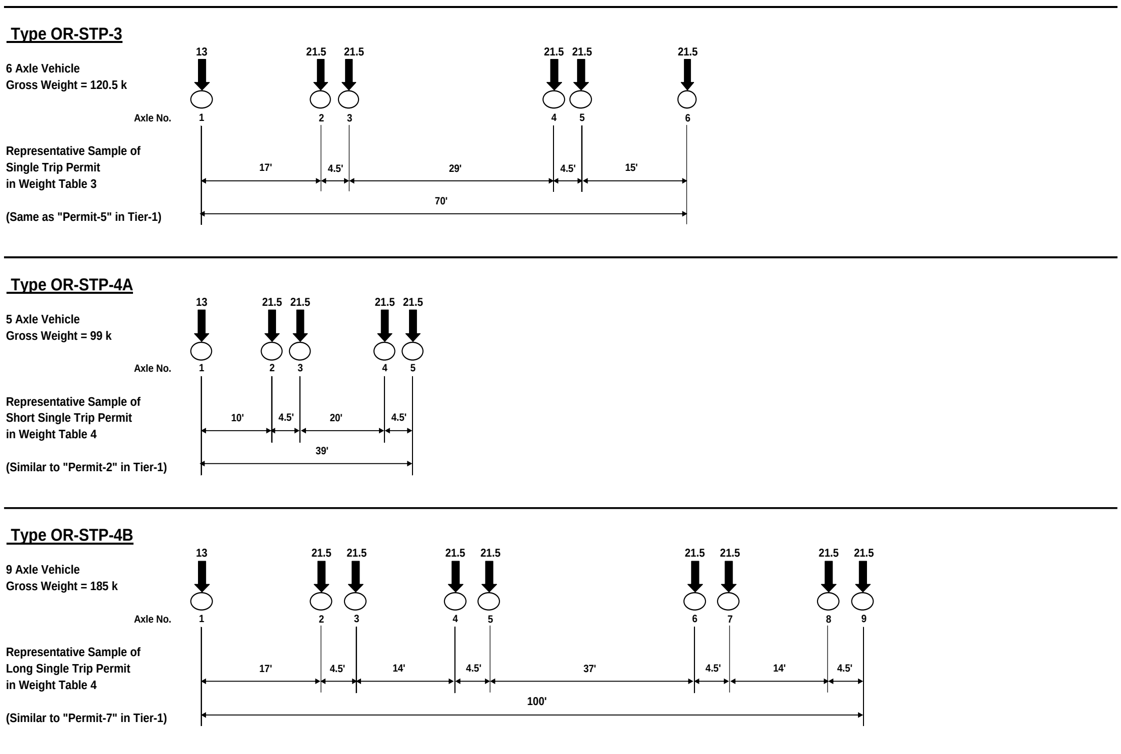


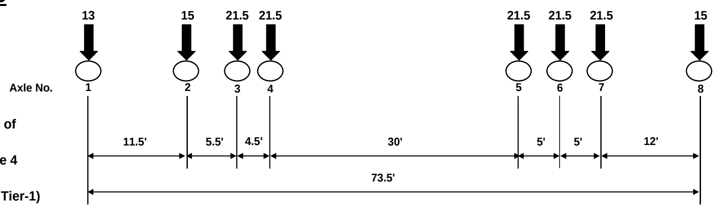
Figure 23.b Oregon Single trip permit trucks

Type OR-STP-4C

8 Axle Vehicle
Gross Weight = 150.5 k

Representative Sample of
Single Trip Permit
in Revised Weight Table 4

(Same as "Permit-6" in Tier-1)

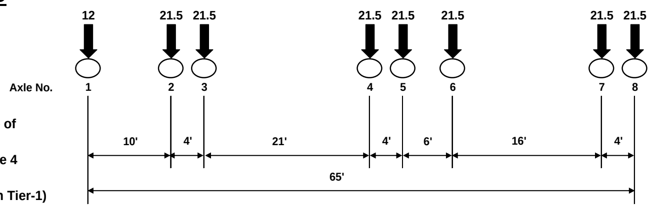


Type OR-STP-4D

8 Axle Vehicle
Gross Weight = 162.5 k

Representative Sample of
Single Trip Permit
in Revised Weight Table 4

(Similar to "Permit-3" in Tier-1)

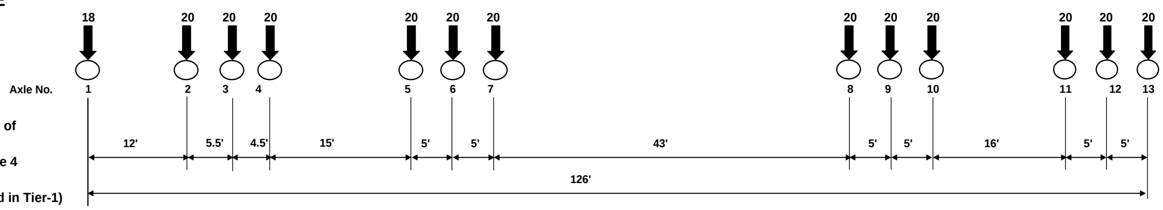


Type OR-STP-4E

13 Axle Vehicle
Gross Weight = 258 k

Representative Sample of
Single Trip Permit
in Revised Weight Table 4

(This load was not used in Tier-1)



Type OR-STP-5BW

9 Axle Vehicle
Gross Weight = 204 k

Representative Sample
Single-Trip Permit
in Revised Weight Table 5
("Bonus Weight" Confiuration)
(Replaces "Permit-4" in Tier-1)

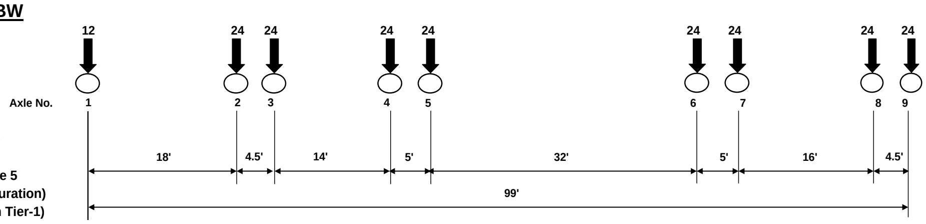


Figure 23.b (Ct'd) Oregon Single Trip Permit Vehicles

Wisconsin LRFR

Permit load rating is the level of load rating analysis required for all structures when performing the Wisconsin Standard permit Vehicle Design Check. The Wis-250 vehicle (fig. 15) is evaluated for both multilane (annual trip permit) as well as single lane (single lane permit) assuming that the vehicle is mixing with normal traffic and that the full dynamic allowance is utilized. An additional check is made to verify that the bridge is able to carry a gross vehicle load of 205 kips utilizing the single trip permit results. In this case, the interior strip or interior girder factors are used. When performing a permit load analysis for negative moments or for spans longer than 200 ft, an additional lane load of 0.2kip/ft in each lane is applied.

Annual Trip Permits: The Wis-250 vehicle is developed to envelope the load effects of all possible annual permit vehicle configurations. In addition, it represents the truck most frequently used to carry loads requiring a single trip permit although some single trip trucks may not be represented by Wis-250. Only non-divisible loads such as machines, self-propelled vehicles are allowed to have Annual Permits. The permit vehicles may mix in the traffic stream and move at normal speeds without any restrictions. The maximum annual permit weight is 170,000 lbs and the axle weight limitations are specified in Table 15.

Table 15: Allowable Axle Weights for Annual Permits

Axle Configuration	Load (Pounds)
Single Axle	20,000 (2 Tires)
Single Axle	30,000 (3 Tires)
2-Axle Tandem	55,000
3-Axle Tandem	70,000
4-Axle Tandem	80,000

In addition, for Single Vehicles, the length limitation is 50 feet and for Vehicle combinations, the length limitation is 75 feet.

Single Trip Permit: Single Trip Permits are issued to non-divisible vehicles, which exceed the annual permit restrictions. All Federal Aid bridges are rated to determine the maximum weight they can carry on a Standard Permit Vehicle (Figure 15). The Standard Permit Vehicle represents the truck most frequently used to carry loads requiring a single trip permit. For other configurations, each single trip permit vehicle is individually analyzed for all structures it encounters on the designated permit route. The live load factors recommended are those of the AASHTO LRFR with ADTT>5000.

The load factors for Permit Load Rating are shown in Table 16.

Table 16: Permit Load Factors for LRFR

Permit Type	Loading Condition	Distribution Factor	Live Load Factor
Annual	Mixed with Normal Traffic	Two or more lanes	1.30
Single Trip	Mixed with Normal Traffic	One Lane	1.50
Single Trip	Escorted with no other vehicles on the bridge	One Lane	1.15

Hawaii LRFR

The Hawaii LRFR load permitting procedure closely follows the AASHTO LRFR with a few modifications to the live load factors. The permits are divided into single trip permits and continuous operation permits.

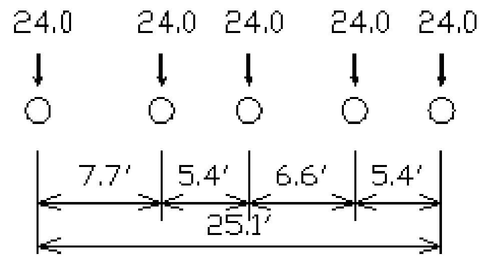
Single Trip Permits: Permits for single trip movements are issued for one-way or round-trip movement of overweight vehicles. Single trip permit analysis is performed for a single lane loading. This is used because these permit loads are infrequent and are likely to be the only heavy loads on the structure during the crossing. When the one-lane LRFD distribution factor is used, the built-in 1.2 multiple-presence factor should be divided out. In special cases, the dynamic load allowance may be neglected provided that the maximum vehicle speed can be reduced to 5 MPH prior to crossing the bridge. Also, in some cases, the truck may be escorted across the bridge with no other vehicles allowed on the bridge during the crossing. In this case, the live load factor can be reduced from 1.5 to 1.15 as shown in Table 17. The set of single trip permit truck configurations shown in Figure 24 were chosen by reviewing past permit applications received by HIDOT and by comparing the load effects induced by the various truck configurations to extract a small number of representative vehicles as Standard Permits.

