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NCHRP REPORT 700

**A Comparison of AASHTO Bridge
Load Rating Methods**

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Mark Mlynarski, Project Manager, Michael Baker Jr., Inc., and Wagdy G. Wassef, Ph.D., Modjeski and Masters, Inc. were the co-principal investigators. The other authors of this report were Andrzej S. Nowak, Ph.D., Professor of Engineering, University of Nebraska, and Vanessa L. Storlie, Modjeski and Masters, Inc.

The work was performed under the general supervision of Mr. Mlynarski and Dr. Wassef. The software development and data extraction work at Michael Baker Jr., Inc. was performed under the supervision of Mr. Mlynarski with the assistance of Mehrdad Ordoobadi, Technical Manager-Bridge Software, and Scott Peterson, Programmer/Database Specialist. The work at Modjeski and Masters was performed under the supervision of Dr. Wassef with the assistance of Vanessa Storlie in all tasks of the project and Mr. Chad Clancy in the solicitation and review of state legal and permit vehicles. The development of the statistical parameters used in the calculation of the reliability index factors was performed at the University of Nebraska under the supervision of Dr. Nowak with the assistance of Piotr Paczkowski and Przemyslaw Rakoczy.

Special thanks to Brian Goodrich of BridgeTech, Inc. and the Wyoming Transportation Department for modifications made to the BRASS software to allow the research team to extract additional data in NCHRP Process 12-50 format needed for reviewing the permit live loading results.

FOREWORD

By **Waseem Dekelbab**

Staff Officer

Transportation Research Board

This report documents an analysis of 1,500 bridges that represent various material types and configurations using AASHTOWare™ Virtis® to compare the load factor rating to load and resistance factor rating for both moment and shear induced by design vehicles, AASHTO legal loads, and eight additional permit/legal vehicles. The report includes proposed revisions to the *AASHTO Manual for Bridge Evaluation* based on a review of the analysis results. The material in this report will be of immediate interest to bridge engineers.

The *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*, 1st Edition and 2005 Interim, was developed under NCHRP Project 12-46. Before the Guide Manual was endorsed by the AASHTO Highway Subcommittee on Bridges and Structures (HSCOBs), some additional research was requested to explain differences between the new manual and the established load factor rating (LFR) requirements. NCHRP 20-07/Task 122 provided explicit comparisons between ratings produced by the LRFR methods of the Guide Manual and the load factor ratings from the latest edition of the *AASHTO Manual for Condition Evaluation of Bridges*. Nevertheless, because the Task 122 scope was limited to flexural ratings and the number of structures rated was small, HSCOBs wanted more bridges rated and LRFR/LFR comparisons made for moment and shear induced by a variety of loads.

The choice of load rating method may affect the transportation of goods and services over the nation's highways by restricting routes that were previously unrestricted. Additional comparisons of LRFR and LFR ratings were needed to (1) develop refinements to the load rating process that maintain an acceptable level of bridge reliability without unnecessary restrictions on commerce and (2) explain changes in truck weight restrictions to the public.

The research was performed under NCHRP Project 12-78 by Michael Baker Jr., Inc, with the assistance of Modjeski and Masters, Inc., and the University of Nebraska. The objectives of NCHRP Project 12-78 were to propose refinements to the LRFR methods in the *AASHTO Manual for Bridge Evaluation* and to explain potential changes in truck weight restrictions of existing bridges.

A number of deliverables are provided as appendices. These are not published herein but are available on the TRB website at <http://www.trb.org/Main/Blurbs/165576.aspx>. The appendices are:

- Appendix A—Final Bridge/Girder List
- Appendix B—Simple Span Steel Girder Bridges
- Appendix C—Simple Span Prestressed I-Girder Bridges
- Appendix D—Simple Span Prestressed Box Girder Bridges

- Appendix E—Simple Span Reinforced Concrete T-Beam Bridges
- Appendix F—Simple Span Reinforced Concrete Slab Bridges
- Appendix G—Steel I-Girder Continuous Span Bridges
- Appendix H—Continuous Reinforced Concrete Slab Bridges
- Appendix I—Continuous Prestressed I-Girder Bridges
- Appendix J—Calculated Reliability Indices
- Appendix K—Effect of Permit Type and ADTT on LRFR Ratings
- Appendix L—Effect of LRFR Rating on Operating Rating
- Appendix M—Effect of LRFR on Rating Using Proposed Load Factors
- Appendix N—MBE Examples
- Appendix O—Review of the NBI/Virtis Databases
- Appendix P—Final Survey
- Appendix Q—Changes required for NCHRP 12-50/Software Documentation
- Appendix R—Format of CSV output produced by RIO software

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Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the Web at www.trb.org) retains the color versions.

S U M M A R Y

A Comparison of AASHTO Bridge Load Rating Methods

This report highlights the work accomplished under the AASHTO sponsored project, “Evaluation of Load Rating by Load and Resistance Factor Rating.” Administered by NCHRP as Project 12-78, the research has provided proposals for changes to the AASHTO *Manual for Bridge Evaluation* through the extensive data analysis of 1,500 bridges of varying material types and structure configurations. The bridges were analyzed using the AASHTO-Ware™ Virtis® software. To select the 1,500 bridge sample and the live loadings considered in the study, the following tasks were performed:

- Reviewed the FHWA National Bridge Inventory (NBI) database to determine an appropriate cross section of material types and bridge configurations. The inventory was broken down into several categories including year built, bridge type, material type, and maximum span length.
- Solicited AASHTOWare™ data files from states during the survey period. In all, states provided over 18,000 bridge data sets for use on the project. From this set, the 1,500 final bridge domain was selected.
- Developed utility software to breakdown the 18,000 bridge database and select bridges that closely matched the cross section determined from the NBI data.
- Determined a set of live load vehicles to be used in the analysis of the 1,500 bridges based on legal and permit vehicle configurations submitted by the states surveyed. From the over 300 vehicles submitted, eight vehicles were selected in the final phase in addition to the AASHTO legal loads (Type 3, Type 3S2, and Type 3-3) and the Load Factor Design (LFD) and Load and Resistance Factor Design (LRFD) vehicles (HS-20 and HL-93, respectively).

Once the bridge domain and vehicles were selected, the AASHTOWare™ Virtis® was used to analyze the bridges. Parameters for using the software were:

- Version 6.1.0 of the AASHTOWare™ Virtis® software with the Wyoming Transportation Department BRASS GIRDER software engine for both load factor rating (LFR) and load and resistance factor rating (LRFR).
- Both LFR and LRFR ratings were performed on all bridges and the results for the critical moment and shear ratings were reviewed.
- The software was run in both LFR and LRFR for the eight permit/legal loads, the three AASHTO legal loads, and the design loads.
- In all, 3,043 girders from 1,500 bridges were analyzed using 12 vehicles for both LFR and LRFR, constituting more than 73,000 separate analysis runs.
- The results of these analysis runs were organized by critical shear and moment ratings and relevant data was extracted using the process developed in NCHRP Project 12-50 (NCHRP Report 485).

The bridge types analyzed using Virtis® for the basis of this research are:

- Simple Span
 - Steel Rolled Beam
 - Steel Built-Up Beam
 - Steel Plate Girder
 - Prestressed I-beam
 - Prestressed Box Beam
 - Reinforced Concrete T-Beam
 - Reinforced Concrete Slabs
- Multiple Spans
 - Steel Rolled Beam
 - Steel Plate Girder
 - Reinforced Concrete Slab
 - Prestressed I-beam

Exporting the critical rating and corresponding results to an external spreadsheet permitted the analysis of the data for trends. The bulk of this research discusses the results of that analysis and the changes suggested to the AASHTO *Manual for Bridge Evaluation*.

CHAPTER 1

Background

The *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*, 1st Edition and 2005 Interim was developed under NCHRP Project 12-46. Before the Guide Manual was endorsed by the AASHTO Highway Subcommittee on Bridges and Structures (HSCOBs) to become a full manual, some additional research was requested to explain differences between the new manual and the established load factor rating (LFR) requirements. NCHRP 20-07/Task 122 provided explicit comparisons between ratings produced by the LRFR methods of the Guide Manual and the LFR from the latest edition of the *AASHTO Manual for Condition Evaluation of Bridges*. Nevertheless, because the Task 122 scope was limited to flexural ratings and the number of structures rated was small, HSCOBs wanted more bridges rated and LRFR/LFR comparisons made for moment and shear induced by a variety of loads. The choice of load rating method may affect the transportation of goods and services over the nation's highways by restricting routes that were previously unrestricted. Additional comparisons of LRFR and LFR ratings are needed to (1) develop refinements to the load rating process that maintain an acceptable level of bridge reliability without unnecessary restrictions on commerce and (2) explain changes in truck weight restrictions to the public.

The objectives of this project are to recommend refinements to the LRFR methods in the *AASHTO Manual for Bridge Evaluation* and to explain changes in truck weight restrictions. A comprehensive database of rating comparisons was established to identify differences in ratings and to develop the proposed refinements.

To accomplish this objective, a BRIDGEWare™ database of 1,500 bridges was compiled for the purposes of calculating the LFR and LRFR ratings for both moment and shear using 12 different vehicles: one design vehicle for each rating method (HS-20, HL-93), three AASHTO legal loads (Type 3, Type 3S2,

and Type 3-3), and eight permit/legal loads selected by region from over 300 submitted by the states.

The method of selecting the 1,500 bridge database, the eight additional vehicles, and the analysis/review of the results are described in the following sections.

1.1 Surveying/Soliciting Data

To determine the overall impressions of the states regarding LRFR and also to solicit data for the building of the bridge sample domain, a survey was prepared and reviewed by the project panel. The survey was then sent via e-mail to all 50 states, the District of Columbia, and Puerto Rico. In several cases at the option of the state agency, follow-up interviews regarding the surveys were conducted.

Completed surveys were received from 33 state agencies and one university. A sample of the survey sent to the states is provided in Appendix P.

Part of the survey of the individual states included a solicitation of electronic data in the form of AASHTOWare™ Virtis® bridge data. During this solicitation, the research team collected data for over 18,000 bridge structures. This data came from a wide geographic range and would be the pool from which the final 1,500 bridge domain would be extracted. The breakdown of bridge datasets by state is shown in Table 1.

The cooperation of the participating AASHTOWare states in donating these data sets was critical to the successful completion of this project. A sample this large enabled the research team to selectively choose data for the final 1,500 bridge set that most closely resembled the cross section of the NBI database.

In addition to the bridge data solicitation, the survey participants were asked to submit a list of vehicles used by their respective agencies for possible inclusion in the study. More than 300 vehicles were submitted from the agencies responding to the survey.

Table 1. AASHTOWare Datasets collected for this research.

State/Agency	Number of AASHTOWare Data Sets
Alabama	3,139
Illinois	3,232
Michigan	378
Missouri	4,644
New York	5,412
Oklahoma	44
South Dakota	1,135
Tennessee	53
Total	18,037

CHAPTER 2

Research Approach

To successfully complete the objectives of this project, a 1,500 bridge BRIDGEWare database needed to be established. Ideally the database would reflect the cross section of bridges found in the FHWA NBI database and would include as many material types and bridge configurations as possible. An early limiting factor in the development of the 1,500 bridge domain was the types of bridges that Virtis could analyze. Ultimately, the final breakdown of the bridge types used for this research were:

- Simple Span
 - Steel Rolled Beam
 - Steel Built-Up Beam
 - Steel Plate Girder
 - Prestressed I-beam
 - Prestressed Box Beam
 - Reinforced Concrete T-Beam
 - Reinforced Concrete Slabs
- Multiple Spans
 - Steel Rolled Beam
 - Steel Plate Girder
 - Reinforced Concrete Slab
 - Prestressed I-beam

In addition to this information, the bridges were categorized by year built, maximum span length, and girder spacing to obtain the best variety of bridges that also shadowed the cross section obtained from the NBI database.

Also, vehicles submitted by the survey participants were analyzed to determine the set to be used for the final rating/analysis.

The following sections detail the methods used for selecting the bridge domain and the live load vehicles used to analyze the domain.

2.1 Selection of Bridges

In order to select an appropriate number of bridge types to include in the final 1,500 bridge domain, a review of the percentage of bridges in the NBI database was needed. The NBI

2008 data was downloaded from the FHWA website and was queried using a software tool developed for this project. For a description of the NBI querying tool and its dual usage to query the collected Virtis data, see Appendix Q. The software enabled the project team to query not only the NBI database but the collected BRIDGEWare data as well.

During the survey portion of this research, the project team also solicited BRIDGEWare datasets. The breakdown of the 18,038 data sets by state is shown in Table 1.

The preparation of the initial bridge domain provided was based on a review of the FHWA NBI database 2008 and a subsequent comparison with the Virtis data collected from the states.

2.1.1 NBI/Virtis Data Review/Comparison with Virtis Data

In order to provide a 1,500 bridge domain that would appropriately model the cross section of the current national bridge inventory, some means was needed to quickly analyze not only the NBI data but also the solicited Virtis data. Each of the 18,000 Virtis bridges submitted contain a large amount of information and was thus not easily analyzed without drilling down to some basic information.

To breakdown the Virtis data into a format that allowed for an easier comparison with the NBI data, the project team created a query using a commercial data-analysis tool to extract pertinent information from the 18,000 bridge Virtis database. Once a simplified form of the data was extracted, the data-analysis software developed for this project was used to help select a sample bridge domain that reflected the cross section of the NBI data.

Some sample figures of the data analysis are shown in Figures 1 and 2. A more detailed breakdown of the data analyzed along with a final set of comparative tables of the various structure types reviewed for this project are provided in Appendix O. A description of the software used to analyze the results for this research project is provided in Appendix Q.

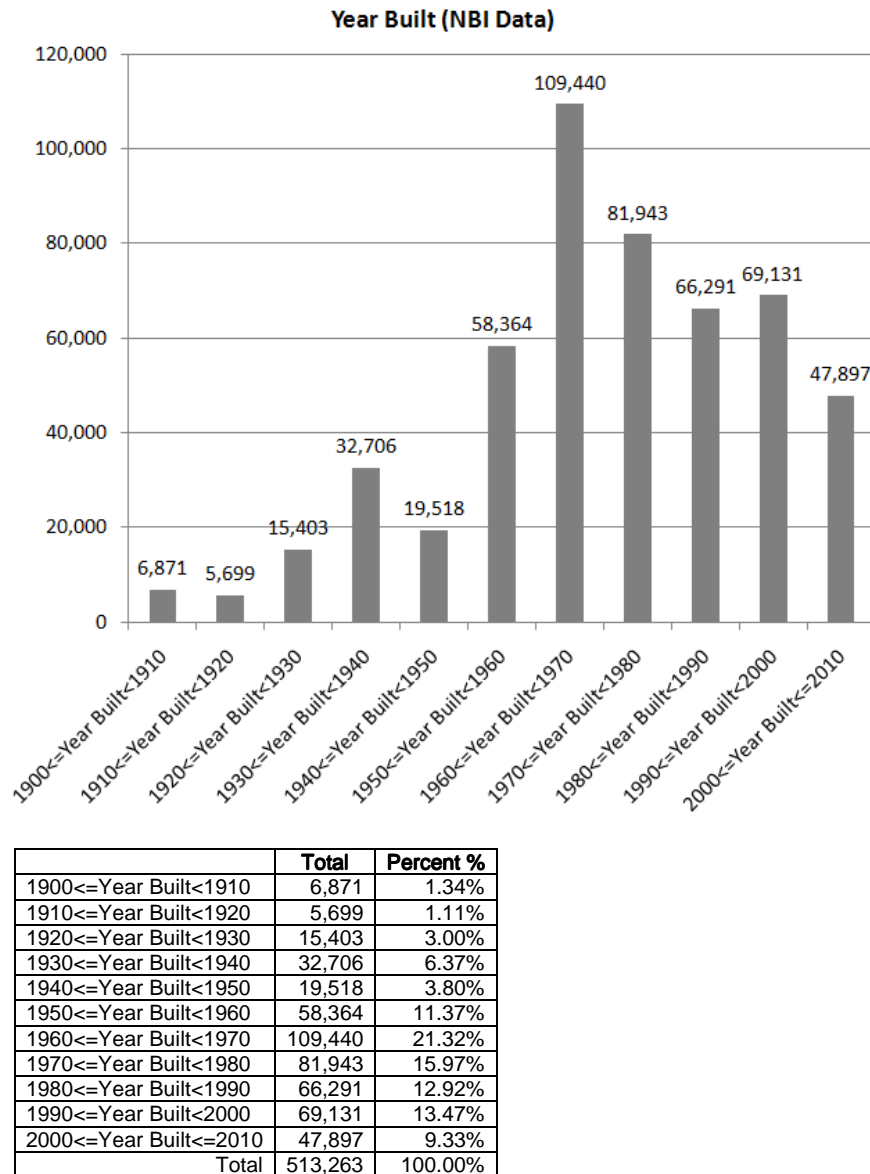


Figure 1. Breakdown of the NBI data by year built.

Using this software, an initial sample bridge domain was developed based on these comparisons. The sample was intentionally left larger than 1,500 bridges needed as additional winnowing of the data would take place in the second phase of the research. The entire process for the initial selection of the bridge domain for this research is illustrated in Figure 3.

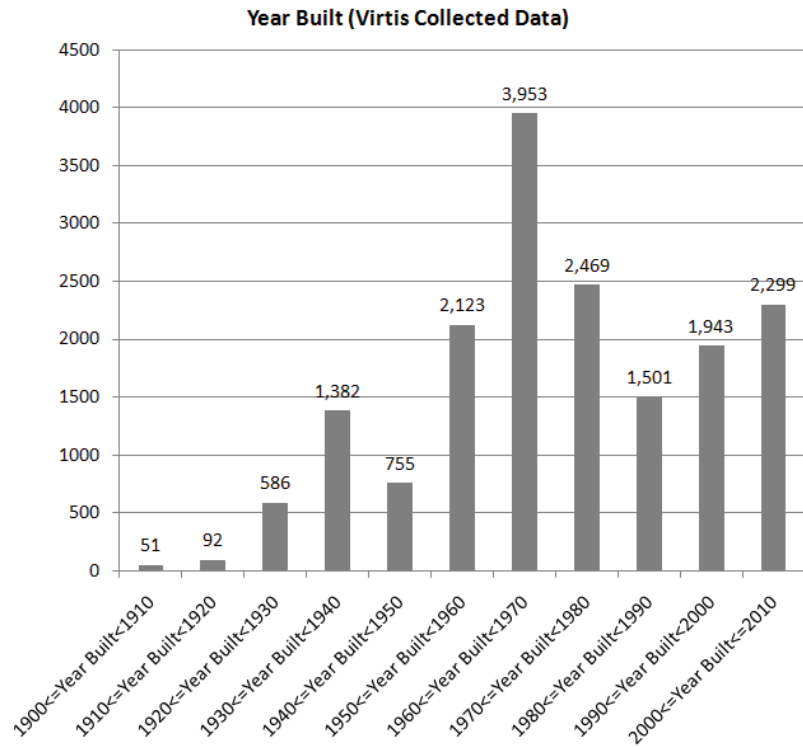
2.1.2 Refinement of the Bridge Selections—Final Bridge Domain

To refine the bridge selection and to obtain the final 1,500 bridge domain, the software tools that were developed to interact with the NBI data and the state provided Virtis database

were combined to refine the selection/modification process. An illustrated flow chart of the entire process from bridge selection to production of results used for the review is shown in Figure 4. The steps of the procedure are in the figure.

