U.S. Department
of Transportation
Federal Highway Administration

Memorandum

Subject: **<u>INFORMATION:</u>** Bridge Load Ratings for the National Bridge Inventory

Date: October 30, 2006

/s/Original Signed by

From: M. Myint Lwin, P.E., S.E.
Director, Office of Bridge Technology

Reply to
Attn. of: HIBT-30

To: Directors of Field Services
Division Administrators
Federal Lands Highway Division Engineers

Several State and FHWA Bridge Engineers have suggested that we clarify our policy regarding the appropriate methodology and loads to be used in reporting operating and inventory rating data (Items 63, 64, 65 and 66 of the 1995 Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (Coding Guide), Report No. FHWA-PD-96-001) to the National Bridge Inventory (NBI). An overview of our past bridge load rating policies are provided in the attached appendix and our current policy and future direction is provided herein.

With the adoption of the AASHTO Load and Resistance Factor Design (LRFD) Specifications, our June 28, 2000, policy memorandum requiring all new bridges to be designed by the LRFD Specifications after October 1, 2007, and the ongoing effort to merge the Manual for Condition Evaluation of Bridges and the Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges (LRFR Manual), we believe that it is necessary to accommodate and support Load and Resistance Factor Rating (LRFR), while continuing to accept Load Factor Rating (LFR) for the large inventory of in-service bridges that have been designed by another method other than LRFD. The FHWA does not intend to mandate re-rating existing and valid bridge load ratings by LRFR.

Therefore, FHWA's policy for Items 63, 64, 65, and 66 of the Coding Guide is as follows (see **Table 1** for more information):

1. For bridges and total replacement bridges designed by LRFD Specifications using HL-93 loading, prior to October 1, 2010, Items 63, 64, 65 and 66 are to be computed and reported to the NBI as either a Rating Factor (RF) or in metric tons. Rating factors shall be based on LRFR methods using HL-93 loading (see **Appendix A – Example 1**) or LFR methods using MS18 loading (see **Appendix A – Example 2**). Metric ton rating values

shall be reported in terms of MS18 (32.4 metric tons) loading derived from a RF calculated using LRFR methods and HL-93 loading, or LFR methods using MS18 loading (see **Appendix A – Example 3**).

2. For bridges and total replacement bridges designed by LRFD Specifications using HL-93, after October 1, 2010 Items 63, 64, 65 and 66 are to be computed and reported to the NBI as a RF based on LRFR methods using HL-93 loading (see **Appendix A – Example 1**).

3. For bridges designed or reconstructed by either Allowable Stress Design (ASD) or Load Factor Design (LFD) Specifications, Items 63, 64, 65 and 66 are to be computed and reported to the NBI as a RF or in metric tons. Rating factors shall be based on LRFR methods using HL-93 loading (see **Appendix A – Example 1**) or LFR methods using MS18 loading (see **Appendix A – Example 2**). Metric ton rating values shall be reported in terms of MS18 (32.4 metric tons) loading derived from a RF calculated using LRFR methods and HL-93 loading, or LFR methods using MS18 loading (see **Appendix A – Example 3**)

4. For bridges partially reconstructed resulting in the use of combination specifications (e.g. a reconstructed superstructure designed by LRFD supported by the original substructure designed by ASD) or unknown specifications, Items 63, 64, 65 and 66 are to be computed and reported to the NBI as a RF or in metric tons. Rating factors shall be based on LRFR methods using HL-93 loading (see **Appendix A – Example 1**) or LFR methods using MS18 loading (see **Appendix A – Example 2**). Metric ton rating values shall be reported in terms of MS18 (32.4 metric tons) loading derived from a RF calculated using LRFR methods and HL-93 loading, or LFR methods using MS18 loading (see **Appendix A – Example 3**)

5. For bridges designed or reconstructed by either ASD or LFD Specifications and for bridges partially reconstructed resulting in the use of combination specifications or unknown specifications, after October 1, 2010, Items 63, 64, 65 and 66 are to be computed and reported to the NBI as a RF or in metric tons. Rating factors shall be based on LRFR methods using HL-93 loading (see **Appendix A – Example 1**) or LFR methods using MS18 loading (see **Appendix A – Example 2**). Metric ton rating values shall be based on LFR methods using MS18 loading (see **Appendix A – Example 3**). The NBI Code of 3 (Load and Resistance Factor Rating reported in metric tons using MS loading) for Items 63 and 65 will no longer be valid for new load ratings of new or existing bridges after October 1, 2010 (see **Appendix C**).

6. For bridges load rated by load testing methods, Items 63, 64, 65 and 66 are to be computed and reported to the NBI as Load Testing in metric tons based on MS18 loading, even though the actual load test was likely performed with another vehicle configuration.

7. For those cases where the condition or the loading of a bridge warrants a re-rating (existing load rating is invalid), follow the Load Rating Methodology Options presented in **Table 1** for computing and reporting Items 63, 64, 65 and 66.

It is recognized that there will be situations that require engineering judgment with respect to the selection of an appropriate rating method for computing and reporting Items 63, 64, 65 and 66. For example, States have the option of LRFR, LFR, or Allowable Stress Rating (ASR) for timber and masonry bridges. Please work with your State DOT to develop consistent procedures for these exceptions to policy.

Policy exceptions and reporting procedures to the NBI may be revised in the future once LRFR methods and software are further developed and the Coding Guide is updated. For example, as proposed in the update to the Coding Guide, future reporting of load ratings in the NBI will likely be based entirely on RF rather than tons. The options in Table 1 will be revised to accommodate future changes as they occur.

As in the past, the load rating used to report NBI Item 70, Bridge Posting may be computed either by LRFR, LFR, or ASR methods using the maximum unrestricted legal loads to establish load limits for the purpose of load posting. Item 70 evaluates the load capacity of a bridge in comparison to the State legal loads. For load ratings based on LRFR methods using an HL-93 loading, this item represents the minimum LRFR of all legal load configurations in the State (e.g. if the minimum LRFR of all State legal loads = 0.85, then by using the current Coding Guide table, Item 70 would be coded a 3).

Please share this clarification with your State DOT counterparts and feel free to contact either Everett Matias (202) 366-6712 (everett.matias@dot.gov) or Gary Moss (202) 366-4654 (gary.moss@dot.gov) if any further questions arise.

Attachments

Table 1: RATING METHODS FOR COMPUTING AND REPORTING CODING GUIDE ITEMS 63, 64, 65 AND 66

DESIGN OR RECONSTRUCTION SPECIFICATION USED	EXISTING AND VALID LOAD RATING	LOAD RATING OR RE-RATING METHODOLOGY OPTIONS	LOADING	CODING GUIDE ITEMS			
				63	**64**	**65**	**66**
Load and Resistance Factor Design (LRFD)	None or Invalid	LRFR	HL-93	8	Rating Factor (RF)	8	Rating Factor (RF)
		LRFR	MS18[5]	3 [2]	Metric Tons	3 [2]	Metric Tons
		LFR [1]	MS18	6	Rating Factor (RF)	6	Rating Factor (RF)
		LFR [1]	MS18	1	Metric Tons	1	Metric Tons
		ASR [4]	MS18	7	Rating Factor (RF)	7	Rating Factor (RF)
		ASR [4]	MS18	2	Metric Tons	2	Metric Tons
	Load and Resistance Factor Rating (LRFR)	LRFR	HL-93	8	Rating Factor (RF)	8	Rating Factor (RF)
		LRFR	MS18[5]	3 [2]	Metric Tons	3 [2]	Metric Tons
	Load Factor Rating (LFR) or Allowable Stress Rating (ASR)	LRFR	HL-93	8	Rating Factor (RF)	8	Rating Factor (RF)
		LRFR	MS18[5]	3 [2]	Metric Tons	3 [2]	Metric Tons
		LFR	MS18	6	Rating Factor (RF)	6	Rating Factor (RF)
		LFR	MS18	1	Metric Tons	1	Metric Tons
		ASR [3,4]	MS18	7	Rating Factor (RF)	7	Rating Factor (RF)
		ASR [3,4]	MS18	2	Metric Tons	2	Metric Tons
	Load Testing	Load Testing	Equivalent MS18	4	Metric Tons	4	Metric Tons
Load Factor Design (LFD) or Allowable Stress Design (ASD)	None or Invalid	LRFR	HL-93	8	Rating Factor (RF)	8	Rating Factor (RF)
		LRFR	MS18[5]	3 [2]	Metric Tons	3 [2]	Metric Tons
		LFR	MS18	6	Rating Factor (RF)	6	Rating Factor (RF)
		LFR	MS18	1	Metric Tons	1	Metric Tons
		ASR [4]	MS18	7	Rating Factor (RF)	7	Rating Factor (RF)
		ASR [4]	MS18	2	Metric Tons	2	Metric Tons
	Load and Resistance Factor Rating (LRFR)	LRFR	HL-93	8	Rating Factor (RF)	8	Rating Factor (RF)
		LRFR	MS18[5]	3 [2]	Metric Tons	3 [2]	Metric Tons
	Load Factor Rating (LFR) or Allowable Stress Rating (ASR)	LRFR	HL-93	8	Rating Factor (RF)	8	Rating Factor (RF)
		LRFR	MS18[5]	3 [2]	Metric Tons	3 [2]	Metric Tons
		LFR	MS18	6	Rating Factor (RF)	6	Rating Factor (RF)
		LFR	MS18	1	Metric Tons	1	Metric Tons
		ASR [3,4]	MS18	7	Rating Factor (RF)	7	Rating Factor (RF)
		ASR [3,4]	MS18	2	Metric Tons	2	Metric Tons
	Load Testing	Load Testing	Equivalent MS18	4	Metric Tons	4	Metric Tons
Combination of Specifications (LRFD, LFD, ASD) or Unknown	None or Invalid	LRFR	HL-93	8	Rating Factor (RF)	8	Rating Factor (RF)
		LRFR	MS18[5]	3 [2]	Metric Tons	3 [2]	Metric Tons
		LFR	MS18	6	Rating Factor (RF)	6	Rating Factor (RF)
		LFR	MS18	1	Metric Tons	1	Metric Tons
		ASR [4]	MS18	7	Rating Factor (RF)	7	Rating Factor (RF)
		ASR [4]	MS18	2	Metric Tons	2	Metric Tons
		Load Testing	Equivalent MS18	4	Metric Tons	4	Metric Tons
	Load and Resistance Factor Rating (LRFR)	LRFR	HL-93	8	Rating Factor (RF)	8	Rating Factor (RF)
		LRFR	MS18[5]	3 [2]	Metric Tons	3 [2]	Metric Tons
	Load Factor Rating (LFR) or Allowable Stress Rating (ASR)	LRFR	HL-93	8	Rating Factor (RF)	8	Rating Factor (RF)
		LRFR	MS18[5]	3 [2]	Metric Tons	3 [2]	Metric Tons
		LFR	MS18	6	Rating Factor (RF)	6	Rating Factor (RF)
		LFR	MS18	1	Metric Tons	1	Metric Tons
		ASR [3,4]	MS18	7	Rating Factor (RF)	7	Rating Factor (RF)
		ASR [3,4]	MS18	2	Metric Tons	2	Metric Tons
	Load Testing	Load Testing	Equivalent MS18	4	Metric Tons	4	Metric Tons

[1] Bridges and Total Replacement Bridges Designed by LRFD prior to October 1, 2010. Bridges and Total Replacement Bridges Designed by LRFD after October 1, 2010 are to be computed and reported based on LRFR methods.
[2] The NBI Code of 3 for Items 63 and 65 will no longer be valid for new load ratings of new or existing bridges after October 1, 2010.
[3] Non-NHS Bridges Constructed, Replaced, Rehabilitated and Load Rated prior to January 1, 1994. Bridges Load Rated or Re-Rated after January 1, 1994 are to be computed and reported based on LFR or LRFR methods.
[4] Policy exceptions such as timber and masonry bridges.
[5] Report metric tons in terms of MS18 (32.4 metric tons) loading derived from a RF calculated using LRFR methods and HL-93 loading.

Appendix – A
Examples

Input options for Coding Guide Items 63 and 65 (March 22, 2004 memorandum)

Code	Description
1	Load factor (LF) reported in metric tons using MS18 loading.
2	Allowable Stress (AS) reported in metric tons using MS18 loading.
3	Load and Resistance Factor Rating (LRFR) reported in metric tons using MS18 loading.
4	Load testing reported in metric tons using equivalent MS18 loading.
5	No rating analysis performed.
6	Load Factor (LF) rating reported by rating factor (RF) method using MS18 loading.
7	Allowable Stress (AS) rating reported by rating factor (RF) method using MS18 loading.
8	Load and Resistance Factor Rating (LRFR) rating reported by rating factor (RF) method using HL-93 loading.

Example 1:

Given:
LRFR of HL-93 Loading
Computed Operating Rating Factor = 1.17
Computed Inventory Rating Factor = 0.90

Therefore:
Code Item 63: 8
Code Item 64: 117
Code Item 65: 8
Code Item 66: 090

Example 2:

Given:
LFR of MS18 Loading by Rating Factor
Computed Operating Rating Factor = 54.1/32.4 = 1.67
Computed Inventory Rating Factor = 32.4/32.4 = 1.00

Therefore:
Code Item 63: 6
Code Item 64: 167
Code Item 65: 6
Code Item 66: 100

Example 3:

Given:
LFR of MS18 Loading in metric tons
Computed Operating Rating = 54.1 metric tons
Computed Inventory Rating = 32.4 metric tons

Therefore:
Code Item 63:1
Code Item 64: 541
Code Item 65: 1
Code Item 66: 324

Appendix – B
Background and History

The FHWA memoranda issued on November 5, 1993 and December 22, 1993, and the Coding Guide established a policy whereby the operating and inventory ratings (Items 64 and 66) of all bridges constructed, replaced, or rehabilitated after January 1, 1994, as reported to the NBI were to be computed by the LFR method using MS loading (HS metric equivalent) as the national standard. In addition, the load ratings of all bridges that did not have a valid load rating or required a re-rating due to changes in condition or loading were to be computed by the LFR method. Through our field offices, target dates were established with the State DOT's for updating all NBI load ratings using the LFR method, starting with all bridges on the National Highway System (NHS). For bridges off of the NHS that were constructed, replaced, or rehabilitated prior to January 1, 1994, a valid load rating computed by LFR, ASR or LRFR was acceptable. For any bridge that required posting or overweight load permits, States had the option of using LFR, ASR, or LRFR methods to establish load limits.

With the adoption of the AASHTO LRFD Specifications, FHWA issued a proposal letter, dated April 19, 2000, to the Chairman of the AASHTO Technical Subcommittee on Bridge Management, Evaluation, and Rehabilitation (T-18) requiring all new load ratings to be computed and reported to the NBI by the LRFR method. Bridges previously designed and currently under design using LRFD were to be rated by the LFR or LRFR methods, until adoption of the LRFR Manual. FHWA also proposed that within 10 years of adoption, all load ratings in the NBI would be in accordance with the LRFR Manual. In recognition of the state-of-development and understanding of the LRFR methodology, and concern by the AASHTO State members over the resources required to re-rate all bridges once again, FHWA rescinded the April 19, 2000, proposal letter via a second letter to the Chairman of T-18 on November 15, 2001.

Since that time, the bridge community's understanding of LRFD and LRFR methods has improved and several State DOT's have started using the load and resistance factor method for design and rating of bridges. On June 28, 2000, FHWA issued a policy memorandum that required all new bridges be designed by LRFD Specifications after October 1, 2007, and all new culverts, retaining walls and other standard structures be designed by LRFD Specifications after October 1, 2010. For modification to existing structures, States had the option of using LRFD Specifications or the specifications that were used for the original design.

Our March 22, 2004, memorandum revised the Coding Guide by providing three additional codes to the Method Used to Determine Operating Rating and Method Used to Determine Inventory Rating (Items 63 and 65). The additions were made to accommodate the reporting of RF determined by LRFR, LFR, or ASR methods. This memorandum did not require bridges to be rated or re-rated using LRFR methods, nor did it change our position on using LFR with MS loading as the preferred method for bridges designed by LFD or ASD. Instead, this memorandum provided the additional option of reporting RF and encouraged the use of LRFR methods with HL-93 loading for all new and reconstructed bridges that were designed by LRFD Specifications.

At the request of T-18, FHWA produced a report in June 2005 titled, the *Impact of Load Rating Methods on Federal Bridge Program Funding*. A copy of the report is available at http://www.fhwa.dot.gov/bridge/bridgeload01.cfm. The report concluded that there would be less than a 2 percent change in deck area on deficient bridges if all the inventory ratings were suddenly based on LRFR. The report also indicated that implementation of LRFR is likely to occur gradually, making any changes in deck area on deficient bridges, and therefore Federal bridge funding levels, difficult to detect.

Based on the results of our study, the advancement and development of LRFD and LRFR methodologies, and our March 22, 2004, memorandum, FHWA's current practice is to accept the reporting of operating and inventory load ratings as outlined in Table 1.

Copies of each of the referenced memorandums are available on our website at http://www.fhwa.dot.gov/bridge/memos.htm.

Appendix – C
Code of 3 - Load and Resistance Factor Rating (LRFR) reported in metric tons using MS loading

The code of 3 (Load and Resistance Factor Rating (LRFR) reported in metric tons using MS loading) for Items 63 and 65 had been included in the Coding Guide prior to the full development of the LRFD Specifications and the LRFR Manual. With the adoption of the LRFR Manual, load ratings computed by LRFR methods produce a RF based on HL-93 loading. An HL-93 loading cannot be equated to an MS loading, therefore a direct conversion from RF to MS loading is not possible.

A valid code of 3 involves reporting metric tons in terms of MS18 (32.4 metric tons) loading derived from a RF calculated using LRFR methods and HL-93 loading. This procedure does not produce an equivalent MS18 load. This procedure enables those States that utilize PONTIS, which currently does not support RF's, to input a LRFR as a tonnage value.

The methodology for LRFR allows the user to verify bridge safety and serviceability through a number of distinct procedures (LRFR Manual APPENDIX A.6.1, LOAD AND RESISTANCE FACTOR RATING FLOW CHART). Following these procedures, a RF based on an HL-93 loading will always be calculated while the RF for Legal Loads may be calculated. Therefore, it is the intent of FHWA that the NBI Code of 3 for Items 63 and 65 will no longer be valid for new load ratings of new or existing bridges after October 1, 2010. Bridges currently and correctly coded a 3 are not required to be re-rated.

Alaska DOT

FEDERAL HIGHWAY ADMINISTRATION

LOAD & RESISTANCE FACTOR RATING OF HIGHWAY BRIDGES

April 2008

Thomas Saad, P.E.

FHWA

FHWA MEMORANDUM: OCT. 30, 2006

BRIDGE LOAD RATING FOR THE NATIONAL BRIDGE INVENTORY

For bridges and total replacement bridges designed by LRFD Specifications using HL-93 loading, prior to October 1, 2010, Items 63, 64, 65 and 66 are to be computed and reported to the NBI as either a Rating Factor (RF) or in metric tons. Rating factors shall be based on LRFR methods using HL-93 loading (see **Appendix A - Example 1**) or LFR methods using MS18 loading (see **Appendix**

For bridges and total replacement bridges designed by LRFD Specifications using HL-93, after October 1, 2010 Items 63, 64, 65 and 66 are to be computed and reported to the NBI as a RF based on LRFR methods using HL-93 loading (see **Appendix A - Example 1**).

NHI Course 130092 – LRFR of Highway Bridges

- Comprehensive 5-day course
- Modular Delivery
  - Module I (2-day): Intro. to LRFR (Load Rating Process, Rating for Design, Legal and Permit Loads and Load Posting)
  - Module II (1-day): Load Rating of Concrete Superstructure Bridges
  - Module III (1-day): Load Rating of Steel Superstructure Bridges
  - Module IV (1-day): Load Rating of Bridge Substructures
- Under development - Available January 2009

LOAD & RESISTANCE FACTOR RATING OF HIGHWAY BRIDGES

OUTLINE OF PRESENTATION

SESSION 1:	INTRODUCTION TO LRFR
SESSION 2:	LOAD MODELS FOR LRFR
SESSION 3:	LRFR LOAD RATING PROCESS & LOAD RATING EQUATION
SESSION 4:	LRFR LIMIT STATES, RELIABILITY INDICES & LOAD FACTORS
SESSION 5:	THE NEW AASHTO MANUAL FOR BRIDGE EVALUATION
SESSION 6:	P/S GIRDER BRIDGE LRFR RATING
SESSION 7:	STEEL GIRDER BRIDGE LRFR RATING

LOAD & RESISTANCE FACTOR RATING OF HIGHWAY BRIDGES

SESSION 1

INTRODUCTION TO LRFR

Effect of LRFD and LRFR Specs on Bridges

1- More Reliable and Safer Bridges

2- Increased Bridge Life

2- Meaningful Load Ratings!

LRFD / LRFR

•RELIABILITY-BASED LIMIT STATES SPECIFICATIONS

•USE PROBABILISTIC METHODS TO DERIVE LOAD & RESISTANCE FACTORS

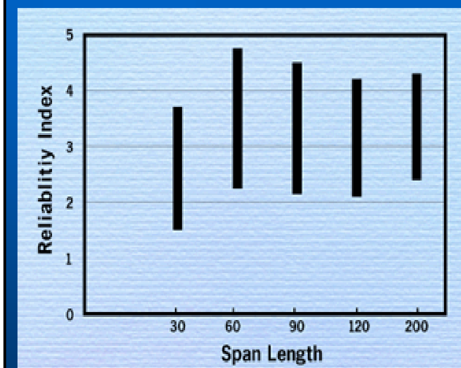
•UNIFORM RELIABILITY IN DESIGN & LOAD RATINGS / POSTINGS

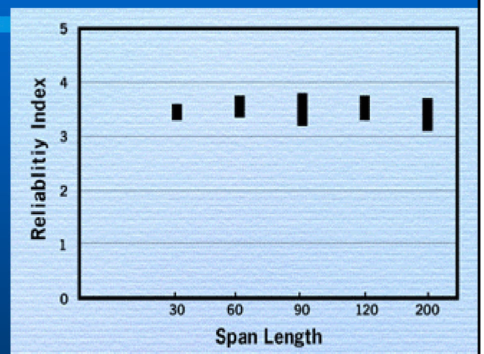
•PRESENTATION SUCH THAT PRIOR KNOWLEDGE OF RELIABILITY WILL NOT BE NECESSARY.

LOAD FACTOR RATING METHOD

- A Strength-based load rating method
- Uncalibrated code. Load factors were established based on engineering judgment (Unknown reliability)
- No guidance on adjusting load and resistance factors for changed uncertainty in loadings or member resistance.

LRFR GOAL : UNIFORM RELIABILITY





LFD

LFR

LRFD

LRFR

CALIBRATION OF LIMIT STATES

- Only the Strength Limit State was calibrated based upon structural reliability theory. Other limit states were calibrated to current practice
- Reliability indices of bridges designed by the Standard Specs ranged from 1.5 to 4.5
- Target reliability index of 3.5 was selected for new designs.
- Design Reliability $\beta = 3.5$; 1 in 10,000 notional failure probability.
- For evaluation $\beta = 2.5$ or a 1 in 100 notional failure probability.

LIMIT STATE EQUATION

FACTORED LOAD EFFECT ≤ FACTORED RESISTANCE

$$\eta_D \, \eta_R \, \eta_I \sum \gamma_i \, Q_i \leq \phi \, R_n$$

where:

η_D	=	ductility factor
η_R	=	redundancy factor
η_I	=	operational importance factor
γ_i	=	load factor
ϕ	=	resistance factor
Q_i	=	force effect
R_n	=	nominal resistance

LRFR RATING EQUATION

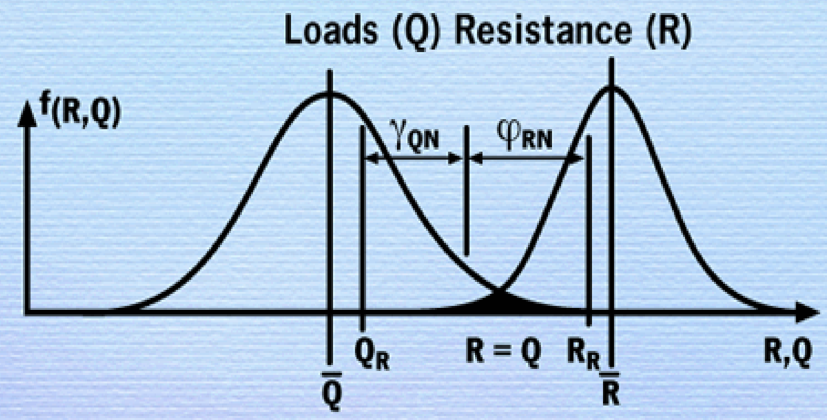
$$RF = \frac{\phi_C \phi_S \phi R - \gamma_{DC} DC - \gamma_{DW} DW}{\gamma_L (\mathrm{LL+IM})}$$

$$\phi_C \phi_S \geq 0.85$$

ϕ_S SYSTEM FACTOR FOR REDUNDANCY

ϕ_C MEMBER CONDITION FACTOR

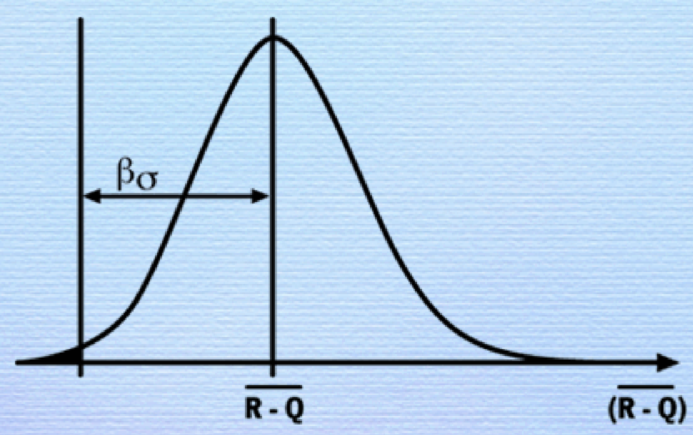
PROBABILISTIC DESIGN & EVALUATION



Each variable represented by mean and standard deviation.