Continuous Operation Permit: Continuous operation permits are issued for the movement of overweight vehicles over a specified route or within a restricted area for a one year period. Continuous operation permits are usually valid for unlimited trips over a period not to exceed one year. The permit vehicle may mix in the traffic stream and move at normal speeds without any restrictions. The evaluation live-load factors for permits for the Strength II limit state is taken as given below in Table 17.

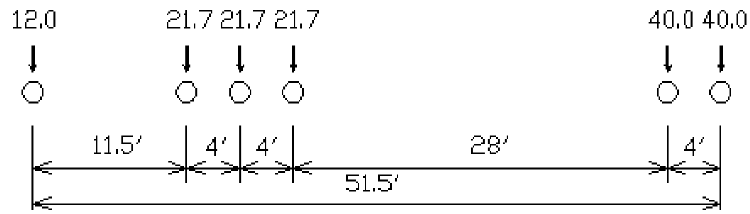
Table 17 Hawaii Permit Load Factors

Permit Type	Loading Condition	ADTT	LRFD Distribution Factor	Live Load Factor
Continuous Operation	Mixed with Normal Traffic	> 5000	Two or more lanes	1.30
Continuous Operation	Mixed with Normal Traffic	1000	Two or more lanes	1.20
Single Trip	Mixed with Normal Traffic	> 5000	One Lane	1.50
Single Trip	Mixed with Normal Traffic	1000	One Lane	1.40
Single Trip	Escorted with no other vehicles on the bridge	N/A	One Lane	1.15

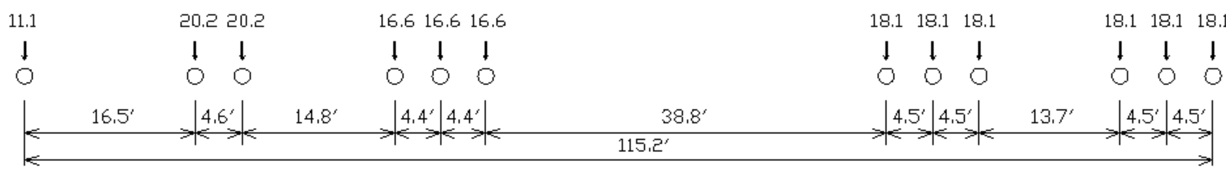
Note: When one-lane distribution factor is used, the built-in 1.2 multiple presence factor should be divided out. Linear interpolation is permitted for other ADTT



Hawaii Permit HP1: 120.0 kips (For Simple Spans Up to 200 Ft)



Hawaii Permit HP2: 157.0 kips (For Simple Spans Up to 200 Ft and Continuous Spans)



Hawaii Permit HP3: 209.9 kips (For Simple Spans > 200 Ft and Continuous Spans)

Figure 24. Hawaii Standard Single Trip Permit Trucks

Michigan LRFR

The recommendations made in preparation of the future adoption of the LRFR method for load permits consist of a set of live load factors calibrated based on the procedure followed by Moses (2001) during the calibration of the AASHTO LRFR. The calculations take into consideration the truck traffic data collected at Michigan WIM sites, the probability of side-by-side events based on typical data from a representative Michigan site, and the type and weights of overload vehicles which are represented into twenty typical configurations and three classes of overloads. As an example, Table 17 shown below provides the recommended live load factors for the evaluation of Class A vehicles for different ADTT. A minimum value of γ_L=1.1 is recommended when the calculations lead to lower values.

Table 17 Michigan Overload Class A Vehicle Load Factors

Michigan Overload Class A Vehicle Load Factors for Strength Limit States				
Vehicle Number	GVW (kips)	5000 ADTT γ_{LL}	1000 ADTT γ_{LL}	100 ADTT γ_{LL}
1	120.0	1.39	1.36	1.30
2	120.0	1.39	1.36	1.30
3	120.0	1.39	1.36	1.30
4	120.0	1.39	1.36	1.30
5	120.0	1.39	1.36	1.30
6	126.0	1.35	1.32	1.27
7	138.0	1.28	1.25	1.20
8	149.6	1.22	1.19	1.15
9	158.4	1.18	1.16	1.12
10	177.0	1.12	1.10	1.10
11	180.0	1.11	1.10	1.10
12	190.6	1.10	1.10	1.10
13	195.0	1.10	1.10	1.10
14	211.2	1.10	1.10	1.10
15	238.0	1.10	1.10	1.10
16	244.4	1.10	1.10	1.10
17	272.6	1.10	1.10	1.10
18	283.4	1.10	1.10	1.10
19	277.2	1.10	1.10	1.10
20	264.0	1.10	1.10	1.10

4. CONCLUSIONS

The review of national practice in load rating, permitting and posting indicates that currently most states use the same basic AASHTO MCBE document for load rating processes. However, major differences are observed in the methods followed to post bridges with low ratings or to provide permits for truck overloads. The AASHTO LRFR proposes a set of load rating, load posting and load permitting criteria which were calibrated to provide uniform levels of reliability so that bridges rated using the legal trucks would provide uniform levels of safety as expressed by a reliability index $\beta=2.5$ for operating level ratings. The reliability calibration of the AASHTO LRFR was based on the following assumptions:

- The reliability index $\beta=2.5$ for operating ratings is consistent with the rating of typical bridge configurations which produced LFR operating ratings RF=1.0.
- Current truck traffic load spectra are consistent with the random truck load data used during the LRFR and LRFD calibrations which assume that the upper 20% of the static truck load effects can be represented by a Normal probability distribution.
- The probability of side-by-side heavy truck events is 1/15.
- The probabilistic models for bridge member resistance, load distribution, and dynamic allowance can be represented using the same models applied when calibrating the AASHTO LRFD.

While a few states have developed or are in the process of developing their own state-specific customized versions of the LRFR procedures, most of the states tend to simply select the parts of the AASHTO LRFR that seem to best coincide with their experience, current practice, and level of comfort. Oregon recalibrated the Oregon LRFR live load factors based on truck Weigh-In-Motion data collected at representative sites. However, the same assumptions on the shape of the upper 20% of the data and the probability of multiple-presence were kept as implied in the AAHTO LRFR. Michigan's proposed LRFR not only used the Michigan WIM truck weight data but also modified the probability of multiple presence to 1/30 which is found to be more consistent with headway data collected at a representative Michigan site. However, Michigan's approach still kept the assumption on the shape of the truck weight spectra implied in the AASHTO LRFR.

In a recent paper, Kulicki (2005) mentioned the importance of "identifying the characteristics of the data used for calibration" and reviewing the "system administration aspects" of the LRFR and LRFD specifications to ensure that the specifications are "providing adequate reserve for legal and permit loads" and to assess "the manner in which service, overload and permitting are treated". For example, recent data and findings collected as part of NCHRP 12-76 [Sivakumar et al; 2008] have shown that the basic assumptions regarding the shape of the truck weight spectra and the probability of multiple presence previously used during the calibration of the AASHTO LRFR may no longer represent current truck traffic conditions. Similar observations on the multiple-presence data were made by Gindi and Nassif (2007). Also, a review of current procedures in several states may possibly show that the implied reliability index could differ from the reliability index β=2.5 used as the base line for the calibration of the AASHTO LRFR live load factors. Therefore, for developing an LRFR version consistent with a State's current procedures it may be necessary to revisit the assumptions implied during the AASHTO LRFR based on state-specific data on truck weights and multiple presence. This is currently possible due to the abundance of currently installed Weigh-In-Motion systems that are capable of providing data on current truck weights and weigh spectra in combination with information on truck multiple presence information that was not available to previous LRFR and LRFD studies. [Sivakumar et al, 2008]

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APPENDIX IV

COMPARISON OF RATINGS FROM PROPOSED PROCEDURE AND CURRENT NYSDOT METHOD

1. INTRODUCTION

This Report presents the rating results for 17 New York State bridges. The bridges were selected by the New York State DOT Technical Working Group overseeing the project and give a range of LFR Inventory level ratings. The purpose of this analysis is to compare the LFR Ratings to those that would be obtained if the NYS-LRFR live load factors calibrated in this study are adopted by NYSDOT. The analyses performed in this report compare the LFR Operating ratings to the proposed NYS-LRFR Legal Load Ratings. In addition, the LFR Permit Load Ratings are compared to the proposed NYS-LRFR Permit Load Ratings. The ratings are performed using the program Virtis.

It is noted that the proposed NYS-LRFR for Legal Loads was calibrated to match on the average close to a target reliability index β_{target}=2.0 while remaining above β_{min}=1.50 if the rating Factor R.F.=1.0. This average and minimum targets are significantly higher than those observed from current NYS-LFR methods. Thus, the results of the analyses to be performed in this Report are expected to produce lower NYS-LRFR Legal Load ratings than those obtained by current methods. This would be especially true for short span bridges with close beam spacing as is the case with the bridges rated in this document.

The proposed NYS-LRFR permit load ratings were also calibrated to produce reliability index values close to β_{target}=2.0. However, in most cases a higher value was obtained due to a condition that was imposed which required keeping a lower bound Permit load factor γ_L=1.10. Although the proposed NYS-LRFR live load factors ranging between 1.1 and 1.2 are lower than the live load γ_L=1.30 their effect on the Rating factors for short span bridges with closely spaced beams may be offset by applying the AAHTO LRFD load distribution factors. The combination of lower Permit load factors and higher load distribution factors for closely spaced beams is expected to lead to calculating NYS-LRFR Permit Load ratings that are somewhat similar to the ratings obtained when using the current LFR methods but that may be higher or lower depending on the span length and beam spacing.

It is noted that the program Virtis gives only the controlling ratings for a bridge without giving a detailed listing of moment and shear ratings or if serviceability governs. Therefore, a direct one on one comparison of LFR versus LRFR results is not possible. The overall objective of the comparison performed in this report is to show that the proposed NYS-LRFR approach gives reasonable results when compared with LFR Operating Ratings.

2. METHOD OF ANALYSIS AND RESULTS

Load Factor Ratings (LFR) and Load and Resistance Factor Ratings (LRFR) were performed for 17 representative bridges that were selected from the NYSDOT bridge inventory. This set of bridges included conventional reinforced concrete (T-beam and slab), prestressed concrete (I-beam, box beam), and steel (rolled I-beam, welded/riveted plate girder, built-up girder) superstructures supporting concrete decks.