Step 1. Using the software developed for this project, determine the cross section by the year built of a particular bridge type (e.g., steel plate girders) from the NBI data. Using the same software with the solicited Virtis data, build a list of available bridge types, and move that list of data to a spreadsheet where it can be sorted and manipulated.

Step 2. After building a final list of bridges for a particular bridge type, organize them in the spreadsheet to meet the criteria as closely as possible for the year breakdown for



	Total	Percent %
1900<=Year Built<1910	51	0.30%
1910<=Year Built<1920	92	0.54%
1920<=Year Built<1930	586	3.42%
1930<=Year Built<1940	1,382	8.06%
1940<=Year Built<1950	755	4.40%
1950<=Year Built<1960	2,123	12.38%
1960<=Year Built<1970	3,953	23.04%
1970<=Year Built<1980	2,469	14.39%
1980<=Year Built<1990	1,501	8.75%
1990<=Year Built<2000	1,943	11.33%
2000<=Year Built<=2010	2,299	13.40%
Total	17,154*	100.00%

*Note: Date not provided for all Virtis bridge IDs

Figure 2. Breakdown of the collected Virtis database by year built.

that bridge type and span length. This is done by creating several tabs within the spreadsheet to hold a pool of available bridges by span length and moving the selected bridges to a “final” bridge tab. Once the final tab has been established, data are moved to a CSV file for use by the XML converter software.

Step 3. Run the XML converter software to find the list of bridges built in Step 2. All 18,000+ bridges that have been provided for this project have been exported from Virtis in their raw XML form just as they were donated. The software locates and modifies the XML file by changing the bridge ID and bridge description before copying the file to a holding area. Once the files have been copied, they can be imported by using the batch XML import option provided in Virtis.

Step 4. Once in Virtis the bridges are modified to remove any extraneous information. If multiple simple spans exist, only the span of interest and the associated alternative is maintained. Afterwards, the entire list of bridges is automatically modified for missing LRFD input information such as effective slab width and P/S strand development length. This is done by an external process that was developed for this research project that reads the Virtis database directly.

Step 5. Once the bridges are modified, all are run in both LFR and LRFR and issues addressed for each bridge. While they are running, bridge rating and analysis of structural systems (BRASS) produces the Process 12-50 results for each member alternative. The implementation of Process 12-50 into Virtis for the purposes of this research project is provided in Appendix Q of this report. A description of

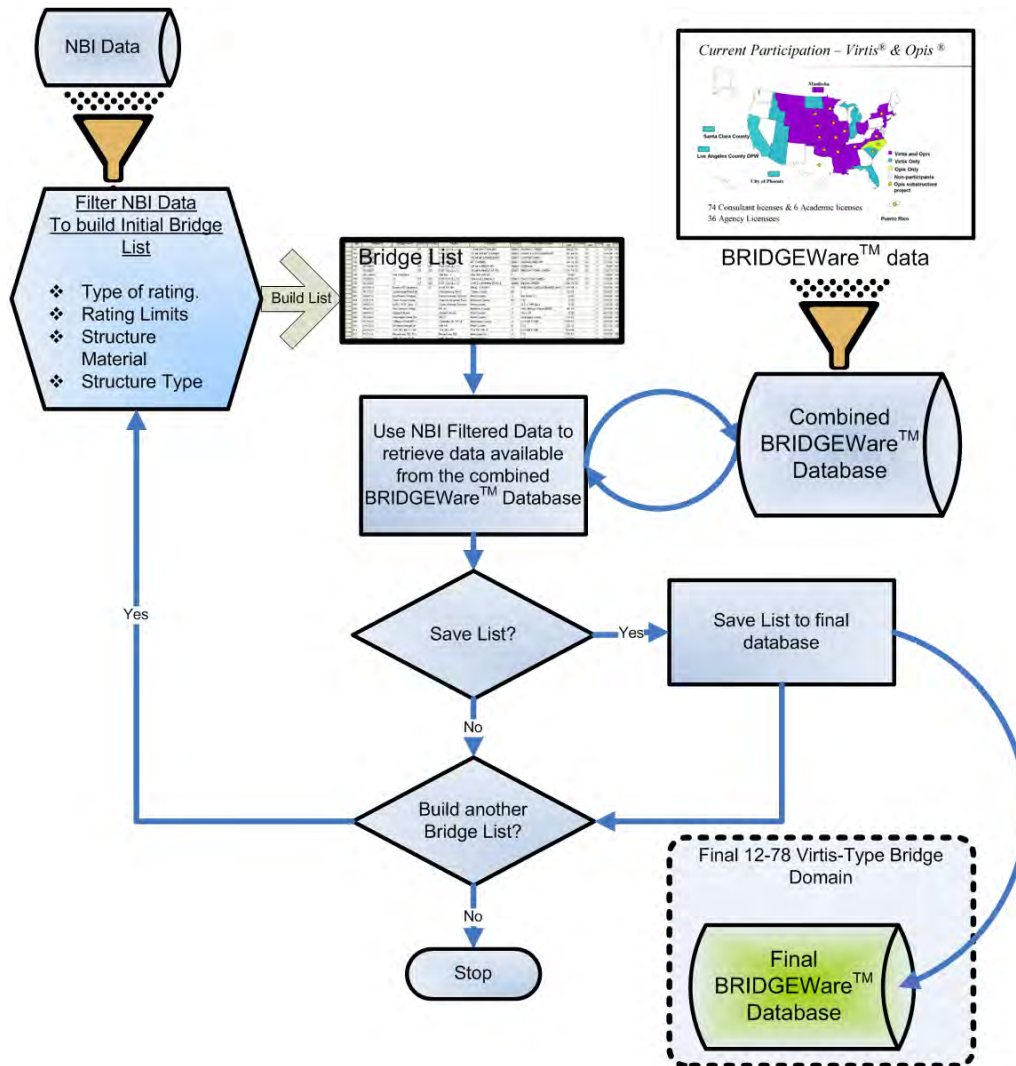


Figure 3. Process for selecting initial 12-78 bridge database.

Process 12-50 is available in *NCHRP Report 485: Bridge Software—Validation Guidelines and Examples*. A process was added internally to Virtis for this project to move the Process 12-50 files produced by BRASS to a central location and to modify the internal values for the “SubdomainID” to uniquely identify the records produced with the specific girder in Virtis. Once in the holding area, the CSV Process 12-50 files can be combined into a single file for importing into a relational database.

Step 6. Because BRASS only produces output type results in Process 12-50 format, an additional process was developed to extract information directly from the Virtis database and provide the information in Process 12-50 format. This information (e.g., girder spacing, skew angle, wearing surface thickness, etc.) is related to user input and is documented in detail in Appendix Q.

Step 7. Once all of the Process 12-50 is gathered, it is imported into a Microsoft Access Database. The database is then

queried using a simple software application developed for this project [Reliability Index Output (RIO) generator] into a single line of comma delimited text output for each girder, of each bridge. A single line is produced for the following combinations

- Moment Rating-Inventory
- Moment Rating-Operating
- Shear Rating-Inventory
- Shear Rating-Operating

A sample screen shot of the RIO software is shown in Figure 5. The format of the CSV file produced by the RIO software is provided in Appendix R.

During the process of the selection of the 1,500 bridge domain a format was developed to identify the files in Virtis. The format is shown in Table 2. A more detailed description of the file naming convention is provided in Appendix Q.

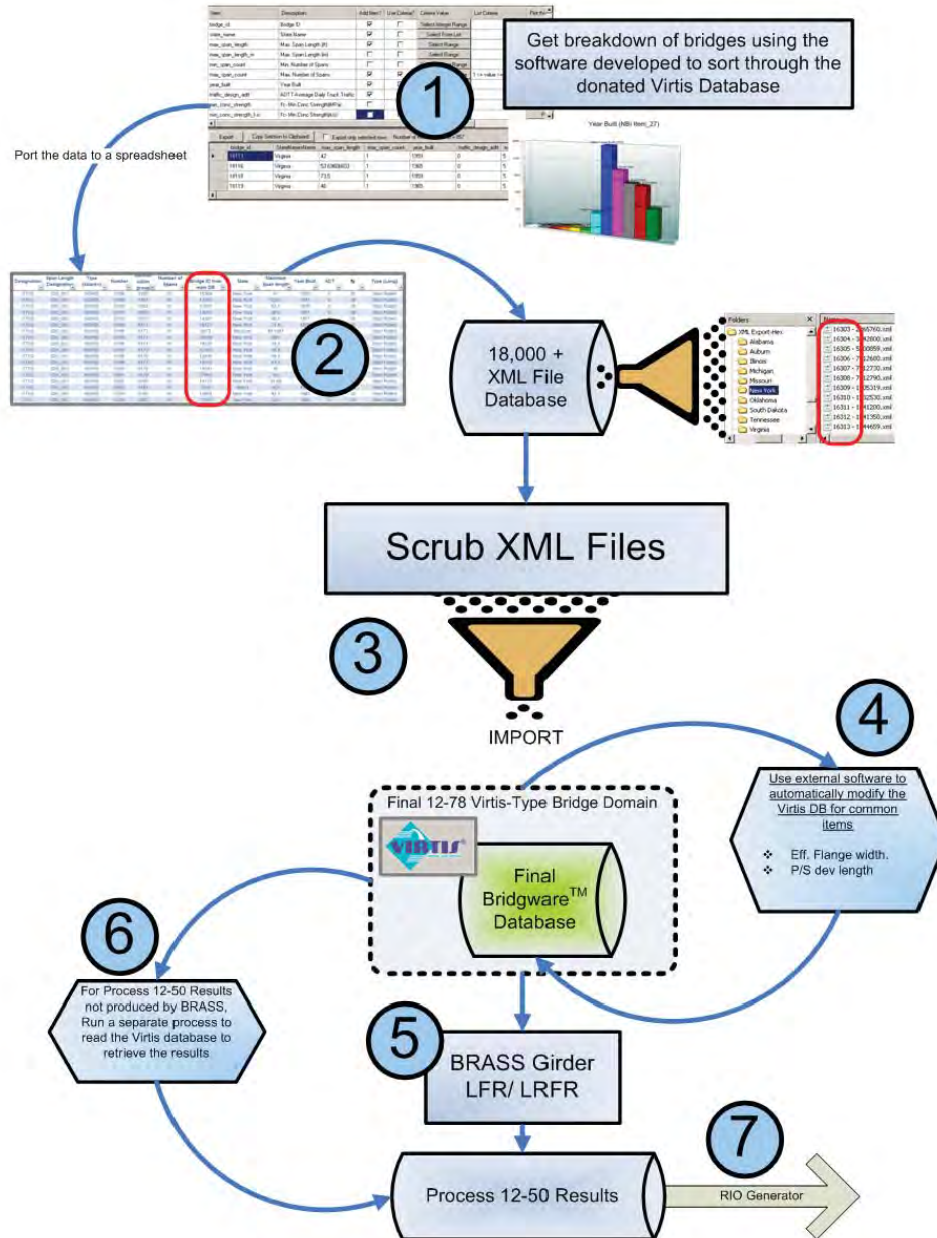


Figure 4. Flowchart of steps in the bridge selection process.

2.2 Development of Live Load Vehicle List

2.2.1 Introduction

The vehicle loadings obtained by the NCHRP Project 12-78 Survey were used to investigate the resulting shears and moments in comparison to the HL-93 live load. The vehicles were divided into geographical regions, which allow an envelope of actions to be obtained. This process allowed for the elimination of vehicles that did not follow the observed trends for that region. The objective was to develop several “regional live loads” that will be used for the additional tasks of the NCHRP Project 12-78.

2.2.1.1 Live Load Comparison Utility

The assorted vehicle loadings were analyzed using BM.exe, a utility developed internally at Modjeski and Masters, Inc. This utility computes live load moments and shears for simple span beams as well as continuous beams consisting of two equal spans. The program generates shears and moments for a range of spans as input by the user. The shears and moments generated are:

- Mss—Simple span moment;
- +M 0.4L—positive moment at the four tenths point;
- -M 0.4L—negative moment at the four tenths point;

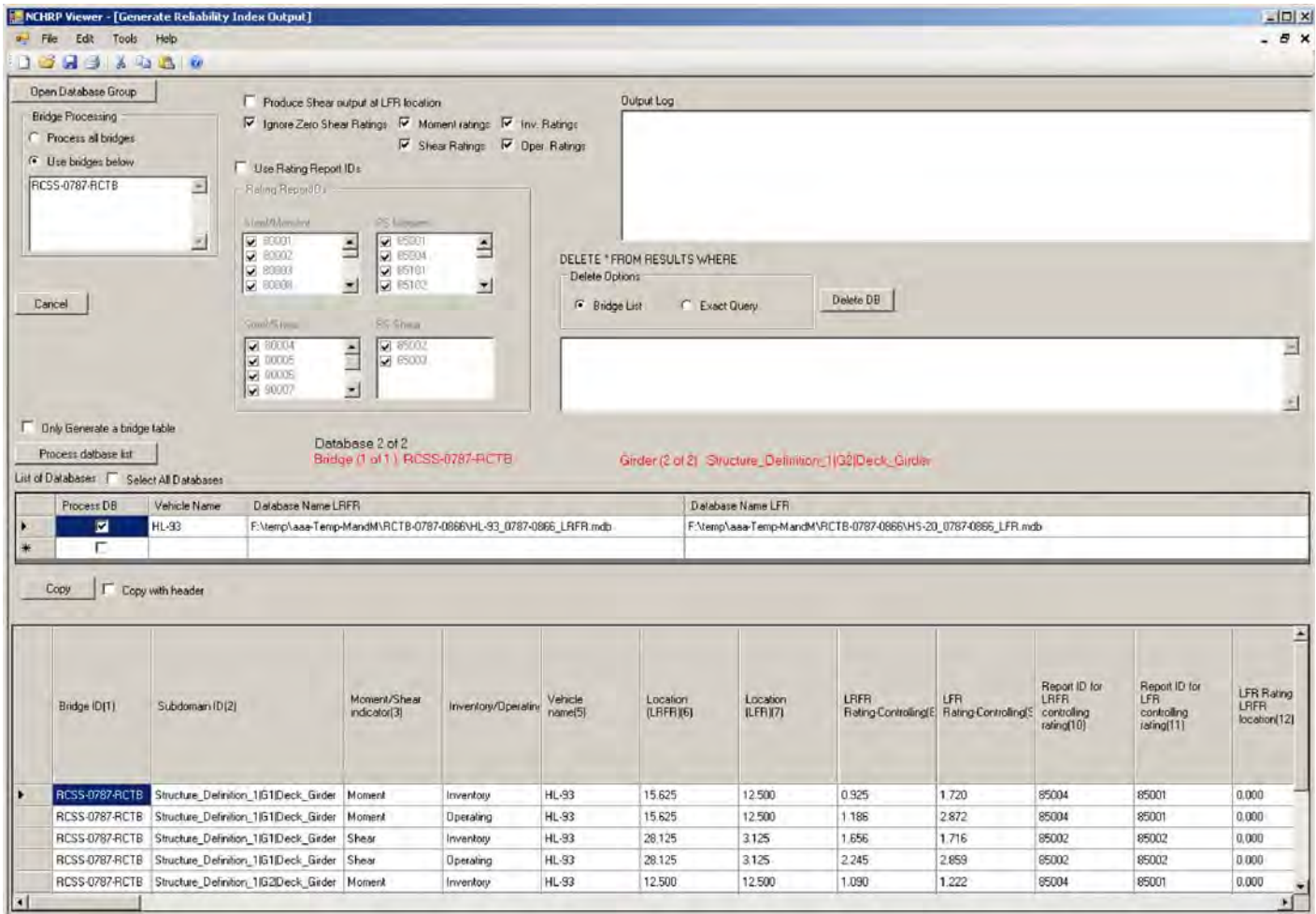


Figure 5. Reliability Index Output (RIO) generator.

Table 2. NCHRP Project 12-78 file naming conventions.

Component	Description
Material-Span Configuration	This component defines the material type and type of span configuration. The first two letters specify the material type: ST – Steel PS – Prestress RC – Reinforced Concrete The second two letters indicate the span configuration and is simply defined as: SS – Simple span MS – multiple spans
Bridge ID	This component defines a unique 4-digit bridge ID within this Material-Span Configuration category.
Structure type	This up to 4 character component defines the type of structure for this bridge. The definitions are as follows: MGBU – Multi-girder built-up member MGRB – Multi-girder rolled beam MGSP – Multi-girder steel plate girder PSIB –P/S I-beam PSBX –girder box beam PSTB – P/S T-beam RCSL – Reinforced concrete slab RCTB – Reinforced concrete T-beam RCIB – reinforced concrete I-beam

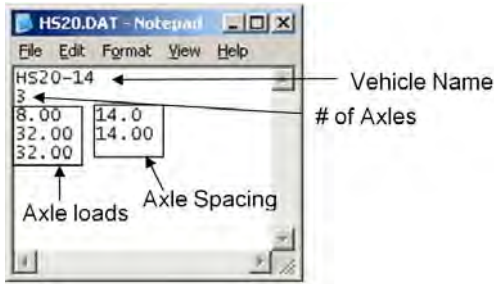


Figure 6. BM.exe Input.

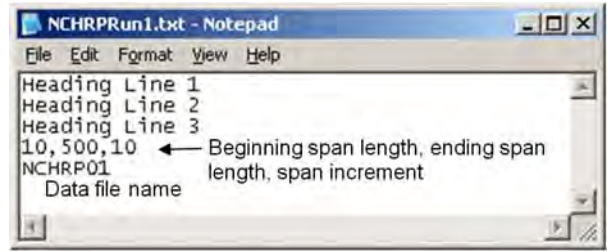


Figure 7. Text file input into BM.exe.

- -M_{supt}—negative moment at the interior support for continuous spans (considers one truck and two “spaced” trucks);
- V_{ab}—shear at exterior supports; and
- V_{ba}—shear at interior support.

The program requires input data files that contain the vehicle name, number of axles, axle loads, and axle spacing. Figure 6 shows the input for an HS20 Truck with the rear axles spaced at 14 ft. A text file indicating the span lengths and increments to be analyzed along with the input data file name is fed into BM.exe (Figure 7). BM.exe returns the shears and moments for these quantities mentioned earlier at each span length for each vehicle in a tab-delimited file that can be imported into Excel.

2.2.1.2 Analysis Results:

The results obtained from the BM.exe program were input into Excel to be analyzed and compared to the HL-93 loading

(including the lane load). Prior to comparing the results to the HL-93 load, the vehicles obtained in the survey were divided into four geographic regions.