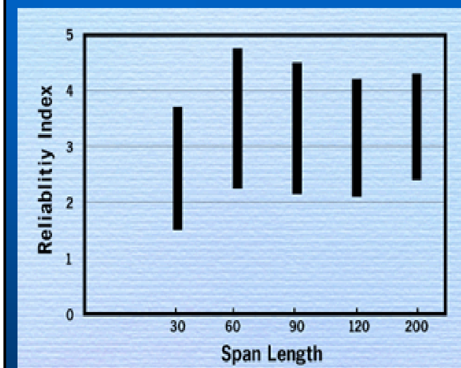
RELIABILITY INDEX β

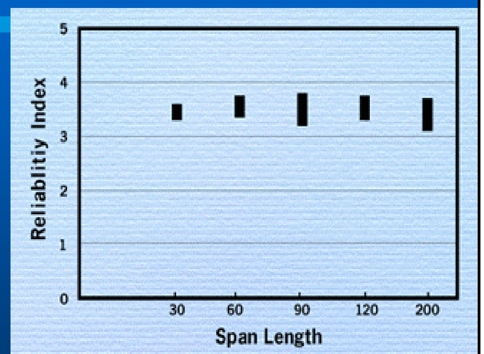
New Measure of Safety



R-Q = SAFETY MARGIN

LRFR GOAL : UNIFORM RELIABILITY





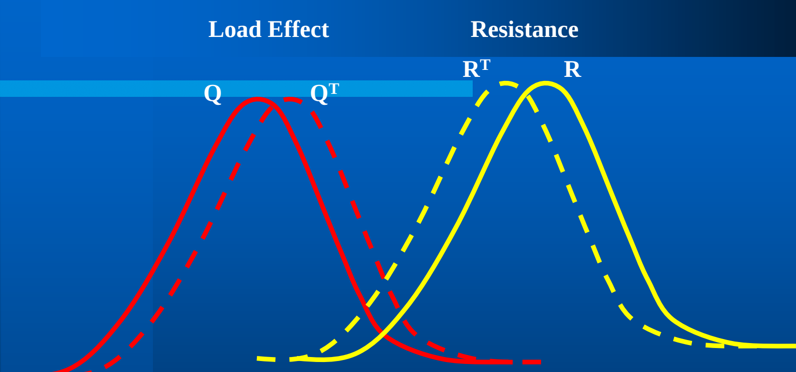
LFD

LFR

LRFD

LRFR

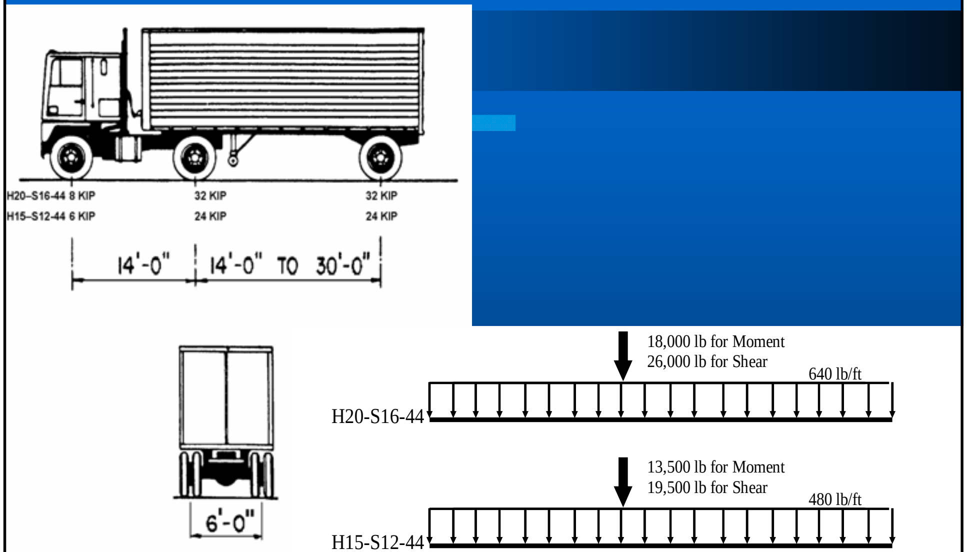
Time Dependant Reliability



Loads (Q^T) Increase, Resistance (R^T) Decreases

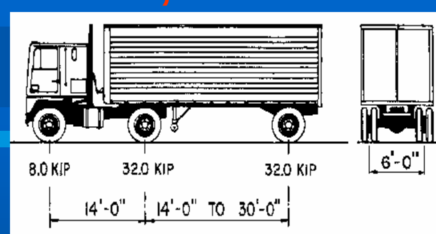
Reliability Decreases with Time

1944 Live Loads



LRFD Live Load, HL-93

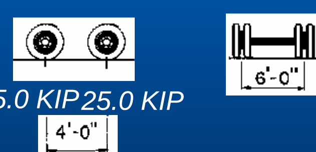
- ***Design Truck:*** ⇒



or

or

- ***Design Tandem:***

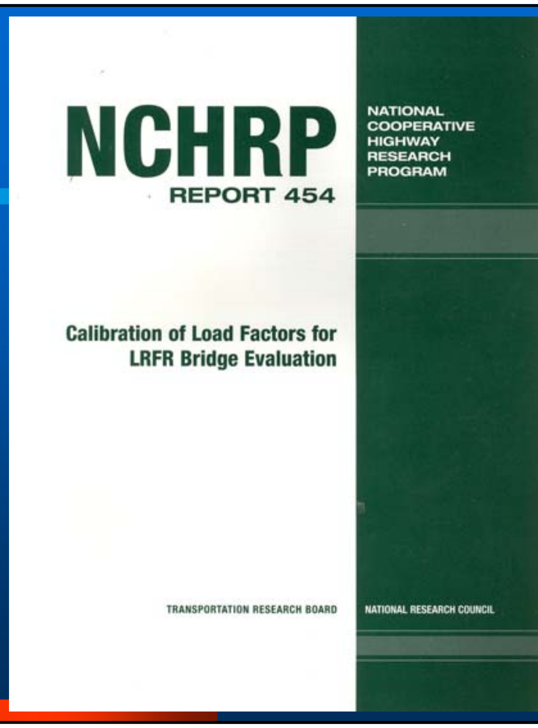


superimposed on

\+

- ***Design Lane Load***

0.64 Kip/ft


NCHRP

REPORT 454

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Calibration of Load Factors for LRFR Bridge Evaluation

TRANSPORTATION RESEARCH BOARD

NATIONAL RESEARCH COUNCIL

LRFR CALIBRATION

- The calibration uses a data base consistent with the calibration of the AASHTO LRFD Bridge Design Specifications.
- Live load factors are given for legal load rating, posting, and permit load checking.
- Live load factors could utilize, where available, site specific traffic information.
- Target safety indices are calibrated to bridge performance history using AASHTO LFR rating methods.
- Uniform consistent target reliabilities are achieved with the new LRFR bridge evaluation format.

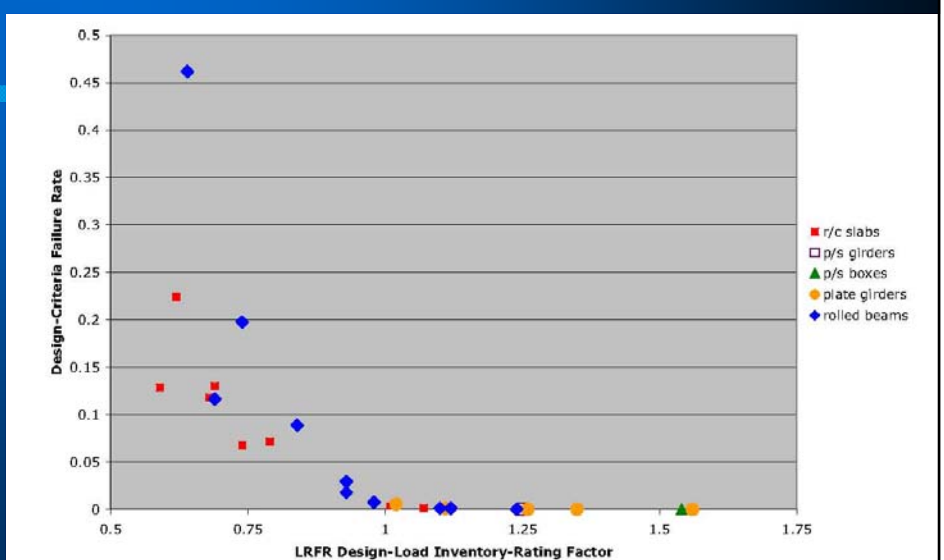
NCHRP PROJECT 20-07/Task 122

LOAD RATING BY LOAD AND RESISTANCE FACTOR EVALUATION METHOD

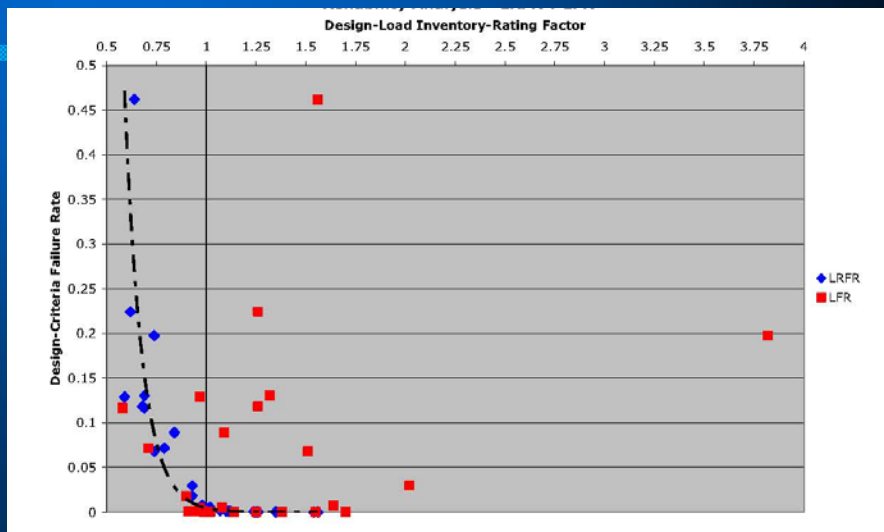
FINAL REPORT JUNE 2005

- The objective of this project is to provide explicit comparisons between the ratings produced by the LRFR method and Load Factor ratings (LFR).
- The comparisons of 74 bridges are based upon flexural-strength ratings.
- Design criteria failure rates & reliability indices determined using Monte Carlo simulations.

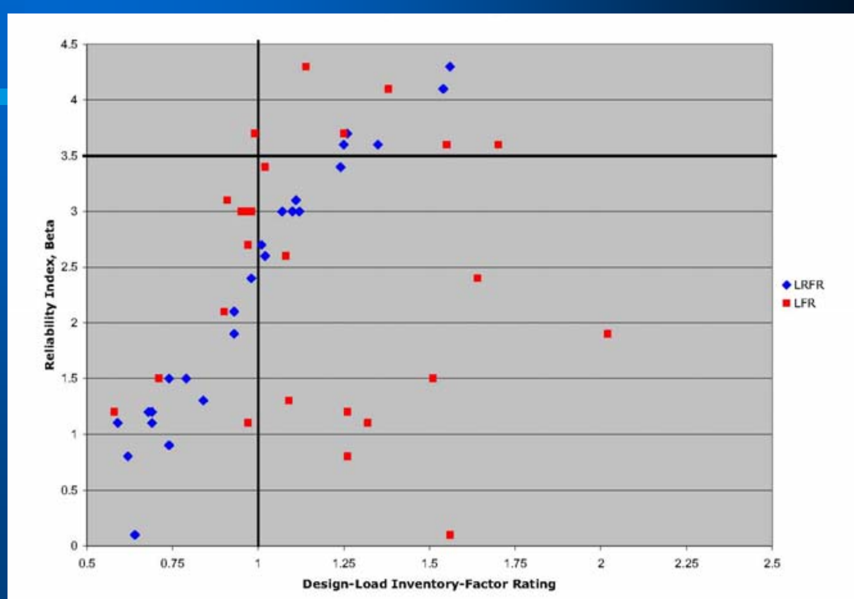
FAILURE RATE VERSUS LRFR HL-93 INVENTORY RATING FACTOR



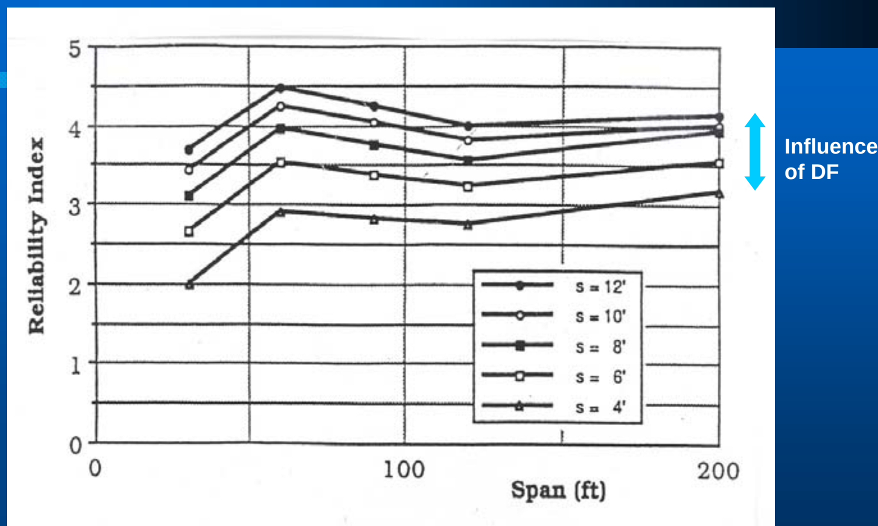
FAILURE RATE VERSUS LRFR & LFR DESIGN LOAD INVENTORY RATING FACTORS



RELIABILITY INDEX VERSUS LRFR & LFR DESIGN LOAD INVENTORY RATING FACTORS

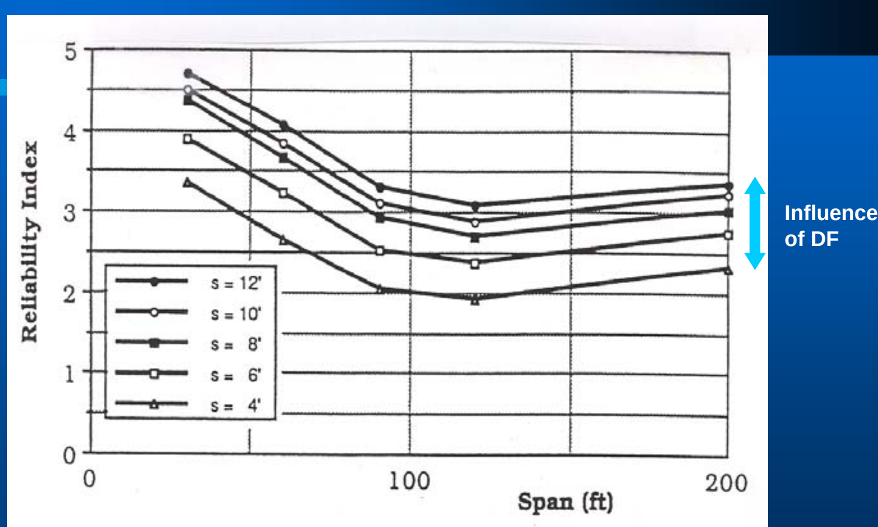


RELIABILITY INDICES FOR AASHTO STD. SPECS.
SIMPLE SPAN MOMENTS IN STEEL GIRDERS



Ref: NCHRP 12-33 Calibration of LRFD Bridge Design Code

RELIABILITY INDICES FOR AASHTO STD. SPECS.
SIMPLE SPAN SHEAR IN STEEL GIRDERS



Ref: NCHRP 12-33 Calibration of LRFD Bridge Design Code

RELIABILITY-BASED EVALUATION OF EXISTING BRIDGES

CAN THE LRFD CODE BE USED FOR EVALUATION?

EVALUATION IS NOT THE INVERSE OF DESIGN

DESIGN vs EVALUATION

	Design	Evaluation
Added cost of conservatism	Marginal	Can be prohibitive
Exposure period	75 years	2 years
Live load uncertainty	High	Lower
Resistance uncertainty	Low	Higher

CAN THE LRFD CODE BE USED FOR EVALUATION?

- Is the design level reliability of 3.5 appropriate for evaluation?
- Existing bridges have a beta in the 2.5 to 4.5 range (many below 3.5).
- Should all serviceability limit states be imposed on existing bridges?
- How should overweight permit loads be evaluated?
- Design impact factors (33%) may be too conservative for evaluation.

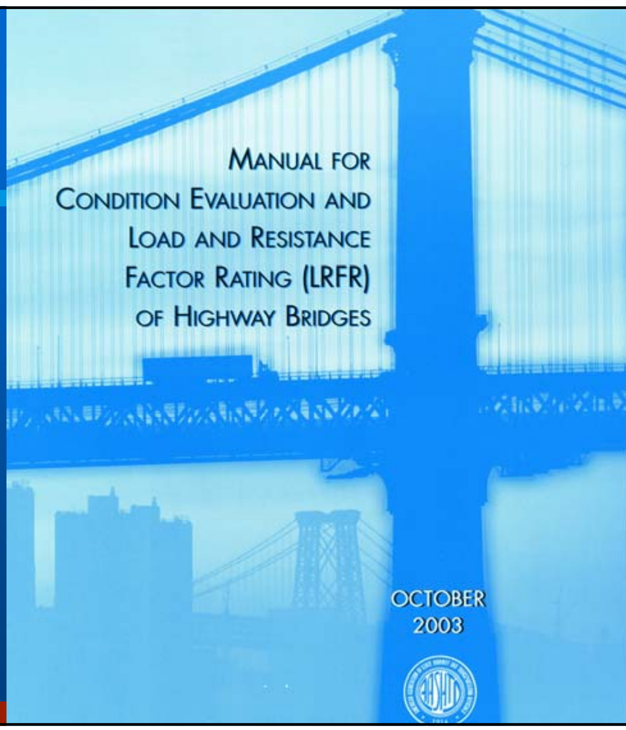
CAN THE LRFD CODE BE USED FOR EVALUATION?

- The HL-93 loading does not bear any resemblance to actual trucks.
- HL-93 is a notional load not suitable for posting.
- LRFD only addresses redundant superstructure systems. Many existing bridges are non-redundant.
- LRFD is focused on new bridges, not degraded bridges.
- Older materials & connections (rivets) are not covered in the LRFD Specs.

EXTENDING THE LRFD PHILOSOPHY TO EXISTING BRIDGES

AASHTO LRFR Manual --- Oct 2003

- Companion Document to the LRFD Specifications.
- Calibrated Load and Resistance Factors for Existing Bridges
- Provides Guidance on Older Materials and Bridge Types
- Includes Allowable Stress & Load Factor Ratings in Appendix.



National Cooperative Highway Research Program
Project 12-46
1997 -- 2002

AASHTO *GUIDE SPECIFICATIONS FOR CONDITION EVALUATION & LOAD AND RESISTANCE FACTOR RATING (LRFR) OF HIGHWAY BRIDGES*

Published October 2003

GOALS FOR THE NEW LRFR MANUAL

- Maintain consistency with LRFD philosophy & codes.
- Presentation such that prior knowledge of reliability methods will not be necessary.
- Replace the 1994 AASHTO *Manual for Condition Evaluation of Bridges* (MCE).

MAJOR OVERHAUL OF 1994 AASHTO CONDITION EVALUATION MANUAL

2003 LRFR GUIDE MANUAL

- Only inspection & material testing sections were retained
- New sections:
  - Load and Resistance Factor Rating (LRFR)
  - Fatigue evaluation of bridges
  - Non-destructive load testing of Bridges
  - Introduction to Bridge Management Systems
- Parallel commentary & many Illustrative examples.
- Includes Allowable Stress & Load Factor rating methods as alternate rating methods.

AASHTO LRFR MANUAL

TABLE OF CONTENTS

LOAD & RESISTANCE FACTOR RATING

Can be used for the load rating of:

- Existing Bridges Designed Using the Standard Specifications:
- New Bridges Designed Using the LRFD Specifications

LRFR Rating of New Bridges Designed Using the LRFD Specifications

- Ratings can be checked for:
  - In-service LRFD bridges, or
  - For bridges still in the design phase.
- Rating consistent with the design approach.
- Maintains uniform reliability as the basis for design and evaluation
- Consistent treatment of serviceability criteria over the Life of the bridge.

LRFR Rating of Existing Bridges Designed Using the Standard Specifications

- Introduces uniform reliability as the basis for evaluation, load posting, and overload permitting.
- Promotes more confidence in rating and posting values.
- Introduces superior serviceability criteria that could guide inspections and enhance long-term maintainability.
- Provides guidance for evaluation of overloads
- Introduces state-of-the-art technologies that could benefit existing bridges.

2005 AASHTO BRIDGE MEETING

AASHTO Adopted the LRFR Manual to replace the 1994 *Manual for Condition Evaluation* with the following modifications:

- Change title to " The Manual for Bridge Evaluation"
- Include LRFR, Load Factor and Allowable Stress rating methods in Section 6 of the new Manual.
- Update to be consistent with LRFD Latest Edition.
- New Manual was completed in Spring 2007.

THANK YOU

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LOAD & RESISTANCE FACTOR RATING OF HIGHWAY BRIDGES

ADOT / FHWA

SESSION 2

LOAD MODELS FOR LRFR

BALA SIVAKUMAR, P.E.

LIVE LOADS ON OUR HIGHWAYS

- FEDERAL LEGAL LOADS } < HS20
- EXCLUSION VEHICLES (Grandfathered Trucks)
- OVERWEIGHT PERMIT VEHICLES

FEDERAL TRUCK WEIGHT LIMITS

- FOUR BASIC FEDERAL WEIGHT LIMITS APPLY:
  - SINGLE AXLE (20,000 #)
  - TANDEM AXLE (34,000 #)
  - BRIDGE FORMULA B
  - GROSS VEHICLE WEIGHT (80,000 #)

$$W = 500\left[\frac{LN}{N-1} + 12N + 36\right]$$

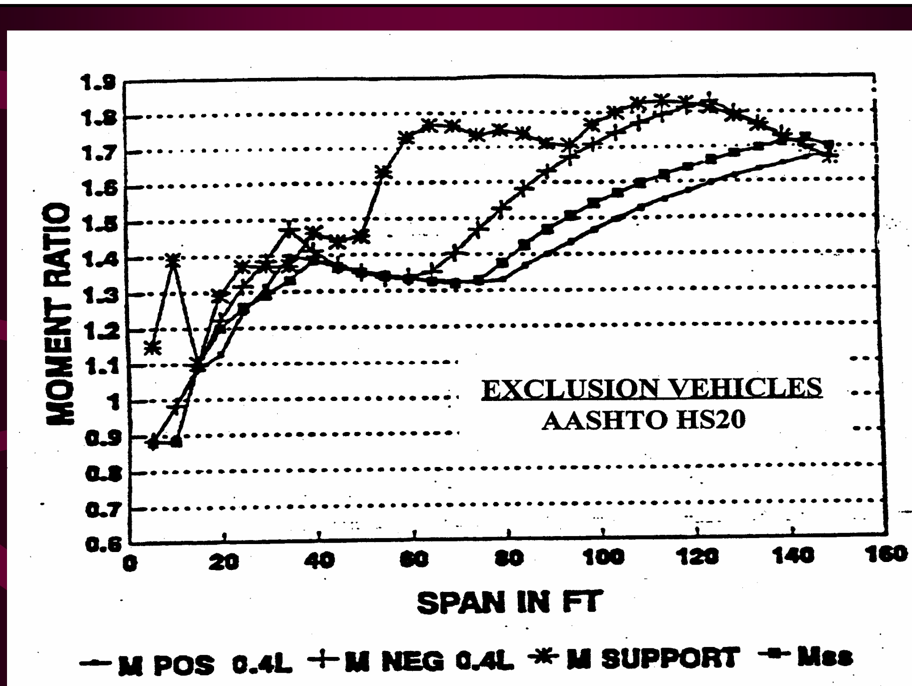
- ONLY SEVEN STATES APPLY THESE LIMITS STATEWIDE WITHOUT MODIFICATION.
- OTHER STATES ALLOW TRUCKS EXCEEDING THESE LIMITS UNDER THE "GRANDFATHER PROVISIONS".