Load ratings were performed using AASHTOWARE's "Virtis v.6.1" software, using the BRASS GIRDER rating engine. Each bridge was load rated for HS20 (LFR) and HL93 (LRFR) design loads, Type 3, Type 3S2, Type 3-3 and SU4 legal vehicles, in addition to 5 divisible (NYP_1 to NYP_5) and 8 non-divisible load permits (NYP6 to NYP13) in accordance with Draft NYSDOT EI based on the LRFR methodology. LFR ratings were performed per AASHTO MBE for comparison purposes both at the inventory (INV) and operating (OPR) levels. Live load factors used in the analysis are given in Table 1 below.

Table 2 gives the listing of the bridges analyzed in this report along with information on their configuration, span length, beam spacing, material type, ADTT, and overall condition.

Table 3 compares the 3S2 legal load ratings for LFR and the proposed NYS-LRFR methodologies. The values provided are those for the controlling ratings as reported by Virtis. Virtis does not specify whether the ratings are governed by shear or moment or whether the serviceability conditions which are used in conjunction with the NY-LRFR govern. It should also be noted that the proposed NYS-LRFR uses the SU-4 for rating short span bridges. Therefore, the ratings shown in Table 3 may not be the governing ratings.

Table 4 gives the ratings for the design loading HS-20 for LFR and HL-93 for NYS-LRFR as well as the ratings for each of the AASHTO Legal trucks and the SU-4 truck. Here gain the ratings are those reported by Virtis without specifying which is the controlling limit state.

Table 5 provides the ratings for the NYSDOT non-divisible loads while Table 6.a and 6.b gives those for the divisible loads as reported by Virtis. The configurations for the permit loads are provided in Chapter 3 of the body of the report.

The figures provided below give plots of the tabulated ratings for easy visual comparisons.

TABLE 1 – Live Load Factors used in Load Ratings

	Live Load Factors		
All LFR Ratings	Inventory		2.17
	Operating		1.30
LRFR HL-93 Design Load Rating	Inventory		1.75
	Operating		1.35
NYSDOT Legal Load Rating for Routine Commercial Traffic Multi-lane Bridges	Type 3, Type 3S2, Type 3-3 and SU4	Unknown	1.95
		ADTT ≥ 5000	1.95
		ADTT = 1000	1.85
		ADTT ≤ 100	1.65
NYSDOT Annual Divisible Loads Unlimited Trips, Multi-lane Bridges Mix with Traffic (DF = Multi-lane)	NYP_6, NYP_7, NYP_8, NYP_9, NYP_10, NYP_11, NYP_12, NYP_13	Unknown	1.20
		ADTT ≥ 5000	1.20
		ADTT = 1000	1.15
		ADTT ≤ 100	1.10
NYSDOT Non-Divisible Loads Multiple Trips, Multi-lane Bridges Mix with Traffic (DF = Multi-lane)	NYP_1, NYP_2, NYP_3, NYP_4, NYP_5		1.10

TABLE 2 - BRIDGE DATA

Bridge No. & MP	Span(s)	Span Length of Rated Span (ft.)	Material Type	Superstructure Framing	Redundant or Non-Redundant	Girder Spacing (ft.)	Superstructure Condition Rating	Deck Type	ADTT
3344660	1	26.50	Concrete	Slab	N/A	N/A	Fair	Concrete	75
3342420	1	37.83	Concrete	T-Beam	All Other Bridges/Slabs	5' - 0"	Good or Satisfactory	Concrete	5
1012860	3	47.08	Prestressed Concrete	Adjacent Box Girder	Redundant Stringers	3' - 1/2"	Poor	Concrete	187
1034970	3	74.83	Prestressed Concrete	Adjacent Box Girder	All Other Bridges/Slabs	4' - 7/8"	Fair	Concrete	154
1046200	1	81.00	Prestressed Concrete	Adjacent Box Girder	All Other Bridges/Slabs	4' - 1/2"	Poor	Concrete	78
1061381	1	100.00	Prestressed Concrete	I-Beam	Redundant Stringers	6' - 0"	Good or Satisfactory	Concrete	1965
1094740	3	42.00	Prestressed Concrete	Adjacent Box Girder	All Other Bridges/Slabs	6' - 4" & 4' - 0"	Good or Satisfactory	Concrete	322
3323020	3	38.13	Prestressed Concrete	Adjacent Box Girder	All Other Bridges/Slabs	3' - 6" & 3' - 1/2"	Good or Satisfactory	Concrete	85
1010160	1	65.00	Steel	Rolled I-Beam	Redundant Stringers	6' - 4"	Good or Satisfactory	Concrete	101
1019600	3	103.00	Steel	Welded Plate Girder	Redundant Stringers	6' - 6" & 7' - 9"	Good or Satisfactory	Concrete	1117
1032170	5	76.50	Steel	Rolled I-Beam	Redundant Stringers	6' - 9"	Poor	Concrete	73
1040802	2	95.00	Steel	Welded Plate Girder	Redundant Stringers	6' - 11" & 7' - 1"	Good or Satisfactory	Concrete	1368
1045470	1	33.00	Steel	Welded Plate Girder	Redundant Stringers	4' - 6"	Fair	Concrete	201
1055740	1	87.72	Steel	Welded Builtup Girder	Redundant Stringers	8' - 0"	Good or Satisfactory	Concrete	225
2217300	1	30.75	Steel	Rolled I-Beam	Redundant Stringers	2' - 1/8"	Good or Satisfactory	Concrete	1
2262130	1	26.00	Steel	Welded Plate Girder	Redundant Stringers	2' - 3"	Good or Satisfactory	Concrete	Unknown
3301860	1	25.25	Steel	Rolled I-Beam	Redundant Stringers	3' - 5"	Fair	Concrete	20

TABLE 3 - COMPARISON OF 3S2 LEGAL LOAD RATINGS FOR LFR AND LRFR METHODOLOGIES (CONTROLLING RATINGS REPORTED BY VIRTIS)

Bridge No. & MP	Material Type	Superstructure Framing	LFR RATING		LRFR LEGAL RATING	Legal Type 3S2 Comparison	
			INV	OPR		LFR INV / LRFR	LFR OPR / LRFR
3344660	Concrete	Slab	0.824	1.375	0.869	0.95	1.58
3342420	Concrete	T-Beam	0.722	1.205	0.938	0.77	1.28
1012860	Prestressed Concrete	Adjacent Box Girder	0.73	1.22	0.449	1.63	2.72
1034970	Prestressed Concrete	Adjacent Box Girder	1.451	2.424	2.222	0.65	1.09
1046200	Prestressed Concrete	Adjacent Box Girder	0.009	0.018	1.298	0.01	0.01
1061381	Prestressed Concrete	I-Beam	0.754	1.907	1.194	0.63	1.60
1094740	Prestressed Concrete	Adjacent Box Girder	0.829	1.455	0.938	0.88	1.55
3323020	Prestressed Concrete	Adjacent Box Girder	0.741	1.238	1.157	0.64	1.07
1010160	Steel	Rolled I-Beam	1.019	1.703	1.047	0.97	1.63
1019600	Steel	Welded Plate Girder	0.738	1.233	2.273	0.32	0.54
1032170	Steel	Rolled I-Beam	1.087	1.832	1.160	0.94	1.58
1040802	Steel	Welded Plate Girder	1.434	2.395	1.619	0.89	1.48
1045470	Steel	Welded Plate Girder	0.788	1.317	1.079	0.73	1.22
1055740	Steel	Welded Builtup Girder	1.663	2.777	1.584	1.05	1.75
2217300	Steel	Rolled I-Beam	0.824	1.376	0.357	2.31	3.85
2262130	Steel	Welded Plate Girder	0.776	1.296	0.496	1.57	2.61
3301860	Steel	Rolled I-Beam	0.368	0.615	0.214	1.72	2.88

TABLE 4 - RATING FOR DESIGN AND LEGAL LOADS (CONTROLLING RATINGS REPORTED BY VIRTIS)

Bridge No. & MP	Material Type	Superstructure Framing	Design Load Rating				Legal Load Rating for Routine Commercial Traffic											
			HS20		HL93		Type 3			Type 3S2			Type 3-3			Type SU4		
			LFR		LRFR		LFR		LRFR	LFR		LRFR	LFR		LRFR	LFR		LRFR
			INV	OPR	INV	OPR	INV	OPR	RF	INV	OPR	RF	INV	OPR	RF	INV	OPR	RF
3344660	Concrete	Slab	0.696	1.162	0.468	0.607	0.797	1.33	0.840	0.824	1.375	0.869	0.967	1.615	1.021	0.667	1.114	0.707
3342420	Concrete	T-Beam	0.546	0.912	0.495	0.641	0.685	1.144	0.888	0.722	1.205	0.938	0.843	1.408	1.089	0.582	0.972	0.751
1012860	Prestressed Concrete	Adjacent Box Girder	0.508	0.848	0.243	0.315	0.668	1.116	0.410	0.73	1.22	0.449	0.804	1.342	0.495	0.577	0.963	0.559
1019600	Prestressed Concrete	Adjacent Box Girder	0.928	1.567	1.025	1.618	1.313	2.209	2.607	1.451	2.424	2.222	1.092	1.84	2.254	1.756	2.636	2.31
1046200	Prestressed Concrete	Adjacent Box Girder	0.008	0.015	0.674	0.987	0.011	0.02	1.484	0.009	0.018	1.298	0.01	0.018	1.326	0.01	0.018	1.327
1061381	Prestressed Concrete	I-Beam	0.663	1.687	0.254	0.895	0.924	2.326	1.828	0.754	1.907	1.194	0.761	1.842	1.110	0.836	2.128	1.383
1094740	Prestressed Concrete	Adjacent Box Girder	0.6	1.054	0.407	0.696	0.772	1.356	0.870	0.829	1.455	0.938	0.933	1.639	1.055	0.663	1.165	0.746
3323020	Prestressed Concrete	Adjacent Box Girder	0.639	1.066	0	0.912	0.901	1.504	1.405	0.741	1.238	1.157	0.754	1.26	1.178	0.818	1.366	1.387
1010160	Steel	Rolled I-Beam	0.789	1.318	0.693	0.898	1.07	1.786	1.102	1.019	1.703	1.047	1.09	1.82	1.118	0.95	1.586	1.132
1032170	Steel	Rolled I-Beam	0.629	1.051	0.216	0.28	0.874	1.46	0.433	1.05	1.753	0.365	0.736	1.228	0.365	0.785	1.312	0.389
1034970	Steel	Rolled I-Beam	0.857	1.432	0.678	0.878	1.175	1.963	1.301	1.087	1.832	1.160	1.078	1.8	1.196	1.049	1.752	1.161
1040802	Steel	Welded Plate Girder	1.251	2.089	1.055	1.368	1.751	2.924	1.958	1.434	2.395	1.619	1.406	2.348	1.603	1.598	2.669	1.788
1045470	Steel	Welded Plate Girder	0.631	1.054	0.741	0.963	0.774	1.292	1.053	0.788	1.317	1.079	0.965	1.612	1.314	0.642	1.072	1.024
1055740	Steel	Welded Builtup Girder	1.464	2.444	1.054	1.366	2.063	3.445	1.853	1.663	2.777	1.584	1.62	2.706	1.594	1.886	3.15	1.847
2217300	Steel	Rolled I-Beam	0.651	1.087	0.228	0.295	0.826	1.379	0.356	0.824	1.376	0.357	1.013	1.692	0.434	0.677	1.131	0.338
2262130	Steel	Welded Plate Girder	0.654	1.092	0.268	0.348	0.745	1.244	0.476	0.776	1.296	0.496	0.905	1.511	0.578	0.627	1.047	0.421
3301860	Steel	Rolled I-Beam	0.317	0.529	0.143	0.185	0.355	0.593	0.204	0.368	0.615	0.214	0.431	0.72	0.248	0.297	0.496	0.202