Figure 8 shows the geographic regions along with the number of vehicles obtained from each state; the number does not include “standard” vehicles used by the states (H20, HS20, HL-93, etc). The regions were created to allow for a similar number of trucks in each region. Michigan and Oklahoma were not included in a region but are investigated individually due to the disproportionate number of vehicles obtained. Additionally, Michigan is divided into two groups: (1) legal weight trucks used for rating bridges and (2) overload permit trucks. The distribution of trucks per region is shown in Table 3.

After placing each truck into a geographic region, the actions (shears and moments) for each vehicle, including an impact factor of 1.33, were divided by the HL-93 actions to obtain a ratio. The ratios were grouped by region and plotted for each vehicle versus the span length. All graphs created during this analysis are presented in Appendix O.

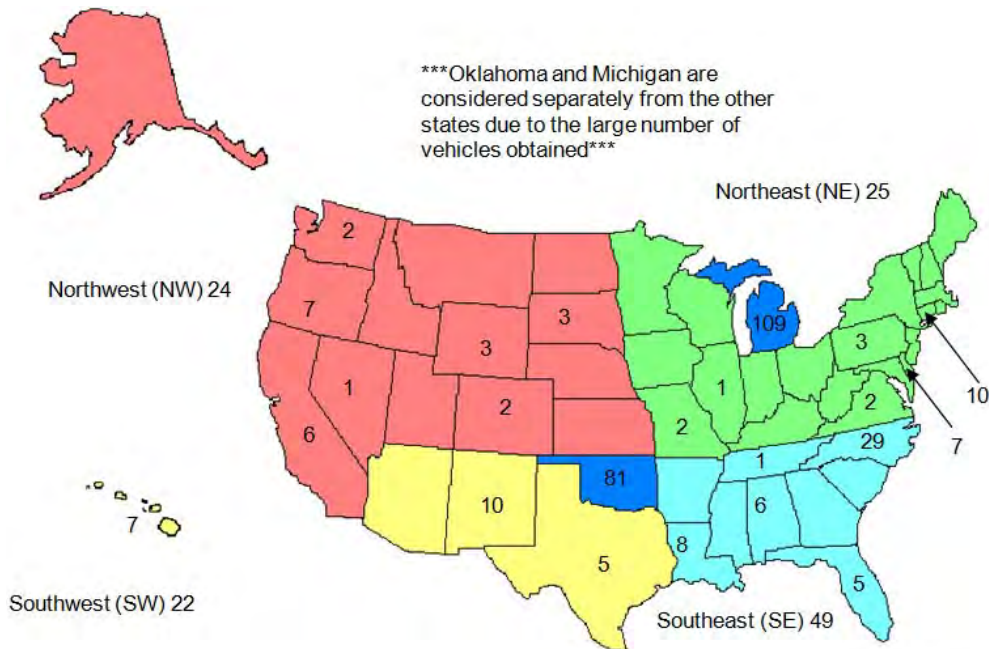


Figure 8. Geographic regions of trucks.

Table 3. Truck distribution.

Region	Number of Trucks:
Northwest (NW)	24
Southwest (SW)	22
Southeast (SE)	49
Northeast (NE)	25
Michigan	109
Oklahoma	81
Total	310

Graphing the ratio versus span length was completed to aid in eliminating vehicles that did not follow the trends observed within each region. An example of this is shown for the southwest region in Figures 9 and 10. Figure 9 shows the results for the southwest region for all vehicles. Figure 10 shows the results after eliminating NM-10, HI-01, and HI-02 (vehicles that did not follow the regional trend). Overall, 14 outlying vehicles were eliminated from the initial set of vehicle loadings obtained by the survey.

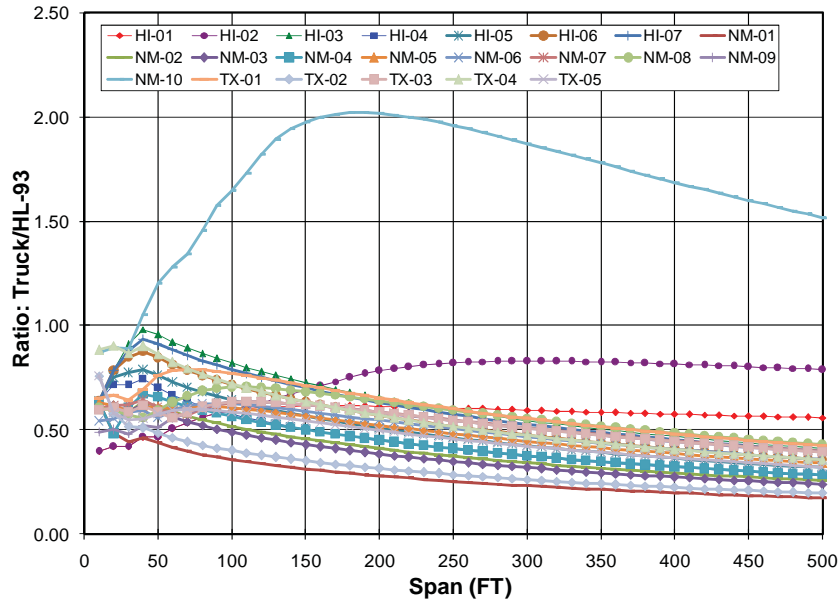


Figure 9. Results from Southwest region.

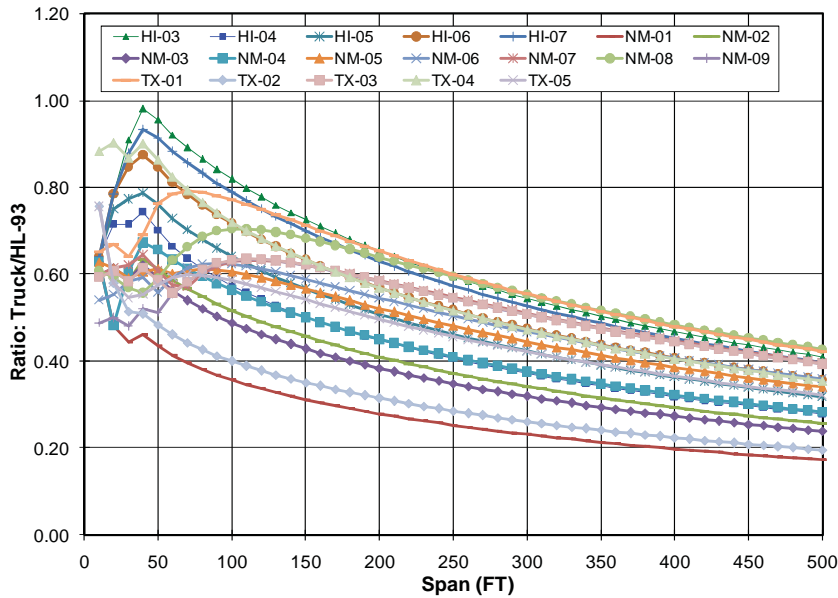


Figure 10. Results from southwest region eliminating NM-10, HI-01, and HI-02.

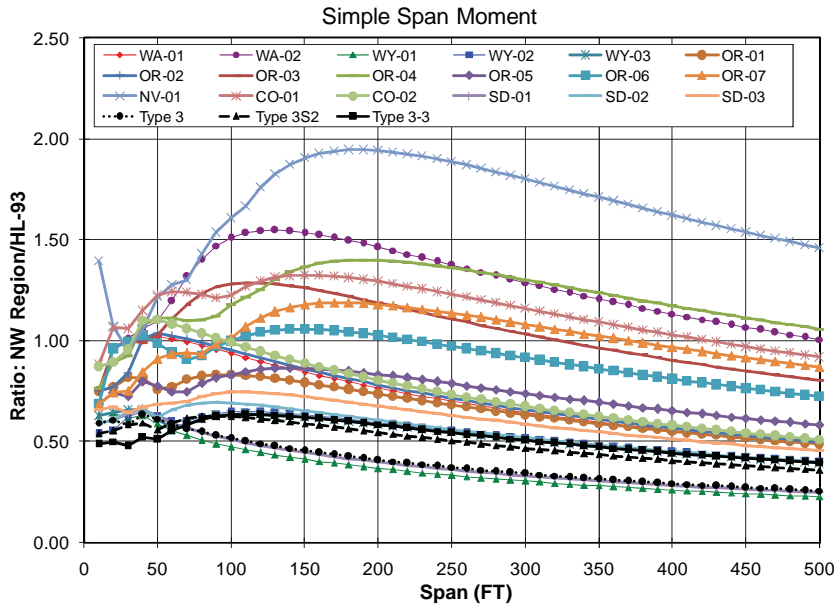


Figure 11. Northwest region live loads including AASHTO legal loads for simple span moment.

2.2.1.3 Comparison to AASHTO Legal Loads

The AASHTO legal loads (Type 3, Type 3S2, and Type 3-3) were compared to the HL-93 loading. The results of this investigation indicate that the actions caused by the legal loads are significantly lower than those caused by the HL-93 loading. Figure 11 shows the graph of the results for the northwest region and AASHTO legal loads. The figure shows that rating vehicles used by states in the northwest region cause greater actions than the AASHTO legal loads for rating bridges. The

trends in the simple span moment are consistent with those seen in the other actions that were examined.

Looking at the southwest region (Figure 12), the AASHTO legal loads tend to encompass a large portion of the state loadings used for rating bridges. The state loadings generally result in greater actions (moments and shears) for span lengths from 10 ft to 150 ft, but for longer spans the AASHTO legal loads provide actions that are very similar to the state vehicles. The AASHTO legal loads envelope approximately 50% of the state vehicles in the southwest, southeast, and

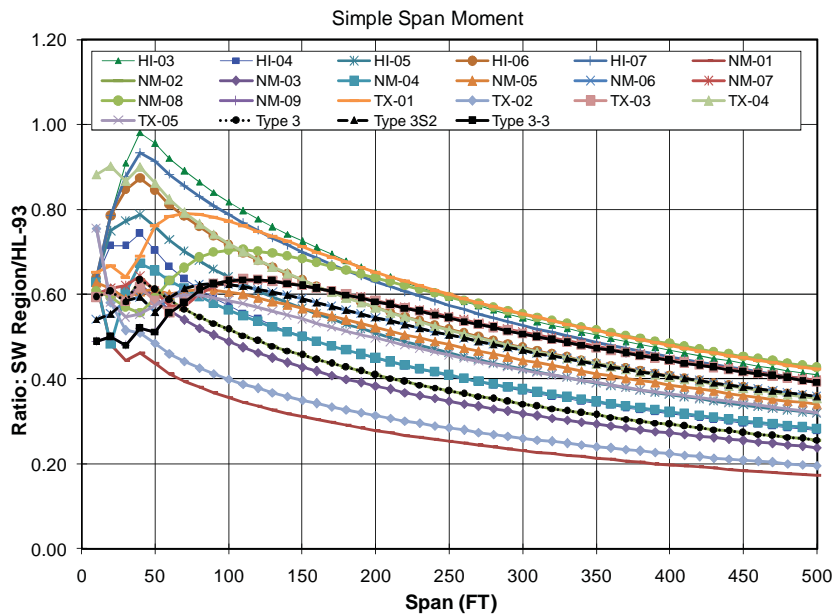


Figure 12. Southwest region live loads including AASHTO legal loads for simple span moment.

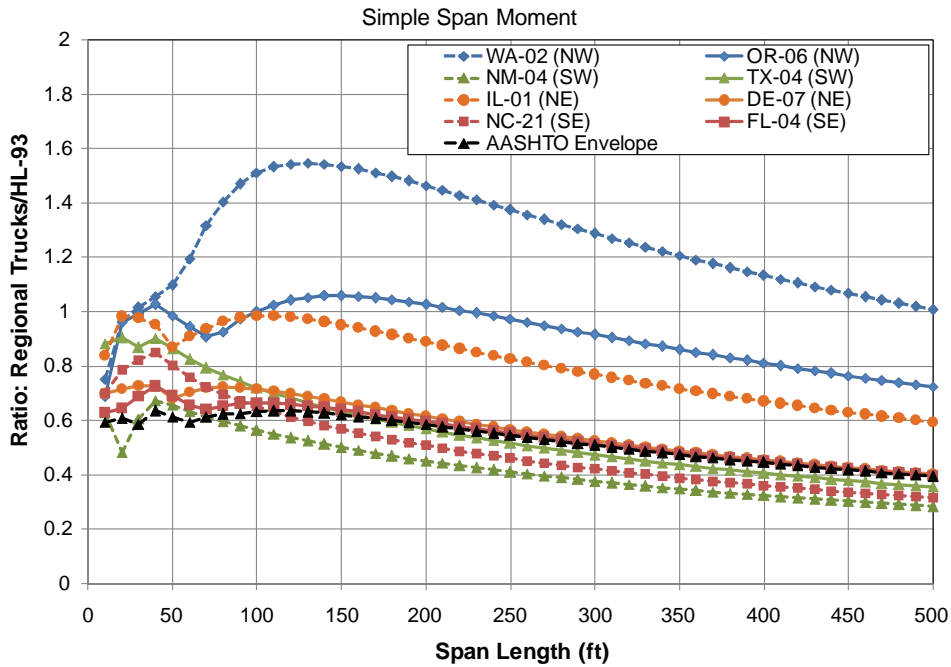


Figure 13. Simple span moments ratioed to HL-93 for regional vehicles and AASHTO legal loads.

northeast regions, indicating that the vehicles used for rating bridges result in similar actions to the AASHTO legal loads, but still do not approximate the HL-93 loading very well.

2.2.2 Conclusion

Analysis of results indicates a wide range of truck loads used to rate bridges throughout the United States. The project investigators have narrowed down the large number of trucks received via the survey to two regional trucks for each region shown in Figure 8, resulting in a total of eight trucks as described later in this chapter. The trucks selected for each region represent an “average truck” and a “heavy truck” for each region while encompassing a majority of the vehicles received in the survey.

The selected regional loadings were ratioed to both HL-93 and HS20 and plotted against span length. The envelope of the AASHTO legal loads was also included in these plots. A complete set of plots is included in Appendix O and a representative plot for simple span moments ratioed to HL-93 is shown in Figure 13.

These vehicles were recommended for use based on this analysis and the resulting live load actions used in the “Findings and Applications” section of this report.

2.2.3 Vehicle Options

Based on the analysis results shown in Appendix O, the following permit loadings were chosen from the various regions: Northwest (Figure 14), Southwest (Figure 15), Northeast (Figure 16), and Southeast (Figure 17).

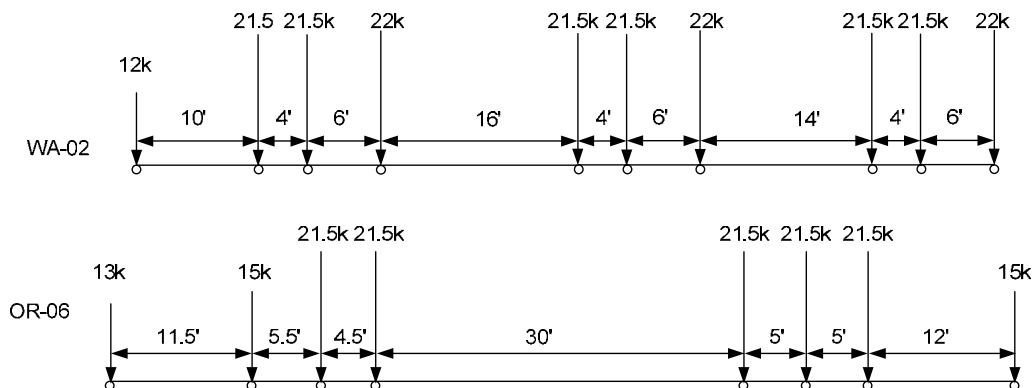


Figure 14. Northwest region vehicles.

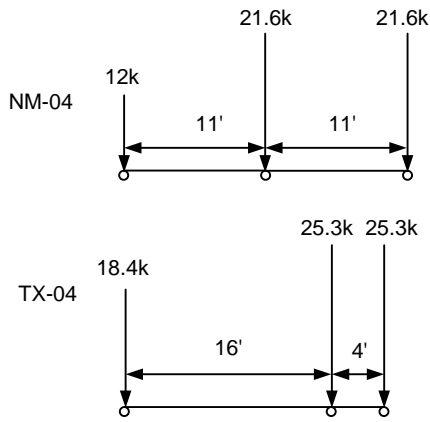


Figure 15. Southwest region vehicles.

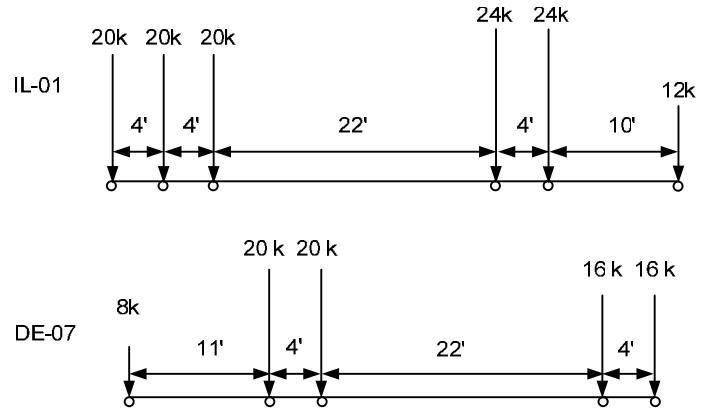


Figure 16. Northeast region vehicles.

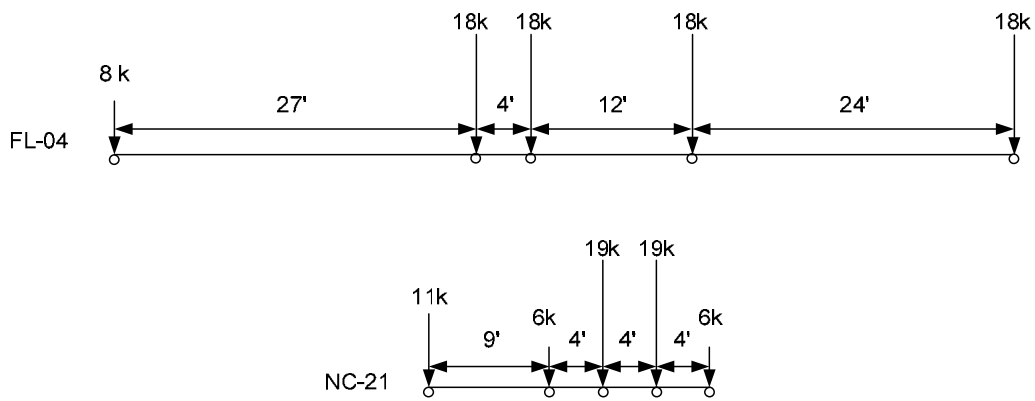


Figure 17. Southeast region vehicles.

CHAPTER 3

Findings and Applications

This section summarizes the findings of the final analysis runs for the 1,500 bridges selected for this research.