EXCLUSION TRUCKS

MICHIGAN EXCLUSION LOAD: 3-S3-5

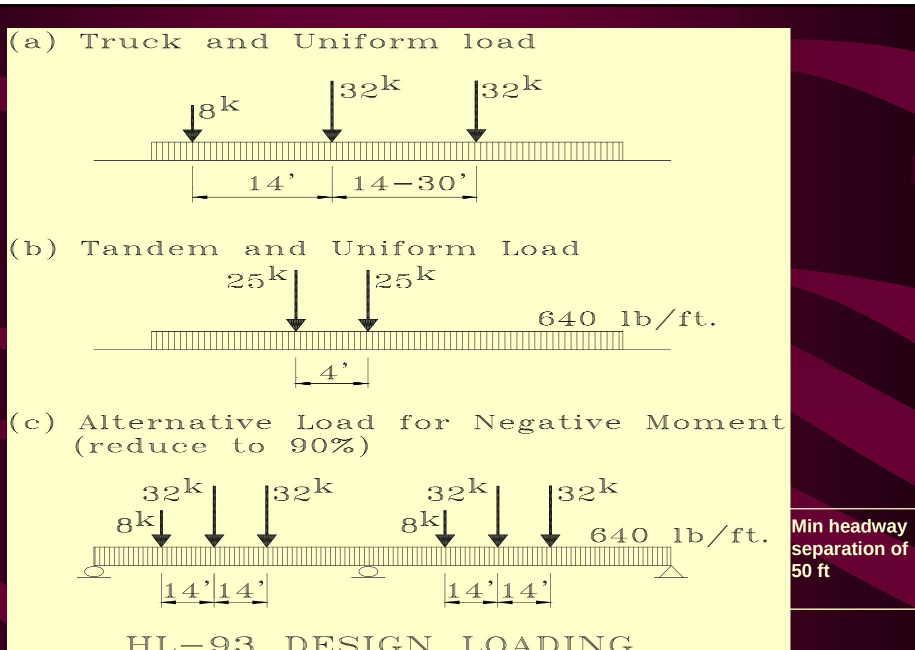
Total Weight = 149.4 Kips   Total length = 72.4 Ft

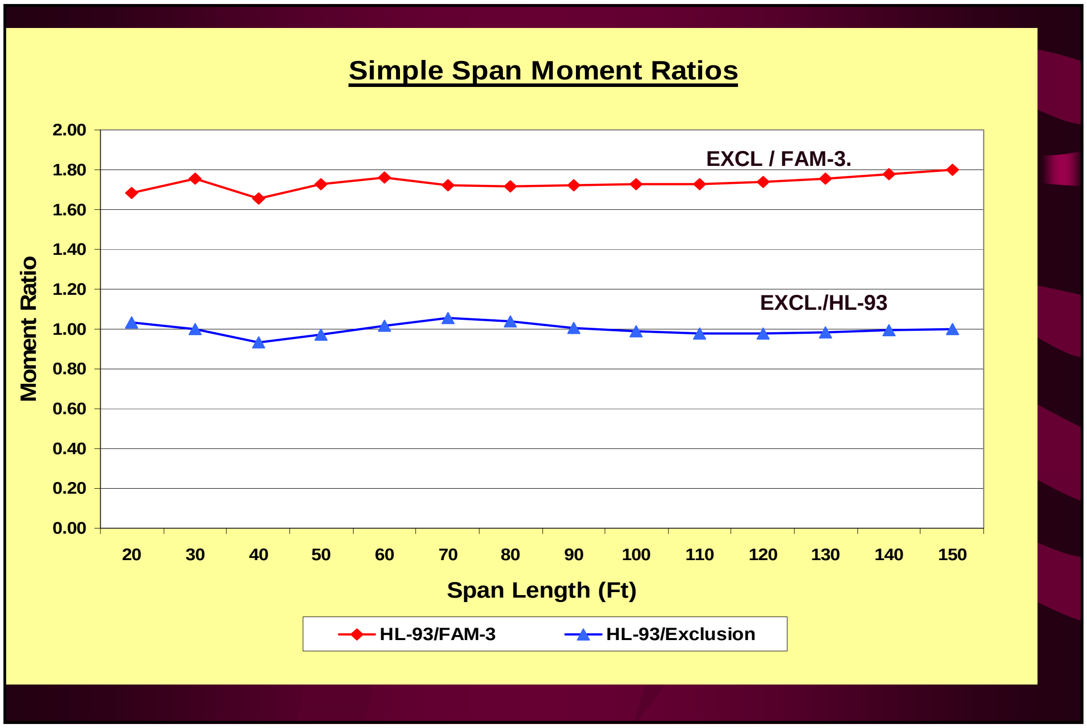
Vehicle Load Effects Enveloped by HL-93



UNIFORM RELIABILITY IN DESIGN & EVALUATION

- TO ACHIEVE UNIFORM RELIABILITY YOU NEED UNIFORM BIAS (MOMENT & SHEAR RATIOS) FOR LOAD EFFECTS ACROSS ALL SPAN LENGTHS.
- HS20 LOAD MODEL DOES NOT PROVIDE A UNIFORM BIAS
- A NEW LOAD MODEL WAS NEEDED TO ACHIEVE UNIFORM RELIABILITY





LIVE LOADS ON OUR HIGHWAYS

- FEDERAL / STATE LEGAL LOADS
- EXCLUSION VEHICLES (Grandfathered Trucks)
- OVERWEIGHT PERMIT VEHICLES

HL-93

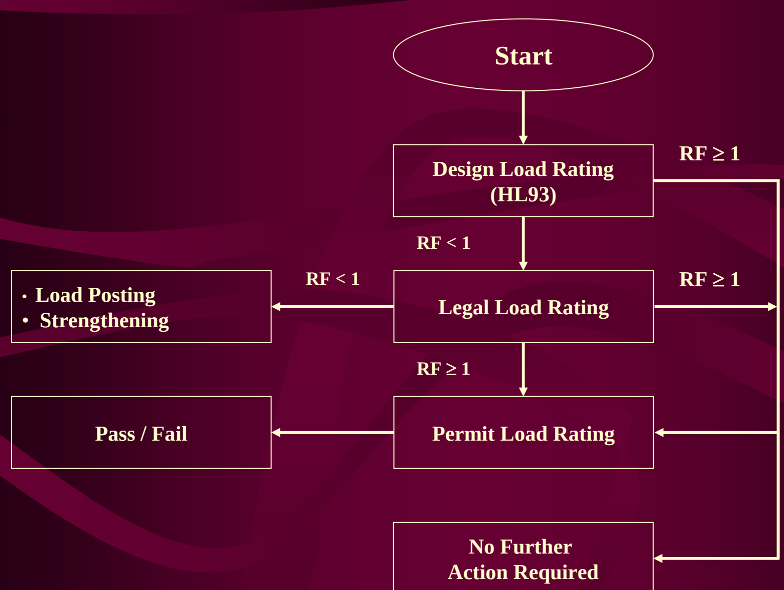
THE LRFR LOAD RATING PROCESS

1) DESIGN LOAD RATING (HL-93)

2) LEGAL LOAD RATING (POSTING)

3) PERMIT LOAD RATING (OVERWEIGHT TRUCKS)

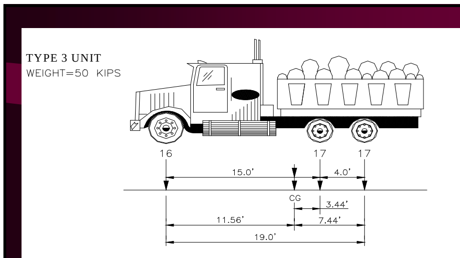
Flow Chart for LRFR Load Rating

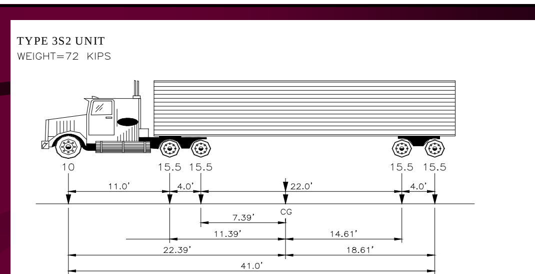


LEGAL LOAD RATING

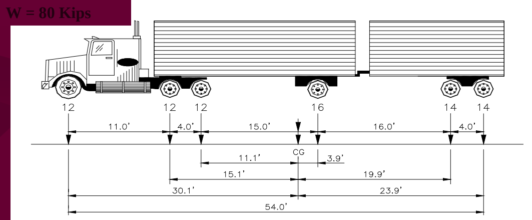
- Bridges with RF < 1.0 for HL-93 should be load rated for AASHTO & State legal loads. (Similar to HS20 check in LFR).
- Bridges with RF < 1.0 for legal loads should be posted.
- Single load rating at $\beta = 2.5$ for legal loads.
- LRFR Provides a <u>single safe load capacity for indefinite use.</u>

<u>Load Factor Operating Rating</u> --- Maximum permissible live load for the structure, suitable for one-time or Limited Crossings.



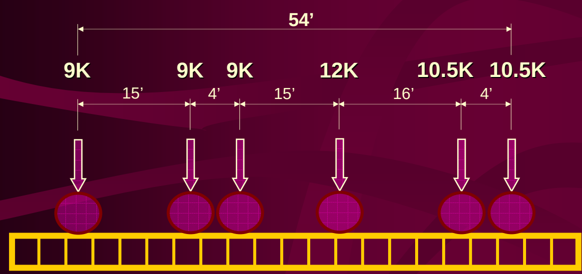


TYPE 3-3
W = 80 Kips



AASHTO LEGAL TRUCKS

LRFR LEGAL LANE LOAD MODEL FOR SPANS BETWEEN 200 FT. and 300 FT.



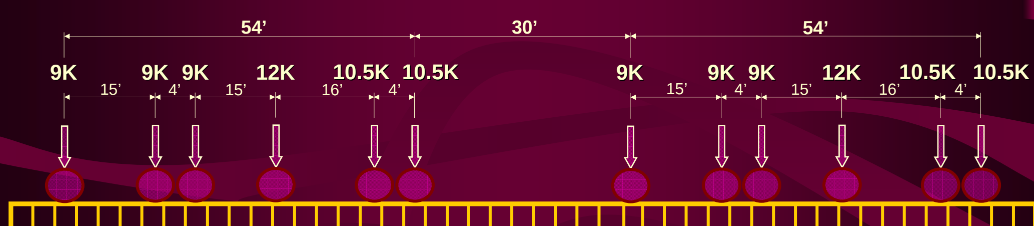
LEGEND

Truck = 75% of Type 3-3

= 60 Kips

Lane Load = 0.2 KLF

LRFR LEGAL LANE LOAD MODEL FOR NEGATIVE MOMENTS



LEGEND

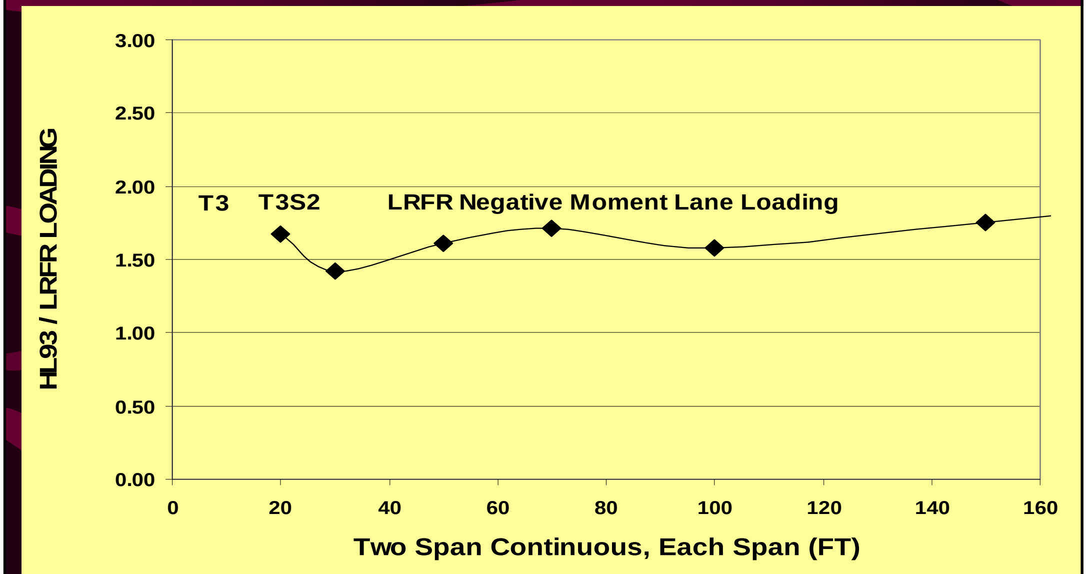
Each Truck = 75% of Type 3-3

= 60 Kips

Lane Load = 0.2 KLF

Headway Distance = 30 Ft

NEGATIVE MOMENT LOADING BIAS



CHECKING OVERWEIGHT PERMITS

STRENGTH II LIMIT STATE

LRFR FOR OVERWEIGHT PERMIT CHECKING

- LRFR provides permit load factors by permit type:
  - Routine/Annual Permits < 150 K
  - Special Permits > 150 K
- Load factors calibrated to provide uniform reliability: β = 2.5 for Routine Permits, β = 3.5 for Special permits.

ROUTINE PERMIT -- OREGON-TRAIN

8-AXLE CONTINUOUS TRIP PERMIT 105.5Kip

SPECIAL PERMIT

Ohio Superload: TOTAL WEIGHT 848.6 k

NCHRP Project 12-63

NEW AASHTO LEGAL LOADS

SPECIALIZED HAULING VEHICLES (SHV)

- AASHTO legal loads were adopted in the 1970s
- Trucking industry has in recent years introduced Specialized Hauling Vehicles with closely-spaced multiple axles:
  - Dump trucks, construction vehicles, solid waste trucks and other hauling trucks.
- Certain SHVs are under 80,000 # and satisfy Bridge Formula B. They are legal in all states.
- Some SHVs are exempted from Formula B under the grandfather clause. Legal in some states.

FEDERAL BRIDGE FORMULA B

$$W = 500\left[\frac{LN}{N-1} + 12N + 36\right]$$

W = Maximum weight in pounds that can be carried on a group of two or more axles

L = Distance in feet between the outer axles of any two or more consecutive axles.

N = Number of axles being considered.

SPECIALIZED HAULING VEHICLE

5-AXLE SHV

SPECIALIZED HAULING VEHICLE

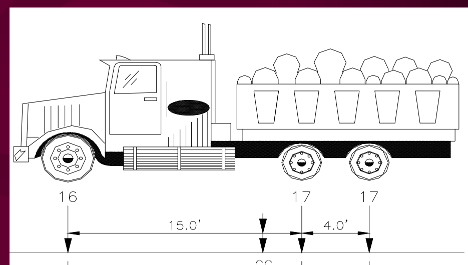
7-AXLE SHV

THE PROBLEM

- AASHTO Type 3 posting load is not representative of these newer legal loads.

TYPE 3

Weight = 50 Kips



- It is considered likely that these specialized vehicles may be severely overstressing some non-posted bridges.

NCHRP PROJECT 12-63 OBJECTIVE

- Project timeline : 2003 to 2006
- Investigate recent developments in specialized truck configurations and State legal loads
- Recommend revisions to the AASHTO legal loads as depicted in the current *AASHTO Manual for Condition Evaluation and the new AASHTO LRFR Guide Manual.*

2005 AASHTO BRIDGE MEETING

AASHTO SCOBS ADOPTED:

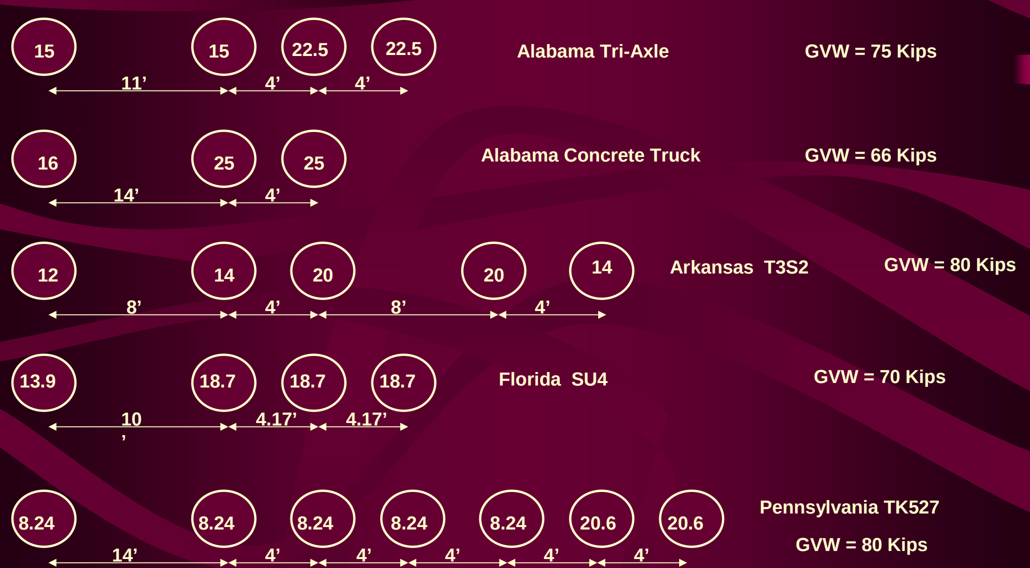
- A NEW NOTIONAL RATING TRUCK FOR LOAD RATING OF BRIDGES
- NEW POSTING LOAD MODELS FOR SINGLE UNIT TRUCKS
- APPLIES TO: ALLOWABLE STRESS, LOAD FACTOR, LOAD AND RESISTANCE FACTOR METHODS

POSTING LOADS FOR SPECIALIZED HAULING VEHICLES THAT MEET BRIDGE FORMULA B

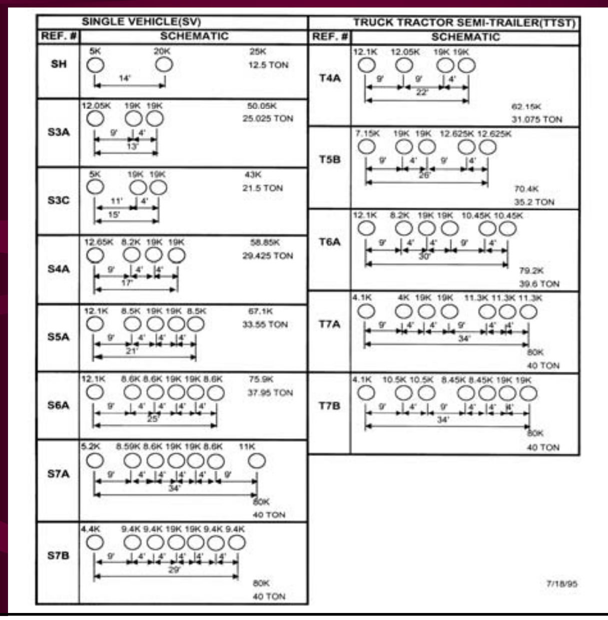
SHORT HAULING TRUCKS

In response to changing truck configurations, several states have adopted a variety of short multi-axle vehicles as rating and posting loads.

STATE LEGAL LOADS



NORTH CAROLINA LEGAL LOADS

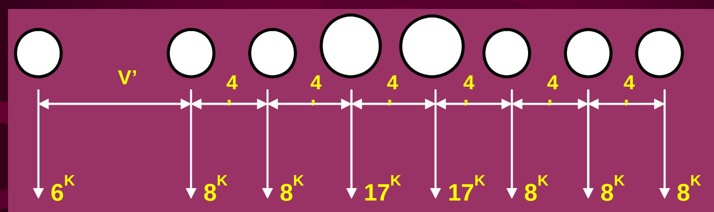


A SINGLE "NOTIONAL" RATING LOAD FOR ALL FORMULA B TRUCKS

- DEVELOP A SINGLE ENVELOPE RATING LOAD MODEL FOR ALL FORMULA B TRUCKS:
  - MAY NOT REPRESENT AN ACTUAL TRUCK (NOTIONAL TRUCK)
  - SIMPLIFIES THE RATING ANALYSIS

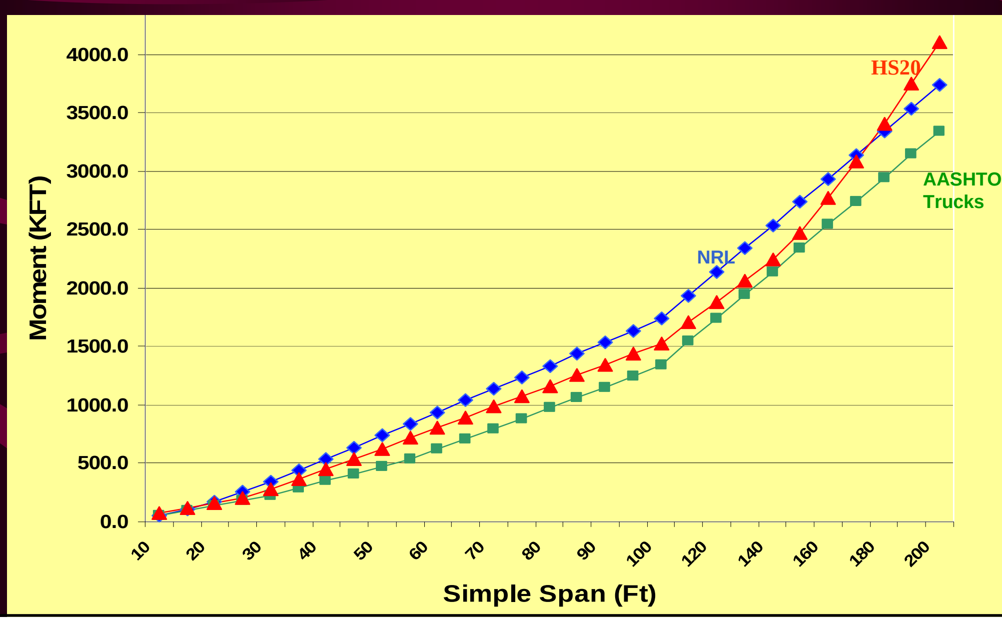
2005 AASHTO BRIDGE MEETING

AASHTO ADOPTS: “NOTIONAL RATING LOAD” NRL



- GVW = 80 KIPS
- V — 6’0” TO 14’-0”. SPACING
- AXLES THAT DO NOT CONTRIBUTE TO THE MAXIMUM LOAD EFFECT UNDER CONSIDERATION SHALL BE NEGLECTED.

SIMPLE SPAN MOMENTS

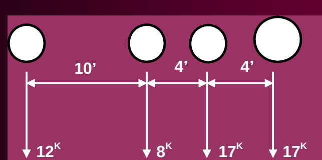


BRIDGE POSTING LOADS

- A wide variety of vehicle types cannot be effectively controlled by any single posting load
- A single posting load based on a short truck model would be too restrictive
- Setting weight limits for posting should consider legal truck types that operate within a state.