TABLE 5 - RATING FOR NYSDOT NON-DIVISIBLE PERMIT LOADS (CONTROLLING RATINGS REPORTED BY VIRTIS)

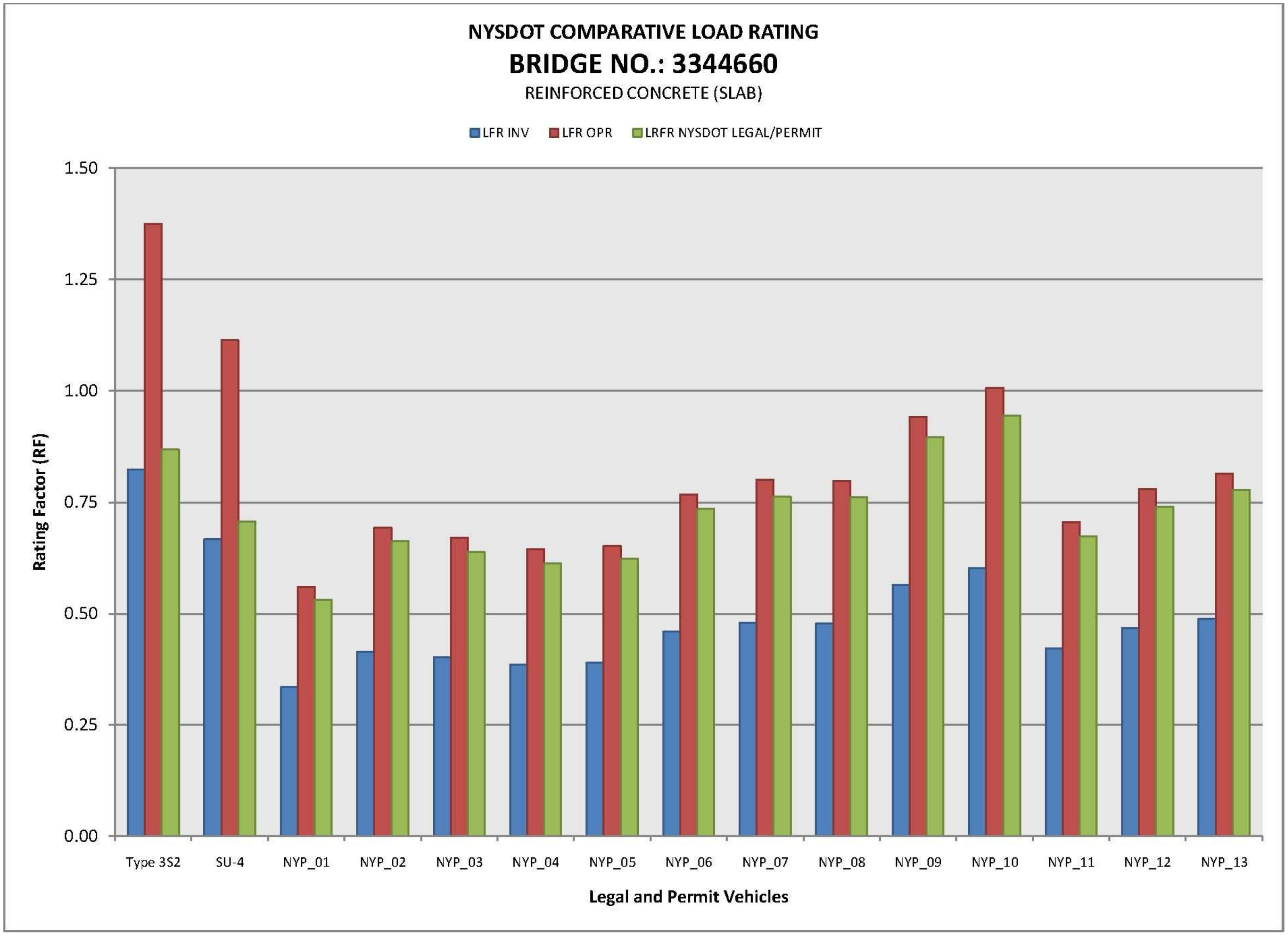
Bridge No. & MP	Material Type	Superstructure Framing	Non-Divisible Loads														
			NYP_1			NYP_2			NYP_3			NYP_4			NYP_5		
			LFR		LRFR	LFR		LRFR	LFR		LRFR	LFR		LRFR	LFR		LRFR
			INV	OPR	RF	INV	OPR	RF	INV	OPR	RF	INV	OPR	RF	INV	OPR	RF
3344660	Concrete	Slab	0.335	0.56	0.531	0.415	0.693	0.663	0.402	0.671	0.638	0.386	0.645	0.613	0.39	0.652	0.624
3342420	Concrete	T-Beam	0.275	0.459	0.532	0.385	0.644	0.752	0.363	0.607	0.711	0.317	0.53	0.611	0.282	0.471	0.545
1012860	Prestressed Concrete	Adjacent Box Girder	0.258	0.431	0.243	0.371	0.62	0.352	0.309	0.515	0.295	0.309	0.516	0.288	0.27	0.451	0.251
1034970	Prestressed Concrete	Adjacent Box Girder	0.431	0.737	1.034	0.494	0.804	1.367	0.454	0.775	1.175	0.616	1.045	1.706	0.474	0.809	1.183
1046200	Prestressed Concrete	Adjacent Box Girder	0.004	0.007	0.661	0.004	0.008	0.787	0.004	0.008	0.760	0.005	0.01	1.039	0.004	0.008	0.762
1061381	Prestressed Concrete	I-Beam	0.309	0.781	0.468	0.327	0.871	0.467	0.335	0.795	0.540	0.434	1.047	0.941	0.34	0.844	0.576
1094740	Prestressed Concrete	Adjacent Box Girder	0.305	0.536	0.503	0.433	0.76	0.741	0.396	0.695	0.673	0.354	0.623	0.605	0.31	0.544	0.525
3323020	Prestressed Concrete	Adjacent Box Girder	0.295	0.492	0.690	0.342	0.572	0.818	0.322	0.538	0.753	0.418	0.698	0.978	0.324	0.54	0.757
1010160	Steel	Rolled I-Beam	0.38	0.634	0.458	0.47	0.785	0.570	0.442	0.737	0.535	0.499	0.834	0.604	0.403	0.674	0.488
1024320	Steel	Rolled I-Beam	0.295	0.492	0.179	0.328	0.547	0.199	0.324	0.54	0.196	0.41	0.684	0.248	0.319	0.532	0.195
1032170	Steel	Rolled I-Beam	0.406	0.678	0.612	0.489	0.817	0.740	0.459	0.766	0.691	0.55	0.918	0.803	0.437	0.729	0.660
1040802	Steel	Welded Plate Girder	0.6	1.002	1.042	0.618	1.031	1.076	0.647	1.081	1.132	0.844	1.409	1.448	0.66	1.103	1.151
1045470	Steel	Welded Plate Girder	0.317	0.529	0.503	0.415	0.693	0.662	0.398	0.665	0.635	0.356	0.595	0.563	0.332	0.555	0.529
1055740	Steel	Welded Builtup Girder	0.698	1.166	0.728	0.708	1.183	0.836	0.751	1.254	0.805	0.989	1.652	1.006	0.768	1.283	0.791
2217300	Steel	Rolled I-Beam	0.333	0.556	0.168	0.431	0.719	0.219	0.413	0.69	0.210	0.376	0.627	0.190	0.358	0.598	0.183
2262130	Steel	Welded Plate Girder	0.316	0.528	0.201	0.392	0.654	0.249	0.376	0.627	0.239	0.368	0.614	0.233	0.369	0.616	0.235
3301860	Steel	Rolled I-Beam	0.153	0.255	0.103	0.186	0.311	0.127	0.18	0.301	0.122	0.178	0.297	0.120	0.177	0.296	0.121

TABLE 6a - RATING FOR NYSDOT DIVISIBLE LOADS (CONTROLLING RATINGS REPORTED BY VIRTIS)