3.1 Summary of Bridge Rating Analysis

The work presented herein involved the analysis and rating of the superstructure elements in 1,500 bridges (3,043 girders); an investigation of the rating factors for LFR and LRFR; and an analysis of the differences of those ratings. Additionally the reliability index was calculated to determine if it meets that assumed in the development of the Manual for Bridge Evaluation (MBE). The target reliability index assumed in the development of the MBE is dependent upon the type of vehicle. Table 4 shows the assumed target reliability index for each vehicle type.

The bridges were analyzed using the BRASS analysis engines within Virtis. BRASS produces results using the NCHRP Project 12-50 Process Report IDs; these results are then concatenated and imported into a Microsoft Access database. For each of the 12 vehicles used in the analysis presented herein, there are 32 databases containing LFR results and 32 databases containing LRFR results. Data were then extracted from the databases using software developed for this research to review critical rating factors for LFR and LRFR for each girder. Rating factors were obtained for moment and shear, and inventory, operating, legal, or permit depending upon the vehicle type.

3.1.1 Software Used for the Analysis/Data Gathering

The number of bridges reviewed for this project required a great deal of automation, not only from the standpoint of analyzing the structures but also the sifting through the results. In all, given the 1,500 bridges, 12 vehicles and two specifications, the data results of more than 73,000 analysis runs of the Virtis-

BRASS engine were processed. A combination of existing analysis software and a data gatherer written for this project were used to accomplish these requirements. The two key software components are briefly described in the following sections.

3.1.2 Virtis/BRASS

The software used for the analysis of the bridges is the AASHTO Virtis software version 6.1 which incorporates the Wyoming Department of Transportation BRASS analysis engine. Wyoming BRASS has a separate version of software for LFR and LRFR rating and analysis. The versions used for this project were:

- BRASS Girder (LRFD) version 2.0.3
- BRASS Girder (LFD) version 6.0.0

Version 2.0.3 of BRASS Girder (LRFD) is not the version delivered with AASHTO Virtis 6.1.0. During the analysis of the Process 12-50 report IDs, it was noticed that BRASS was not producing the 12-50 analysis results for permit vehicles. Wyoming DOT graciously provided the revisions needed for the additional 12-50 results on a later version of the BRASS LRFR software than that provided in Virtis 6.1.0.

3.1.2.1 Reliability Index Output Generator

The RIO generator was developed to sort the Process 12-50 data produced by Virtis and BRASS. The detail of the format of the CSV file produced by this software is described in more detail in Appendix R. The interface of the software is fairly simple and is illustrated in Figure 18. The software provides the ability to extract information as needed for the Process 12-50 results so that the raw data can be imported into a spreadsheet for use in calculating the reliability indices as well as comparing and sorting the data to develop trends. The data can also exclude specific rating report IDs

Table 4. Assumed target reliability index, β .

Vehicle Type	Reliability Index
Design Load – Inventory Rating	3.5
Legal Loads	2.5
Routine Permits	2.5
Special Permits – Escorted	2.5
Special Permits – Single or Multiple Trips mixed with traffic	3.5

if necessary and can process multiple databases at a time. The output of the software is a 138-column comma-delimited file with 4 lines of data produced for each bridge girder as follows:

- Moment-Inventory
- Moment-Operating
- Shear-Inventory
- Shear-Operating

The final format for the comma-delimited file is shown in Appendix R. Each vehicle database is run through RIO.

3.1.3 Final Bridge Breakdown

This section and Appendix A serve as a summary of the final bridge/girder selection used for this project. Using the methods described in previous sections, the research team attempted to break down the bridge types and materials used for the analysis on this project based on the 2008 FHWA NBI. Using a cross section of the NBI along with the available Virtis bridge data provided by the states, 1,500 bridges (3,043 girders) were selected for the final analysis and review.

The bridges were subdivided by material type, configuration, year built and span length in an attempt to match the NBI

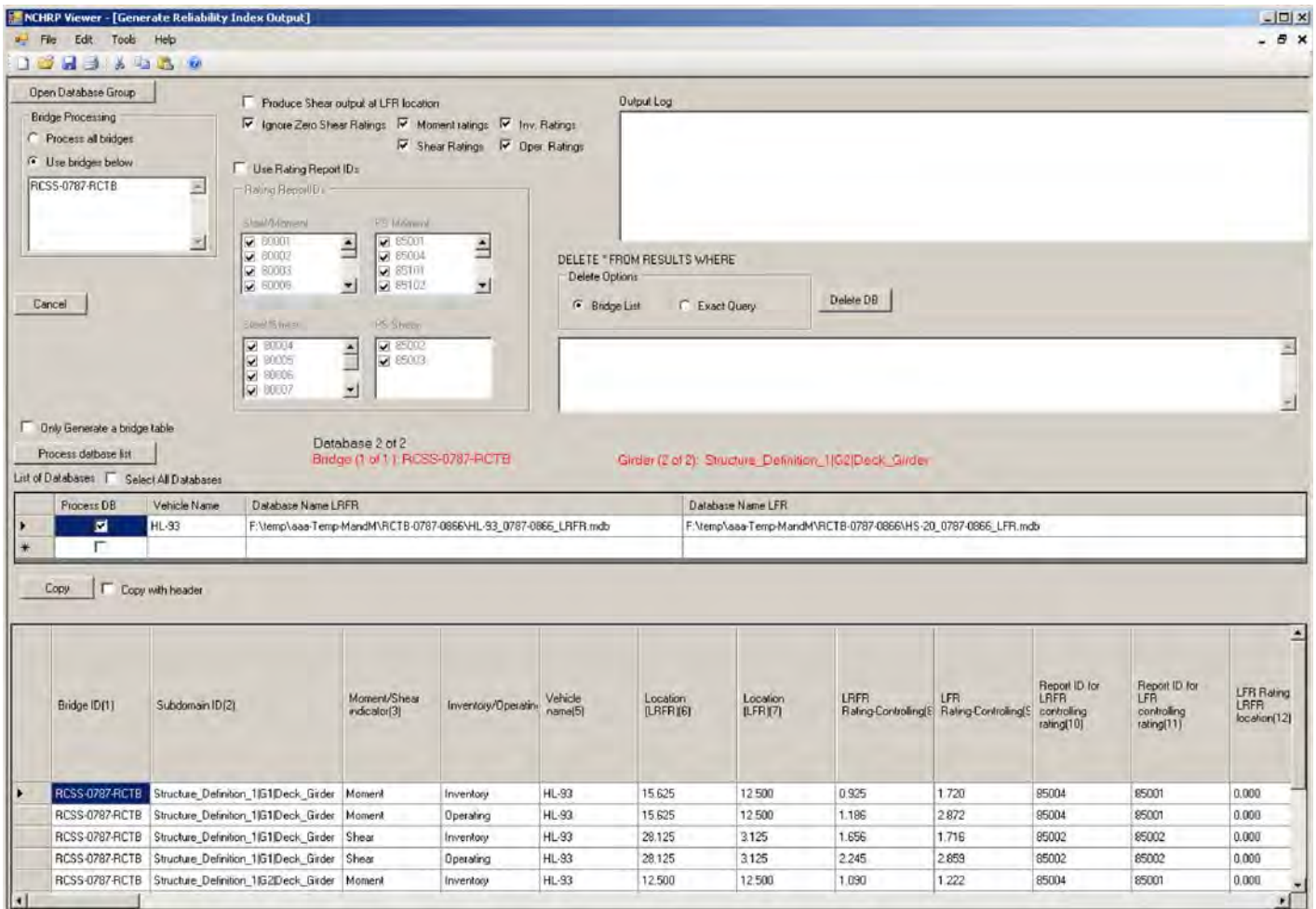


Figure 18. RIO generator.

as closely as possible. During the analysis process, bridges were discarded if the process required hand calculations to provide missing LRFR input. An example would be the manual calculation of LRFR distribution factors for supports with unequal skews. The distribution factor calculation provided in BRASS was used unless the input provided by the states included distribution factors.

The final bridge breakdown for various parameters is provided in the Figures 19–23. The final list of bridges (by girder) is provided in Appendix A.

3.1.4 Live Loads

The ratings were performed for the design loads, HS20 for LFR and HL-93 for LRFR, as well as for eight other vehicles that were selected earlier in the project. The eight state vehicles selected for this project were divided into different categories present in the MBE. Five vehicles were classified as “Routine Permit” vehicles and are shown in Table 5. The other three vehicles were classified as “Special or Limited Permit” vehicles and are shown in Table 6. The three AASHTO legal loads, Type 3, Type 3-3, and Type 3S2, are now included and shown in Table 7.

Table 8 shows how the eight vehicles are used in completing ratings by their respective states. Five of the vehicles (DE-07,

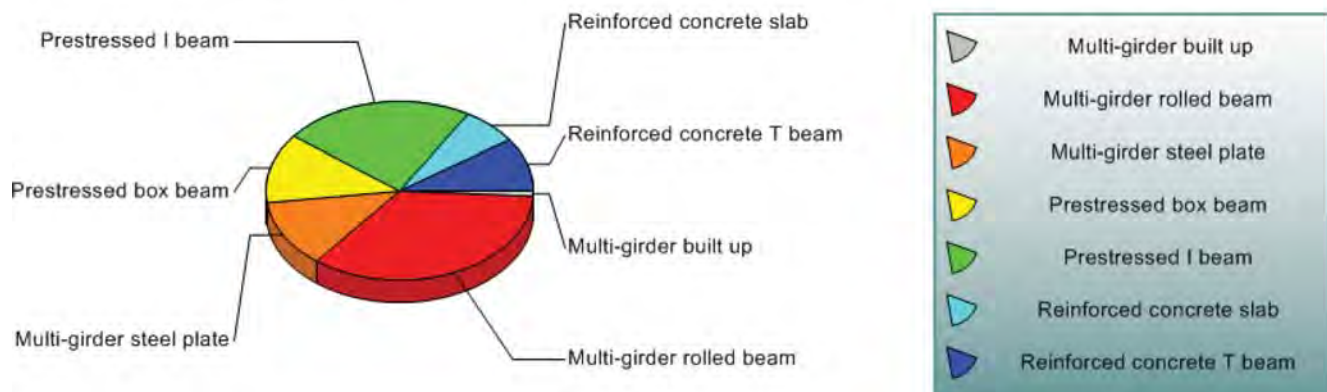
FL-04, NC-21, NM-04, and TX-04) are used as posting vehicles. Three of the vehicles are used as vehicles for determining the operating rating (FL-04, NM-04, and OR-06) and NM-04 also is used for inventory rating. Two vehicles are classified as “permit trucks” by their respective state (IL-01 and WA-02).

The current MBE rating provisions specify the number of lanes to be loaded by routine permit and special permit vehicles, but does not explicitly specify the number of lanes loaded by the HL-93 loading and AASHTO legal trucks. As the HL-93 loading and the AASHTO legal trucks represent typical commercial traffic, the critical distribution factor, single lane or multiple lanes loaded, was used for these loads. For routine permit vehicles, the MBE provisions require that the multiple lane distribution factor be used. For special permit vehicles, the MBE provisions require using a single lane distribution factor without the 1.2 multiple presence factor.

Table 9 shows the live load factors applicable for what the MBE terms “routine commercial vehicles,” which are the AASHTO Legal Loads shown in Table 7. Table 10 shows the live load factors for “routine or annual permit vehicles” and for “special or limited permit vehicles.” When the truck weight is less than 100 kips, the routine permit load factors approach those used for the legal load rating.

The live load factors used for the various types of vehicles in this study are provided in Table 11. An ADTT of 1,000 is used

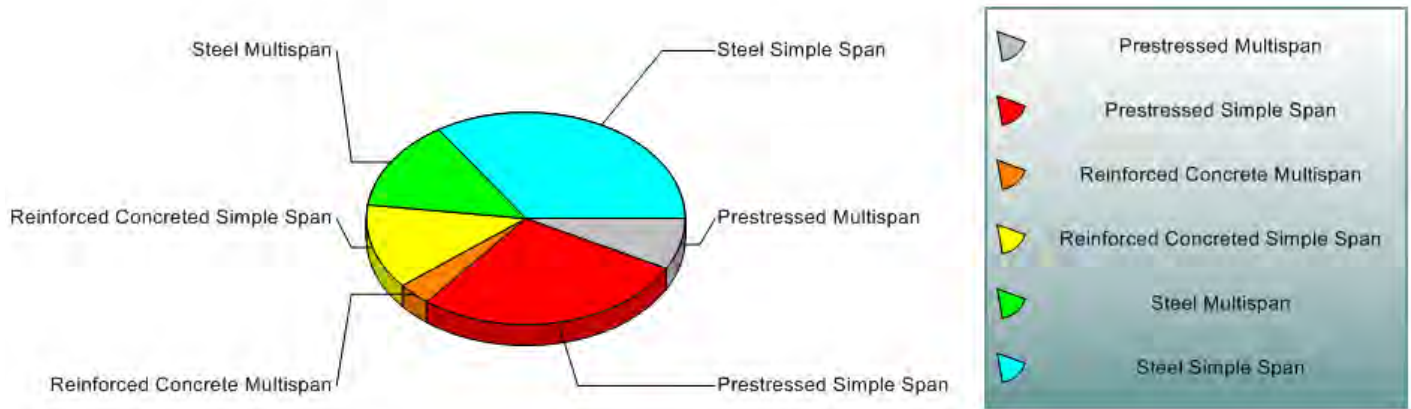
Bridge Type (Final 12-78 Breakdown)



Bridge Type	Total # Girders	Percent %
Multi-girder built up	29	0.95
Multi-girder rolled beam	1,056	34.70
Multi-girder steel plate	374	12.29
Prestressed box beam	381	12.52
Prestressed I beam	704	23.14
Reinforced concrete slab	204	6.70
Reinforced concrete T beam	295	9.69
Total	3,043	100.00

Figure 19. Bridge type (final 12-78 breakdown).

Bridge Material / Span Configuration



Bridge Material/ Span Configuration	Total # Girders	Percent %
Prestressed multispans	238	7.82
Prestressed simple span	847	27.83
Reinforced concrete multispans	105	3.45
Reinforced concrete simple span	394	12.95
Steel multispans	418	13.74
Steel simple span	1,041	34.21
Total	3,043	100.00

Figure 20. Bridge material/span configuration (final 12-78 breakdown).

Exterior-Interior Girder (Final 12-78 Breakdown)

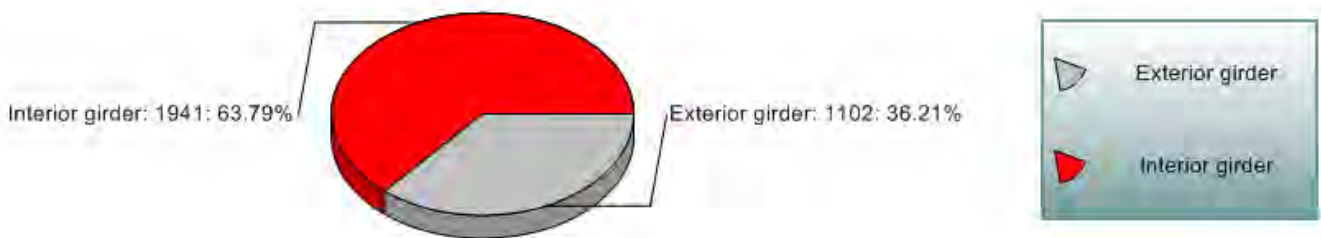
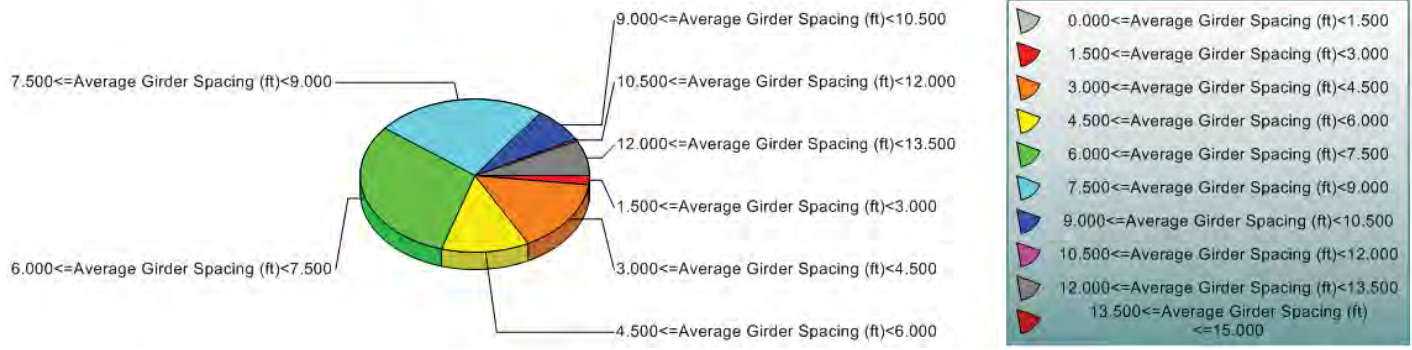


Figure 21. Exterior-interior girder (final 12-78 breakdown).

Average Girder Spacing (ft) (Interior Girders Only)



Average Girder Spacing	Total # Girders	Percent%
0.0000 ≤ Avg. Girder Spacing (ft) < 1.500	0	0.00
1.500 ≤ Avg. Girder Spacing (ft) < 3.000	36	1.92
3.000 ≤ Avg. Girder Spacing (ft) < 4.500	289	15.40
4.500 ≤ Avg. Girder Spacing (ft) < 6.000	233	12.41
6.000 ≤ Avg. Girder Spacing (ft) < 7.500	580	30.90
7.500 ≤ Avg. Girder Spacing (ft) < 9.000	452	24.08
9.000 ≤ Avg. Girder Spacing (ft) < 10.500	138	7.35
10.500 ≤ Avg. Girder Spacing (ft) < 12.000	8	0.43
12.000 ≤ Avg. Girder Spacing (ft) < 13.500	141	7.51
13.500 ≤ Avg. Girder Spacing (ft) ≤ 15.000	0	0.00
Total	1,877	100.00

Figure 22. Average girder spacing-interior girders only (final 12-78 breakdown).

for all graphs except where all different ADTTs are presented to show the effect of the ADTT.