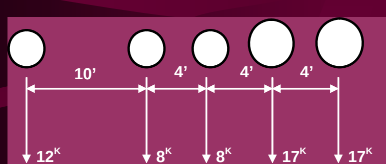
2005 AASHTO BRIDGE MEETING

ADOPTS NEW POSTING LOADS RECOMMENDED BY NCHRP 12-63



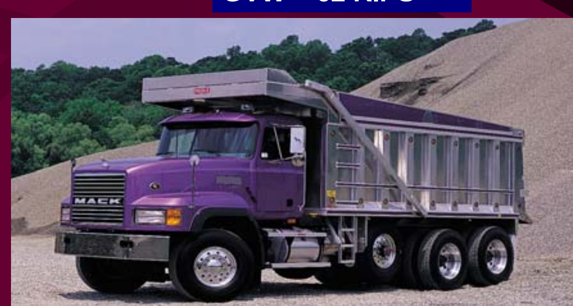
SU4 TRUCK

GVW = 54 KIPS



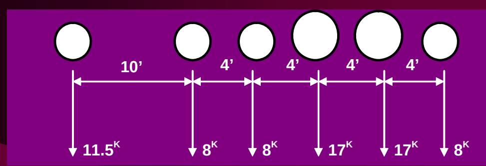
SU5 TRUCK

GVW = 62 KIPS



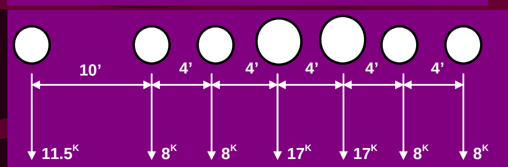
2005 AASHTO BRIDGE MEETING

ADOPTS NEW POSTING LOADS RECOMMENDED BY NCHRP 12-63



SU6 TRUCK

GVW = 69.5 KIPS



SU7 TRUCK

GVW = 77.5 KIPS

LOAD & RESISTANCE FACTOR RATING OF HIGHWAY BRIDGES

ADOT / FHWA

SESSION 3

LRFR LOAD RATING PROCESS & LOAD RATING EQUATION

BALA SIVAKUMAR, P.E.

What is Load Rating?

The safe live load carrying capacity of a highway structure is called its load rating.

It is usually expressed as a Rating Factor (RF) or in terms of tonnage for a particular vehicle

TYPES OF RATINGS

Inventory Rating

Operating Rating

- **Axial**
- **Moment**
- **Shear**
- **Serviceability**
- **Moment Shear Interaction**

IN LRFR INVENTORY & OPERATING RATINGS ARE DEFINED IN TERMS OF ASSOCIATED RELIABILITY INDICES (β=3.5 INV, β=2.5 OPR)

RATING FACTOR

$$RF = \frac{C - A_1 * D}{A_2 * L*(1 + I)}$$

A_1 = Factor for dead loads

A_2 = Factor for live load

C = Capacity of the bridge

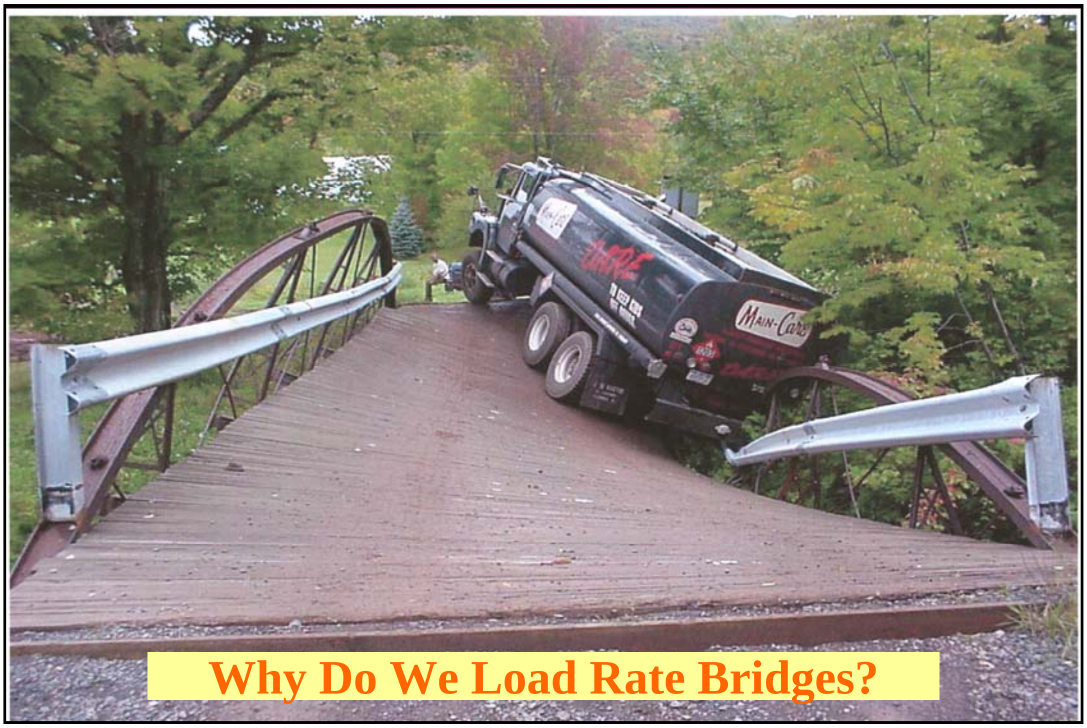
D = Dead load effect

I = Impact factor (Dynamic Load Allowance)

L = Live load effect

Why Do We Load Rate Bridges?

- Different design vehicles have been used in the past for the design of bridges (e.g., H-15, HS20-44, HS25, HL-93, ...).
- Some bridges have aged, deteriorated or become structurally deficient during the course of their life.
- To have a consistent summary of the load carrying capacities, all bridges are rated using a standard set of vehicles, called Legal Loads.
- For the safety of general public and traffic using highway structures, the loading rating is performed.
- Bridges that have insufficient load capacity are posted for restricted loads.



STRENGTH RATING METHODS

- LOAD RATING DONE AT STRENGTH LIMIT STATE, SERVICEABILITY IS CHECKED SEPARATELY
- LFR & LRFR ARE SIMILAR IN THIS RESPECT

LOAD FACTOR RATING METHOD

- LOAD RATING DONE AT STRENGTH LIMIT STATE, SERVICEABILITY IS CHECKED SEPARATELY
- LOAD FACTORS WERE NOT CALIBRATED USING STATISTICAL DATA, BUT WERE ESTABLISHED BASED ON ENGINEERING JUDGMENT (UNKNOWN RELIABILITY)
- NO GUIDANCE ON ADJUSTING LOAD FACTORS FOR CHANGED UNCERTAINTY IN LOADING CONDITIONS

THE LRFR PHILOSOPHY

- Reliability-based, limit states approach consistent with LRFD.
  - Rating done at Strength limit state and checked for serviceability.
- Provides more flexible rating procedures with uniform reliability.
  - Calibrated live load factors for different live load models and loading uncertainties.

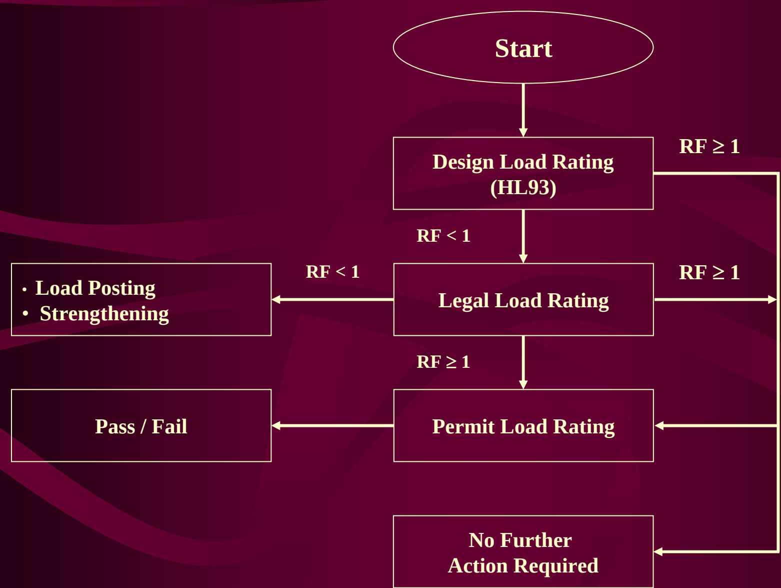
THE LRFR LOAD RATING PROCESS

1) DESIGN LOAD RATING (HL-93)

2) LEGAL LOAD RATING (AASHTO & STATE LEGAL LOADS)

3) PERMIT LOAD RATING (OVERWEIGHT TRUCKS)

Flow Chart for LRFR Load Rating



DESIGN LOAD RATING (HL 93)

- **HL93 is a notional representation of trucks permitted under "Grandfather" exclusion to weight laws.**
- **Bridges that rate for HL93 are safe for all legal loads (including grandfather trucks).**
- **RF < 1.0 Identifies vulnerable bridges for further evaluations (need for posting).**
- **Results suitable for NBI reporting of LRFR Ratings (Similar to HS20).**

DESIGN LOAD RATING (HL 93)

- **Do not convert HL-93 rating factors to tonnage. It's a notional load that includes a lane load.**
- **Report ratings as rating factors to the NBI.**
- **Provides a metric for assessing existing bridges to current (LRFD) design standards.**

FHWA REPORTING

- **Recent revisions to the Coding Guide to allow reporting of HL-93 LRFR Rating Factors (Items 63 and 65)**
- **Allows reporting of Rating Factors instead of Tons.**
- **FHWA encourages the reporting of LRFR ratings using HL-93 for new or reconstructed bridges designed using LRFD. (FHWA Memo. Dated March 22, 2004)**

RELIABILITY LEVELS FOR HL-93

1) For States that allow “Exclusion Loads”

β = 3.5 (Inventory Level)

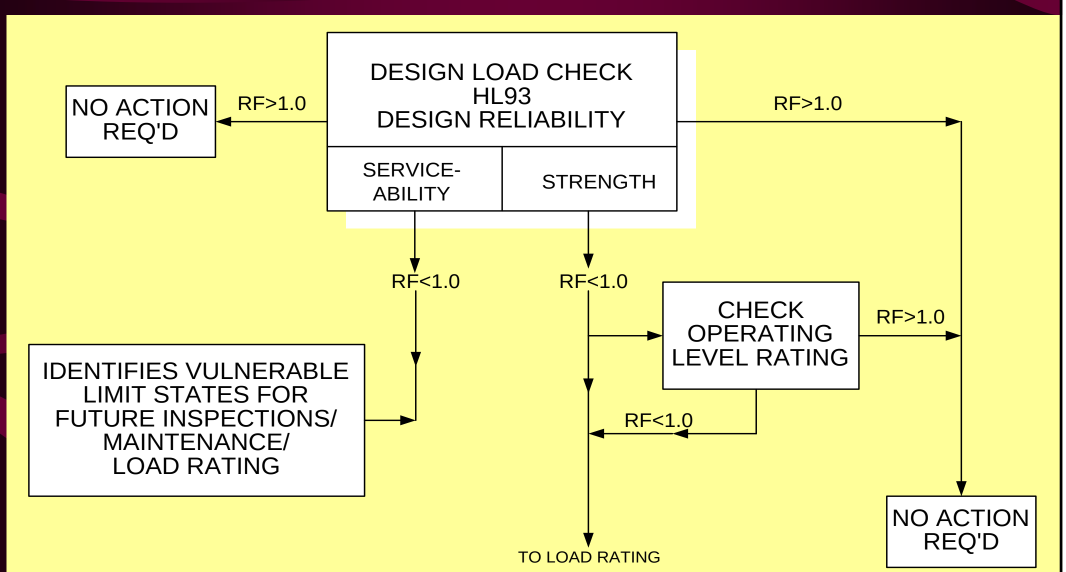
Live Load Factor = 1.75

2) For States that comply with federal weight laws (incl. Formula B):

β = 2.5 (Operating Level)

Live Load Factor = 1.35

LRFR FLOW CHART FOR HL-93



LEGAL LOAD RATING

- Bridges with RF < 1.0 for HL-93 should be load rated for AASHTO & State legal loads.
- Bridges with RF < 1.0 for legal loads should be posted.
- Single load rating at $\beta = 2.5$ for legal loads.
- LRFR Provides a <u>single safe load capacity for indefinite use.</u>

<u>Load Factor Operating Rating</u> --- Maximum permissible live load for the structure, suitable for one-time or Limited Crossings.

<u>LFR INVENTORY & OPERATING RATINGS</u>

Inventory Rating --- Safe Load Capacity for Indefinite Crossings of a Bridge. Generally corresponds to the customary design level of stresses. Allows comparisons with the capacity of new structures.

Operating Rating --- Maximum permissible live load for the structure, suitable for one-time or Limited Crossings. (OKAY FOR PERMITS, BUT NOT FOR POSTING)

LEGAL LOAD RATING

- Single load rating ($\beta = 2.5$) for a given legal load. Departure from current practice of INV & OPR ratings.
- Provides Load Ratings using AASHTO Legal Loads (Type 3, Type 3-3, Type 3S2) & Specialized hauling Vehicles
- Legal load ratings are used to establish need for posting (RF < 1.0) or bridge strengthening
- Do not use HL-93 results for posting purposes

STRENGTH LIMIT STATE EVALUATION

General Load Rating Equation

$$RF = \frac{\phi_C \phi_S \phi R - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_P P}{\gamma_L L(1 + IM)}$$

Where:

- RF = Rating Factor
- 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
- ϕ_C = Condition factor
- ϕ_S = System factor
- ϕ = LRFD resistance factor
- R = Nominal member resistance
- 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.
- L = Live Load effect
- IM = Dynamic Load Allowance

CAPACITY C

$C = \phi_C \phi_S \phi R$ For the STRENGTH Limit States

$C = f_R$ For the SERVICE Limit States

Where:

- ϕ_C = LRFR Condition factor (Optional)
- ϕ_S = LRFR System factor (Optional)
- ϕ = LRFD resistance factor
- f_R = Allowable Stress Specified in the LRFD Code

LRFR LOAD FACTORS FOR HL-93

β = 3.5 (Inventory Level)

Live Load Factor = 1.75

β = 2.5 (Operating Level)

Live Load Factor = 1.35

LOAD FACTORS FOR LEGAL LOADS

β = 2.5

TRAFFIC VOLUME	LOAD FACTOR
ADTT > 5000	1.80
ADTT = 1000	1.65
ADTT < 100	1.40

- For ADTT between 100 and 5000 interpolate the load factor.

STRENGTH LIMIT STATES

STRENGTH I ---- Design Load Rating
Legal Load Rating

STRENGTH II ---- Permit Load Rating

LRFR RATING EQUATION

$$RF = \frac{\phi_C \phi_S \phi R - \gamma_{DC} DC - \gamma_{DW} DW}{\gamma_L (LL + IM)}$$

$$\phi_C \phi_S \geq 0.85$$

ϕ_S OPTIONAL SYSTEM FACTOR FOR REDUNDANCY

ϕ_C OPTIONAL MEMBER CONDITION FACTOR

ϕ LRFD RESISTANCE FACTOR

LRFD ϕ Factors for New Designs

R/C Concrete Flexure	$\phi = 0.90$
P/S Concrete Flexure	$\phi = 1.00$
Concrete Shear	$\phi = 0.90$
Steel Flexure & Shear	$\phi = 1.00$

Applies to new members in good condition.

What are the resistance factors for evaluation of existing members in deteriorated condition?

ϕ Is not a strength reduction factor

BRIDGE SAFETY AND REDUNDANCY

- LRFD IS CALIBRATED TO PROVIDE UNIFORM MEMBER SAFETY FOR REDUNDANT PARALLEL GIRDER SUPERSTRUCTURE SYSTEMS (CONSIDERED REPRESENTATIVE OF CURRENT AND FUTURE TRENDS IN BRIDGE CONSTRUCTION)
- MANY EXISTING BRIDGES HAVE NON-REDUNDANT SUPERSTRUCTURE SYSTEMS
- REDUNDANT SYSTEMS:
  - o SYSTEM SAFETY > MEMBER SAFETY
- NON-REDUNDANT SYSTEMS:
  - o SYSTEM SAFETY = MEMBER SAFETY

SYSTEM FACTOR ϕ_s

- SYSTEM FACTOR ϕ_s IS RELATED TO THE DEGREE OF REDUNDANCY IN THE TOTAL STRUCTURAL SYSTEM. IN LRFR, BRIDGE REDUNDANCY IS DEFINED AS THE CAPABILITY OF A STRUCTURAL SYSTEM TO CARRY LOADS AFTER DAMAGE OR FAILURE OF ONE OR MORE OF ITS MEMBERS.
- MEMBERS DO NOT BEHAVE INDEPENDANTLY, BUT INTERACT WITH ONE ANOTHER TO FORM ONE STRUCTURAL SYSTEM.
- NCHRP REPORT 406 *Redundancy in Highway Superstructures* ESTABLISHED SYSTEM FACTORS FOR BRIDGE MEMBERS

SYSTEM FACTOR ϕ_s

$$C = \phi\phi_c\,\phi_s\,R$$

- System Factors are Multipliers to the Nominal Resistance to Reflect the Level of Redundancy of the Complete Superstructure System.
- Non-Redundant Bridges will Have Their Factored Member Capacities Reduced, and, Accordingly, will Have Lower Ratings.
- System Factors are Used to Maintain an Adequate Level of System Safety.
- The Aim of ϕ_s is to Add Reserve Capacity (to non-redundant member) such That System Reliability is Increased from an Operating Level Reliability to an Inventory Level Reliability

SYSTEM FACTOR ϕ_s

$$C = \phi\phi_c\,\phi_s\,R$$

- Redundant Bridges ϕ_s = 1.00
- Non-redundant Bridges ϕ_s = 0.85

NON-REDUNDANT MEMBERS WITH INTERNAL REDUNDANCY

- Riveted Two-Girder/Truss Bridges f_s= 0.90
- Multiple Eyebar Members in Trusses f_s = 0.90
- Floorbeams with Spacing > 12 ft f_s = 0.85

SYSTEM FACTORS ARE NOT APPROPRIATE FOR SHEAR AS SHEAR FAILURES TEND TO BE BRITTLE. WITHOUT DUCTILITY SYSTEM RESERVE IS NOT POSSIBLE.

CONDITION FACTOR ϕ_c

$$C = \phi\phi_c\,\phi_s\,R$$

Resistance of Deteriorated Bridges:

- LRFD Resistance Factors for New Members Must be Reduced When Applied to Deteriorated Members
- There is Increased Uncertainty and Variability in Resistance of Deteriorated Members
- They are Prone to Accelerated Future Deterioration. (increased additional losses between inspection cycles)
- Improved Inspections will Reduce, but not Totally Eliminate, the Increased Resistance Variability in Deteriorated Bridges.

CONDITION FACTOR ϕ_c

- Condition Factor ϕ_c is Tied to The Condition of The Member Being Evaluated (element level data preferred):
  - Good or Satisfactory ϕ_c = 1.00
  - Fair ϕ_c = 0.95
  - Poor ϕ_c = 0.85
- If Element Level Condition Data is not Collected, NBI Ratings for the Superstructure May be Used to Set f_c

  ϕ_c = 0.85 for NBI Rating of 4

  ϕ_c = 0.95 for NBI Rating of 5

  ϕ_c = 1.00 for NBI Rating 6 or higher

Increases beta from 2.5 to a target of 3.5 to account for the increased variability of resistance of deteriorated members.

LRFD DYNAMIC LOAD ALLOWANCE (IM)

Component	IM
All Components • Fatigue & Fracture Limit State • All Other Limit States	 15% 33%

- Standard IM specified for use with all load models
- IM is Applied to HL-93 Design Truck Only … *Not* to Design Lane Load
- This simple approach is based upon a study which revealed that the most influential factor is roadway surface roughness (not span length).

LRFR DYNAMIC LOAD ALLOWANCE

IM = 33% Is Standard. Following Values are Optional:

- ***Legal Load rating***

Riding surface conditions	IM
smooth	*10%*
minor surface irregularities	*20%*
major surface irregularities	*33%*

- ***Permits** – same, except for:*

slow moving (< 5mph) vehicles	*0%*

LRFR DEAD LOAD FACTORS

DC - Dead Load, except wearing surfaces and utilities

DW - wearing surfaces and utilities, acting on the long-term composite section.

$\gamma_{DC} = 1.25$

$\gamma_{DW} = 1.50$

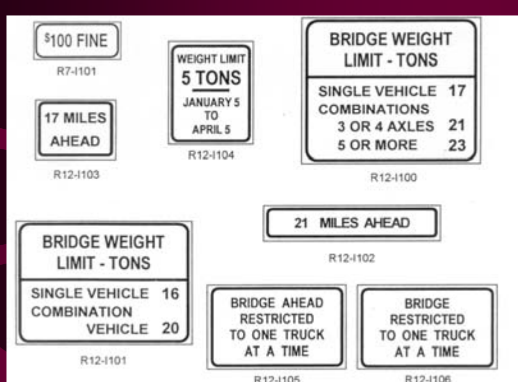
$\gamma_{DW} = 1.25$ When overlay thickness is filed measured.

Data Collection for Load Rating

Loading and Traffic Data

- Actual wearing surface thickness, if present
- Non-structural attachments and utilities
- Depth of fill (buried structures)
- Number and positioning of traffic lanes
- ADTT or traffic volume and % trucks
- Posted load limit, if any
- Roadway surface conditions.

LRFR LOAD POSTING OF BRIDGES



Illinois

Alabama

SETTING A POSTING WEIGHT LIMIT
Current Practice (LFR)

Bridge Safe Load Capacity or posting load is commonly based on the following factors:

- Condition of the bridge
- Bridge redundancy
- Site traffic conditions
- Sometimes the inspection frequency

Very little consistency among various jurisdictions in this regard. These factors are not currently considered in the LFR load rating process.

POSTING BRIDGES FOR UNIFORM RELIABILITY

- Post Bridges when RF < 1.0 for legal loads.
- The LRFR philosophy of uniform reliability applies to load ratings & posting.
- Bridges should also be posted to maintain uniform reliability.
- The rating analysis is aimed at determining a rating factor or tonnage for a specific truck.
- The posting analysis provides the safe posting level for a bridge when RF < 1.0.

POSTING BRIDGES FOR UNIFORM RELIABILITY

- Relationship between posting load and rating factor is not linear in a reliability based evaluation.
- The lower the posting load, the greater the possibility of illegal overloads and multiple presence.
  - 5 Ton Capacity vs 20 Ton capacity; which bridge has a greater probability of illegal overloads?
- The overload probabilities are higher for low rated bridges, which should be considered in the posting analysis.
- A more conservative posting is required for low rated bridges to maintain the same level of reliability used in the rating analysis.
- Posting analysis translates rating factors into posting loads. Provides a more rational assessment of bridge safe load capacity.

LRFR POSTING ANALYSIS

1) When $0.3 < RF < 1.0$

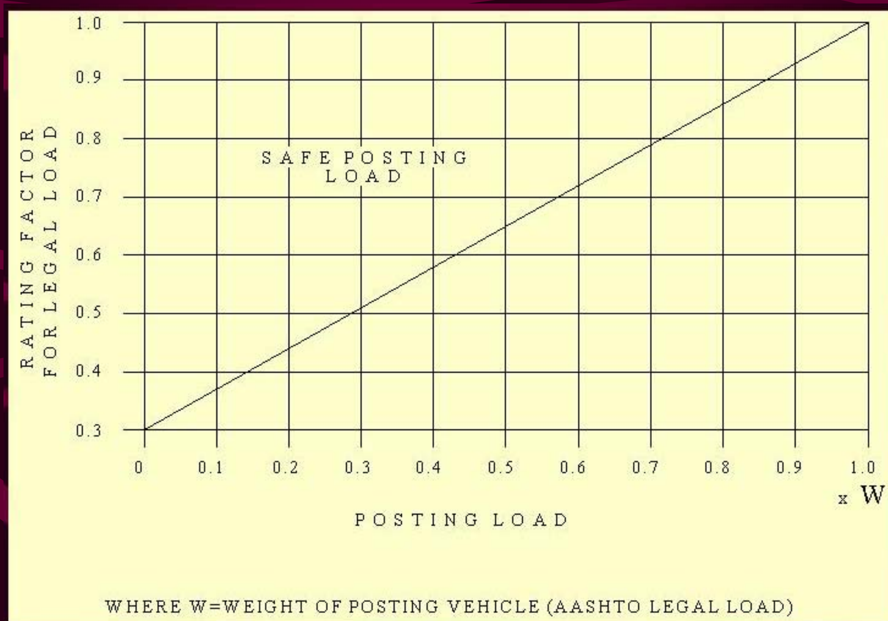
Posting Load = $(W/0.7)\,[\,(RF) - 0.3\,]$

W = Weight of rating vehicle
RF = Legal load rating factor

2) When $RF < 0.3$ for all three AASHTO loads, bridge should be closed

If a bridge cannot support the empty weight of a legal truck, the bridge should be closed.

POSTING LOAD vs RATING FACTOR



LRFR POSTING CURVES

Criteria: A bridge with acceptable capacity less than 3 tons should be closed.

Increase beta from operating target of 2.5 to 3.5 to account for historical failure occurrences in posted bridges

Allow for load shifting among axles

Allow an overweight "cushion" of 10,000 lbs

MANUAL PROVIDES AN INERPOLATION OF POSTING LOADS VS. R.F. FOR VALUES BETWEEN 0.3 AND 1.0.

LOAD & RESISTANCE FACTOR RATING OF HIGHWAY BRIDGES

ADOT / FHWA

SESSION 4

LRFR LIMIT STATES, RELIABILITY INDICES & LOAD FACTORS

BALA SIVAKUMAR, P.E.