Bridge No. & MP	Material Type	Superstructure Framing	Divisible Loads											
			NYP_6			NYP_7			NYP_8			NYP_9		
			LFR		LRFR	LFR		LRFR	LFR		LRFR	LFR		LRFR
			INV	OPR	RF	INV	OPR	RF	INV	OPR	RF	INV	OPR	RF
3344660	Concrete	Slab	0.46	0.768	0.735	0.48	0.801	0.763	0.478	0.798	0.762	0.564	0.942	0.896
3342420	Concrete	T-Beam	0.43	0.718	0.833	0.382	0.637	0.744	0.442	0.739	0.855	0.536	0.895	1.044
1012860	Prestressed Concrete	Adjacent Box Girder	0.421	0.702	0.548	0.366	0.611	0.476	0.426	0.712	0.554	0.544	0.908	0.707
1034970	Prestressed Concrete	Adjacent Box Girder	0.598	1.086	1.937	0.565	0.986	1.743	0.832	1.406	2.510	0.871	1.471	2.753
1046200	Prestressed Concrete	Adjacent Box Girder	0.006	0.011	1.144	0.005	0.01	1.036	0.007	0.013	1.414	0.009	0.016	1.697
1061381	Prestressed Concrete	I-Beam	0.462	1.123	1.203	0.421	1.047	1.045	0.587	1.376	2.053	0.611	1.402	1.877
1094740	Prestressed Concrete	Adjacent Box Girder	0.489	0.86	0.864	0.417	0.733	0.737	0.494	0.868	0.871	0.626	1.1	1.105
3323020	Prestressed Concrete	Adjacent Box Girder	0.448	0.748	1.133	0.401	0.67	1.016	0.572	0.955	1.445	0.675	1.127	1.701
1010160	Steel	Rolled I-Beam	0.597	0.997	0.725	0.534	0.891	0.649	0.684	1.143	0.826	0.891	1.487	1.085
1024320	Steel	Rolled I-Beam	0.452	0.754	0.276	0.405	0.676	0.247	0.556	0.928	0.338	0.639	1.067	0.390
1032170	Steel	Rolled I-Beam	0.635	1.06	0.966	0.564	0.942	0.855	0.75	1.252	1.102	0.952	1.59	1.438
1040802	Steel	Welded Plate Girder	0.867	1.449	1.735	0.806	1.346	1.616	1.123	1.876	2.234	1.097	1.832	2.225
1045470	Steel	Welded Plate Girder	0.46	0.768	0.741	0.458	0.766	0.735	0.478	0.799	0.770	0.57	0.952	0.917
1055740	Steel	Welded Builtup Girder	1.004	1.677	1.142	0.936	1.562	1.021	1.316	2.198	1.386	1.263	2.109	1.637
2217300	Steel	Rolled I-Beam	0.477	0.797	0.243	0.492	0.822	0.249	0.496	0.829	0.252	0.591	0.987	0.300
2262130	Steel	Welded Plate Girder	0.434	0.725	0.328	0.453	0.756	0.341	0.451	0.753	0.341	0.528	0.881	0.400
3301860	Steel	Rolled I-Beam	0.206	0.344	0.141	0.215	0.359	0.146	0.214	0.357	0.146	0.252	0.42	0.171

TABLE 6b - RATING FOR NYSDOT DIVISIBLE LOADS (CONTROLLING RATINGS REPORTED BY VIRTIS)

Bridge No. & MP	Material Type	Superstructure Framing	Divisible Loads											
			NYP_10			NYP_11			NYP_12			NYP_13		
			LFR		LRFR	LFR		LRFR	LFR		LRFR	LFR		LRFR
			INV	OPR	RF	INV	OPR	RF	INV	OPR	RF	INV	OPR	RF
3344660	Concrete	Slab	0.603	1.007	0.945	0.422	0.705	0.673	0.467	0.78	0.740	0.488	0.815	0.778
3342420	Concrete	T-Beam	0.511	0.853	0.994	0.366	0.612	0.705	0.434	0.724	0.844	0.458	0.764	0.887
1012860	Prestressed Concrete	Adjacent Box Girder	0.494	0.825	0.640	0.35	0.584	0.455	0.442	0.738	0.576	0.457	0.763	0.606
1034970	Prestressed Concrete	Adjacent Box Girder	0.92	1.554	2.711	0.58	0.985	1.697	0.769	1.3	2.407	0.872	1.473	2.677
1046200	Prestressed Concrete	Adjacent Box Girder	0.007	0.014	1.493	0.005	0.01	0.999	0.007	0.013	1.448	0.007	0.014	1.505
1061381	Prestressed Concrete	I-Beam	0.616	1.655	2.344	0.418	1.015	1.024	0.556	1.297	1.621	0.615	1.436	2.231
1094740	Prestressed Concrete	Adjacent Box Girder	0.574	1.008	1.010	0.401	0.704	0.704	0.504	0.886	0.890	0.532	0.935	0.935
3323020	Prestressed Concrete	Adjacent Box Girder	0.603	1.007	1.524	0.401	0.669	1.014	0.565	0.943	1.426	0.606	1.012	1.532
1010160	Steel	Rolled I-Beam	0.735	1.228	0.891	0.519	0.867	0.628	0.75	1.253	0.912	0.739	1.234	0.890
1024320	Steel	Rolled I-Beam	0.582	0.972	0.353	0.399	0.667	0.244	0.557	0.93	0.340	0.585	0.977	0.358
1032170	Steel	Rolled I-Beam	0.787	1.315	1.199	0.551	0.921	0.837	0.784	1.309	1.189	0.794	1.326	1.200
1040802	Steel	Welded Plate Girder	1.236	2.063	2.504	0.794	1.327	1.592	1.013	1.691	2.033	1.169	1.953	2.349
1045470	Steel	Welded Plate Girder	0.575	0.96	0.923	0.417	0.696	0.672	0.463	0.773	0.749	0.489	0.816	0.786
1055740	Steel	Welded Builtup Girder	1.439	2.403	1.452	0.927	1.548	1.006	1.173	1.959	1.405	1.375	2.297	1.469
2217300	Steel	Rolled I-Beam	0.606	1.011	0.310	0.438	0.732	0.223	0.485	0.81	0.245	0.507	0.847	0.258
2262130	Steel	Welded Plate Girder	0.564	0.942	0.422	0.398	0.665	0.301	0.437	0.73	0.331	0.46	0.769	0.348
3301860	Steel	Rolled I-Beam	0.266	0.444	0.180	0.189	0.316	0.129	0.208	0.347	0.142	0.219	0.365	0.149