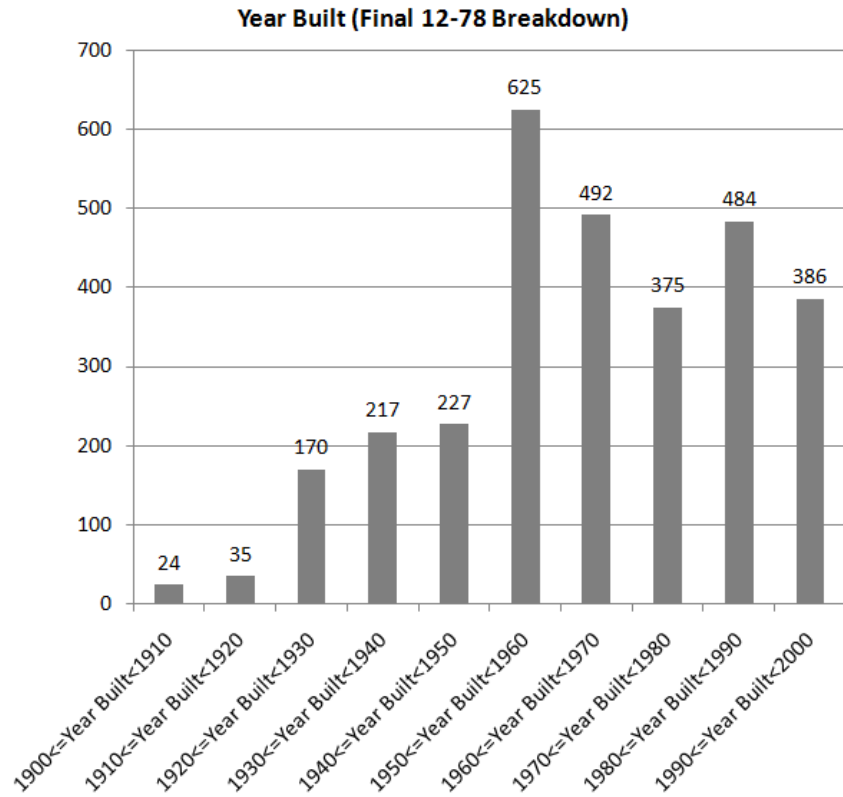
In performing the analysis, the BRASS/Virtis input files provided by several states were analyzed without any revisions, except those necessary for the LRFR rating to be completed, as they were thought to represent a cross-section of the existing bridge population. A problem with these input files was discovered during review of the results; the ADTT was input as zero for 91.5% of the girders analyzed. Figure 24 shows the percentage of girders in each ADTT range for each bridge type. Virtis/BRASS uses this ADTT to determine the load factor for the legal and permit load ratings. The bridges were first analyzed for the 11 legal or permit vehicles used in this work using the applicable load factor based on ADTT for LRFR, generally 1.4, from Table 9. This load factor was applied by Virtis/BRASS as it corresponds to the load factor for commercial traffic when low ADTT exists.

To include the effect of the ADTT on the rating factor comparisons, the rating factors were modified in the spreadsheets developed for this project by multiplying by a ratio of load factors to adjust all ratings to the same ADTT. IL-01, OR-06, and WA-02 were treated as single trip special permit vehicles allowed to mix with other traffic. The other five permit vehi-

cles are considered to be routine permit vehicles. Both types of permit vehicles as well as legal vehicles are assumed to have an ADTT = 1000. The effect of ADTT on the LRFR rating factor is discussed in Section 3.4, but all results contained in the appendices are for ADTT = 1000.

3.2 Reliability Index Calculation Spreadsheet

A spreadsheet was developed to compute reliability indices using the data obtained from BRASS. The reliability index calculation spreadsheet consolidates the results for each girder onto a single line representing the controlling rating factor and reliability index calculation. The data used in this spreadsheet is obtained from the RIO Generator program described earlier. Figure 25 shows a small portion of the Reliability Index Calculation spreadsheet. A similar spreadsheet was created for each type of bridge girder that was examined: simple span steel, simple span prestressed I-beam, simple span prestressed boxes, simple span reinforced concrete T-beams, simple span reinforced concrete slabs, continuous span steel, continuous span reinforced concrete slabs, and continuous span prestressed I-beams.

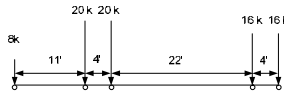
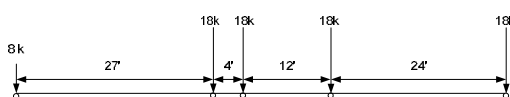
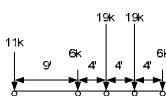
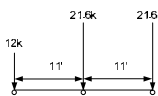
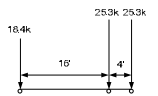


Year Built	Total # Girders	Percent %
1900<=Year built<1911	24	0.79
1911<=Year built<1922	35	1.15
1922<=Year built<1933	170	5.60
1933<=Year built<1944	217	7.15
1944<=Year built<1955	227	7.48
1955<=Year built<1966	625	20.59
1966<=Year built<1977	492	16.21
1977<=Year built<1988	375	12.36
1988<=Year built<1999	484	15.95
1999<=Year built<=2010	386	12.72
Total	3,035*	100.00

*Note: A small number of bridges had no "Year Built" input in Virtis

Figure 23. Year built (final 12-78 breakdown).

Table 5. Routine permit vehicles used in analysis.

Vehicle	GVW (kips)	Length (ft)	Schematic
DE-07	80	41	
FL-04	80	67	
NC-21	61	21	
NM-04	55.2	22	
TX-04	69	20	

GVW=gross vehicle weight

Table 6. Special or limited crossing permit vehicles.

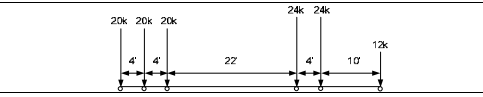
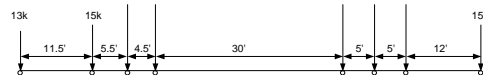
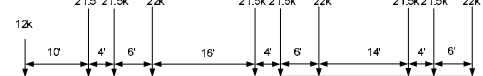
Vehicle	GVW (kips)	Length (ft)	Schematic
IL-01	120	44	
OR-06	150.5	73.5	
WA-02	207	70	

Table 7. AASHTO legal vehicles.

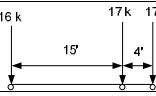
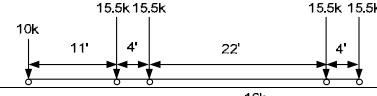
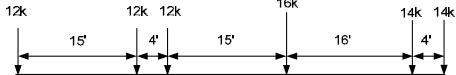
Vehicle	GVW (kips)	Length (ft)	Schematic
Type 3	50	19	
Type 3-3	80	54	
Type 3S2	72	41	

Table 8. Use of rating vehicles.

	Inventory	Operating	Posting	Permit
Routine Permit Vehicles				
DE-07				
FL-04				
NC-21				
NM-04				
TX-04				
Special or Limited Crossing Permit Vehicles				
IL-01				
OR-06				
WA-02				

Table 9. Generalized live load factors, γ_L for routine commercial traffic (MBE Table 6A.4.4.2.3a-1).

Traffic Volume (One direction)	Load Factor for Type 3, Type 3S2, Type 3-3 and Lane Loads
Unknown	1.80
ADTT \geq 5,000	1.80
ADTT = 1,000	1.65
ADTT \leq 100	1.40

ADTT=average daily truck traffic

Table 10. Permit load factors, γ_L (MBE Table 6A.4.5.4.2a-1).

Permit Type	Frequency	Loading Condition	DF ^a	ADTT (one direction)	Load Factor by Permit Weight ^b	
					Up to 100 kips	\geq 150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	> 5000	1.80	1.30
				= 1000	1.60	1.20
				< 100	1.40	1.10
					All Weights	
Special or Limited Crossing	Single Trip	Escorted with no other vehicles on 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		

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

^b For routine permits between 100 kips and 150 kips, interpolate the load factor by weight and ADTT value. Use only axle weights on the bridge.

Table 11. Live load factors used in analysis.

Vehicle	LRFR Live Load Factor			LFR Live Load Factor
	ADTT \leq 100	ADTT = 1,000	ADTT \geq 5,000	
Design Vehicle (HL-93 or HS20)	1.75			2.17
Routine Permit Vehicles (DE-07, FL-04, NC-21, NM-04, TX-04)	1.4	1.6	1.8	1.3
Special Permit Vehicles (IL-01, OR-06, WA-02)	1.35	1.4	1.5	1.3
Legal Vehicles (Type 3, Type 3-3, Type 3S2)	1.4	1.65	1.8	1.3

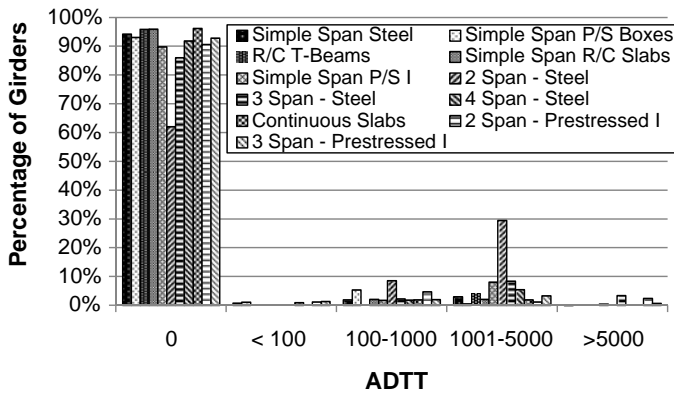


Figure 24. Distribution of girders to ADTT categories.

3.2.1. Raw Data

The raw data is obtained from RIO for each set of databases. This data includes the bridge name, girder number, a moment/shear indicator, an inventory/operating rating, the vehicle name, and the controlling rating factors for LRFR and LFR. Using the controlling rating factors, the moments and shears are determined at the critical location. The raw data for each vehicle is combined on its own separate tab (#1 in Figure 25) and sorted by whether the results are for moment or shear.

3.2.2 Sorted Raw Data

After the data is sorted, the data pertaining to the flexure and shear ratings are copied into separate spreadsheets for each vehicle (#2 in Figure 25). These spreadsheets are an intermediate step between the raw data and calculating the reliability

index. These spreadsheets are also linked to the “Rating Factor Comparison.”

3.2.3 Reliability Index Calculation

A separate spreadsheet is used to calculate the reliability index for moment and shear (#3 in Figure 25). This spreadsheet references the appropriate columns from the spreadsheet containing the relevant sorted raw data (#2).

The spreadsheet calculates the reliability index following the process used in calibrating the design specification and as presented in *NCHRP Report 368*. The main equation used to calculate the reliability index is shown here. The reliability index is calculated based upon the current section capacity for the actual reliability index. The required reliability index, shown only for the Design Vehicle, is the reliability index that would be obtained if the girder was designed for the load and load factors in the MBE.

$$\beta = \frac{(1 - 2 V_R) \left(1 + LN \left(\frac{1}{1 - 2 V_R} \right) \right) - \frac{\mu_Q}{\lambda_R R_n}}{\sqrt{(V_R - 2V_R^2)^2 + \frac{\sigma_q^2}{(\lambda_R R_n)^2}}}$$

Equation 1

where:

V_R = coefficient of variation of resistance

μ_Q = mean total applied load

$\lambda_R R_n$ = mean unfactored resistance (actual) or mean factored applied load (required)

σ_q = standard deviation of total applied load

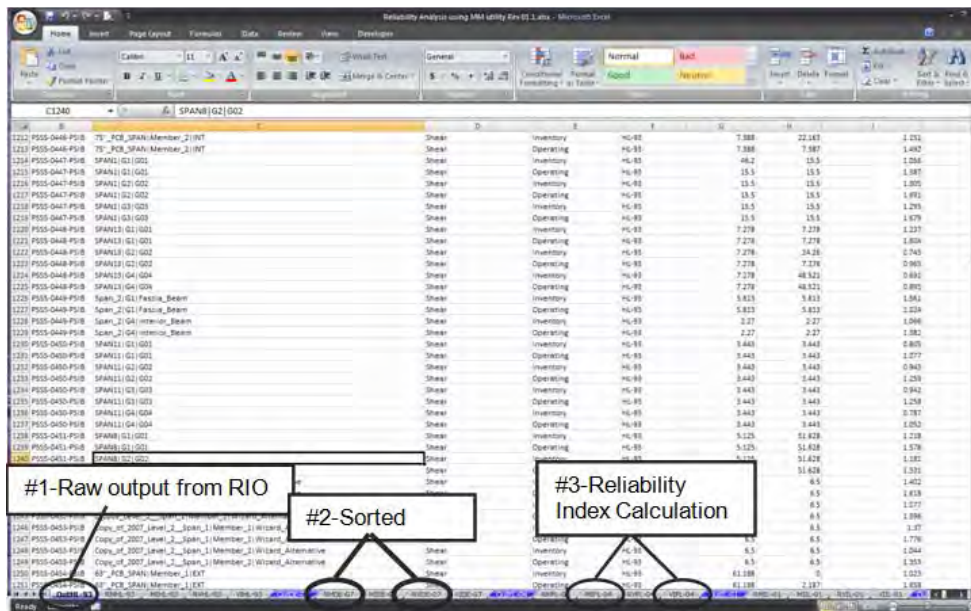


Figure 25. Reliability index calculation spreadsheet.

The mean total load is calculated by adding the mean live load plus impact load, the mean non-composite and composite dead loads, and the mean wearing surface dead load. Equation 2 shows the mean total load equation; μ_{DC1} is taken as the mean due to girder self-weight and μ_{slab} is taken as mean due to all other dead loads except for the wearing surface.

$$\mu_Q = \mu_{LL+IM} + \mu_{DC1} + \mu_{slab} + \mu_{DW} \quad \text{Equation 2}$$

The standard deviation of the total load is calculated using Equation 3. The standard deviations for each load component are calculated using the mean load and coefficient of variation.

$$\sigma_Q = \sqrt{\sigma_{LL+IM}^2 + \sigma_{DC1}^2 + \sigma_{slab}^2 + \sigma_{DW}^2} \quad \text{Equation 3}$$

The reliability index calculation used the following statistical parameters for the resistance of the cross section, as shown in Table 12.

The bias and COV for rolled shapes, prestressed I beams, and reinforced concrete T-beams are from *NCHRP Report 368*. The bias and COV for steel plate girders and built-up shapes, prestressed boxes, and reinforced concrete slabs were determined for this project using available plans of existing example bridges and the statistical parameters for dimensions and materials. The bias and COV have not yet been determined for shear in built-up sections; the shear parameters for rolled shapes were used for the built-up shapes. Statistical parameters for shear in reinforced concrete slabs have not been provided as shear does not have to be checked as long as the flexural design is completed according to AASHTO LRFD 4.6.2.3. The reliability index for shear in reinforced concrete slabs utilizes the same statistical parameters provided for shear in reinforced concrete T-beams.

The live load bias used in the calculation of the reliability index for the design, legal, and routine permit vehicles is provided in the tables below. The column of interest is “5 Years,” for the legal and routine permit vehicles while the 75-year column is used for the design load to be consistent with the AASHTO LRFD Design Specification. The appropriate columns are applicable for all loading situations (single or multiple lanes loaded). The bias values from *NCHRP Report 368* include a multiple presence factor of 1.2; the bias

(shown in Tables 13, 14, and 15) used in this research has been divided by 1.2 as the multiple presence factor is included in the girder distribution factor. Tables 13, 14, and 15 are for ADTT = 1,000; additional tables are presented in *NCHRP Report 368* for ADTT = 5,000. Such load is estimated to produce the highest load effect in the specified time period. The bias is determined for each bridge by interpolating between span lengths. Spans greater than 200 feet long use the bias for spans that are 200 feet long. Special permit vehicles were assumed to be deterministic and use a bias of 1.0 and a coefficient of variation of 0.0.

The statistical parameters used for the different vehicle types are indicated in Table 16. The effect of using higher statistical parameters (bias and coefficient of variation) is a higher mean load and lower reliability index. To obtain the same target reliability index when higher statistical parameters are used, larger load factors will be required.

The values necessary to calculate the reliability indices are shown in Appendix J for 20 girders/slabs of each type of bridge. For steel girders and prestressed boxes, 20 composite girders and 20 non-composite girders are shown. Results for both moment and shear are shown for all types even though shear does not have to be rated for concrete girders when a design rating is performed.

The current load factors in the MBE assume that for routine permit vehicles multiple lanes loaded will control in all situations. Therefore, the live loads from BRASS had to be corrected such that the load was from multiple lanes loaded even though for a particular girder a single lane loaded may control. For special or limited crossing permit vehicles, the MBE assumes that a single lane loaded will control all situations and the live loads from BRASS were adjusted accordingly. The load factors for special or limited crossing permit vehicles in the MBE account for the possibility of other vehicles being alongside the permit vehicle when on the bridge. The corrections were made as all ratings using Virtis were completed assuming that the vehicles were legal vehicles, not permit vehicles, and it was more efficient to multiply by a ratio than to rerun all the analyses. The design and legal vehicles used the critical number of lanes loaded. The following list shows the critical number of lanes for each type of vehicle:

- Design and Legal Vehicles—single or multiple lanes loaded, whichever is critical

Table 12. Statistical parameters for resistance.

Bridge type	Moment		Shear	
	bias	COV	bias	COV
Multi-girder steel rolled shapes	1.12	0.10	1.14	0.105
Multi-girder steel plate girders	1.08 M+, 1.05 M-	0.11 M+, 0.10 M-	1.13	0.16
Multi-girder steel built-up shapes	1.11 M+, 1.12 M-	0.123	X	X
Prestressed I Beam	1.05	0.075	1.15	0.14
Prestressed Box Beam	1.05	0.075	1.16	0.14
Reinforced concrete T beam	1.14	0.13	1.20	0.155
Reinforced concrete slab	1.13	0.13	--	--

Table 13. Live load positive moment statistical parameters.

Mean Maximum Moments for Simple Spans Due to Multiple Trucks in One Lane (Divided by Corresponding HL-93 Moment)										
Span (ft)	1 day	4 days	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.73	0.83	0.85	0.89	0.93	0.98	1.04	1.11	1.25	1.25
20	0.75	0.80	0.82	0.85	0.89	0.92	0.96	1.02	1.10	1.10
30	0.79	0.85	0.87	0.90	0.93	0.96	1.00	1.04	1.13	1.13
40	0.84	0.89	0.91	0.93	0.96	0.99	1.01	1.05	1.11	1.11
50	0.83	0.88	0.90	0.92	0.96	0.98	1.01	1.04	1.10	1.10
60	0.84	0.89	0.90	0.94	0.96	0.98	1.01	1.04	1.10	1.10
70	0.85	0.88	0.90	0.93	0.96	0.98	1.00	1.04	1.09	1.09
80	0.85	0.88	0.90	0.93	0.95	0.97	1.00	1.04	1.10	1.10
90	0.85	0.89	0.90	0.92	0.95	0.97	1.00	1.04	1.09	1.09
100	0.85	0.88	0.90	0.92	0.94	0.96	1.00	1.03	1.09	1.09
110	0.85	0.88	0.89	0.92	0.94	0.96	0.99	1.03	1.09	1.09
120	0.84	0.88	0.89	0.91	0.93	0.96	0.98	1.02	1.08	1.08
130	0.84	0.87	0.88	0.91	0.92	0.94	0.97	1.00	1.06	1.06
140	0.82	0.85	0.86	0.89	0.90	0.92	0.94	0.98	1.03	1.03
150	0.82	0.85	0.86	0.88	0.89	0.91	0.94	0.97	1.03	1.03
160	0.82	0.85	0.86	0.89	0.90	0.92	0.94	0.98	1.03	1.03
170	0.82	0.85	0.86	0.89	0.91	0.93	0.95	0.98	1.04	1.04
180	0.82	0.85	0.86	0.88	0.90	0.92	0.94	0.98	1.04	1.04
190	0.81	0.84	0.85	0.88	0.90	0.92	0.94	0.98	1.03	1.03
200	0.80	0.83	0.85	0.88	0.90	0.91	0.94	0.97	1.03	1.03

Table 14. Live load shear statistical parameters.