LRFR LIMIT STATES AND LOAD FACTORS

Bridge Type	Limit State	Dead Load DC	Dead Load DW	Design Load A6.4.3.2.1 Inventory LL	Design Load A6.4.3.2.1 Operating LL	Legal Load A6.4.4.2.1 LL	Permit Load A6.4.5.4.1 LL
Steel	STRENGTH I	1.25	1.50	1.75	1.35	Table 6.4.4.2.3-1	-
	STRENGTH II	1.25	1.50	-	-	-	Table 6.4.5.4.2-1
	SERVICE II	1.00	1.00	1.30	1.00	1.30	1.00
	FATIGUE	0.00	0.00	0.75	-	-	-
Reinforced Concrete	STRENGTH I	1.25	1.50	1.75	1.35	Table 6.4.4.2.3-1	-
	STRENGTH II	1.25	1.50	-	-	-	Table 6.4.5.4.2-1
	SERVICE I	1.00	1.00	-	-	-	1.00
Prestressed Concrete	STRENGTH I	1.25	1.50	1.75	1.35	Table 6.4.4.2.3-1	-
	STRENGTH II	1.25	1.50	-	-	-	Table 6.4.5.4.2-1
	SERVICE III	1.00	1.00	0.80	-	1.00	-
	SERVICE I	1.00	1.00	-	-	-	1.00

STRENGTH LIMIT STATES

STRENGTH I ---- Design Load Rating
Legal Load Rating

STRENGTH II ---- Permit Load Rating

RELIABILITY LEVELS FOR LOAD RATING

1) **DESIGN LOAD RATING (HL-93)**

β = 3.5 (Inventory Level)

β = 2.5 (Operating Level)

2) **LEGAL LOAD RATING AND ROUTINE PERMIT RATING**

Only a Single Safe Load Capacity

(minimum β = 2.5)

- **Past practice of Computing Inventory & Operating ratings will not be continued in LRFR for posting loads.**
- **Promotes more consistent ratings and load postings.**

RELIABILITY INDICES FOR HL-93

1) For States that allow "Exclusion Loads"

β = 3.5 (Inventory Level)

Live Load Factor = 1.75

2) For States that comply with federal weight laws & Formula B:

β = 2.5 (Operating Level)

LRFR Live Load Factor = 1.35

RELIABILITY INDEX FOR LEGAL LOADS

LRFR Evaluation: β = 2.5 for redundant systems

- **β = 2.5 comparable to average reliability inherent in Load Factor Operating ratings.**
- **β = 2.5 has been shown to be an acceptable minimum level of safety for bridge evaluation.**
- **Exposure period for evaluation is 2 to 5 years versus 75 years for design.**

CALIBRATION OF LOAD FACTORS FOR LRFR LOAD MODELS

- HL-93 Design Load
- AASHTO Legal Loads
- Permit Loads
- Specialized Hauling Vehicles

Calibration of LRFD Live Load Factors

LRFD Bridge Design Specifications:

❖ **Design Live Load used for Calibration is HL-93**

❖ **75 year design life**

❖ **Target safety index (beta) = 3.5**

❖ **Recommended Live Load Factor = 1.75**

❖ **1970's Ontario truck weight data used; 5000 ADTT**

LRFD MULTIPLE PRESENCE OF LIVE LOADS

Number of Loaded Lanes	Multiple Presence Factor "m"
1	1.20
2	1.00
3	0.85
>3	0.65

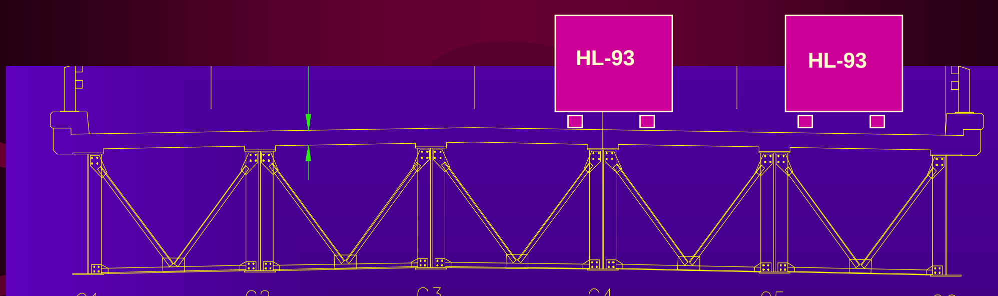
- The Multiple Presence Factors were Developed on the Basis of an ADTT of 5000 Trucks. For Sites with Lower ADTT Reduce Force Effects as Follows:

  100 < ADTT < 1000, Use 95%

  ADTT < 100, Use 90%

CALIBRATION OF STRENGTH I LIMIT STATE

LRFD Specifications: HL-93 Two Lanes Loaded



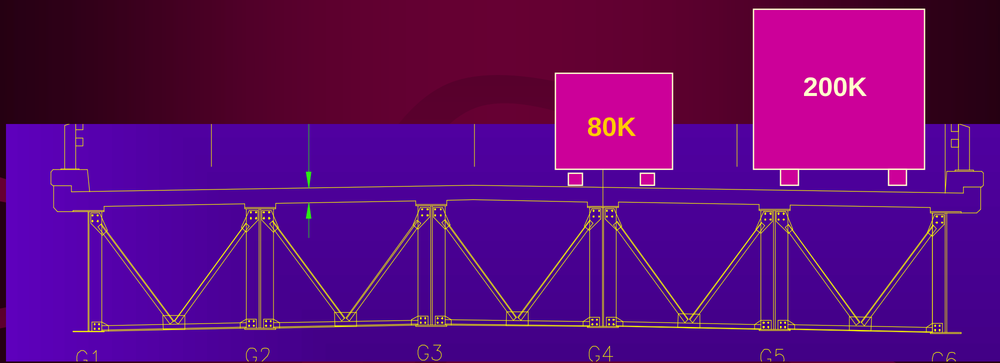
Governing Load Case for LRFD: ADTT = 5000

GENERALIZED LRFR LIVE LOAD FACTORS FOR AASHTO LEGAL LOADS

- For system-wide use nationally, tied to ADTT at the site.
- Target BETA used for calibration = 2.5
TRAFFIC VOLUME	LOAD FACTOR
ADTT > 5000	1.80
ADTT = 1000	1.65
ADTT < 100	1.40
- For ADTT between 100 and 5000 interpolate the load factor.
- MP probability for ADTT = 1000 reduced to 1%
- MP probability for ADTT = 100 reduced to 0.1%

LRFR LIVE LOAD FACTORS FOR OVERLOAD PERMITS

MULTIPLE PRESENCE DURING PERMIT CROSSINGS



AASHTO TWO-LANE DISTRIBUTION FACTORS ASSUME TWO EQUAL TRUCKS SIDE-BY-SIDE

Overweight Permit Distribution Analysis

- LRFD Two-Lane Distribution Factors assume Side-by-Side Presence of Two Equally Heavy Loads.
- Applying LRFD Distribution Factor Analysis to Permit Load Evaluation can be Overly Conservative as the loads are unequal.
- LRFR Manual Provides Specially Calibrated Load Factors for Overweight Permits that Account for the Side-by-Side Presence of Two Unequal Trucks.

LRFR LOAD FACTORS FOR OVERLOAD PERMITS

- Permit Load Factors by Permit Type
  - Routine/Annual Permits < 150 K
  - Special Permits > 150 K
- Load factors calibrated to provide uniform reliability
  Routine/Annual Permits : $\beta = 2.5$
  Special Permits : $\beta = 3.5$
- LRFR Permit Load Factors are Tied to the Permit Type, No. of Crossings, and Site ADTT.
- One-Lane Distribution Factor is Used With Special Permits.

LRFR PERMIT LOAD FACTORS

Routine Permits – Up to 150 K

$\beta = 2.5$

DF	ADTT (one direction)	Load Factor: Weight: Up to 100 KIPS	Load Factor: Weight: $\geq$ 150 KIPS
Two or more lanes	> 5000	1.80	1.30
	= 1000	1.65	1.20
	< 100	1.40	1.10

LRFR PERMIT LOAD FACTORS

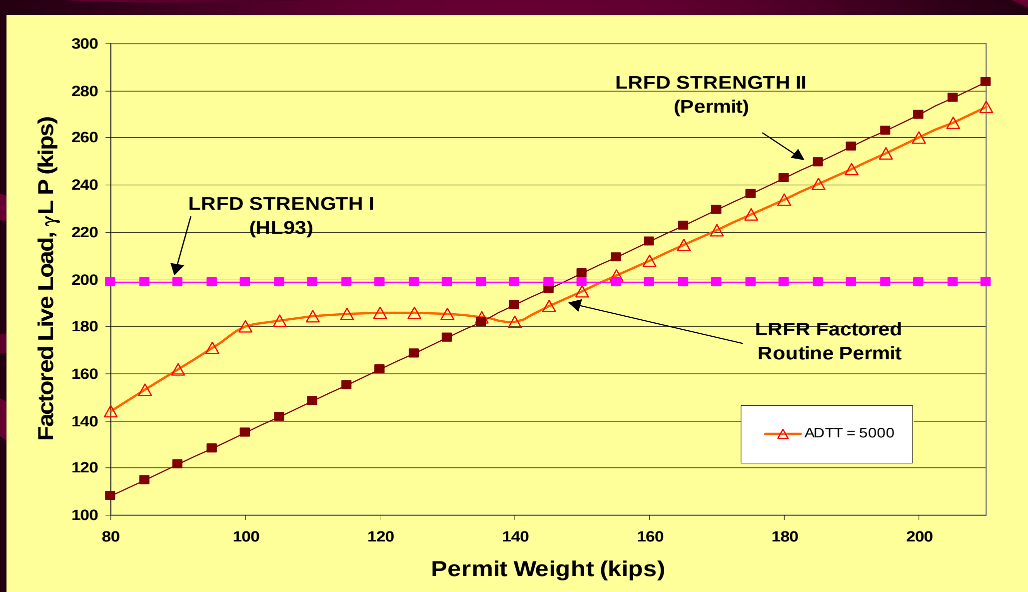
Special Permits

$\beta = 3.5$

TRIPS	OTHER TRAFFIC	DF	ADTT	Load Factor
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

* DIVIDE OUT THE 1.2 MPF FROM THE ONE LANE DF WHEN CHECING PERMIT LOADS.

FACTORED LOAD EFFECTS FOR ROUTINE PERMITS (Span = 65 Ft)



LRFR LIVE LOAD FACTORS FOR SPECIALIZED HAULING VEHICLES

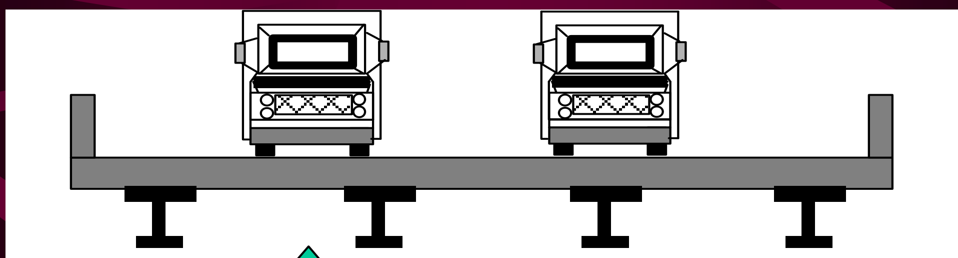
7-AXLE SHV

CALIBRATION OF LRFR LIVE LOAD FACTORS FOR SHVs

- SHVs Usually Constitute a Very Small Percentage of Total Truck Traffic
- Changed Multiple Presence Probabilities for SHVs (Similar to Permit Crossings).
- <u>Reduced Live Load Factors</u> are Likely for SHVs Compared to LRFR Factors for AASHTO Legal Loads.

TRUCK MULTIPLE PRESENCE FOR SHV CROSSINGS

Lane 1	Lane 2
Type 3	SHV



LRFR LOAD FACTORS FOR SHVs (2006 Interim)

Table 6-5-2 Live Load Factors for Specialized Hauling Vehicles

Traffic Volume (One Direction)	Load Factor for NRL, SU4, SU5, SU6, SU7
Unknown	1.60
ADTT ≥ 5000	1.60
ADTT = 1000	1.40
ADTT ≤ 100	1.15

CHANGES TO ACHIEVE UNIFORM RELIABILITY IN LRFD & LRFR

FIVE COMPONENTS OF LRFD LOAD EFFECTS:

- NEW LIVE LOAD MODEL (HL93)
- NEW LOAD FACTORS
- NEW DYNAMIC LOAD ALLOWANCE
- NEW DISTRIBUTION FACTORS
- NEW MULTIPLE PRESENCE FACTORS

DISTRIBUTION FACTOR FOR MOMENT g_m

K_g = Longitudinal Stiffness Parameter
= $n(I + Ae_g^2)$

I = M. I. Of Steel Section
A = Area of Steel Section
e_g = Distance Between C.G. of Steel Section and C.G. of Deck.
Modular Ratio = n

One Lane Loaded:

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

Two or More Lanes Loaded:

$$g_{m1} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

- Check Range of Applicability

$3.5 \le$ Spacing ≤ 16 ft.

$20 \le$ Span ≤ 240 feet

DISTRIBUTION FACTOR FOR SHEAR

One Lane Loaded

$$g_{V_1} = 0.36 + \frac{S}{25}$$

Two or More Lanes Loaded

$$g_{V_2} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

- **Check Range of Applicability**

3.5 ≤ Spacing ≤ 16 ft.

20 ≤ Span ≤ 240 feet

SKEW CORRECTIONS FOR SHEAR

The skew correction factor for shear *increases the live load distribution factor for the end shear* in the exterior girder at the obtuse corner of the bridge. The correction factor is valid for skew angles less than equal to 60 degrees.

$$\text{Correction Factor} = 1 + 0.2\left(\frac{12Lt_s^{\,3}}{K_g}\right)^{0.3} \tan\ \theta$$

In skewed bridges load are transferred by the shortest path. Skew correction is only applied for end shear.

COMPARISON OF DISTRIBUTION FACTORS

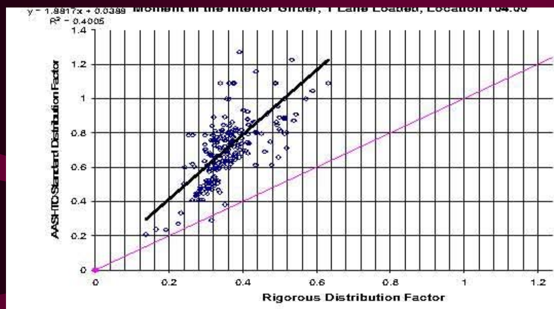


Figure 19 – Comparison of S/D to Grid Analysis

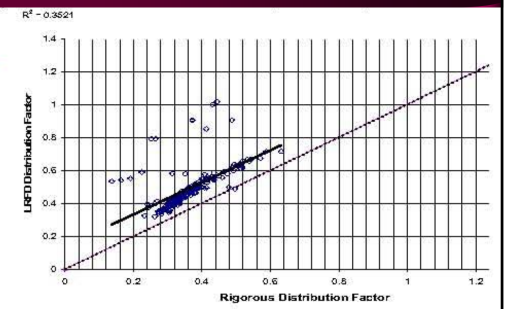


Figure 20 – Comparison of LRFD Distribution Factor to Grid Analysis

LRFD DISTRIBUTION FACTORS (DF)

- DF for flexure is often less than S/D in STD Specs.
- Shear DF are usually higher than STD Specs.
- Exterior girder DF are usually higher than STD Specs.
- LRFD flexure DF are lower for longer spans with wider girder spacings
- LRFD flexure DF can be higher for shorter spans with closely spaced girders.

INCREASED SHEAR IN CONCRETE BRIDGES (Example)

- Increased Live Load in LRFD : HL93 vs HS20
- Increased LRFD DF for shear compared to bending
  - DF_V = 1.145
  - DF_M = 0.840
- Increased LRFD skew correction for shear compared to bending
  - $SCorr_V$ = 1.121 (end shear)
  - $SCorr_M$ = 0.953
- Corrected LRFD Distribution Factors
  - DF_V = 1.284
  - DF_M = 0.801
- Shear effect in LRFD & LRFR may be over 30% higher than in LFD & LFR

LRFR SHEAR RATING FOR CONCRETE BRIDGES

LRFD SPECIFICATIONS

Major Changes to Concrete Provisions

- Unified Concrete Provisions for Reinforced and Prestressed Concrete (Section 5).
- Single Equation for Bending and Shear Resistance for all Concrete Sections (Reinforced or Prestressed)
- Introduces Two New Shear Design Methods:
  - Modified Compression Field Theory (MCFT)
  - Strut-and-Tie Model

LRFD SECTIONAL DESIGN METHOD

NOMINAL SHEAR RESISTANCE V_n (A5.8.3.3)

V_n shall be the lesser of:

$$V_n = V_c + V_s + V_p$$

$$V_n = 0.25 f'_c\, b_v\, d_v + V_p$$

$$V_c = 0.0316\, \beta \sqrt{f'_c}\, b_v\, d_v$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{S}$$

β = factor indicating ability of diagonally cracked concrete to transmit tension.
θ = angle of inclination of diagonal compressive stresses.
s = spacing of stirrups
d_v = effective shear depth
b_v = effective web width

SHEAR DESIGN
AASHTO LRFD SPECIFICATIONS

- V_C IS COMPUTED IN AN ENTIRELY DIFFERENT MANNER. USES THE MODIFIED COMPRESSION FIELD THEORY (MCFT).
- COMPUTING V_C and V_s REQUIRES β AND θ AT A GIVEN SECTION, OBTAINED THROUGH AN ITERATIVE PROCESS.
- β IS A MEASURE OF THE CONCRETE CONTRIBUTION, θ IS A MEASURE OF THE TRANSVERSE STEEL CONTRIBUTION.
- θ AND β ARE FUNCTIONS OF THE SHEAR STRESS AT A SECTION AND THE LONGITUDINAL STRAIN AT MID-DEPTH.
- MCFT USES THE LONGITUDINAL STRAIN AT MID-DEPTH ε_X AS THE DAMAGE INDICATOR TO EVALUATE SHEAR-SLIP RESISTANCE ALONG DIAGONAL CRACKS.

STRAIN AT MIDDEPTH ε_x **(A5.8.3.4.2)**

If the section contains at least the minimum transverse reinforcement as per A5.8.2.5:

ε_x =

$$\frac{\frac{M_u}{d_v} + 0.5N_u + (0.5)(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2\left(E_s A_s + E_p A_{ps}\right)} \le 0.002$$

A_c = area of concrete on the flexural tension side of the member.
N_u = factored axial force
f_{po} = for pretensioned members f_{po} can be taken as the jacking stress (C5.8.3.4.2). (A value of $0.7f_{pu}$ is appropriate.)

Table 5.8.3.4.2-1: Values of θ and β for Sections with Transverse Reinforcement.

$\frac{v_u}{f'_c}$	$\varepsilon_x \times 1{,}000$								
	≤−0.20	≤−0.10	≤−0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00
≤0.075	22.3 6.32	20.4 4.75	21.0 4.10	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤0.100	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.50	34.0 2.32	36.7 2.18
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37.0 2.13
≤0.150	21.6 2.88	23.3 2.79	24.2 2.78	25.0 2.72	26.9 2.60	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08
≤0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.60	28.0 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96
≤0.200	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29.0 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79
≤0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.40	30.0 2.34	30.8 2.14	32.3 1.86	34.0 1.73	35.7 1.64
≤0.250	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.70	34.3 1.58	35.8 1.50

SHEAR DESIGN

AASHTO LRFD SPECIFICATIONS

- HIGHER THE STRAIN ε_x , WIDER THE SHEAR CRACKS, SMALLER β AND IN TURN SMALLER THE CONCRETE CONTRIBUTION.
- SMALLER θ WILL RESULT IN MORE STIRRUPS CROSSING EACH DIAGONAL CRACK (HIGHER V_s).
- MCFT IS A "SECTIONAL DESIGN MODEL" APPLICABLE TO FLEXURAL DESIGN REGIONS (B-REGIONS) OF MEMBERS. MOST DESIGNERS USE IT FOR SHEAR DESIGN OF ALL REGIONS, INCLUDING DISTURBED REGIONS (D-REGIONS).
- SHEAR CAUSES TENSION IN THE LONGITUDINAL STEEL. AT EACH SECTION THE CAPACITY OF THE LONGITUDINAL STEEL MUST BE CHECKED

SHEAR DESIGN

- THE LOGITUDINAL STRAIN (AND SHEAR RESISTANCE) IS A FUNCTION OF FACTORED SHEAR FORCE AND BENDING MOMENT AT A SECTION. HIGH MOMENTS WILL REDUCE SHEAR RESISTANCE. THEREFORE, PERFORM SHEAR CHECKS AT TENTH POINTS OF THE SPAN.

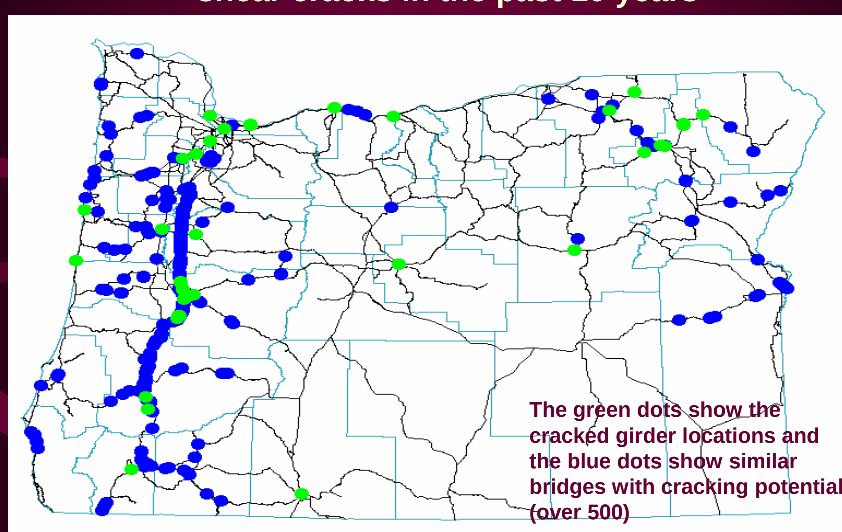
- AS A SIMPLIFICATION, FOR NONPRESTRESSED MEMBERS THE FOLLOWING VALUES MAY BE USED (A5.8.3.4.1):

β= 2.0 θ = 45º

- SECTIONS SHALL MEET MINIMUM REQUIREMENTS FOR TRANSVERSE STEEL

I-5 and I-84 Bridges in Oregon

Numerous reinforced concrete girder bridges have developed shear cracks in the past 10 years



Example of Cracked Bridge (ODOT)

BR 08429E Grande Ronde River, Quarry Bridge

LRFR SERVICE LIMIT STATES

LRFR LIMIT STATES AND LOAD FACTORS

Bridge Type	Limit State	Dead Load DC	Dead Load DW	Design Load A6.4.3.2.1 Inventory LL	Design Load A6.4.3.2.1 Operating LL	Legal Load A6.4.4.2.1 LL	Permit Load A6.4.5.4.1 LL
Steel	STRENGTH I	1.25	1.50	1.75	1.35	Table 6.4.4.2.3-1	-
	STRENGTH II	1.25	1.50	-	-	-	Table 6.4.5.4.2-1
	SERVICE II	1.00	1.00	1.30	1.00	1.30	1.00
	FATIGUE	0.00	0.00	0.75	-	-	-
Reinforced Concrete	STRENGTH I	1.25	1.50	1.75	1.35	Table 6.4.4.2.3-1	-
	STRENGTH II	1.25	1.50	-	-	-	Table 6.4.5.4.2-1
	SERVICE I	1.00	1.00	-	-	-	1.00
Prestressed Concrete	STRENGTH I	1.25	1.50	1.75	1.35	Table 6.4.4.2.3-1	-
	STRENGTH II	1.25	1.50	-	-	-	Table 6.4.5.4.2-1
	SERVICE III	1.00	1.00	0.80	-	1.00	-
	SERVICE I	1.00	1.00	-	-	-	1.00
Wood	STRENGTH I	1.25	1.50	1.75	1.35	Table 6.4.4.2.3-1	-
	STRENGTH II	1.25	1.50	-	-	-	Table 6.4.5.4.2-1

Notes

Shaded cells of the table indicate optional checks.

SERVICE I is used to check the 0.9Fy stress limit in reinforcing steel.

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

FATIGUE limit state is checked using the LRFD fatigue truck (see Article 6.6.4.1)

LRFR SERVICEABILITY RATING PHILOSHOPY

DIFFERENCES BETWEEN LRFD DESIGN AND LRFR EVALUATION

- Past performance of an existing bridge is a good indicator of adequate serviceability.
- An existing bridge is very likely to have experienced a more extreme load than is likely to occur in the evaluation exposure period (2 to 5 years).
- Posting a bridge to satisfy serviceability criteria is generally not economically justifiable.
- Evaluation requires a more selective application of serviceability checks than design.