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BRIDGE NO.: 3344660
REINFORCED CONCRETE (SLAB)
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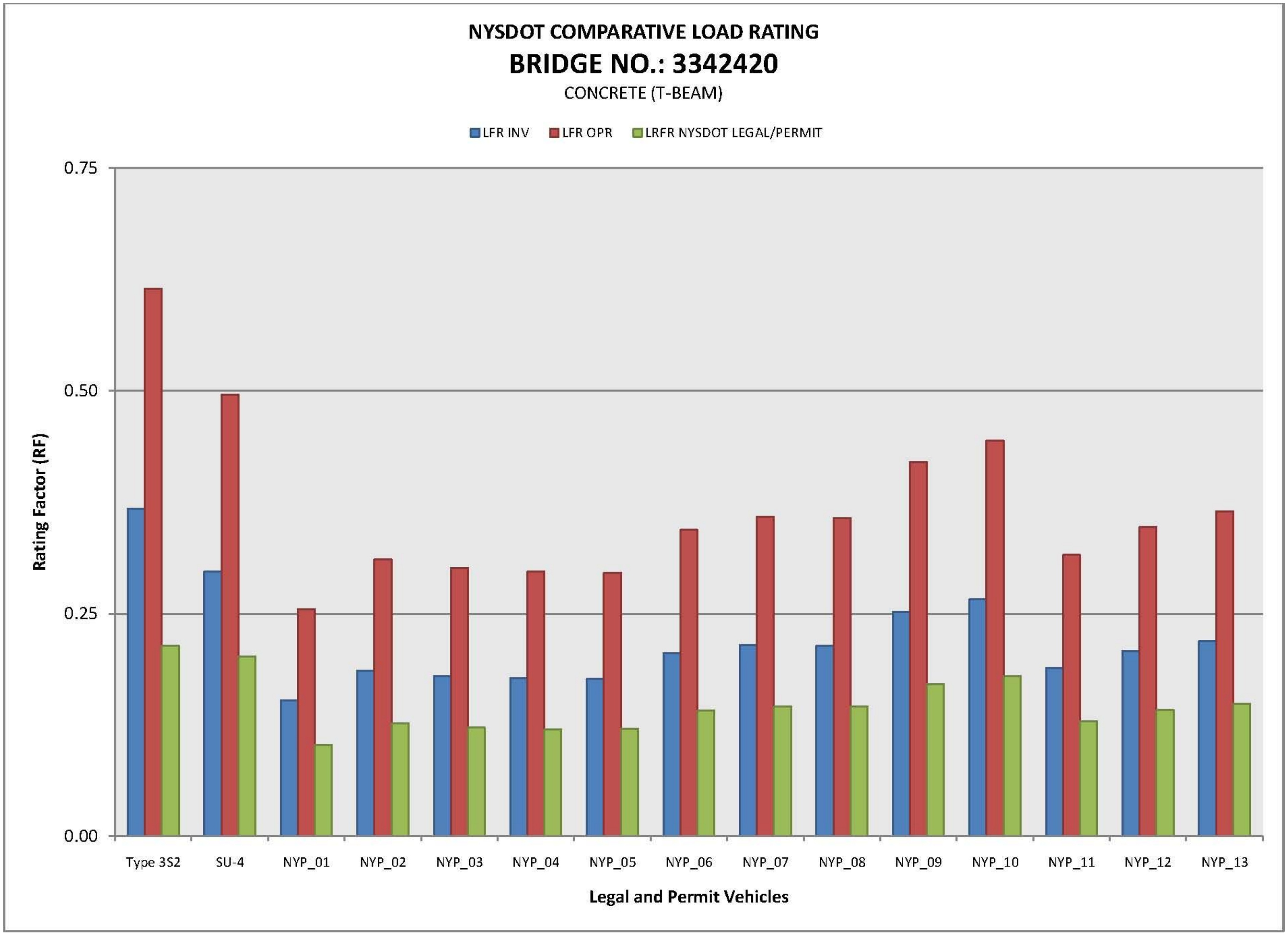
LFR OPR
LRFR NYSDOT LEGAL/PERMIT
Rating Factor (RF)
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Type 3S2
SU-4
NYP_01
NYP_02
NYP_03
NYP_04
NYP_05
NYP_06
NYP_07
NYP_08
NYP_09
NYP_10
NYP_11
NYP_12
NYP_13
Legal and Permit Vehicles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CONCRETE (T-BEAM)
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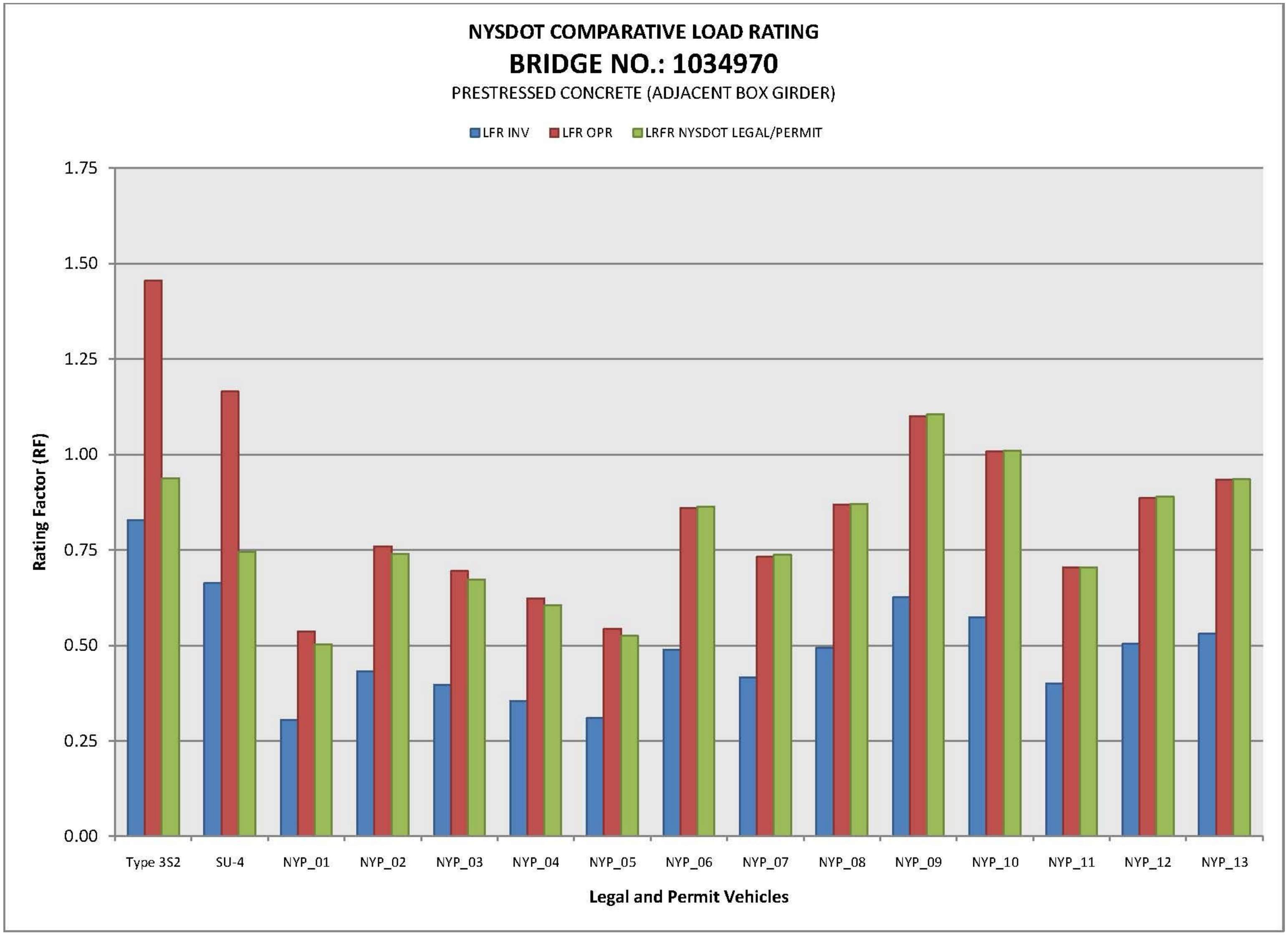
LFR OPR
LRFR NYSDOT LEGAL/PERMIT
0.75
0.50
0.25
0.00
Rating Factor (RF)
Type 3S2
SU-4
NYP_01
NYP_02
NYP_03
NYP_04
NYP_05
NYP_06
NYP_07
NYP_08
NYP_09
NYP_10
NYP_11
NYP_12
NYP_13
Legal and Permit Vehicles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BRIDGE NO.: 1034970
PRESTRESSED CONCRETE (ADJACENT BOX GIRDER)
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LFR OPR
LRFR NYSDOT LEGAL/PERMIT
Rating Factor (RF)
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Type 3S2
SU-4
NYP_01
NYP_02
NYP_03
NYP_04
NYP_05
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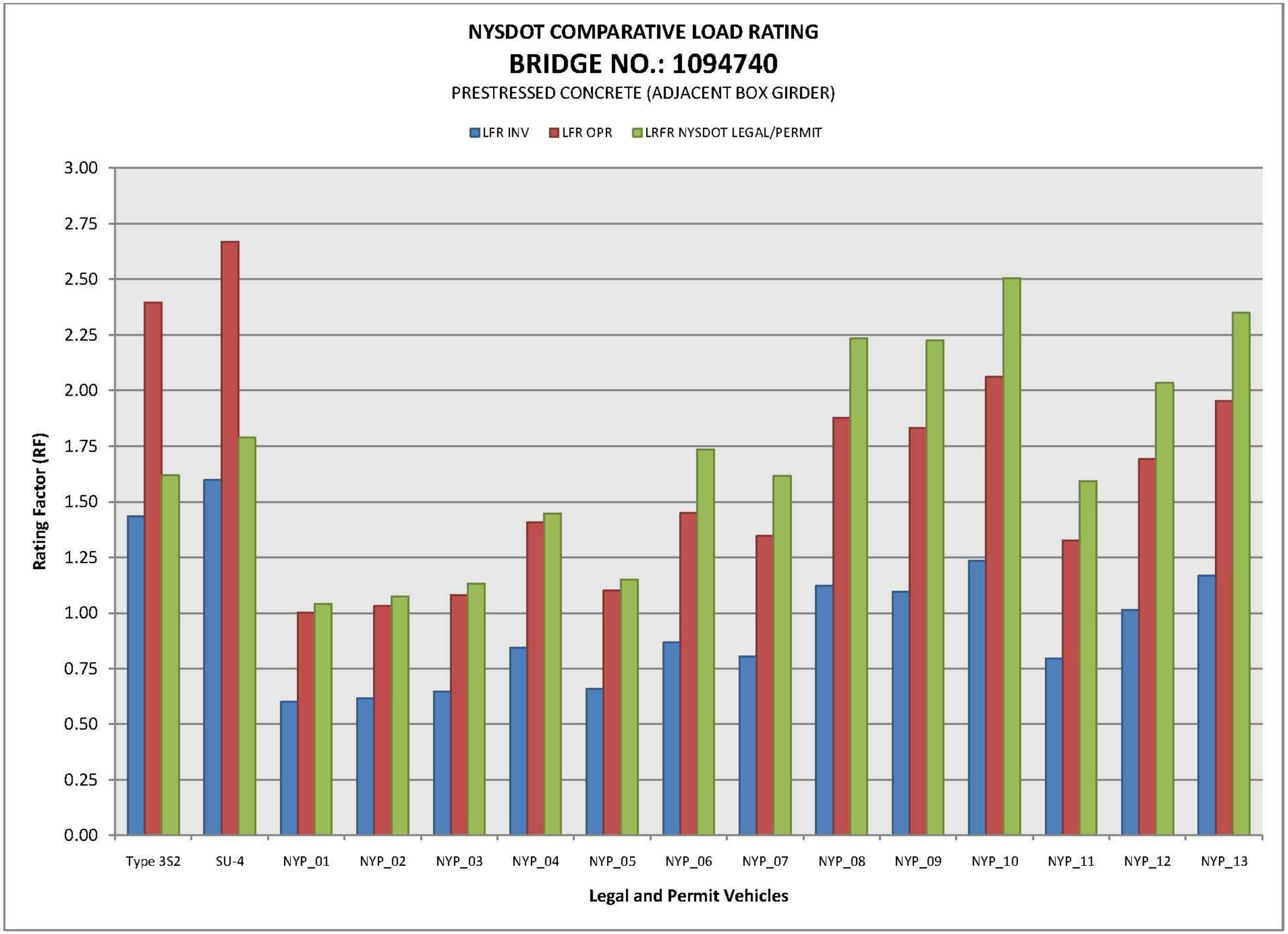
NYP_07
NYP_08
NYP_09
NYP_10
NYP_11
NYP_12
NYP_13
Legal and Permit Vehicles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BRIDGE NO.: 1094740
PRESTRESSED CONCRETE (ADJACENT BOX GIRDER)
LFR INV
LFR OPR
LRFR NYSDOT LEGAL/PERMIT
Rating Factor (RF)
3.00
2.75
2.50
2.25
2.00
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Type 3S2
SU-4
NYP_01
NYP_02
NYP_03
NYP_04
NYP_05
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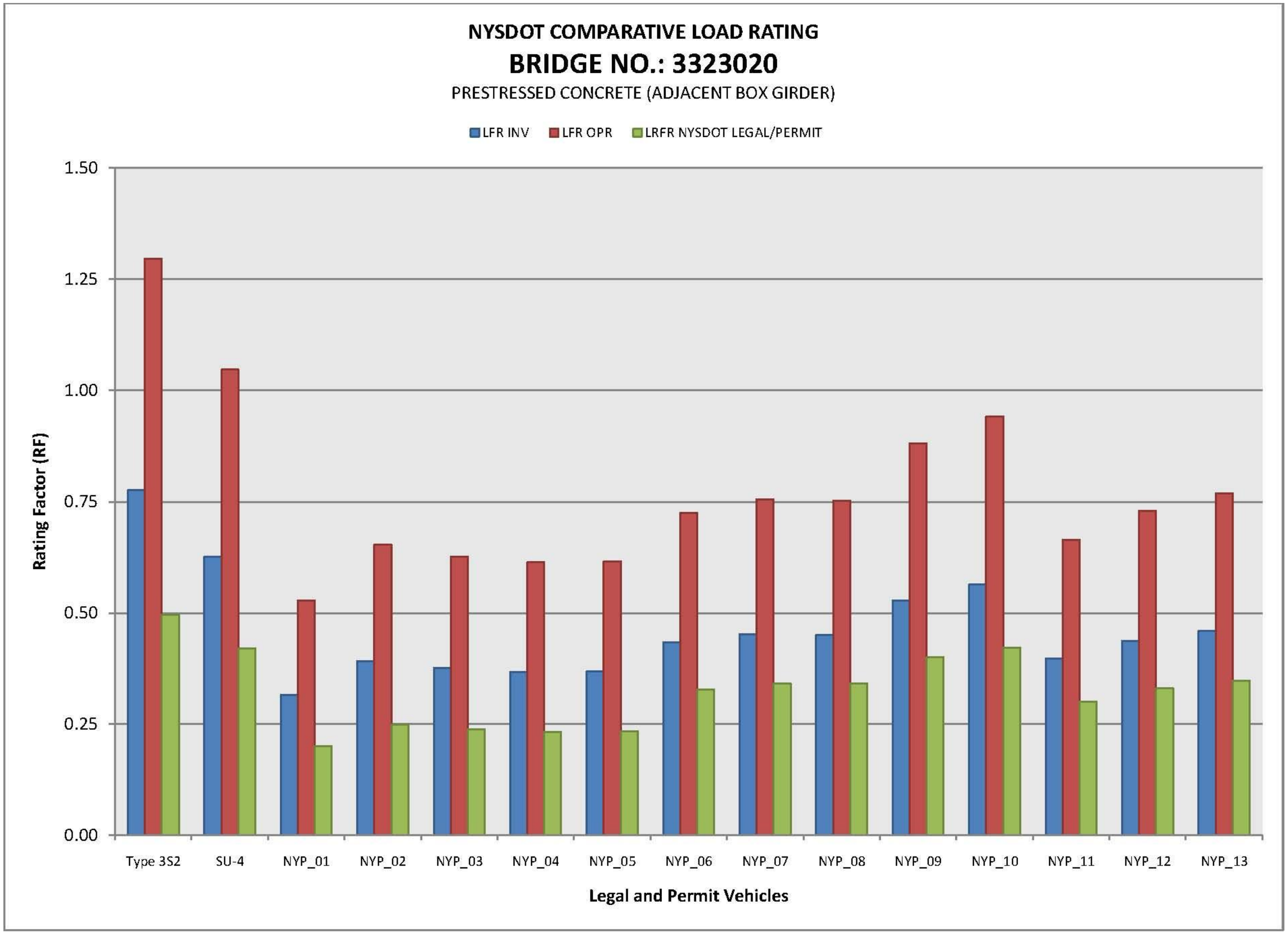
NYP_08
NYP_09
NYP_10
NYP_11
NYP_12
NYP_13
Legal and Permit Vehicles