Mean Maximum Shears for Simple Spans Due to Multiple Trucks in One Lane (Divided by Corresponding HL-93 Shear)										
Span (ft)	1 day	4 days	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.74	0.79	0.81	0.85	0.86	0.89	0.91	0.94	0.99	1.00
20	0.77	0.83	0.84	0.88	0.88	0.92	0.93	0.96	1.02	1.03
30	0.80	0.85	0.87	0.90	0.91	0.94	0.96	0.99	1.03	1.04
40	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
50	0.81	0.86	0.87	0.90	0.91	0.93	0.95	0.97	1.01	1.02
60	0.82	0.88	0.89	0.91	0.92	0.94	0.96	0.99	1.02	1.03
70	0.84	0.89	0.90	0.92	0.93	0.95	0.97	0.99	1.04	1.04
80	0.85	0.89	0.91	0.93	0.94	0.97	0.99	1.01	1.05	1.06
90	0.85	0.90	0.91	0.93	0.94	0.97	0.99	1.01	1.06	1.07
100	0.86	0.90	0.91	0.93	0.95	0.97	0.99	1.02	1.06	1.06
110	0.85	0.89	0.90	0.92	0.93	0.96	0.98	1.00	1.04	1.05
120	0.84	0.88	0.88	0.90	0.91	0.93	0.95	0.98	1.01	1.02
130	0.83	0.87	0.87	0.89	0.90	0.92	0.93	0.96	0.99	1.00
140	0.83	0.87	0.88	0.90	0.91	0.92	0.93	0.96	1.00	1.01
150	0.82	0.86	0.87	0.89	0.90	0.92	0.93	0.96	1.00	1.00
160	0.81	0.85	0.86	0.88	0.89	0.91	0.92	0.95	0.99	1.00
170	0.80	0.84	0.85	0.88	0.89	0.90	0.92	0.95	0.99	1.00
180	0.80	0.83	0.85	0.86	0.87	0.89	0.91	0.95	0.98	0.99
190	0.79	0.83	0.84	0.86	0.87	0.88	0.90	0.93	0.97	0.98
200	0.79	0.83	0.84	0.85	0.87	0.88	0.90	0.93	0.97	0.97

Table 15. Live load negative moment statistical parameters.

Mean Max. Negative Moments for Continuous Spans Due to Multiple Trucks in One Lane (Divided by HL-93 Neg. Moment)										
Span (ft)	1 day	4 days	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.79		0.88	0.92	0.94	0.97	0.99	1.03	1.09	1.09
20	0.86		0.93	0.95	0.95	0.97	0.99	1.02	1.05	1.06
30	0.91		0.96	0.98	0.99	1.01	1.02	1.04	1.06	1.07
40	0.92		0.98	0.99	1.00	1.02	1.04	1.05	1.08	1.09
50	0.94		1.00	1.01	1.02	1.04	1.06	1.07	1.10	1.11
60	0.88		0.94	0.95	0.96	0.98	0.99	1.01	1.04	1.04
70	0.86		0.92	0.93	0.94	0.96	0.97	0.98	1.01	1.02
80	0.85		0.91	0.92	0.93	0.95	0.96	0.98	1.01	1.01
90	0.85		0.90	0.91	0.92	0.94	0.95	0.97	1.00	1.00
100	0.85		0.90	0.91	0.92	0.94	0.95	0.97	1.00	1.00
110	0.85		0.90	0.91	0.92	0.94	0.95	0.97	1.00	1.00
120	0.85		0.90	0.91	0.92	0.94	0.95	0.97	1.00	1.00
130	0.85		0.90	0.91	0.92	0.94	0.95	0.97	0.99	1.00
140	0.85		0.90	0.91	0.92	0.94	0.95	0.97	0.99	1.00
150	0.85		0.90	0.91	0.92	0.94	0.95	0.97	1.00	1.00
160	0.85		0.90	0.91	0.92	0.95	0.95	0.97	1.00	1.00
170	0.85		0.90	0.91	0.92	0.94	0.95	0.97	1.00	1.00
180	0.85		0.90	0.91	0.92	0.94	0.95	0.97	1.00	1.00
190	0.85		0.90	0.91	0.92	0.94	0.95	0.97	0.99	1.00
200	0.86		0.90	0.91	0.92	0.94	0.95	0.97	0.99	1.00

- Routine Permit Vehicles—two or more lanes loaded
- Special or Limited Crossing Permit Vehicles—single lane loaded without multiple presence factor

3.3 Rating Factor Comparison Spreadsheet

This spreadsheet is used to compile the bridge geometric characteristics, dead load moments and shears, and critical moment and shear locations for each girder. Additionally, for each live load (Design Vehicle, eight permit loads, and three legal loads), the LFR rating factor, LRFR rating factor, actual reliability index, and required reliability index for moment and shear are also compiled. The data is pulled into this spreadsheet from the reliability index calculation spreadsheet. LFR and

LRFR Inventory Ratings are used for the Design Vehicle (HS-20 for LFR and HL-93 for LRFR) while LFR Operating Ratings and LRFR permit or legal ratings are used for the eight permit and three legal loads.

This spreadsheet also compares the dead load moments and shears, live load moments and shears, as well as moment and shear resistance. Checking the ratio of unfactored dead load moments and shears between LFR and LRFR allows verifying the results are being obtained at or near the same location and that the same dead loads are used in both methods. The rating factor comparison spreadsheet contains the results for the Design Vehicle as well as for the eight permit and three legal loads.

After obtaining all results, they are copied into a new sheet where they can be sorted without affecting the formulas extracting the data from the reliability index calculation spreadsheet. The results are sorted into four major categories (for some structure types not all four categories are applicable):

- Interior girders with composite decks;
- Interior girders with non-composite decks;
- Exterior girders with composite decks; and
- Exterior girders with non-composite decks.

Following the sorting into the four major categories, the girders in each category are then plotted against the following bridge characteristics:

- Year of construction
- Span length

Table 16. Statistical parameters for different vehicle types.

Vehicle Type	Moment		Shear	
	Bias (λ)	COV	Bias (λ)	COV
Design	Table 9 and Table 11	0.19	Table 10	0.19
Legal	Table 9 and Table 11	0.19	Table 10	0.19
Routine Permit	Table 9 and Table 11	0.19	Table 10	0.19
Special Permit	1.0	0.0	1.0	0.0

- Skew Angle
- Tributary width (for interior girders this is equal to the average girder spacing and for exterior girders this is equal to the overhang width plus one-half the spacing between the exterior girder and first interior girder)

Scatter plots were created in an effort to observe trends within the data and how the results are affected by these different criteria:

- Ratio of DC1 (unfactored dead loads applied to non-composite cross-section) (moment and shear)
- Ratio of DC2 (unfactored dead loads applied to composite cross-section, does not include the wearing surface) and DW (wearing surface applied to composite cross section) (moment and shear)
- Ratio of Unfactored Live Loads (moment and shear)
- Ratio of Factored Live Loads (moment and shear)
- Actual Reliability Indices (moment and shear)
- Location of Critical rating factor (shear only)
- LFR Rating (moment and shear)
- LRFR Rating (moment and shear)
- Ratio of LRFR Rating to LFR Rating (moment and shear)

Table 17 shows the type and number of girders used in the analysis. There were 3,036 girders used in the analysis from eight different types of bridges.

3.3.1 Data Analysis and Trends

As shown in the Appendices of this report, the amount of data analyzed was overwhelming. The results of different types of structures are similar in that there are no clear trends related to the criteria used (i.e., year of construction, skew angle, and tributary width, etc.) that can be used as a basis for possible revisions to the MBE. In the following appendices, the results for simple span steel structures are presented in more detail than for other types of structures presented in the subsequent sections.

The appendices to this report contain comprehensive coverage of the results. For simple span steel girder bridges, the

Table 17. Girder type and number.

<i>Girder Type</i>	<i>Number of Girders</i>
Simple Span Steel	1037
Simple Span Prestressed Concrete I	467
Simple Span Prestressed Concrete Box	377
Simple Span Reinforced Concrete T-Beam	295
Simple Span Reinforced Concrete Slab	99
Continuous Span Steel	418
Continuous Span Reinforced Concrete Slabs	105
Continuous Span Prestressed I	238

results are presented in Appendix B. The results in Appendix B are plotted against: year of construction, span length, tributary width, and skew angle for interior composite girders. Span length and tributary width are used for the remaining categories of simple span steel bridges. The results for other types of simple span structures, Appendix C through Appendix F, are shown plotted versus span length only. The results for continuous spans are shown in Appendix G through Appendix I, with the continuous steel and prestressed I-girder bridges shown plotted versus tributary width and continuous slabs shown plotted versus maximum span length in the bridge. Due to the lack of trends in all cases, it was concluded that the variable used for the horizontal axis of the graph, be it year of construction, skew angle, and tributary width, etc., did not actually matter.

The results are contained in the following appendices:

- Appendix B—Simple Span Steel I Girder Bridges
 - B.1—Interior Composite
 - B.2—Interior Non-Composite
 - B.3—Exterior Composite
 - B.4—Exterior Non-Composite
- Appendix C—Simple Span Prestressed I Girder Bridges
 - C.1—Interior Composite
 - C.2—Exterior Composite
- Appendix D—Simple Span Prestressed Box Girder Bridges
 - D.1—Exterior Composite
 - D.2—Exterior Non-Composite
 - D.3—Interior Composite
 - D.4—Interior Non-Composite
- Appendix E—Simple Span Reinforced Concrete T-Beam Bridges
 - E.1—Exterior
 - E.2—Interior
- Appendix F—Simple Span Reinforced Concrete Slab Bridges
- Appendix G—Continuous Span Steel Girder Bridges
 - G.1—Interior Non-Composite
 - G.2—Interior Composite
 - G.3—Exterior Non-Composite
 - G.4—Exterior Composite
- Appendix H—Continuous Span Reinforced Concrete Slab Bridges
- Appendix I—Continuous Span Prestressed I Girder Bridges
 - I.1—Interior Composite
 - I.2—Exterior Composite

3.3.2 Main Sources of Difference in Rating Factors Between LFR and LRFR

In some cases, significant differences in the rating factors calculated using the LFR and LRFR methodologies were observed

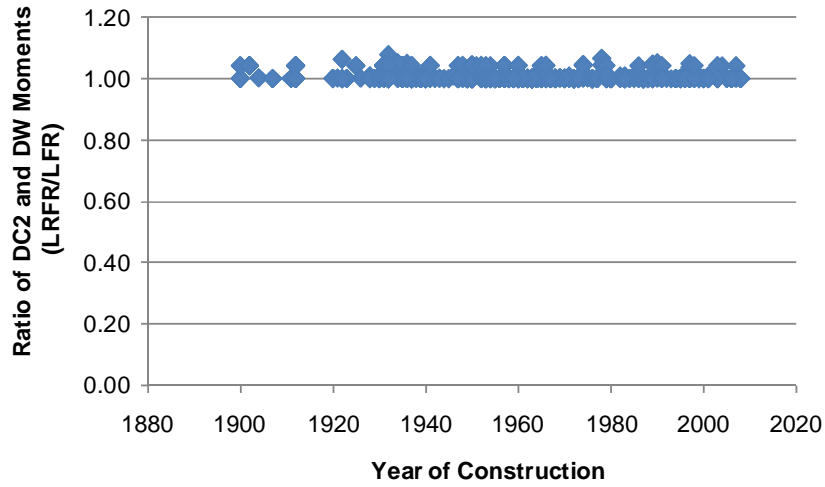


Figure 26. Ratio of unfactored LRFR composite dead loads to unfactored LFR composite dead loads.

for individual bridges. In other cases there was a general trend of a reduction in the rating factors. These differences were investigated to determine the source of the differences. Here are the reasons for the differences:

- Differences in unfactored dead loads: It was expected that for a particular girder the dead loads would be the same in both methods. However, it was observed that in approximately 4% of the simple span steel girders, the distribution of the composite dead loads to the girders in the bridge cross section appeared to be done differently in the two methods (See Figure 26). Further investigation found the values calculated in BRASS were correct, but some of the values extracted using the NCHRP 12-50 process were not.

Using the values calculated by BRASS, the Figure 26 shows that most girders have composite dead loads that are similar; the girders that were not similar have been removed from this graph.

- Differences in live loads: The factored live loads for LRFR are typically greater than the LFR live loads for both moment and shear. Figures 27 and 28 show the ratio of factored positive live load moment and factored negative live load moment, respectively, for the DE-07 vehicle. Figure 29 shows the ratio of factored live load shears for the DE-07 vehicle. The figures show that most girders have LRFR live loads that are greater than the LFR live loads for both moment and shear. The resistances and dead loads were similar for most types of girders resulting in the numerator

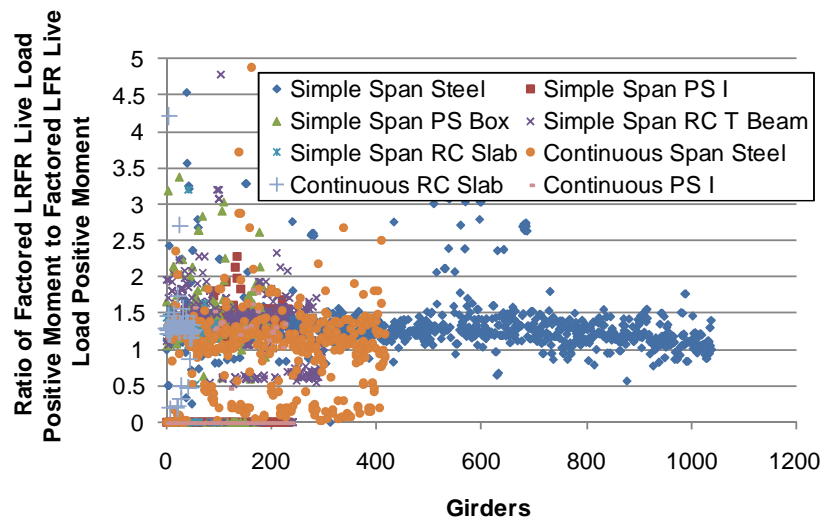


Figure 27. Ratio of factored positive live load moments (LRFR/LFR) for DE-07 vehicle.

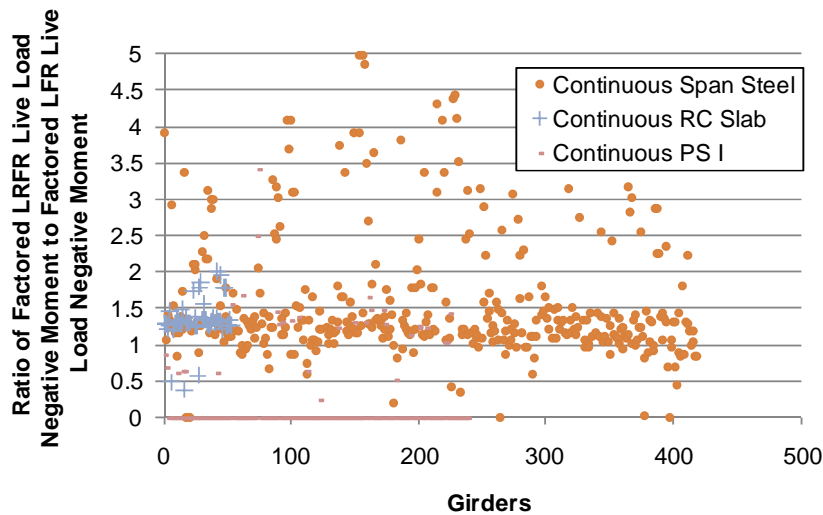


Figure 28. Ratio of factored negative live load moments (LRFR/LFR) for DE-07 vehicle.

of the rating factor equation being similar. This indicates that the factored live loads are likely causing most girders to have lower ratings for LRFR than for LFR, in addition to the new rating criteria discussed below.

- Atypical bridge characteristics: The calculations of loads for bridges with atypical geometry resulted in significant differences. Examples of these bridges are:
 - A simple span bridge with one square (zero skew) abutment and the second abutment having 35 degree skew angle. Figure 30 shows the plan of such a bridge. The results showed that the loads varied from girder to girder because each girder has a different length. This was caused by the load factors for live load in the LFR being the same for all girders (distribution factors are a function of the

girder spacing only) while they were significantly different for LRFR (a function of span length, girder spacing, deck/beam relative stiffness). This bridge was removed from the set.

- Bridges with sidewalks where the input assumed that the sidewalk may be removed and the entire width of the bridge is available to vehicular traffic for LFR and considered the effects of the sidewalk on the loads (lighter live load) in LRFR. Girder G4 in Figure 31 was a case where the presence of a sidewalk was ignored in the calculation of the distribution factor for LFR by the user. For LRFR, the distribution factor is calculated by BRASS and considered the presence of the sidewalk.

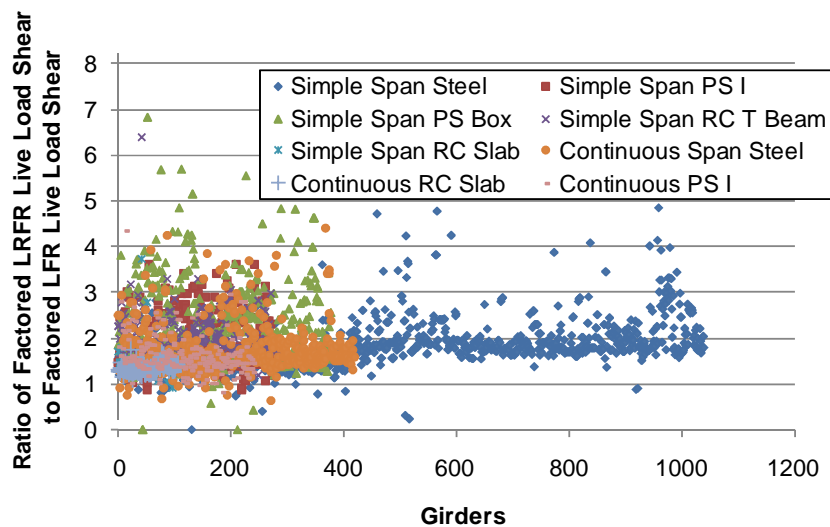


Figure 29. Ratio of factored live load shears (LRFR/LFR) for DE-07 vehicle.