Lichtenstein

SERVICEABILITY CONSIDERATIONS IN LRFR

•PERMANENT DERFORMATION OF STEEL MEMBERS (SERVICE II) : Legal loads, Permit loads.

•FATIGUE OF STEEL MEMBERS (FATIGUE)

•PERMANENT DEFORMATION OF REINFORCING STEEL IN R/C & P/S CONCRETE MEMBERS (SERVICE I): Permit loads

•CRACKING OF P/S CONCRETE MEMBERS (SERVICE III)

SERVICEABILITY LIMIT STATES

Steel Bridges:	**SERVICE II**
	FATIGUE
Concrete Bridges:	**SERVICE I**
	SERVICE III (p/s only)

LRFR SERVICEABILITY RATING EQUATION

$$RF = \frac{f_R - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_P P}{\gamma_L(\mathrm{LL+IM})}$$

f_R = Allowable Stress Specified in the LRFD Code

= 0.95 F_{yf} (composite sections)

= 0.80 F_{yf} (non-composite sections)

LRFR SERVICEABILITY CHECKS FOR CONCRETE BRIDGES

Type	Load	Limit State	Load Factor
Reinf. Conc.	Permit	SERVICE I	1.0 (optional)
P/S Conc.	HL93	SERVICE III	0.8
P/S Conc.	Legal	SERVICE III	1.0 (optional)
P/S Conc.	Permit	SERVICE I	1.0 (optional)

LRFR SERVICEABILITY CHECKS FOR P/S CONCRETE BRIDGES

- SERVICE III is a design level check for crack control in P/S components using an uncracked section analysis. (P/S bridges will crack under first overload)

- SERVICE III check is made optional for legal loads; not required for permit loads.

- *For all concrete bridges, permit loads should satisfy SERVICE I check to ensure that cracks will close once the live load is removed (no yielding in reinforcing steel). check for: $f_{DL} + f_{LL+IM} \leq 0.9\ f_y$*

  (SERVICE I check for R/C and P/S concrete bridges is similar to SERVICE II check for steel bridges -- permanent deformation under overloads)

LRFR SERVICEABILITY CHECKS FOR REINFORCED CONCRETE BRIDGES

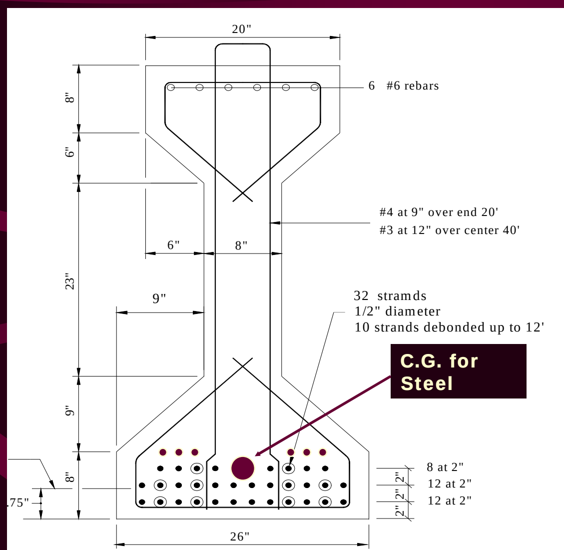
- **None required for HL93 and legal load ratings. Same as Load Factor method.**

- **Permit loads should be checked to ensure there is no yielding in rebars under SERVICE I**

  $f_{DL} + f_{LL+IM} \leq 0.9\ f_y$

- **Avoids permanent deformations in members. Could govern certain heavy single-trip permits.**

CONCRETE MEMBER WITH DISPERSED STRANDS



- Steel area is typically lumped at the CG for ease of analysis.
- Strands in the bottom layer will have higher stresses than the average stress at the CG (Use SERVICE I check).

LRFR SERVICEABILITY RATING OF STEEL BRIDGES

Limit State	*Load Type*	*Load Factor*
SERVICE II	*HL93 (Inv.)*	*1.3*
SERVICE II	*HL93 (Opr.)*	*1.0*
SERVICE II	*Legal*	*1.3*
SERVICE II	*Permit*	*1.0 (optional)*

Allowable Steel Stress:

$f_R = 0.95\ F_{yf}$ (composite sections)

$ = 0.80\ F_{yf}$ (non-composite sections)

NEW LRFR FATIGUE EVALUATION PROVISIONS
CALCULATION OF REMAINING FATIGUE LIFE

- Principals of reliability and uncertainty employed in the LRFD Specifications have been extended to fatigue life evaluation in the new LRFR Manual (Section 7).
- Procedures consistent with LRFD fatigue design provisions.
  - Infinite-life check
  - Finite-life check
- Three levels of finite fatigue life:
  1) Design fatigue life (2% failure probability)
  2) Evaluation fatigue life (16% failure probability)
  3) Mean fatigue life (50% failure probability)

LOAD & RESISTANCE FACTOR RATING OF HIGHWAY BRIDGES

ADOT / FHWA

SESSION 5

THE NEW AASHTO MANUAL FOR BRIDGE EVALUATION

Bala Sivakumar, P.E.

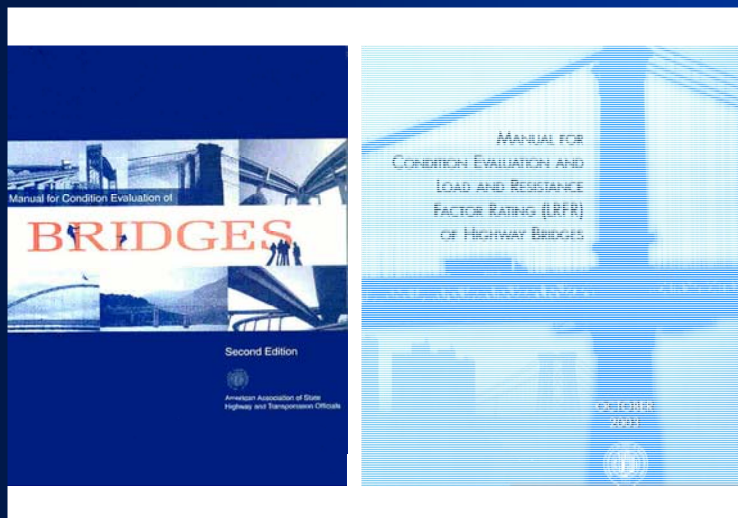
2005 AASHTO BRIDGE MEETING

AASHTO
Subcommittee on
Bridges and Structures

- AASHTO Adopted the LRFR Manual to replace the 1994 *Manual for Condition Evaluation* with the following modifications:
  - Change title to " The Manual for Bridge Evaluation (MBE)"
  - Include Load Factor and Allowable Stress ratings in a new section in the Manual.
  - No priority will be placed on any rating method.
  - Include new legal loads for rating & posting (NCHRP 12-63).
  - Update to be consistent with LRFD 3rd Edition.

  Contractor : Lichtenstein Engineers.

A NEW SINGLE STANDARD FOR BRIDGE EVALUATION



NEW MBE MANUAL TIMELINE

• AASHTO SCOBS VOTE ON MBE	JUNE 2005
• AASHTO CONTRACT AUTHORIZED	JAN 2006
• FIRST DRAFT OF MBE	APRIL 2006
• REVIEW MEETING WITH T18 COMMITTEE	AUG 2006
• REVISED DRAFT OF MBE	OCT 2006
• FIRST DRAFT OF RATING EXAMPLES	NOV 2006
• FINAL DRAFT OF MBE	DEC 2006
• PUBLISH	2008

MBE Sections

1. Introduction
2. Bridge File (Records)
3. Bridge Management Systems
4. Inspection
5. Material Testing
6. Load ~~and Resistance Factor~~ Rating
7. Fatigue Evaluation of Steel Bridges
8. Non-Destructive Load Testing

~~9. Special Topics~~

Appendix A: Illustrative Examples

Section 6. Load Rating

Section 6 is organized as follows:

PART A — LRFR

PART B — Allowable Stress & Load Factor Ratings

Section 6. Load Rating

The Interims for 2005 and 2006 for all three rating methods have been incorporated.

LRFR rating provisions have been updated to be consistent with the 3rd edition of the LRFD Design Specifications.

Includes LRFR provisions for Concrete Segmental Bridges & Steel Curved Girder Bridges.

Section 6. Load Rating

Includes new legal loads for rating & posting –
Notional Rating Load NRL, SU4, SU5, SU6, SU7

NCHRP Project 12-63

AASHTO LOAD MODELS FOR BRIDGE RATING & POSTING

- Investigate recent developments in specialized truck configurations and State legal loads
- Recommend revisions to the AASHTO legal loads as depicted in the current *AASHTO Manual for Condition Evaluation and the new AASHTO LRFR Guide Manual.*
- *Lichtenstein – Prime Contractor*

NCHRP 12-63 – NEW AASHTO LEGAL LOADS

ANALYSIS OF WEIGH-IN-MOTION DATA

RECENT WEIGH-IN-MOTION DATA FROM 18 STATES WERE USED TO IDENTIFY UNUSUAL SHORT MULTI-AXLE TRUCKS MEETING THE FOLLOWING CRITERIA:

- SATISFIES FEDERAL FORMULA B
- WEIGHS 80 KIPS OR LESS
- SINGLE-UNIT TRUCKS

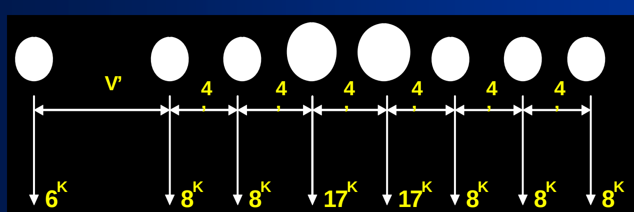
SPECIALIZED HAULING VEHICLES THAT MEET BRIDGE FORMULA B

NOTIONAL RATING LOAD FOR ALL FORMULA B TRUCKS

- DEVELOP A SINGLE ENVELOPE RATING LOAD MODEL FOR ALL FORMULA B TRUCKS:
    - A NOTIONAL TRUCK
    - SIMPLIFIES THE RATING ANALYSIS

2005 REVISIONS TO AASHTO LOADS

NOTIONAL RATING LOAD NRL



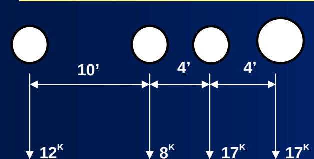
- GVW = 80 KIPS
- V — 6'0" TO 14'-0". SPACING
- AXLES THAT DO NOT CONTRIBUTE TO THE MAXIMUM LOAD EFFECT UNDER CONSIDERATION SHALL BE NEGLECTED.

RATING FOR TRUCKS WITH LIFT AXLES

- WHEN CHECKING FORMULA B REQUIREMENTS FOR LIFT AXLE TRUCKS, ALL AXLES ARE CONSIDERED DEPLOYED.
  - *TRUCKS ARE "ILLEGAL" IF ALL AXLES ARE NOT DEPLOYED DURING HIGHWAY OPERATION. THIS IS AN ENFORCEMENT PROBLEM NOT A LOAD RATING ISSUE.*

2005 AASHTO REVISIONS

ADOPTS NEW POSTING LOADS RECOMMENDED BY NCHRP 12-63

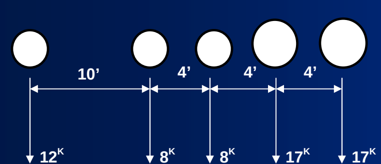


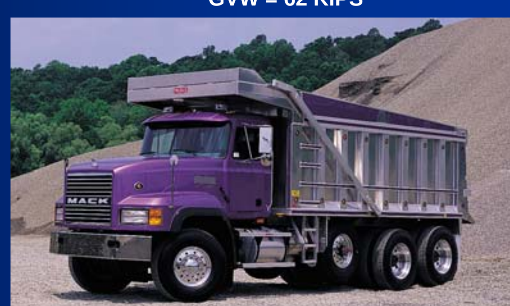
SU4 TRUCK

GVW = 54 KIPS

SU5 TRUCK

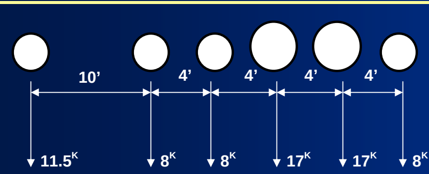
GVW = 62 KIPS





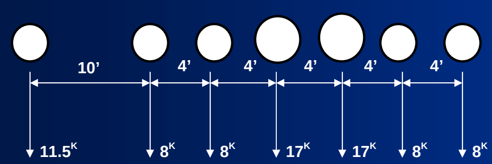
2005 AASHTO REVISIONS

ADOPTS NEW POSTING LOADS RECOMMENDED BY NCHRP 12-63



SU6 TRUCK

GVW = 69.5 KIPS



SU7 TRUCK

GVW = 77.5 KIPS

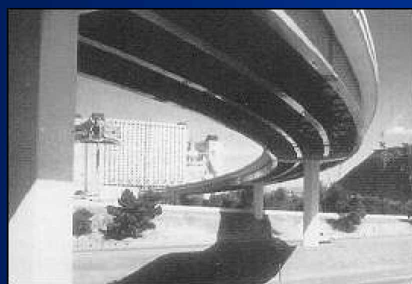
LRFR STEEL BRIDGE RATING

A-6.6.1 Scope

The provisions of this section apply to components of straight or horizontally curved I-girder bridges and straight or horizontally curved single or multiple closed-box or tub girder bridges.

LRFD STEEL FLEXURAL DESIGN

LRFD 3rd Edition Provisions for the Flexural Design of Straight & Curved Steel-Girder Bridges



"One-Third" Rule

$$f_{bu} + \frac{1}{3} f_{\ell} \leq F_n$$

$$M_u + \frac{1}{3} f_{\ell} S_x \leq M_n$$

Unifies equations for curved and straight I-girder design (lateral bending from all sources: curvature, wind, deck placement, skew)

M_u f_{bu} = major axis bending

f_l = lateral bending stress

Analysis

LRFR 6A.3.3 Refined Methods of Analysis

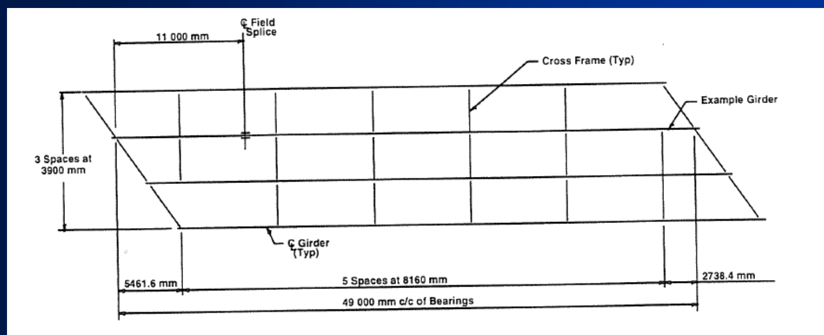
Analysis of bridges curved in plan should be performed using refined methods of analysis, unless the Engineer ascertains that approximate methods of analysis are appropriate (according to the provisions of LRFD Article 4.6.2.2.4).

CURVED STEEL BRIDGES

6A.6.9.7 Diaphragms and Cross-Frames

Diaphragm and cross-frame members in horizontally curved bridges shall be considered to be primary members and should be load rated accordingly

SKEWED STEEL BRIDGES



- **Significant flange lateral bending effects in straight girders maybe caused by the use of discontinuous cross-frames / diaphragms in conjunction with skews exceeding 20°.**

Lateral Bending Stress

- **The f_ℓ term may be included in the load rating analysis by adding to the other appropriate component major-axis bending stresses**
- **A suggested estimate of f_ℓ for skewed straight girder bridges, which may be used in lieu of a direct structural analysis of the bridge, is discussed in LRFD Article C6.10.1.**

LRFR Timber Bridge Rating

- Timber base resistance values have been been changed in LRFD 3rd Edition (2006 Interim) to make it consistent with NDS 2005
- Strength specified in terms of allowable stress, even for LRFD.
- 2nd Edition Dynamic Load Allowance - 16.5% Dropped in 3rd edition
- Major changes to adjustment factors
  - ✓ Time Effect Factor, CT = 0.8, Added in 3rd edition

LRFR Timber Bridge Rating

LRFD 3rd Edition with 2006 Interims

$F_b = F_{bo} C_{KF} C_M \left(C_F \text{ or } C_V\right) C_{fu} C_i C_d C_\lambda$ LRFD Eq. 8.4.4.1-1

F_{bo} = 1.25 Ksi Reference Design Value LRFD Table 8.4.1.1.4-1

$C_{KF} = 2.5/\phi = 2.5 / 0.85 = 2.94$ Format Conversion Factor LRFD 8.4.4.2

C_M = 1.0 Wet service factor LRFD 8.4.4.3

C_F = 1.0 Size Effect factor for sawn lumber LRFD 8.4.4.4

C_{fu} = 1.0 Flat Use Factor LRFD 8.4.4.6

C_i = 0.8 Incising Factor LRFD 8.4.4.7

C_d = 1.0 Deck Factor LRFD 8.4.4.8

C_λ = 0.8 Time Effect Factor for STRENGTH I LRFD 8.4.4.9

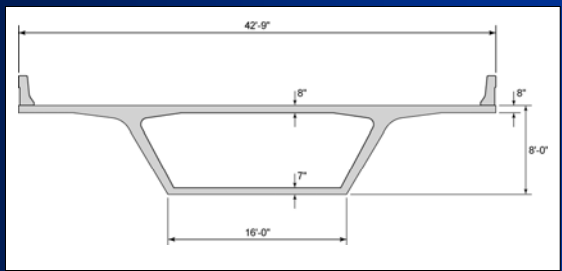
$F_b = 1.25 x 2.94 x 1.0 x 1.0 x 1.0 x 0.8 x 1.0 x 0.8$

Adjusted Design Value = 2.35 Ksi

LRFR MANUAL -- 2005 INTERIM

Load Rating of Post-Tensioned Segmental Concrete Bridges Consistent with LRFR Requirements

GENERAL RATING REQUIREMENTS



The load-rating capacity of post-tensioned concrete segmental bridges shall be checked in the longitudinal and transverse direction.

It is possible for transverse effects in a typical segmental box section to govern a load rating for a bridge. This can be a consequence of the flexural capacity of the top slab at the root of the cantilever wing or interior portion.

SERVICE LIMIT STATES

- Service I and Service III limit states are mandatory for load rating of segmental concrete box girder bridges.
- Service III limit state specifically includes the principal tensile stress check of LRFD Design Article 5.8.6 with a stress limit of $0.126\sqrt{f'c}$.
- The principal tensile stress check is necessary in order to verify the adequacy of webs of segmental box girder bridges for longitudinal shear and torsion.
- The number of live load lanes may be taken as the number of striped lanes.

LRFR RATING EQUATION

$$\mathbf{RF} = \frac{\phi_C \phi_S \phi R - \gamma_{DC} DC - \gamma_{DW} DW}{\gamma_L (\mathrm{LL+IM})}$$

$$\phi_C \phi_S \geq 0.85$$

ϕ_S SYSTEM FACTOR FOR REDUNDANCY (0.85 to 1.0)

ϕ_C CONDITION FACTOR

SYSTEM FACTOR ϕ_s

In the context of post-tensioned segmental box girders, the system factor must properly account for a few significant and important aspects different than other types of bridges:

- ✓ **Longitudinally continuous versus simply supported spans,**
- ✓ **The inherent integrity afforded by the closed continuum of the box section,**
- ✓ **Multiple-tendon load paths,**
- ✓ **Number of webs per box, and**
- ✓ **Types of details and their post-tensioning.**

SYSTEM FACTOR ϕ_s

- **BASED UPON THE APPROACH IN NCHRP REPORT 406 IT IS CONSIDERED THAT SYSTEM FACTORS FOR THE DESIGN OF SIMPLE AND CONTINUOUS SEGMENTAL BRIDGES WITH A MINIMUM OF 4 TENDONS PER WEB COULD BE 1.10 AND 1.20, RESPECTIVELY.**
- **LONGITUDINAL CONTINUITY IS RECOGNIZED THROUGH THE SIMPLE CONCEPT OF THE NUMBER OF PLASTIC HINGES NEEDED TO FORM A COLLAPSE MECHANISM:**
- **A SYSTEM FACTOR FOR TRANSVERSE FLEXURE OF 1.00 IS APPROPRIATE,**

LRFR System Factors for Longitudinal Flexure in Segmental Concrete Box Girder Bridges.

Bridge Type	Span Type	# of Hinges to Failure	System Factors (ϕ_s) No. of Tendons per Web[a] 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
	End or Hinge Span	2	0.85	1.00	1.10	1.15
	Statically Determinate	1	n/a	0.90	1.00	1.10
Precast Span-by-Span Type A Joints	Interior Span	3	n/a	1.00	1.10	1.2
	End or Hinge Span	2	n/a	0.95	1.05	1.15
	Statically Determinate	1	n/a	n/a	1.00	1.10
Precast Span-by-Span Type B Joints	Interior Span	3	n/a	1.00	1.10	1.2
	End or Hinge Span	2	n/a	0.95	1.05	1.15
	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
	End or Hinge Span	2	0.85	1.00	1.10	1.15
	Statically Determinate	1	n/a	0.90	1.00	1.10

LOAD & RESISTANCE FACTOR RATING OF HIGHWAY BRIDGES

ADOT / FHWA

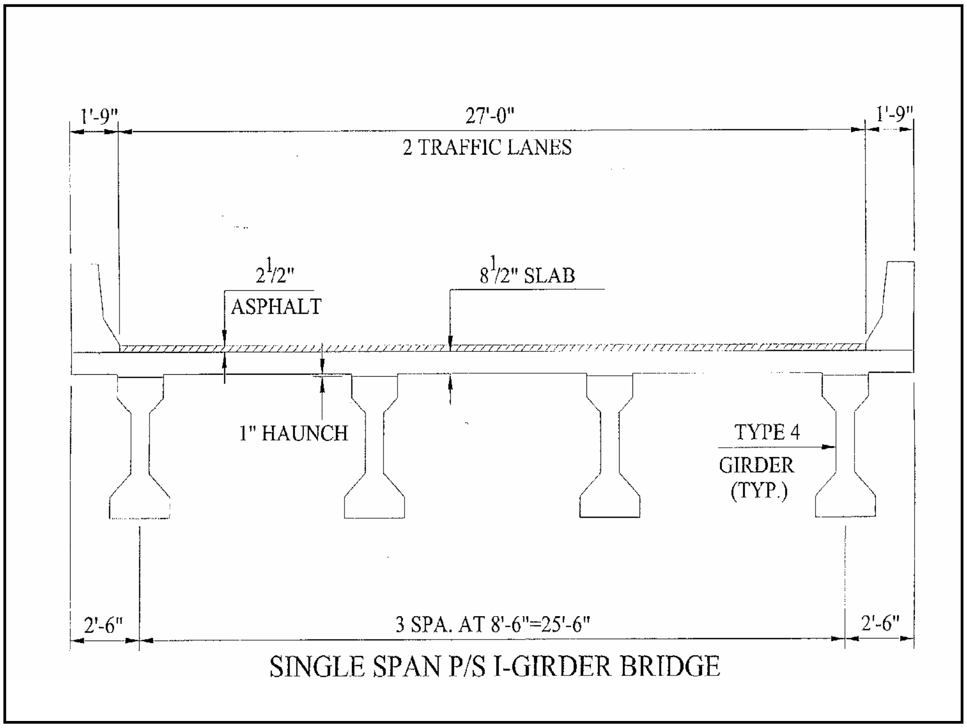
SESSION 6

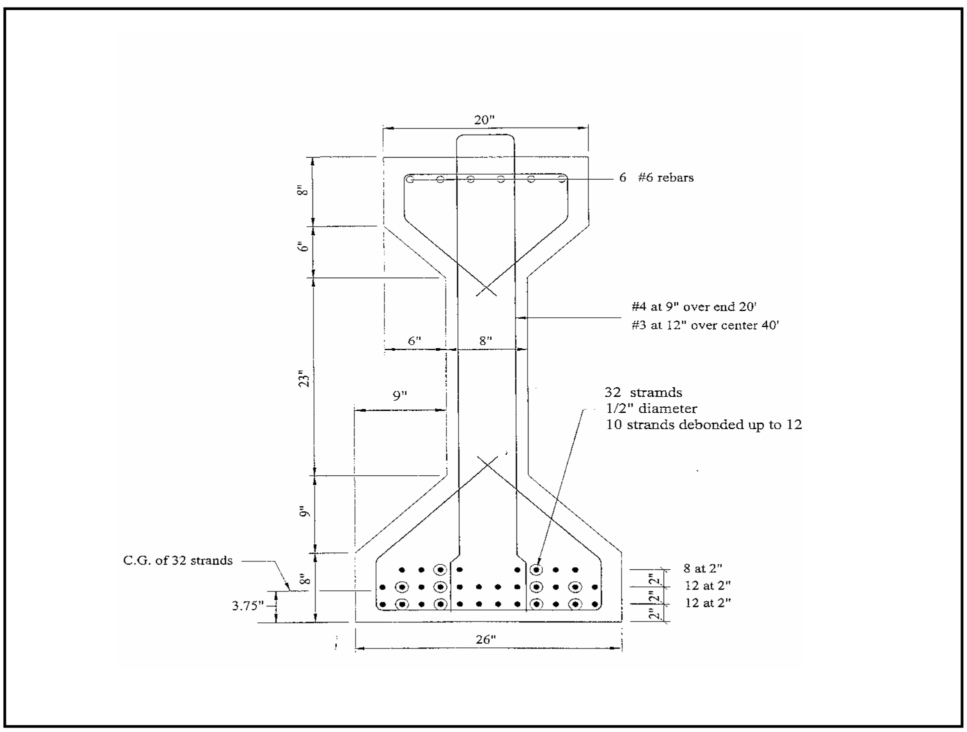
SIMPLE SPAN PRESTRESSED CONCRETE I-GIRDER BRIDGE -- LRFR EVALUATION OF AN INTERIOR GIRDER

SIMPLE SPAN PRESTRESSED CONCRETE I-GIRDER BRIDGE -- LRFR EVALUATION OF AN INTERIOR GIRDER

Bridge Data

Span:	80 ft. (Total Length = 81 ft.)
Year Built:	1985
Materials:	
Concrete:	$f'_c = 4.0$ ksi (Deck)
	$f'_c = 5$ ksi (P/S Beam)
	$f_{ci} = 4$ ksi (P/S Beam at transfer)
Prestressing Steel:	1/2 in. diameter, 270 ksi, Low-Relaxation Strands
	$A_{ps} = 0.153$ in^2 per strand
	32 prestressing strands, 10 are debonded over the last 12 ft on each end
Stirrups:	#4 at 9 in. over end 20 ft.
	#3 at 12 in. over center 40 ft.
Compression Steel:	six #6 Grade 60
Condition:	No Deterioration, NBI Item 59 Code = 6
Riding Surface:	Minor surface deviations (Field verified and documented)
ADTT (one direction)	5000


1'-9"
27'-0"
1'-9"
2 TRAFFIC LANES
2 1/2"
ASPHALT
8 1/2" SLAB
1" HAUNCH
TYPE 4
GIRDER
(TYP.)
2'-6"
3 SPA. AT 8'-6"=25'-6"
2'-6"
SINGLE SPAN P/S I-GIRDER BRIDGE


20"
6 #6 rebars
8"
6"
#4 at 9" over end 20'
#3 at 12" over center 40'
6"
8"
23"
9"
32 stramds
1/2" diameter
10 strands debonded up to 12
9"
C.G. of 32 strands
8"
3.75"
8 at 2"
12 at 2"
12 at 2"
2"
2"
2"
2"
26"

Summary of Section Properties

Type 4 Girder

h = 54 in.