NYSDOT COMPARATIVE LOAD RATING
BRIDGE NO.: 3323020
PRESTRESSED CONCRETE (ADJACENT BOX GIRDER)
LFR INV
LFR OPR
LRFR NYSDOT LEGAL/PERMIT
Rating Factor (RF)
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Type 3S2
SU-4
NYP_01
NYP_02
NYP_03
NYP_04
NYP_05
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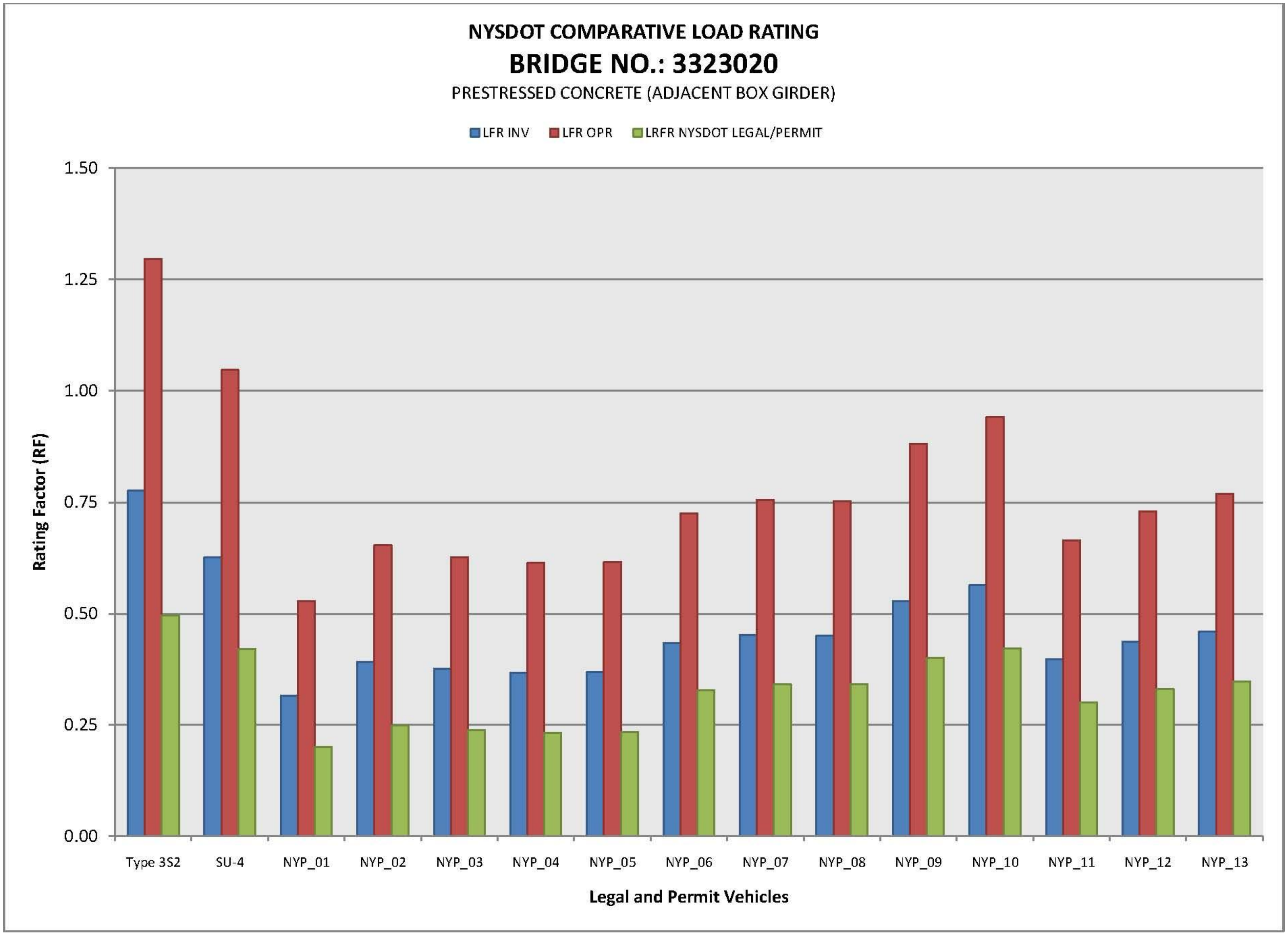
Legal and Permit Vehicles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BRIDGE NO.: 3323020
PRESTRESSED CONCRETE (ADJACENT BOX GIRDER)
LFR INV
LFR OPR
LRFR NYSDOT LEGAL/PERMIT
Rating Factor (RF)
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Type 3S2
SU-4
NYP_01
NYP_02
NYP_03
NYP_04
NYP_05
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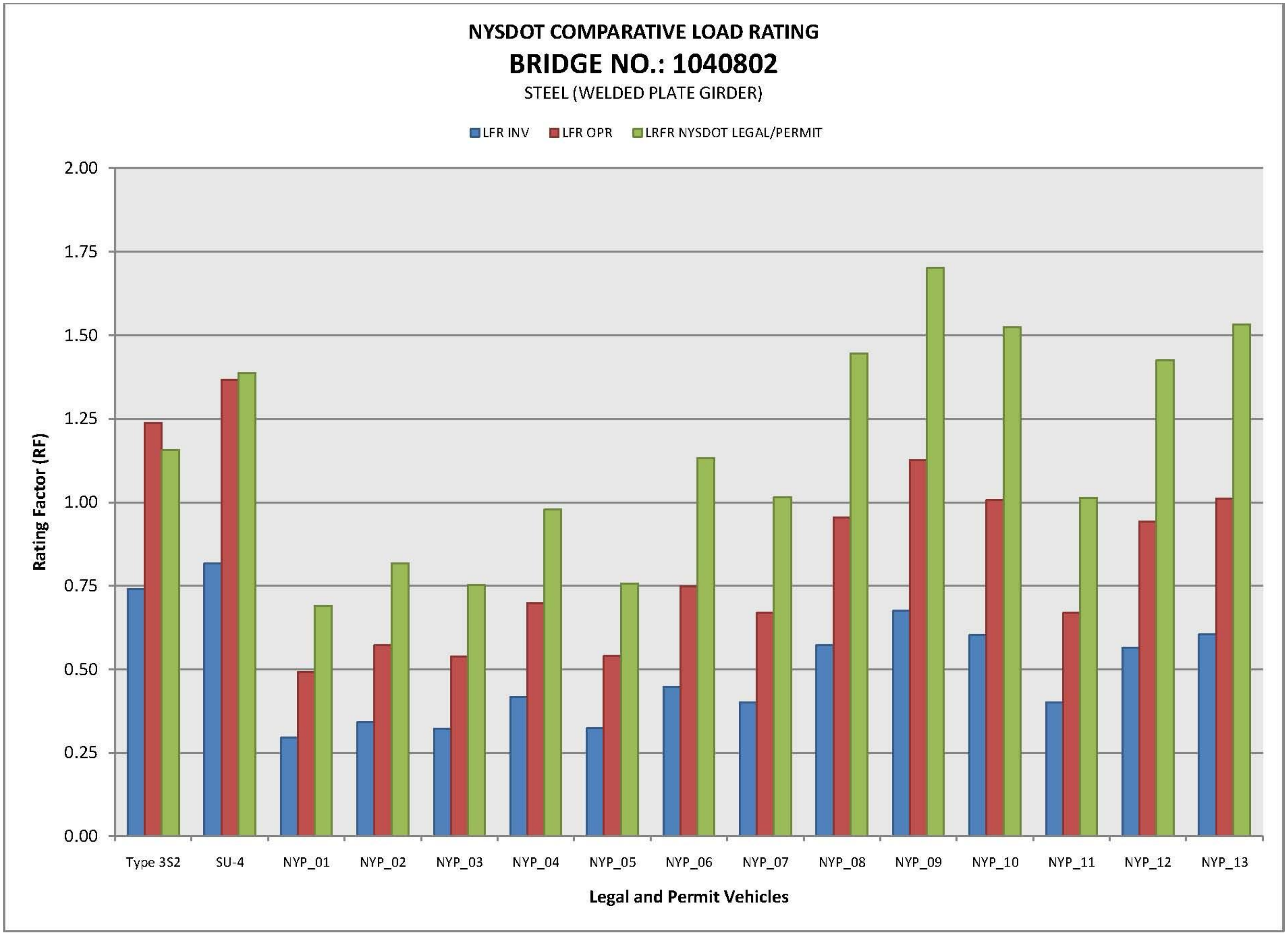
NYP_07
NYP_08
NYP_09
NYP_10
NYP_11
NYP_12
NYP_13
Legal and Permit Vehicles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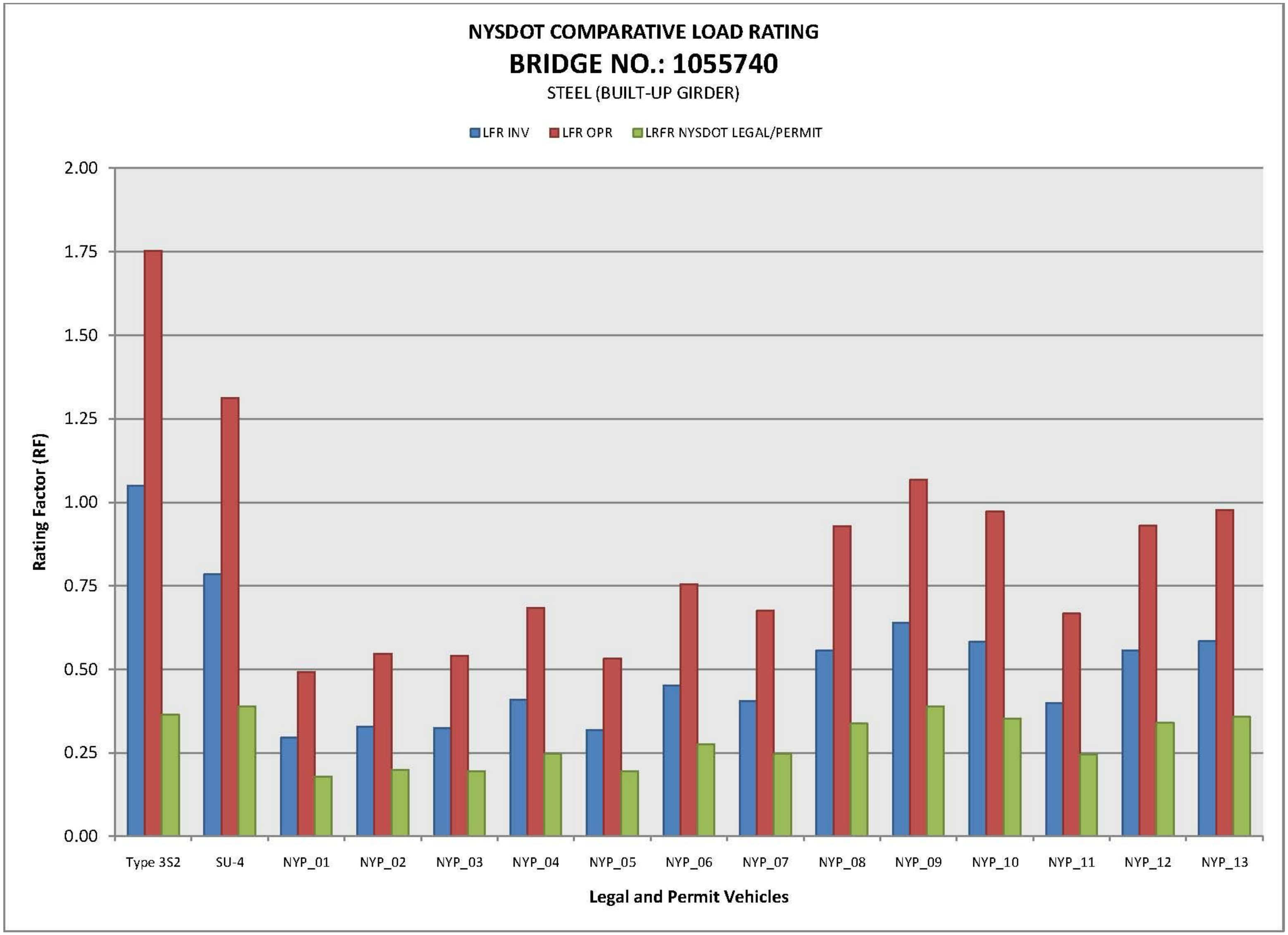
BRIDGE NO.: 1040802
STEEL (WELDED PLATE GIRDER)
LFR INV
LFR OPR