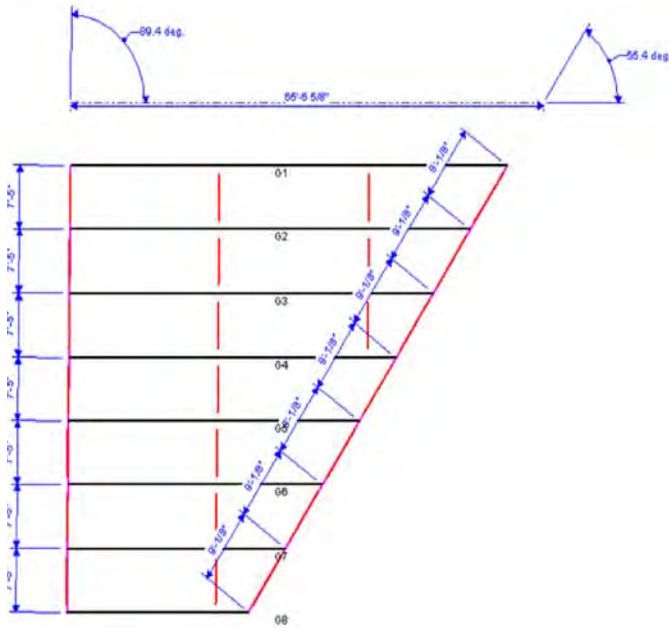


Figure 30. Atypical bridge characteristics.

- Bridges with small girder spacing and thick decks. Such arrangement produces significantly different live load distribution factors for the two rating methods. When the effect of the thick deck and small girder spacing is combined with different treatment of the sidewalk, the difference in the rating factors is extreme.

For example, for the bridge shown in Figure 31, the deck thickness exceeds that for which the LRFD distribution factor equations were developed and the girder spacing is at the limit of the applicability range. Following the distribution factor table in the AASHTO LRFD Design Specification, the distribution factor is determined using the lever rule and considering the limits imposed by sidewalks on the position of the traffic lanes. The LFR distribution factor is still deter-

mined using S/5.5 ignoring the sidewalk and its effect on the location of the traffic lanes. The distribution factor for LFR is 0.64 wheel lines, i.e., 0.32 lanes, and for LRFR is 0.06 lanes.

Within the simple span steel bridges, there were 118 girders with tributary widths less than 3.5 feet, 198 girders with a deck thickness of less than 4.5 inches, and 12 girders with a deck thickness greater than 12 inches. The 3.5 foot spacing and deck thickness limits of 4.5 to 12 inches are the limits on the range of applicability for the distribution factors from Table 4.6.2.2b-1 in the LRFD Design Specification. When the thin decks were investigated, it was found that several were timber decks that were erroneously input as concrete.

- New rating criteria: The LFR required checking far fewer criteria than the LRFR. In many cases when a general trend of lower rating factors was observed, it was noticed that the lower LRFR ratings were generally controlled by a criteria not included or rarely controlled in LFR. Following are five examples for such cases.
 - Concrete structures: Rating of concrete beams and slabs for shear. In the LFR ratings, shear could be ignored for concrete elements but in the LRFR rating shear must be considered for permit vehicles and for the design and legal loads if signs of distress exist. Many of the Virtis input files for bridges with concrete superstructures had the shear rating turned off for both LFR and LRFR. The shear rating was turned on and shear ratings were considered and compared. Figure 32 shows that the prestressed concrete I-girder LRFR ratings for shear are more scattered and of the 274 girders shown, 177 were controlled by the shear rating, not the moment rating.
 - The results for shear in steel girders: The LRFR shear rating factor for approximately one-third of the simple span steel bridge girders was controlled by the capacity of the bearing stiffener (NCHRP 12-50 Report ID 80007).

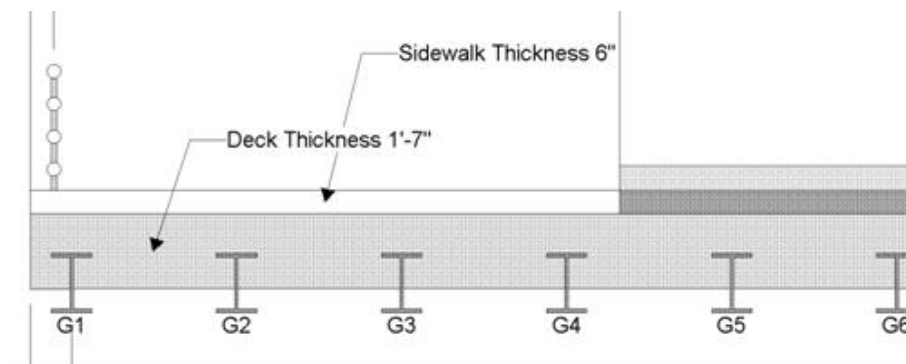


Figure 31. Bridge with characteristics outside of range of applicability for LRFD distribution factor equations.

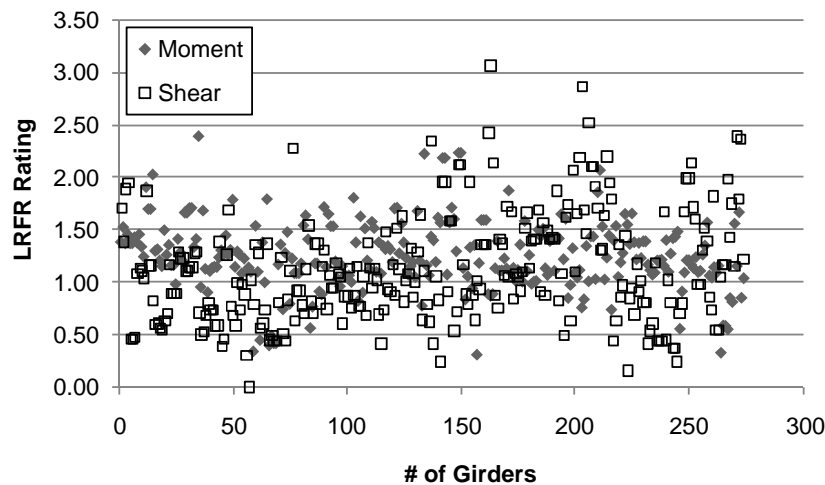


Figure 32. LRFR design load inventory ratings for simple span prestressed I-girders.

This check was not typically performed in the LFR rating. Figure 33 shows that ratings are generally lower when the capacity of the bearing stiffener was included in determining the controlling rating factor. The maximum difference in the value of the controlling rating factor including and ignoring the bearing stiffener rating is 88.3% with the average difference being 13.6%. Further investigation into the bearing stiffener ratings found that for LFR, the version of BRASS used in this project (Version 6.0.0) was calculating both the bearing resistance and axial resistance of the bearing stiffener but was incorrectly calculating the area used in the bearing resistance equation. In addition, further investigation by BridgeTech, the developer of BRASS, determined that the LFR rating was only considering the axial column resistance, not both the bearing resistance and

axial column resistance, in determining the bearing stiffener rating. These errors were corrected in later versions of BRASS (Version 6.0.1 and later).

After these errors were corrected, the resistance of the bearing stiffeners in both methods is very similar when all load factors are taken into account. Therefore, the difference in the rating factor of the bearing stiffener in the two methods will be the same as the difference in the beam reaction. Due to heavier live loads in the LRFR, the reaction under the LRFR loads is expected to be approximately 20-40% higher than for LFD.

- The results for flexure in prestressed I-girders and box girders, reinforced concrete T-beams, and reinforced concrete slabs were affected by including the effect of shear on the force in the longitudinal reinforcement near the ends of girders (NCHRP 12-50 Report ID 85004).

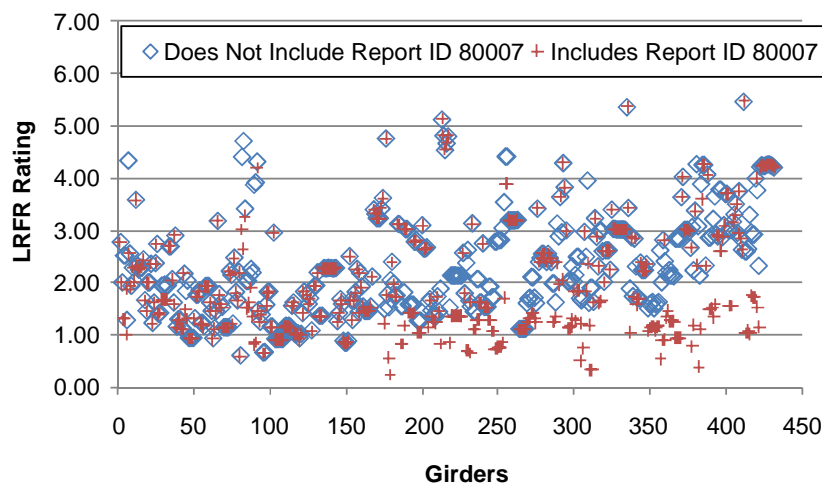


Figure 33. LRFR design load shear rating factors for simple span steel bridges.

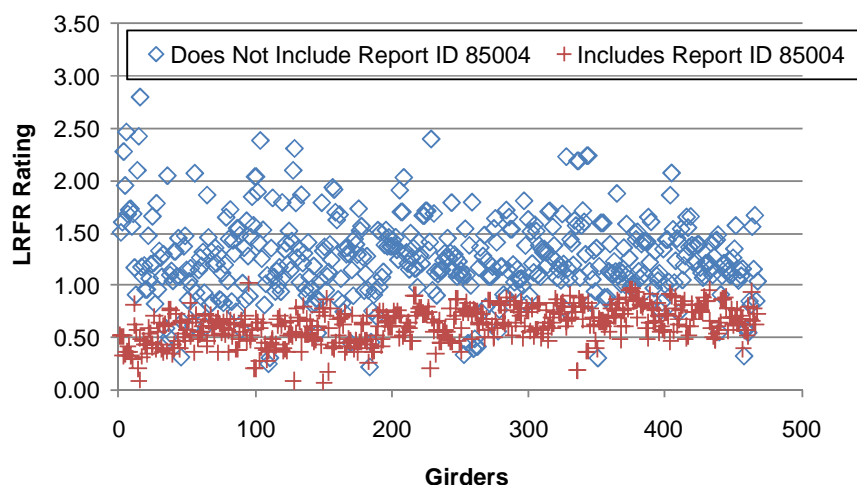


Figure 34. LRFR design load moment ratings for simple span prestressed concrete I-beams.

This Report ID returns the rating of the stress in the longitudinal reinforcement required by Article 5.8.3.5 of the AASHTO LRFD. Figure 34 shows that ignoring the longitudinal steel rating when determining the controlling rating leads to increased ratings for many girders. The maximum percentage difference is 86.3% of the rating ignoring Report ID 85004 and the average percent difference is 20.2%. In addition to changing the ratings, the controlling section shifted towards the support and away from midspan.

- Stresses in prestressed girders under service loads: It was determined that when the longitudinal steel rating is ignored, the service limit states control most girders with LRFR ratings less than 1.0. The service limit states are optional for legal and permit vehicles but are calculated by BRASS. There is an option within Virtis which allows the service stresses to be considered in determining the

critical rating; if this box is not selected the critical rating will not consider the service stress ratings. All ratings are printed in the BRASS output as well as the NCHRP 12-50 Process output. Figure 35 shows the LRFR rating ignoring both the longitudinal steel rating and the Service III tensile stress rating, indicated by the blue diamonds, and the LRFR ratings where only the longitudinal steel is ignored, indicated by the red plus signs. The LRFR rating improves for most girders when the Service III tensile stress rating is ignored. The tensile service stresses are calculated for LFR but control significantly fewer girders.

- Shear friction rating in composite concrete girders: The shear friction resistance between concrete girders and cast-in-place concrete decks was not checked in LFR. In LRFR, this is a design criteria specified in AASHTO LRFD Design Specification 5.8.4. The effect of ignoring

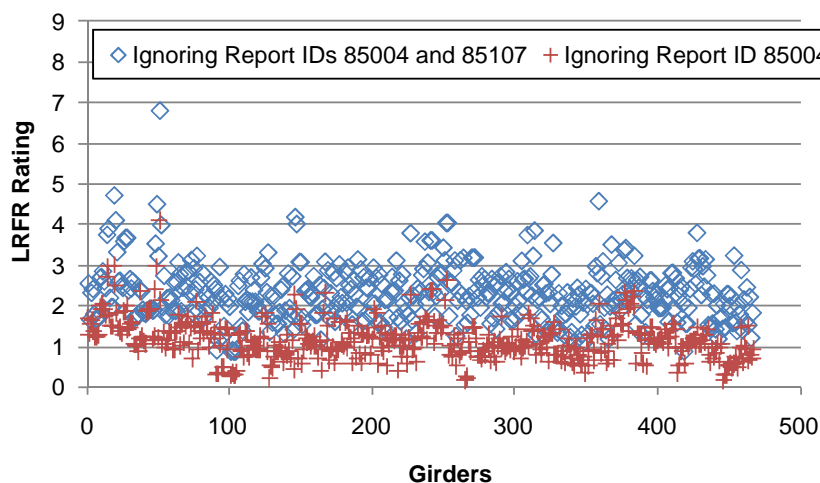


Figure 35. LRFR moment ratings for simple span prestressed concrete I-beams for WA-02 vehicle.

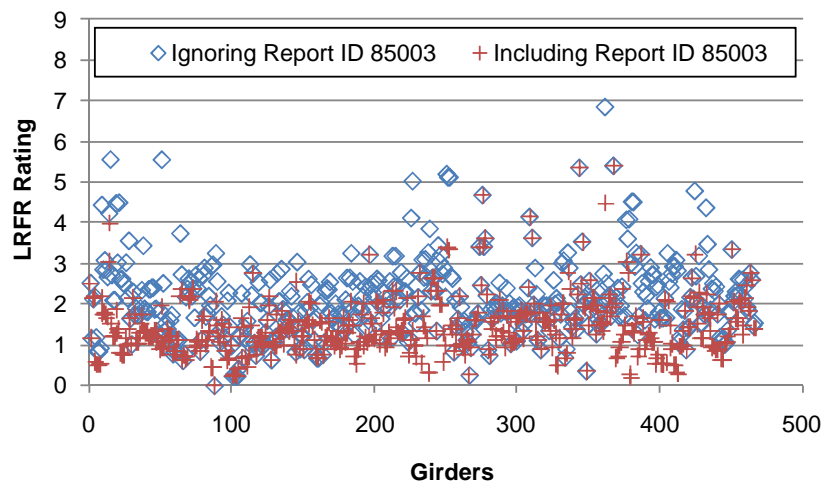


Figure 36. LRF R shear ratings for simple span prestressed concrete I-Beams for WA-02 vehicle.

this criterion is shown in Figure 36. The LRF R rating when the shear friction is ignored is generally higher than when it is included.

3.4 Effect of ADTT and Permit Type on Ratings

The effect of ADTT and type of permit on the LRF R ratings was investigated. Legal and routine permit ratings were determined for each ADTT category: greater than 5,000, equal to 1000, and less than 100. Special permit ratings were determined for each permit type within the special permit category and for each ADTT category. The special category types (listed in Table 10) are escorted, single trip mixed with other traffic, and multiple trip (less than 100 crossings) mixed with other traffic. The following sections present the percentage of girders with rating factors within each range for simple span steel bridges when all rating criteria are considered. The graphs also show the distribution of the LFR rating for comparison of the LFR rating and the possible LRF R ratings using the current load factors in the MBE. Similar graphs for the remaining bridge types included in this study are shown in the appendices. In all cases covered by Tables 18 through 28 the rating factors drop with the increase in ADTT due to the increase in the corresponding load factor for live load. Tables containing the distribution of rating factors for all bridge types and all permit and legal vehicles when all rating criteria and existing load factors are considered are shown in Appendix K.

Tables 18 through 28 show the percentage of girders with LFR and LRF R ratings within each range of rating factor, e.g., RF 0.9 to 1.0. If all girders of a specific girder type and for a particular ADTT category (ADTT < 100) have ratings greater than 1.0, then that bar will reach the top of the graph. As the

ADTT increases (which results in higher load factor), the rating factors drop, resulting in an increase in the percentage of girders in the lower ranges of the rating factors.

3.4.1 AASHTO Legal Vehicles

The results for the three AASHTO legal vehicles are shown in this section. The live load factors used are those shown in Table 9 unless a service limit state was the controlling rating, in which case the rating does not depend upon the ADTT and a live load factor of 1.0 is used.

Table 18 shows the percentage of girders within each range of ratings for the different ADTTs used in the MBE for the AASHTO Type 3 legal load. At least 90% of composite girders had ratings above 1.0 for all ADTTs for both moment and shear. Non-composite girders have poor flexure ratings for all ADTT levels, but approximately 90% have satisfactory ratings for shear.

Table 19 shows the percentage of girders within each range for the different ADTTs used in the MBE for the AASHTO Type 3-3 legal load. At least 90% of composite girders had ratings above 1.0 for all ADTTs for both moment and shear. Non-composite girders have poor flexure ratings for all ADTT levels with only 60% of girders having a flexure rating above 1.0 for an ADTT of less than 100. Approximately 90% of non-composite girders have satisfactory ratings for shear.

Table 20 shows the percentage of girders within each range for the different ADTTs used in the MBE for the AASHTO Type 3S2 legal load. At least 90% of composite girders had ratings above 1.0 for all ADTTs for both moment and shear. Non-composite girders have poor flexure ratings for all ADTT levels with less than 60% of girders having a flexure rating above 1.0 for an ADTT of less than 100. Approximately 85–95% of non-composite girders have satisfactory ratings for shear,

Table 18. AASHTO Type 3—LFR ratings for different ADTT.