A = 789 in.^{2}

I = 260730 in.^{4}

Y_b = 24.73 in.

S_b = 10543 in.^{3}

S_t = 8908 in.^{3}

2) Composite Section

Effective Flange Width b_e = 102 in.

$$\text{Modular Ratio } n = \frac{E_{deck}}{E_{beam}} = \frac{3.64\times10^3}{4.07\times10^3} = 0.89$$

$$\text{Transformed Width, } b_{trans} = 102 \text{ in.} \times 0.89 = 90.8 \text{ in.}$$

Components and Attachments *DC*

a) Non-Composite Dead Loads DC_1

Girder Self Weight: = 0.822 kip/ft.

Diaphragms: = 0.150 kip/ft.

Slab + haunch:

$$\left[\frac{8.5\text{ in.}}{12\text{ in./ft.}}\times 8.5\text{ ft.} + \frac{1\text{ in.}\times 20\text{ in.}}{144\text{ in.}^2/\text{ft.}^2}\right]\times 0.15\text{ kcf} \qquad = 0.925 \text{ kip/ft.}$$

Total per Girder DC_1 = 1.90 kip/ft.

$$V_{DC_1} = 1.90\text{ kip/ft.}\times\frac{80\text{ ft.}}{2} = 76 \text{ kip}$$

$$M_{DC_1} = \frac{1}{8}\times 1.90\text{ kip/ft.}\times(80\text{ ft.})^2 = 1520 \text{ kip-ft.}$$

a) Distribution Factor for Moment g_m

One Lane Loaded:

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

$$= 0.06 + \left(\frac{8.5}{14}\right)^{0.4}\left(\frac{8.5}{80}\right)^{0.3}(2.28)^{0.1}$$

$$= 0.514$$

Two or More Lanes Loaded:

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

$$= 0.075 + \left(\frac{8.5}{9.5}\right)^{0.6}\left(\frac{8.5}{80}\right)^{0.2}(2.28)^{0.1}$$

$$= 0.724 > 0.514$$

use $g_m = 0.724$

b) Distribution Factor for Shear g_v

One Lane Loaded:

$$g_{v1} = 0.36 + \frac{S}{25}$$

$$= 0.36 + \frac{8.50}{25}$$

$$= 0.70$$

Two or More Lanes Loaded:

$$g_{v2} = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2$$

$$= 0.2 + \left(\frac{8.5}{12}\right) - \left(\frac{8.5}{35}\right)^2$$

$$= 0.849 > 0.70$$

use $g_v = 0.849$

Compute Maximum Live Load Effects

a) Maximum Design Live Load (HL-93)

$$IM = 33\%$$
$$M_{LL+IM} = 512 + 1160 \times 1.33$$
$$= 2054.8 \text{ kip-ft.}$$

Distributed Live Load Moment at Midspan

$$M_{LL+IM} = 2054.8 \times g_m$$
$$= 2054.8 \times 0.724$$
$$= 1487.7 \text{ kip - ft.}$$

Compute Nominal Flexural Resistance at Midspan

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$

$$k = 0.28 \text{ for low-relaxation strands}$$
$$f_{pu} = 270 \text{ ksi}$$

$$f_{ps} = 270\left(1 - 0.28 \times \frac{4.39}{59.75}\right)$$
$$= 264.4 \text{ ksi}$$

Nominal Flexural Resistance (Midspan):

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$
$$= 4.896 \times 264.4\left(59.75 - \frac{3.73}{2}\right)\frac{1}{12}$$
$$= 6244.4 \text{ kip-ft.}$$

Maximum Reinforcement

φ factor of compression controlled sections shall be reduced in accordance with LRFD Article 5.5.4.2.1.

The net tensile strain, ε_t, is the tensile strain at nominal strength and determined by strain compatibility.

Given an allowable concrete strain of 0.003 and depth to neutral axis c = 4.39 in.
d_p = 59.75 in.

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_t}{d-c}$$

$$\frac{0.003}{4.39''} = \frac{\varepsilon_t}{59.75''-4.39''}$$

$$\varepsilon_t = 0.0378$$

For ε_t = 0.0378 > 0.005, the **section is tension controlled** and Resistance Factor φ shall be taken as 1.0.

Minimum Reinforcement

Amount of reinforcement must be sufficient to develop M_r equal to the lesser of:

1.2 M_{cr} or 1.33 M_u

$$M_R = \varphi M_n = (1.0)\ (6244.4) = 6244.4 \text{ kip-ft.}$$

1) $1.33\ M_u = 1.33\ [1.75\ (1487.7) + 1.25\ (1520 + 200) + 1.5\ (162)]$

$= 6645.3$ kip-ft. > 6244.4 kip-ft.

2) $M_{cr} = S_c\left(f_r + f_{cpe}\right) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$

$1.2 \times M_{cr} = 1.2 \times 3492.4$ kip-ft = 4190.9 kip-ft.
$1.33\ M_u = 6645.3$ kip-ft. (previously calculated)
$1.2 \times M_{cr} < 1.33\ M_u$ therefore, $1.2 \times M_{cr}$ *governs*
$M_r = 6244.4$ kip-ft. (previously calculated)
$M_r = 6244.4$ kip-ft. > 4190.9 kip-ft. OK

The minimum reinforcement check is satisfied.

Distance from centerline of bearing to critical shear section:

$= 58.4\ \text{in.} + 6\ \text{in.}$

$= 64.4\ \text{in.}$

$= 5.37\ \text{ft.}$

Maximum Shear at Critical Section Near Supports

$\text{Total Shear} \quad = V_{LANE} + V_{TRUCK} \times 1.33 \quad = 100.5\ \text{kips}$

$\text{Distributed } V_{LL+IM} = 100.5\ \text{kips} \times 0.849 \quad = 85.3\ \text{kips}$

Dead Load Shears:

DC_1 = 1.90 kip/ft. and DC_2 = 0.25 kip/ft.

DW = 0.203 kip/ft.

$V_{DC} = (1.90\ \text{klf} + 0.25\ \text{klf})(0.5 \times 80\ \text{ft.} - 5.37\ \text{ft.}) \quad = 74.5\ \text{kips}$

$V_{DW} = (0.203\ \text{klf})\ (0.5 \times 80\ \text{ft.} - 5.37\ \text{ft.}) \quad = 7.03\ \text{kips}$

Compute Nominal Shear Resistance

The nominal shear resistance V_n, shall be the lesser of :

$V_n = V_s + V_c + V_p$

$V_n = 0.25\ f_c' b_v d_v + V_p$

$V_p = 0.0$ as straight tendons are provided

MCFT (Sectional Design Model)

Shear stress on the concrete

$$v = \frac{V_u - \varphi V_p}{\varphi b_v d_v}$$

$$= \frac{252.8}{(0.9)(8)(58.4)} = 0.601\ \text{ksi}$$

$$\frac{v}{f_c'} = \frac{0.601}{5.0} = 0.12 < 0.25$$

At First Critical Section for Shear (64.4 in. from c.l. of bearing)

$g_m \times M_{LL+IM} = 0.724 \times 539.3 = 390.5$ kip-ft.

Dead Load Moments at First Critical Section for Shear:

Load	Load Factor γ
DC	1.25
DW	1.50
LL	1.75

Factored Moment: $M_u = (1.75)\ (390.5) + (1.25)\ (430.8) + (1.50)\ (40.7)$

$= 1282.9$ kip-ft.

Transfer Length 60 strand diameters = 30 in. < 64.4 in.

As the section is outside the transfer length, the full value of f_{po} is used in resistance.

$\frac{v}{f'_c} = 0.12$ (≤ 0.125, row 3 of LRFD Table 5.8.3.4.2-1)

Assume $\varepsilon_x \leq -0.10 \times 10^{-3}$ $(\varepsilon_x \times 1000 \leq -0.10)$

From LRFD Table 5.8.3.4.2-1: (row 3, column 2) $\theta = 21.9°$ $\beta = 2.99$

If ε_x is negative, it must be recalculated including concrete stiffness.

A_c = Area below $h/2$

$= (8)(26) + 1/2\ (8 + 26)(9) + (10)(8) = 441$ in.² = 441 in

$$\varepsilon_x = \frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}}{2\left(E_cA_c + E_sA_s + E_pA_{ps}\right)}$$

$$\varepsilon_x = \frac{\frac{12 \times 1282.9}{58.4} + (0.5)(252.8)(\cot 21.9°) - (3.366)(189)}{2((4030)(441) + 0 + (28500)(3.366))}$$

$= -0.016 \times 10^{-3} >$ assumed $\varepsilon_x \leq -0.10 \times 10^{-3}$

Assume $\varepsilon_x \leq 0$

From LRFD Table 5.8.3.4.2-1: (row 3, column 4) $\theta = 23.7°$ $\beta = 2.87$

$$V_c = 0.0316\ \beta \sqrt{f_c'} b_v d_v$$
$$= (0.0316)(2.87)\sqrt{5}(8)(58.4)$$
$$= 94.75 \text{ kips}$$
$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$
$$= \frac{(0.39)(60)(58.4)(\cot 23.7°)}{9}$$
$$= 345.9 \text{ kips}$$
$$V_n = V_c + V_s$$
$$= 94.75 + 345.9 = 440.7 \text{ kips} < 584 \text{ Kips}$$
$$\phi V_n = 0.9 \text{ x } 440.7 = 396.6 \text{ kips}$$

Summary of Moments and Shears

location	support	critical shear	stirrup change	midspan
x/L	0.0	0.067	0.25	0.5
X (ft.)	0.0	5.37	20	40
V_{DC1} (kips)	76	65.8	38	-
V_{DC2} (kips)	10	8.7	5	-
V_{DW} (kips)	8.12	7.03	4.1	-
$g_m V_{LL+IM}$ (kips)	-	85.3	63.7	-
V_n (kips) simplified	-	221.7	129.1	-
V_n (kips) MCFT	-	440.7	227	-
M_{DC1} (kip-ft.)	-	380.7	1140	1520
M_{DC2} (kip-ft.)	-	50.1	150	200
M_{DW} (kip-ft.)	-	40.7	121.8	162
$g_m M_{LL+IM}$ (kip-ft.)	-	390.5	1087	1487.7
M_n (kip-ft.)	-	-	-	6244.4

General Load Rating Equation

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$

Evaluation Factors (for Strength Limit State)

a) Resistance Factor φ

$\varphi = 1.0$ for flexure (tension controlled section)

$\varphi = 0.9$ for shear

b) Condition Factor φ_c

$\varphi_c = 1.0$ No member deterioration, NBI Item 59 Code = 6

c) System Factor φ_s

$\varphi_s = 1.0$ 4-girder bridge with spacing > 4 ft.

Design Load Rating HL-93

A) Strength I Limit State

a) Inventory Level

LOAD	LOAD FACTOR
DC	1.25
DW	1.50 Overlay thickness was not field measured.
LL	1.75

Flexure at Midspan

$$RF = \frac{(1.0)(1.0)(1.0)(6244.4) - \left[(1.25)(1520 + 200) + (1.5)(162)\right]}{(1.75)(1487.7)}$$

$$= 1.48$$

Shear at Critical Section

$$RF = \frac{(1.0)(1.0)(0.9)(440.7) - \left[(1.25)(65.8 + 8.7) + (1.50)(7.03)\right]}{(1.75)(85.3)}$$

$$= 1.96$$

B) Service III Limit State (Inventory Level)

$$RF = \frac{f_R - (\gamma_D)(f_D)}{(\gamma_L)(f_{LL+IM})}$$

Flexural Resistance $f_R = f_{pb}$ + Allowable tensile stress

f_{pb} = Compressive stress due to effective prestress

= 2.548 Ksi

$$\text{Allowable Tensile Stress} = 0.19\sqrt{f_c'} = 0.19\sqrt{5} = 0.425 \text{ ksi}$$

$$f_R = 2.548 + 0.425 = 2.973 \text{ ksi}$$

Determine Dead Load Stresses At Midspan:

M_{DC1} = 1520 kip-ft. and M_{DC2} = 200 kip-ft.

M_{DW} = 162 kip-ft

$S_{b\ (nc)}$ =10542 in^3 $S_{b\ (comp)}$ =17471 in^3

$$f_{DC} = \frac{1520 \times 12}{10542} + \frac{200 \times 12}{17471} = 1.87 \text{ ksi}$$

$$f_{DW} = \frac{162 \times 12}{17471} = 0.11 \text{ ksi}$$

$$\text{Total } f_D = 1.98 \text{ ksi}$$

Live Load Stress At Midspan:

M_{LL+IM} = 1487.7 kip-ft

$S_{b\ (comp\ )}$ =17471 in^3

$$f_{LL+IM} = \frac{1487.7 \times 12}{17471} = 1.02 \text{ ksi}$$

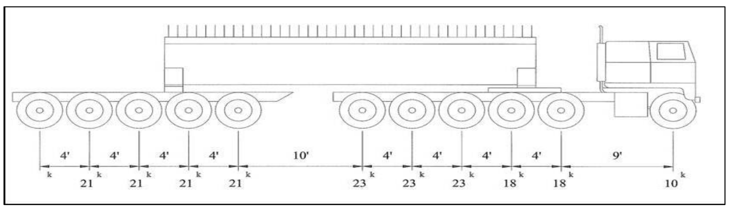
$$RF = \frac{2.973 - (1.0)(1.98)}{(0.8)(1.02)} = 1.22$$

Legal Load Rating

Inventory Design Load Rating RF > 1.0, therefore the legal load ratings do not need to be performed and no posting is required.

Permit Load Rating

Permit Type: Special, Single-Trip, Mix with traffic, No escort
Permit Weight: 220 kips
ADTT (one direction): 5000
Undistributed Maximum M_{LL} = 2950.5 kip-ft.
Undistributed Maximum V_{LL} = 157.9 kips



PERMIT LOAD – 220 K

Use One-Lane Distribution Factor and divide out the 1.2 multiple presence factor.

$$g_{m1} = 0.514 \times \frac{1}{1.2} = 0.428$$

$$g_{v1} = 0.70 \times \frac{1}{1.2} = 0.583$$

IM = 20% (Minor Surface Deviations)

Maximum Live Load Effect:

$$M_{LL+IM} = (2950.5)(0.428)(1.20)$$
$$= 1515.4 \text{ kip-ft.} \quad \text{at Midspan}$$

$$V_{LL+IM} = (157.9)(0.583)(1.20)$$
$$= 110.5 \text{ kips}$$

a) Flexure:

$$RF = \frac{(1.0)(1.00)(1.0)(6244.4) - [(1.25)(1520 + 200) + (1.5)(162)]}{(1.5)(1515.4)}$$
$$= 1.69 > 1.0 \quad \text{OK}$$

b) Shear (Using MCFT):

$$RF = \frac{(1.0)(1.0)(0.9)(440.7) - [(1.25)(72.0) + (1.50)(6.7)]}{(1.5)(110.5)}$$
$$= 1.79 > 1.0 \quad \text{OK}$$

Summary of Rating Factors

Limit State	Design Load Rating (HL-93)		Permit Load Rating
	Inventory	Operating	
Strength I			
Flexure (@ midspan)	1.48	1.92	
Shear (@ 64 in)	1.96		
Shear (@ 20 ft)	1.30		
Strength II			
Flexure (@ midspan)			1.69
Shear			1.79
Service III			
Flexure (@ midspan)	1.22		
Service I			Stress Ratio= 1.08

ADOT / FHWA

SESSION 7

LRFR FOR STEEL BRIDGES

MOVING FROM LFR TO LRFR

WHAT ARE THE CHANGES ON THE RESISTANCE SIDE?

- THE RESISTANCE SIDE OF THE EQUATION FOR STEEL DESIGN HAS NOT SEEN MANY CHANGES, EXCEPT FOR FATIGUE.
- MAJORITY OF LFD AND LRFD STEEL DESIGN EQUATION ARE EQUIVALENT – LRFD EQUATIONS ARE NON-DIMENSIONAL TO ALLOW TRANSITION TO SI.

FLEXURAL RESISTANCE OF COMPOSITE COMPACT-SECTIONS (2ND Edition)

- **If $D_p \leq D'$ then:** $\quad M_n = M_p$
- **If $D' < D_p \leq 5D'$ then:**

$$M_n = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right)$$

D_p = distance from the top of the slab to the neutral axis at the plastic moment (IN)

D' = distance specified in A6.10.4.2.2b (IN)

M_y = moment capacity at first yield

$= F_y \cdot S_{ST}$ (approx)

Flange Flexural Resistance of Non-Compact Sections

(2nd Edition)

$F_n = R_b R_h F_{yf}$

Where:

R_h = hybrid factor

R_b = load shedding factor

F_{yf} = yield strength of the flange

- *For homogeneous sections* $R_h = 1.0$
- ***The load shedding factor R_b accounts for the shedding of flexural stresses from the web to the compression flange as a result of bend buckling of slender webs.***

Nominal Shear Resistance of Unstiffened Webs

Nominal Shear Resistance limited to shear buckling or shear yield force. No Tension Field Action.

$$V_n = C\,V_p$$

$$V_p = 0.58\,F_{yw}\,D\,t_w$$

C = ratio of the shear buckling stress to the shear yield strength

V_p = plastic shear force

Shear Resistance of Interior Panels (Stiffened Section with Tension-field Action)(3RD Edition)

- Shear resistance of interior web panels proportioned such that:

$$\frac{2Dt_w}{b_{fc}t_{fc} + b_{ft}t_{ft}} \leq 2.5$$

Are capable of developing post-buckling shear resistance due to tension field action.

Shear Resistance of Interior Panels of Compact Sections

(Stiffened Section with Tension-field Action)

- **In 3rd Edition Moment-shear interaction is eliminated**

$$V_n = V_p \left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}} \right]$$

ϕ_f = resistance factor for flexure

M_u = factored moment

M_p = plastic moment

D = Web Depth

D_0 = Stiffener Spacing

LRFD 3RD EDITION -- 2004

New LRFD Provisions for the Flexural Design of Straight Steel-Girder Bridges

<u>REVISIONS TO LRFD A6.10</u>

Flexure of I-Girders

- Improve the flow, clarity, logic of the steel provisions. Flow corresponds more closely to the flow of the calculations.

- Reduce the complexity of the provisions applying to the majority of steel I-girder bridges.

- In particular, facilitate the creation of a fully <u>unified design specification</u> for straight and horizontally curved steel bridge structures.

“One-Third” Rule

$$f_{bu} + \frac{1}{3} f_{\ell} \leq F_n$$

$$M_u + \frac{1}{3} f_{\ell} S_x \leq M_n$$

- Unifies equations for curved and straight I-girder design (handling of lateral bending from all sources: curvature, wind, deck placement, skew)

M_u f_{bu} = major axis bending

f_l = lateral bending

Strength Limit State

Composite Sections in Positive Flexure

- Compact Sections

$$M_u + \frac{1}{3} f_\ell S_{xt} \leq \phi_f M_n$$

$$M_n = M_P \left[1.07 - 0.7 \frac{D_P}{D_t} \right]$$

D_p = distance from top of deck to plastic neutral axis of the composite section

D_t = total depth of composite section.

Strength Limit State

Composite Sections in Positive Flexure

- Non Compact Sections
  - Compression Flange

$$f_{bu} \leq \phi_f R_b R_h F_{yc}$$

  - Tension Flange

$$f_{bu} + \frac{1}{3} f_{\ell} \leq \phi_f R_h F_{yt}$$

Permanent Deformation Check

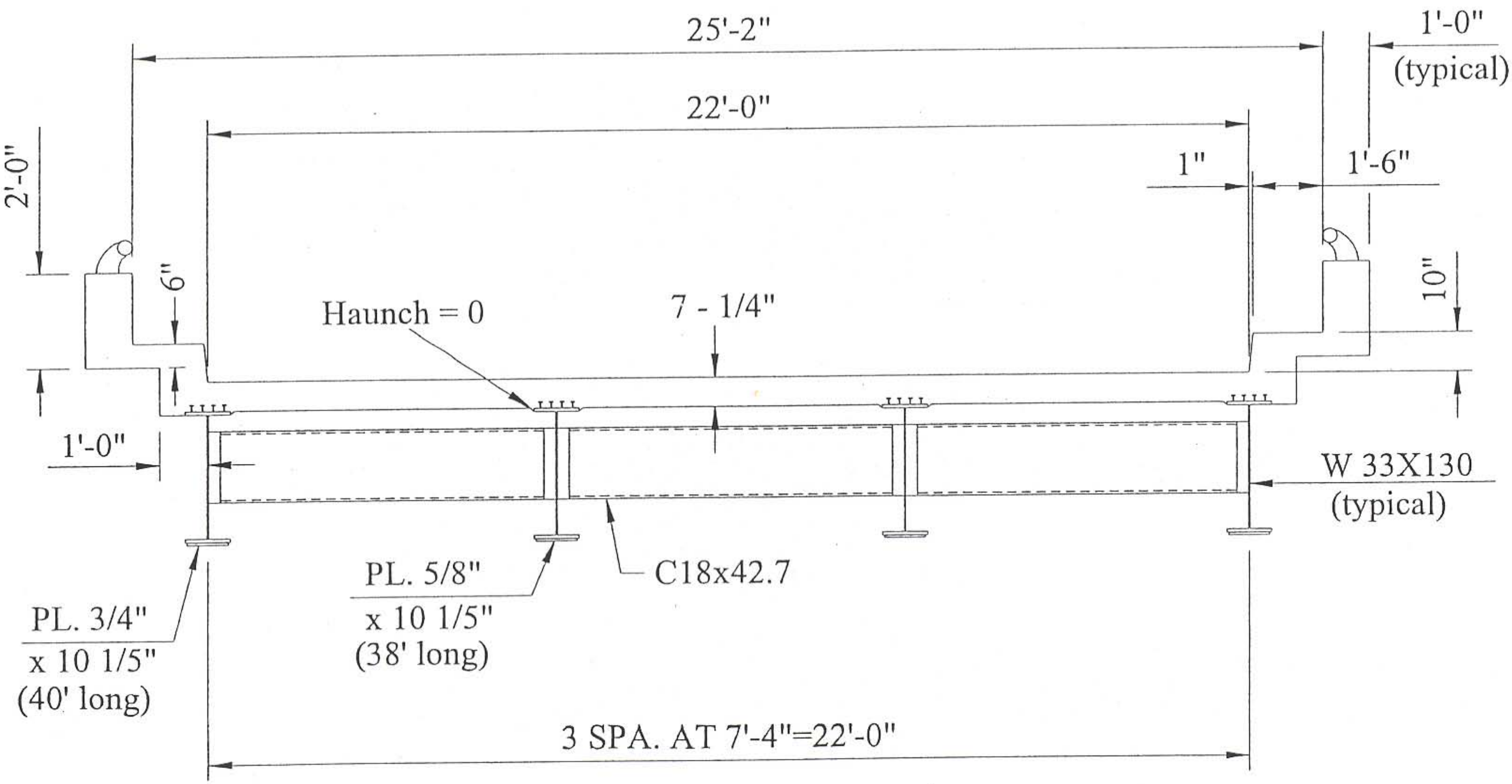
Service II Load Combination

$$1.0\ DL + 1.3\ LL \leq 1.0\ R_n$$

Load combination SERVICE II is applied to steel bridges for two considerations:

- **for members, to limit yielding and objectionable kinking, and**
- **for bolted connections, to prevent repeated slipping or kinking of the connections**

LOAD RATING EXAMPLE



COMPOSITE STEEL STRINGER BRIDGE
Example A1

Example - Simple Span Composite Steel Stringer Bridge Evaluation of Interior Stringers

GIVEN:

Span:	65 FT
Year Built:	1964
Material:	A 36 Steel
	$F_y = 36$ KSI
	$f'_c = 3$ KSI
Condition:	No deterioration (NBI Item 59 = 7)
	Member condition information not available.
Riding Surface:	Minor surface deviations (Field verified and documented)
ADTT (one direction):	1000
Skew:	0 degrees

Note: Same bridge as in example B1, 1994 AASHTO MCE.