LRFR NYSDOT LEGAL/PERMIT
Rating Factor (RF)
2.00
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Type 3S2
SU-4
NYP_01
NYP_02
NYP_03
NYP_04
NYP_05
NYP_06
NYP_07
NYP_08
NYP_09
NYP_10
NYP_11
NYP_12
NYP_13
Legal and Permit Vehicles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BRIDGE NO.: 1055740
STEEL (BUILT-UP GIRDER)
LFR INV
LFR OPR
LRFR NYSDOT LEGAL/PERMIT
Rating Factor (RF)
2.00
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Type 3S2
SU-4
NYP_01
NYP_02
NYP_03
NYP_04
NYP_05
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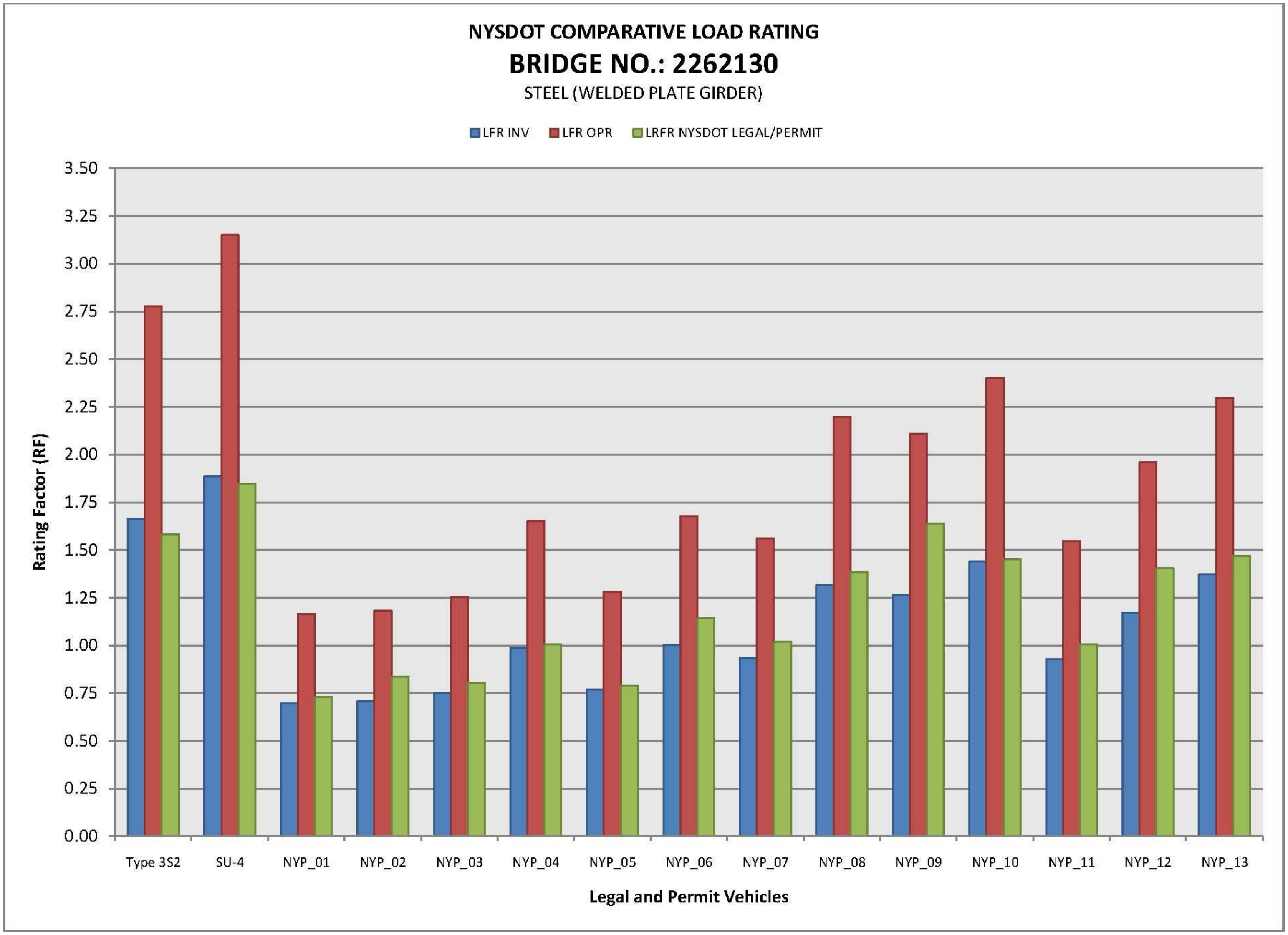
NYP_13
Legal and Permit Vehicles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BRIDGE NO.: 2262130
STEEL (WELDED PLATE GIRDER)
LFR INV
LFR OPR
LRFR NYSDOT LEGAL/PERMIT
Rating Factor (RF)
3.50
3.25
3.00
2.75
2.50
2.25
2.00
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Type 3S2
SU-4
NYP_01
NYP_02
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Legal and Permit Vehicles