Section Properties:

Noncomposite: W33 x 130 & PL 5/8 IN x 10 1/2 IN

$t_f = 0.855$ IN; $b_f = 11.51$ IN; $t_w = 0.58$ IN

$A = 38.26$ IN^2; $I = 6699$ IN^4

THE LRFR LOAD RATING PROCESS

1) DESIGN LOAD RATING (HL-93)

↓

2) LEGAL LOAD RATING (LEGAL LOADS)

↓

3) PERMIT LOAD RATING (OVERWEIGHT TRUCKS)

LRFR LOAD FACTORS FOR LEGAL LOADS

TRAFFIC VOLUME	LOAD FACTOR
ADTT > 5000	1.8
ADTT = 1000	1.6
ADTT < 100	1.4

Lichtenstein

SUMMARY OF SECTION PROPERTIES AT MIDSPAN

a) **Steel Section Only**

$S_{TOP} = 436\ IN^3$

$S_{BOT} = 563.7\ IN^3$

b) **Composite Section – Short Term** $n = 9$

$S_{TOP\ steel} = 4412\ IN^3$

$S_{BOT} = 793\ IN^3$

c) **Composite Section - Long Term** $3\ n = 27$

$S_{TOP\ steel} = 1458\ IN^3$

$S_{BOT} = 725\ IN^3$

DEAD LOAD ANALYSIS – INTERIOR STRINGER

1. Components and Attachments DC

a) **Non-Composite Dead Loads. DC_1**

Deck: $(7.33\ FT)\dfrac{(7.25\ IN)}{12}(0.150\ KCF)$ $= 0.664\ KIP/FT$

Stringer: $(0.130\ KIP/FT)(1.06)$ $= 0.138\ KIP/FT$
(6% increase for connections)

Cover Plate:
$$\frac{(0.625\ IN)(10.5\ IN)(0.490\ KCF/144)(1.06)(38\ FT)}{65\ FT} = 0.014\ KIP/FT$$

Diaphragms: $\dfrac{(3)(0.0427\ \text{KIP / FT})(7.33\ \text{FT})(1.06)}{65\ \text{FT}}$ = 0.015 KIP/FT

Total per stringer **= 0.831 KIP/FT**

$$M_{DC1} = \frac{0.831(65)^2}{8} = 439\ \textbf{KIP FT @ Midspan}$$

$$V_{DC1} = \mathbf{0.831}\left(\frac{65}{2}\right) = 27\ \textbf{KIP @ Bearing}$$

b) **Composite Dead Loads** **DC_2**

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

Curb: $(1\ \text{FT})\left(\dfrac{10\ \text{IN}}{12}\right)(0.150\ \text{KCF})\left(\dfrac{2\ \text{curbs}}{4\ \text{beams}}\right)$ = 0.062 KIP/FT

Parapet:

$\left[\left(\dfrac{6\ \text{IN} \times 19\ \text{IN}}{144}\right)+\left(\dfrac{18\ \text{IN} \times 12\ \text{IN}}{144}\right)\right](0.150\ \text{KCF})\left(\dfrac{2\ \text{parapets}}{4\ \text{beams}}\right)$ = 0.172 KIP/FT

Railing: Assume 0.020 KLF ÷ 2 = 0.01 KIP/FT

Total per Stringer **= 0.244 KIP/FT**

$$\mathbf{M_{DC2}} = \frac{0.244(65)^2}{8}\ \mathbf{= 129\ KIP\ FT\ @\ Midspan}$$

$$\mathbf{V_{DC2} = 0.244}\left(\frac{65}{2}\right)\ \mathbf{= 8\ KIP\ @\ Bearing}$$

2. Wearing Surface DW = 0

LIVE LOAD ANALYSIS – INTERIOR STRINGER

I. Compute Live Load Distribution Factors (Type a cross section). LRFD Table 4.6.2.2.1-1

Longitudinal Stiffness Parameter K_g

LRFD Eq 4.6.2.2.1 - 1

$$K_g = \frac{E_B}{E_D}(I + Ae_g^2)$$

$$E_D = 33{,}000(w_c)^{1.5}\sqrt{f'_c}$$

$$= 33000(0.145)^{1.5}\sqrt{3}$$

$$= 3155.9\ \text{KSI}$$

LRFD Eq 5.4.2.4-1

E_B =29,000 KSI

BEAM + COV. PL

$I = 8293\ IN^4$

$A = 44.82\ IN^2$

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

$K_g = (29000 / 3155.9)(8293 + 44.82 \times 22.65^2)$

$= 287\,493\ IN^4$

a) **Distribution Factor for Moment** g_m

LRFD
Table 4.6.2.2.2b-1

$$\frac{K_g}{12 L t_s^3} = \frac{287493}{12 \times 65 \times 7.25^3} = 0.967$$

One Lane Loaded:

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

$$= 0.06 + \left(\frac{7.33}{14}\right)^{0.4}\left(\frac{7.33}{65}\right)^{0.3}(0.967)^{0.1}$$

$$= \mathbf{0.46}$$

Two or More Lanes Loaded:

$$g_{m1} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$= 0.075 + \left(\frac{7.33}{9.5}\right)^{0.6}\left(\frac{7.33}{65}\right)^{0.2}(0.967)^{0.1}$$

$$= \mathbf{0.626 > 0.46}$$

$\therefore$ use $g_m = \mathbf{0.626}$

b) **Distribution Factor for Shear**

LRFD
Table 4.6.2.2.3a-1

One Lane Loaded

$$g_{V_1} = 0.36 + \frac{S}{25}$$

$$= 0.36 + \frac{7.33}{25}$$

$$= \mathbf{0.653}$$

Two or More Lanes Loaded

$$g_{V_2} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

$$= 0.2 + \frac{7.33}{12} - \left(\frac{7.33}{35}\right)^{2.0}$$

$$= \mathbf{0.767 > 0.653}$$

$\therefore$ use $\quad g_V = \mathbf{0.767}$

II. **Compute Maximum Live Load Effects**

a) Maximum Design Live Load (HL-93) Moment at midspan.

Design Lane Load Moment	=	338 KIP FT	
Design Truck Moment	=	890 KIP FT	Governs
Tandem Axles Moment	=	762.5 KIP FT	

IM = 33% — LRFD Table 3.6.2.1-1

$$\mathbf{M_{LL+IM} = 338 + 890 \times 1.33}$$

$$\mathbf{= 1521.7\ KIP\ FT}$$

b) **Maximum Design Live Load Shear at beam ends.**

Design Lane Load Shear	=	20.8 KIP	
Design Truck Shear	=	61.7 KIP	Governs
Tandem Axles Shear	=	48.5 KIP	

$$\mathbf{V_{LL+IM} = 20.8\ KIP + 61.7\ KIP \times 1.33}$$

$$\mathbf{= 102.9\ KIP}$$

<u>DISTRIBUTED LIVE LOAD MOMENTS AND SHEARS</u>

Design Live Load HL-93

$$\mathbf{M_{LL+IM} = 1521.7 \times g_m}$$

$$\mathbf{= 1521.7 \times 0.626}$$

$= 952.6$ KIP FT

$V_{LL+IM} = 102.9 \times g_V$

$= 102.9 \times 0.767$

$= 78.9$ KIP

COMPUTE NOMINAL RESISTANCE OF SECTION AT MIDSPAN

Locate Plastic Neutral Axis PNA

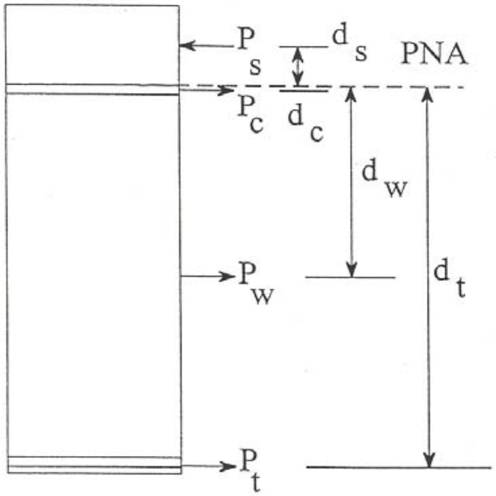
$t_f = 0.855$ IN
$t_w = 0.58$ IN
$b_f = 11.51$ IN
Cov. Pl Area $A_p = 6.56$ IN^2
(PL 5/8 IN x 10 1/2 IN)
Web Depth; D $= 33.10$ IN - 2 (0.855 IN)
$= 31.39$ IN

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

$$A_t = (11.51)(0.855) + (10.5)(0.625) = 16.40 \text{ IN}^2$$

$$y = \frac{(11.51)(0.855)\frac{(0.855)}{2} + (10.5)(0.625)\left(0.855 + \frac{0.625}{2}\right)}{(11.51)(.855) + (10.5)(.625)}$$

$= 0.724$ IN (from top of tension flange to centroid of flange and cover plate)



PLASTIC FORCES

LRFD Appendix A A6.1

P_s $= 0.85\, f_c'\, b_{eff}\, t_s$
$= 0.85 \times 3.0 \times 88 \times 7.25$
$= 1626.9$ KIP

P_c $= F_y\, b_f\, t_f$
$= 36 \times 11.51 \times 0.855$
$= 354.3$ KIP

P_w $= F_y\, D\, t_w$
$= 36 \times 31.39 \times 0.58$
$= 655.4$ KIP

P_t $= F_y\, (b_f\, t_f + A_P)$
$= 36\, (11.51 \times 0.855 + 6.56)$
$= 590.4$ KIP

$P_c + P_w + P_t = 354.3 + 655.4 + 590.4 = 1600.1$ KIP $< P_s = 1626.9$ KIP

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

$$\frac{\bar{y}}{t_s} \times P_s = P_c + P_w + P_t$$

$$\frac{\bar{y}}{7.25} \times 1626.9 = 1600.1$$

$$\bar{y} = 7.13 \text{ IN}$$

CLASSIFY SECTION

Following the I-Sections in Flexure Flowchart

LRFD
Fig. C 6.10.4-1

a) Check web slenderness:

LRFD
A 6.10.4.1.2

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

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

b) Check Ductility Requirement.

LRFD
A 6.10.4.2.2a

$$D_p = \bar{y} = 7.13 \text{ IN}$$

$$DD' = \beta \left(\frac{d + t_s + t_h}{7.5} \right)$$

$$= 0.9 \left(\frac{33.725 + 7.25}{7.5} \right)$$

$$= 4.92 \text{ IN}$$

$$\frac{D_p}{D'} = \frac{7.13}{4.92} = 1.45 < 5$$

The section has adequate ductility.

PLASTIC MOMENT M_P

Moment Arms about the PNA

Slab: $d_s = \dfrac{\bar{y}}{2} = \dfrac{7.13}{2} = 3.56$ IN

Compression Flange: $d_c = \left(t_s - \bar{y}\right) + \dfrac{t_c}{2}$

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

$$= 0.55 \text{ IN}$$

Web: $d_w = \left(t_s - \bar{y}\right) + t_c + \dfrac{D}{2}$

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

$$= 16.67 \text{ IN}$$

Tension Flange: $d_t = \left(t_s - \bar{y}\right) + t_c + D + \dfrac{t_t}{2}$

$$= (7.25 - 7.13) + 0.855 + 31.39 + \frac{0.724}{2}$$

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

$$= 32.73 \text{ IN}$$

The plastic moment M_p is the sum of the moments of the plastic forces about the PNA.

LRFD
A 6.10.3.1.3
& Appendix A 6.1

$$M_p = \frac{\bar{y}}{t_s} P_s d_s + P_c d_c + P_w d_w + P_t d_t$$

$$= \left[\frac{7.13}{7.25} \times 1626.9 \times 3.56 + 354.3 \times 0.55 + 655.4 \times 16.67 + 590.4 \times 32.73\right]\frac{1}{12}$$

$$= 3011.7 \text{ KIP FT}$$

<u>Nominal Flexural Resistance M_n</u>

LRFD
A 6.10.4.2.2a

as $D_p > DD'$ $\quad M_n = \dfrac{5M_p - 0.85M_y}{4} + \dfrac{0.85M_y - M_p}{4}\left(\dfrac{D_p}{D'}\right)$

Yield Moment of a Composite Section M_y:

LRFD
A 6.10.3.1.2
& Appendix A6.2

Assume bottom flange reaches F_y before top flange.

$$F_y = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}}$$

Where M_{D1}, M_{D2} and M_{AD} are moments due to factored loads

$$36 = \frac{1.25 \times 439 \times 12}{563.7} + \frac{1.25 \times 129 \times 12}{725} + \frac{M_{AD}}{793}$$

M_{AD} = 17168 KIP IN = 1430.7 KIP FT

$M_y = M_{DC_1} + M_{DC_2} + M_{AD}$

M_y = 1.25 x 439 + 1.25 x 129 + 1430.7

= 2140.7 KIP FT

alternatively: $M_y = F_yS$ $M_y = (36)(793) = 28548$ KIP IN $M_y = 2378.7$ KIP FT

Approximate method allowed for calculating M_n in LRFD C 6.10.4.2.2a

$$\mathbf{M_n} = \frac{5 \times 3011.7 - 0.85 \times 2140.7}{4} + \frac{0.85 \times 2140.7 - 3011.7}{4}\left(\frac{7.13}{4.92}\right)$$

= 2877.8 KIP FT

Nominal Shear Resistance V_n

LRFD A 6.10.7.2

W 33 x 130 Rolled section, no stiffeners.

Web Depth clear of fillet = 29.75 IN
Total Depth – 2 (Flange thickness) = 31.39 IN

$$\frac{D}{t_w} = \frac{29.75}{0.580} = 51.3$$

$$2.46\sqrt{\frac{E}{F_{yw}}} = 2.46\sqrt{\frac{29000}{36}} = 69.8 > \frac{D}{t_w} = 51.3$$

then

$\mathbf{V_n}$ **= 0.58 F_{yw} D t_w**
= 0.58 x 36 x 29.75 x 0.580
= 360.3 kips

GENERAL LOAD-RATING EQUATION

A 6.4.2

$$\mathbf{RF} = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$

Eq 6.4.2.1-1

EVALUATION FACTORS (for STRENGTH Limit States)

a) Resistance Factor φ

LRFD A 6.5.4.2

φ = 1.0 for flexure and shear

b) Condition Factor φ_c A 6.4.2.3

$\varphi_c = 1.0$

No member condition information available. NBI Item 59 = 7.

c) System Factor φ_s A 6.4.2.4

$\varphi_s = 1.0$ Mulit-girder bridge (for flexure and shear)

1 <u>DESIGN LOAD RATING</u> A 6.4.3

A) **STRENGTH I Limit State** A 6.6.4.1

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

a) **INVENTORY Level**

LOAD	LOAD FACTOR
DC	1.25
LL	1.75

Table 6.4.2.2-1

$$\text{Flexure: } RF = \frac{(1.0)(1.0)(1.0)(2877.8) - (1.25)(439 + 129)}{(1.75)(952.6)}$$

$$= 1.30$$

$$\text{Shear: } RF = \frac{(1.0)(1.0)(1.0)(360.3) - (1.25)(27 + 8)}{(1.75)(78.9)}$$

$$= 2.29$$

b) **OPERATING Level**

LOAD	LOAD FACTOR
DC	1.25
LL	1.35

Table 6.4.2.2-1

$$\text{Flexure: } RF = 1.30 \times \frac{1.75}{1.35}$$

$$= 1.69$$

$$\text{Shear: } RF = 2.29 \times \frac{1.75}{1.35}$$

$$= 2.97$$

Compare to HS20 Ratings – Load Factor Method (see 1994 AASHTO MCE)

INVENTORY RF $= \dfrac{(1.0)(2877.8)-1.3(439+129)}{2.17(751)}$

$= 1.31$ (flexure)

Operating RF $= 1.31 \times \dfrac{2.17}{1.3}$

$= 2.19$(flexure)

B) **SERVICE II Limit State** A 6.6.4.1

$$RF = \frac{f_R - (\gamma_D)(f_D)}{(\gamma_L)(f_{LL+IM})}$$

a) INVENTORY Level

Allowable Flange Stress $f_R = 0.95\, R_b\, R_h\, F_{yf}$ LRFD A 6.10.5.2

Checking the tension flange

$R_b = 1.0$ for tension flanges

$R_h = 1.0$ for non-hybrid sections LRFD A 6.10.4.3.2b

$f_R = 0.95 \times 1.0 \times 1.0 \times 36$ LRFD A 6.10.4.3.1a

$= 34.2$ KSI

$f_D = f_{DC1} + f_{DC2}$

$$= \frac{439 \times 12}{563.7} + \frac{129 \times 12}{725}$$

$= 9.35 + 2.14 = 11.48$ KSI

$$f_{LL+IM} = \frac{952.6 \times 12}{793} = 14.42 \text{ KSI}$$

$\gamma_L = 1.30$ $\gamma_D = 1.0$ Table 6.4.2.2-1

$$\mathbf{RF} = \frac{34.2 - (1.0)(11.48)}{(1.3)(14.42)}$$

$= \mathbf{1.21}$

b) OPERATING Level

γ_L = 1.0 γ_D = 1.0 Table 6.4.2.2-1

$$RF = \frac{34.2-(1.0)(11.48)}{(1.0)(14.42)}$$

$= 1.58$

Compare To HS20 Serviceability Ratings
(Load Factor Method)

$$f_{LL+IM} = \frac{751 \times 12}{793} = 11.44 \text{ KSI}$$

Inventory Level RF $= \frac{34.2-11.48}{(1.67)11.44}$

$= 1.19$

Operating Level RF $= 1.19 \times 1.67$

$= 1.99$

3. **PERMIT LOAD RATING** A 6.6.4.2

Permit Type: Special (Single-Trip, Escorted)
Permit Weight: 220 KIP
The permit vehicle is shown on page
ADTT (one direction): 1000

From Live Load Analysis by Computer Program:

Undistributed Maximum M_{LL} = 2127.9 KIP FT
Undistributed Maximum V_{LL} = 143.5 KIP

1) **STRENGTH II Limit State** A 6.6.4.2.1

γ_L **= 1.15 (Single-Trip, Escorted)** Table 6.4.5.4.2-1

Use One Lane Distribution Factor and divide out the 1.2 multiple presence factor. A 6.4.5.4.2.2

g_{m1} = 0.46/1.2 = 0.383
g_{v1} = 0.653/1.2 = 0.544
IM = 20% (no speed control, minor surface deviations) A 6.4.5.5

Distributed Live Load Effects

M_{LL+IM} = (2127.9) (0.383) (1.20)
= 978.0 KIP FT
V_{LL+IM} = (143.5) (0.544) (1.20)

$$= 93.7 \text{ KIP}$$

Flexure: $RF = \dfrac{(1.0)(1.0)(1.0)(2877.8) - (1.25)(439 + 129)}{(1.15)(978.0)}$

$$= 1.93 \qquad > 1.0 \quad \text{OK}$$

Shear: $RF = \dfrac{(1.0)(1.0)(1.0)(360.3) - (1.25)(27 + 8)}{(1.15)(93.7)}$

$$= 2.94 \qquad > 1.0 \quad \text{OK}$$

2) **SERVICE II Limit State** **(Optional)** A 6.6.4.2.2

$$RF = \frac{f_R - f_D}{\gamma_L (f_{LL+IM})}$$

IM = 20% (no speed control, minor surface deviations)

$\gamma_L = 1.0$ $\gamma_D = 1.0$ Table 6.4.2.2-1

$f_R = 34.2 \text{ KSI}$

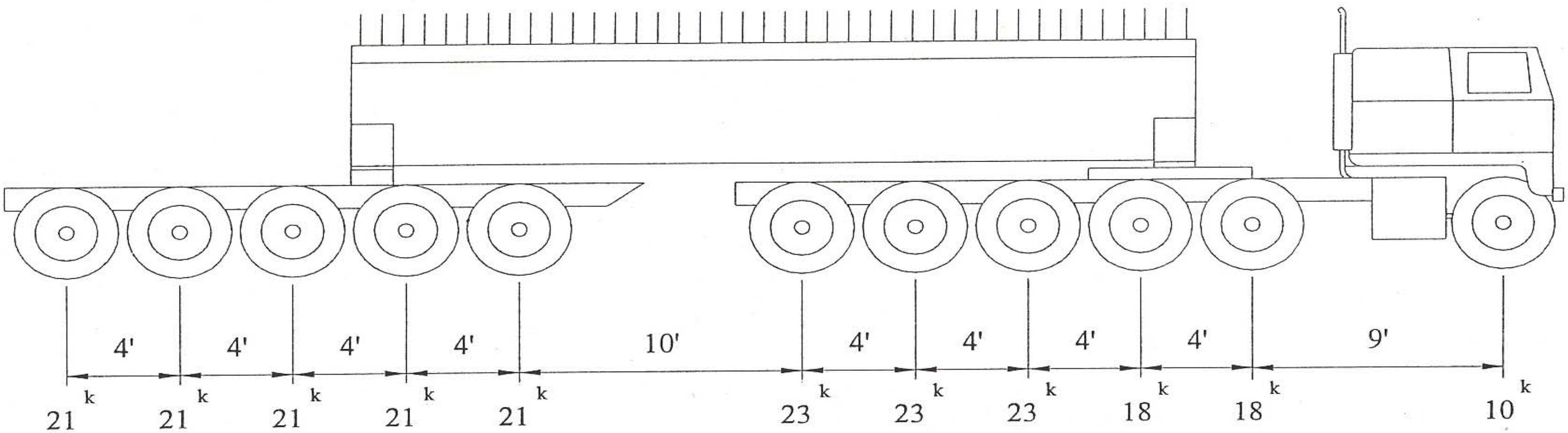
$f_D = 11.48 \text{ KSI}$

$g_m = 0.383$ (m = 1.2 has been divided out) C 6.6.4.2.2

$M_{LL+IM} = (2127.9)\ (0.383)\ (1.2) = 978.0 \text{ KIP FT} = 11736 \text{ KIP IN}$

$$f_{LL+IM} = \frac{M_{LL+IM}}{S_b} = \frac{11736}{793} = 14.8 \text{ KSI}$$

$$RF = \frac{34.2 - (1.0)(11.48)}{(1.0)(14.8)} = 1.53$$



PERMIT TRUCK

LOADING CONFIGURATION

SUMMARY OF RATING FACTORS

EXAMPLE A1
INTERIOR GIRDER

Limit State		Design Load Rating		Legal Load Rating			Permit Load Rating
		Inventory	Operating	T3	T3S2	T3-3	
STRENGTH I	Flexure	1.30	1.69	2.73	2.55	2.76	
	Shear	2.29	2.97				
STRENGTH II	Flexure						1.93
	Shear						2.94
SERVICE II		1.21	1.58	2.33	2.17	2.35	1.53
FATIGUE		0.66					