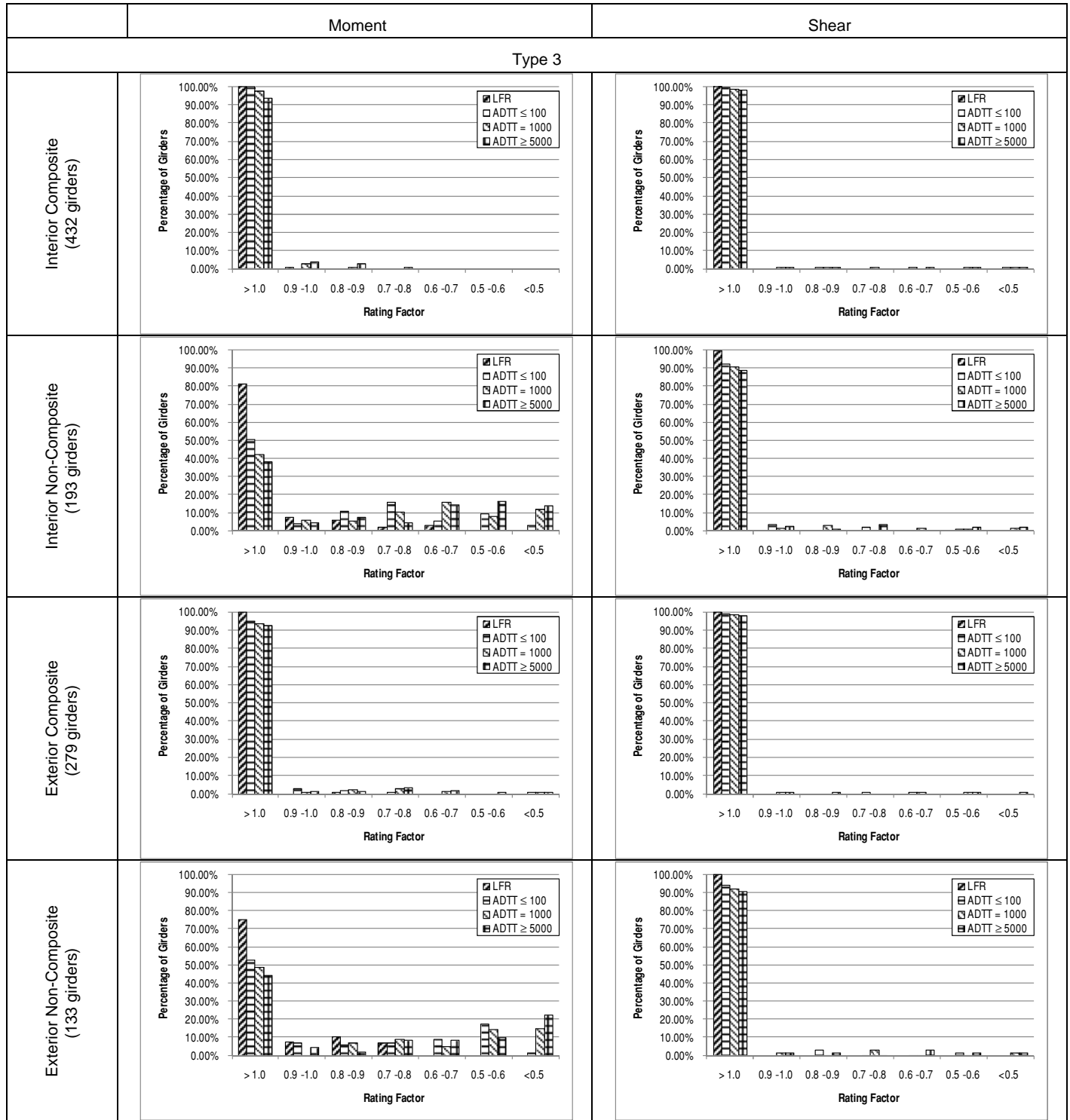


Table 19. AASHTO Type 3-3—LRFR ratings for different ADTT.

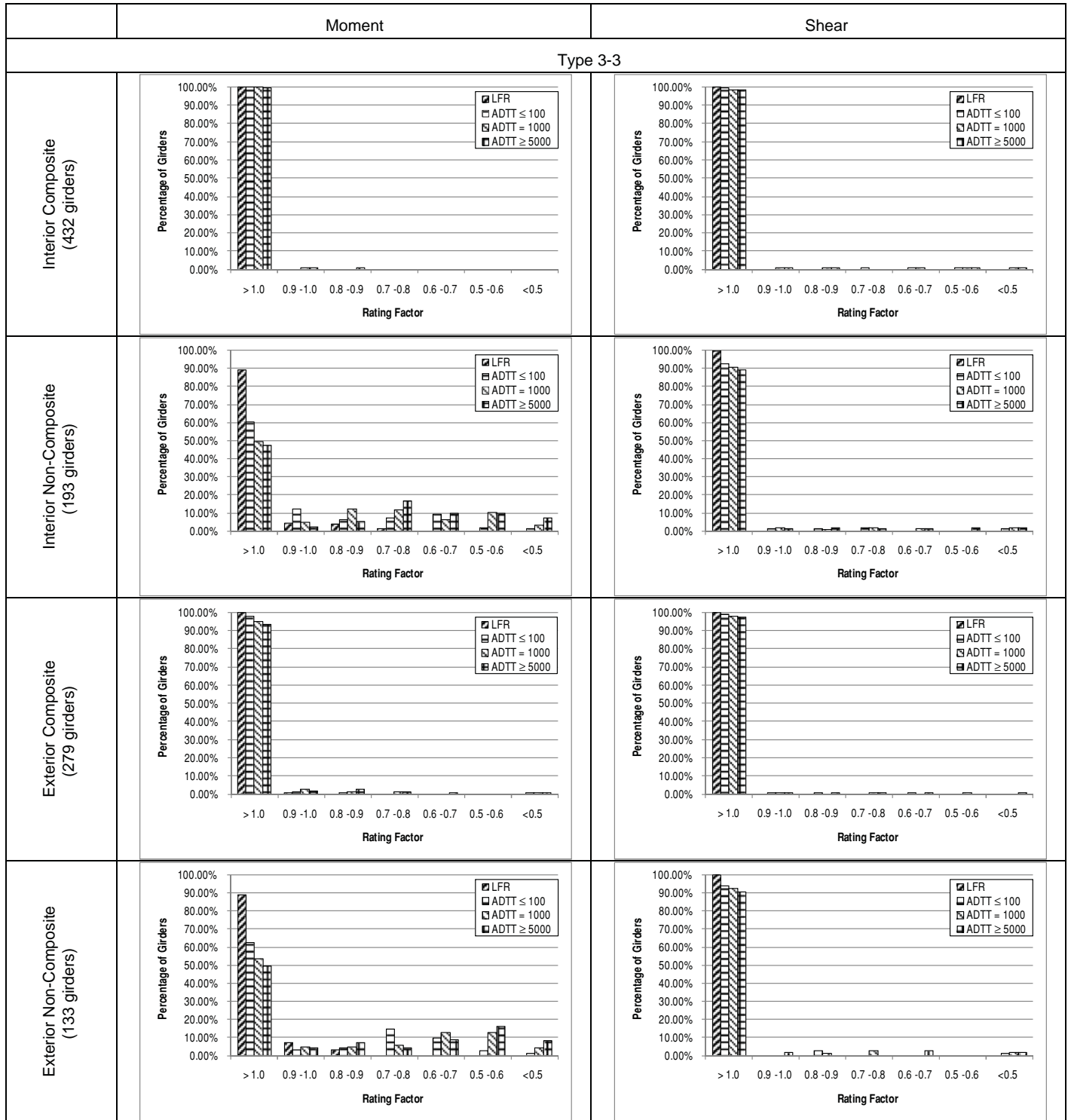
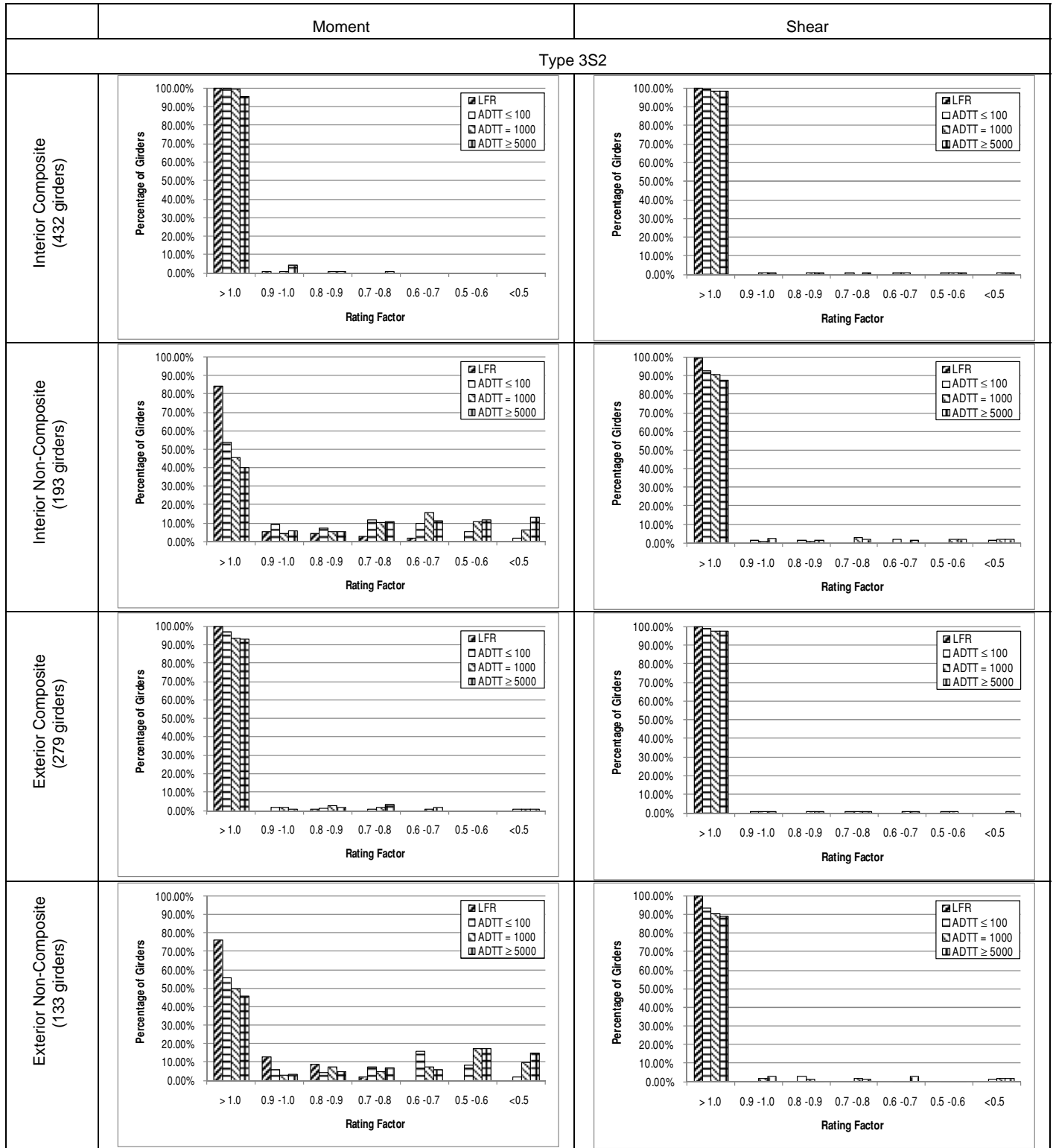


Table 20. AASHTO Type 3S2—LRF ratings for different ADTT.



with higher ADTTs having a lower percentage of girders with ratings above 1.0.

3.4.2 Routine Permit Vehicles

The results for the five vehicles treated as routine permits are shown in this section. The live load factors used are those shown in the upper portion of Table 10 unless a service limit state was the controlling rating, in which case the rating does not depend upon the ADTT and a live load factor of 1.0 is used.

Table 21 shows the percentage of girders within each range for the different ADTTs used in the MBE for the DE-07 vehicle. At least 90% of composite girders had ratings above 1.0 for all ADTTs for both moment and shear. Non-composite girders have poor flexure ratings for all ADTT levels with approximately 50% of the girders having ratings above 1.0 for an ADTT of less than 100. Approximately 80–90% of non-composite girders have satisfactory ratings for shear, depending upon ADTT.

Table 22 shows the percentage of girders within each range for the different ADTTs used in the MBE for the FL-04 vehicle. At least 90% of composite girders had ratings above 1.0 for all ADTTs for both moment and shear. Non-composite girders have poor flexure ratings for all ADTT levels with approximately 50% of the girders having ratings above 1.0 for an ADTT of less than 100. Approximately 80–90% of non-composite girders have satisfactory ratings for shear, depending upon ADTT.

Table 23 shows the percentage of girders within each range for the different ADTTs used in the MBE for the NC-21 vehicle. Approximately 90% of composite girders had ratings above 1.0 for all ADTTs for both moment and shear. Non-composite girders have poor flexure ratings for all ADTT levels with less than 50% of exterior girders and less than 40% of interior girders having ratings above 1.0 for an ADTT of less than 100. Approximately 80–90% of non-composite girders have satisfactory ratings for shear, depending upon ADTT.

Table 24 shows the percentage of girders within each range for the different ADTTs used in the MBE for the NM-04 vehicle. Approximately 90% of composite girders had ratings above 1.0 for all ADTTs for both moment and shear. Non-composite girders have poor flexure ratings for all ADTT levels with less than 60% of exterior girders and 50% of interior girders having ratings above 1.0 for an ADTT of less than 100. Approximately 90% of non-composite girders have satisfactory ratings for shear, depending upon ADTT.

Table 25 shows the percentage of girders within each range for the different ADTTs used in the MBE for the TX-04 vehicle. Approximately 80–90% of composite girders had ratings above 1.0 for all ADTTs for moment and more than 90% have ratings above 1.0 for shear. Non-composite girders have poor flexure ratings for all ADTT levels with less than 50% of exterior girders and less than 40% of interior girders having

ratings above 1.0 for an ADTT of less than 100. Approximately 80–90% of non-composite girders have satisfactory ratings for shear, depending upon ADTT.

3.5 Special Permit Vehicles

The results for the three vehicles treated as special permits are shown in this section. The live load factors used are those shown in the lower portion of Table 10 unless a service limit state was the controlling rating, in which case the rating does not depend upon the ADTT and a live load factor of 1.0 is used.

Table 26 shows the percentage of girders within each range for the different ADTTs used in the MBE for the IL-01 vehicle. At least 90% of composite girders had ratings above 1.0 for all categories for both moment and shear, except for the flexure rating in exterior girders for multiple trip permits with ADTTs of 1,000 and 5,000. Non-composite girders have poor flexure ratings for all ADTT levels with less than 60% of interior girders having ratings above 1.0 when treated as an escorted permit. Exterior non-composite girders have approximately 50% of girders with ratings above 1.0 for flexure. Approximately 80–90% of non-composite girders have satisfactory ratings for shear, depending upon ADTT.

Table 27 shows the percentage of girders within each range for the different ADTTs used in the MBE for the OR-06 vehicle. Approximately 90% of composite girders had ratings above 1.0 for all ADTTs for both moment and shear with the exception of exterior girders assuming multiple trip permits where 80–85% of girders had ratings above 1.0. Non-composite girders have poor flexure ratings for all ADTT levels with approximately 50–55% of the girders having ratings above 1.0 assuming the vehicle was escorted. Approximately 80–90% of non-composite girders have satisfactory ratings for shear, depending upon permit type.

Table 28 shows the percentage of girders within each range for the different permit types used in the MBE for the WA-02 vehicle. Approximately 90% of composite girders had ratings above 1.0 for all permit types for shear. For flexure, a minimum of 88% of girders have satisfactory ratings for interior girders and a minimum of 75% of exterior girders have ratings above 1.0. Non-composite girders have poor flexure ratings for all ADTT levels with a minimum of 20% of the girders having ratings above 1.0 for multiple trip permits with an ADTT greater than 5,000. Between 80% and 90% of non-composite girders have satisfactory ratings for shear, depending upon permit type and ADTT. The number of girders with ratings under 0.5 increases significantly for non-composite girders depending upon the permit type. As an example, for a single-trip escorted permit between 5 and 10% of girders have a rating less than 0.5 while for a multiple trip permit with an ADTT equal to or greater than 5,000 more than 40% of girders have a rating of less than 0.5.

Table 21. DE-07—LRFR ratings for different ADTT.

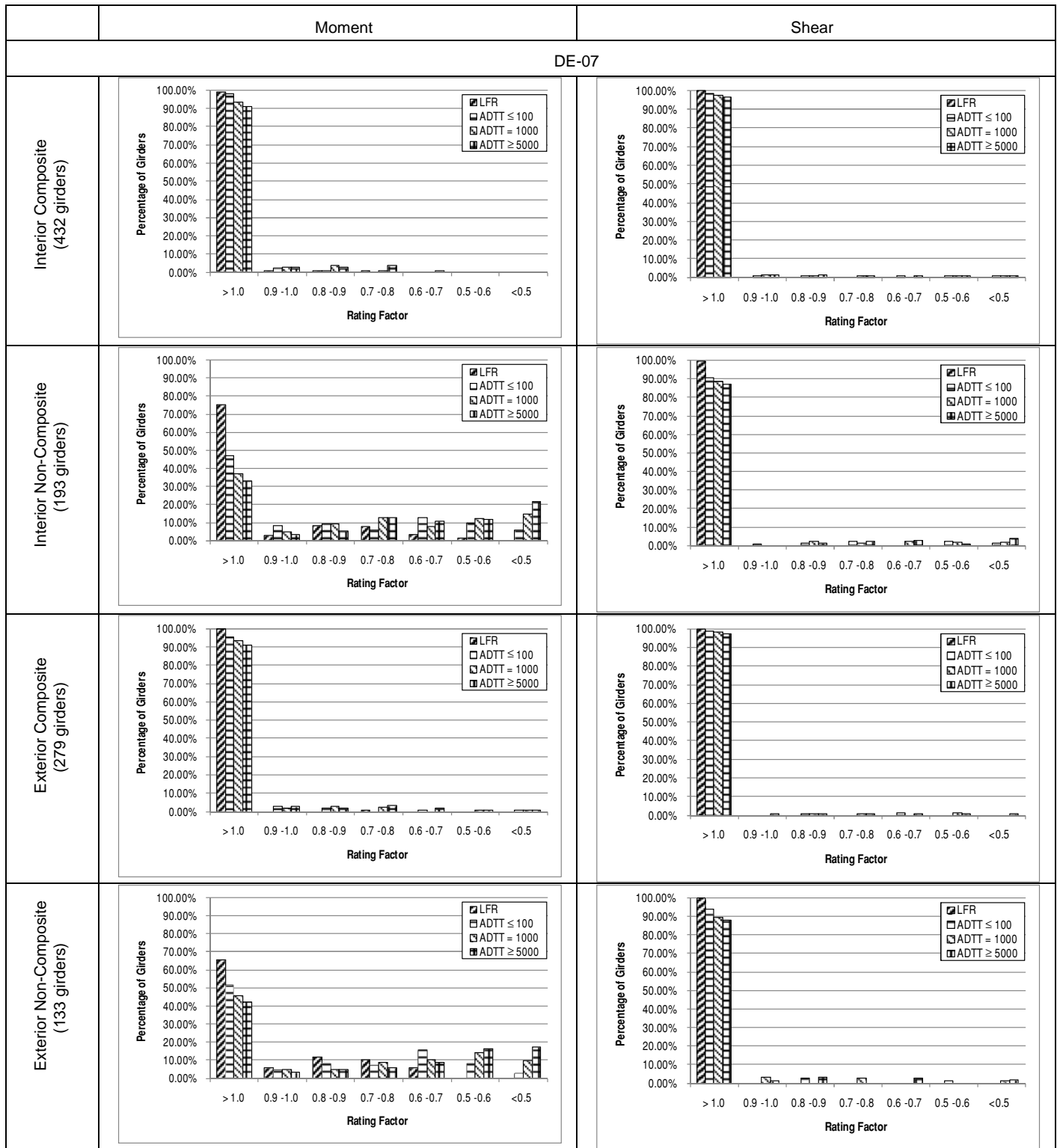


Table 22. FL-04—LRFR ratings for different ADTT.

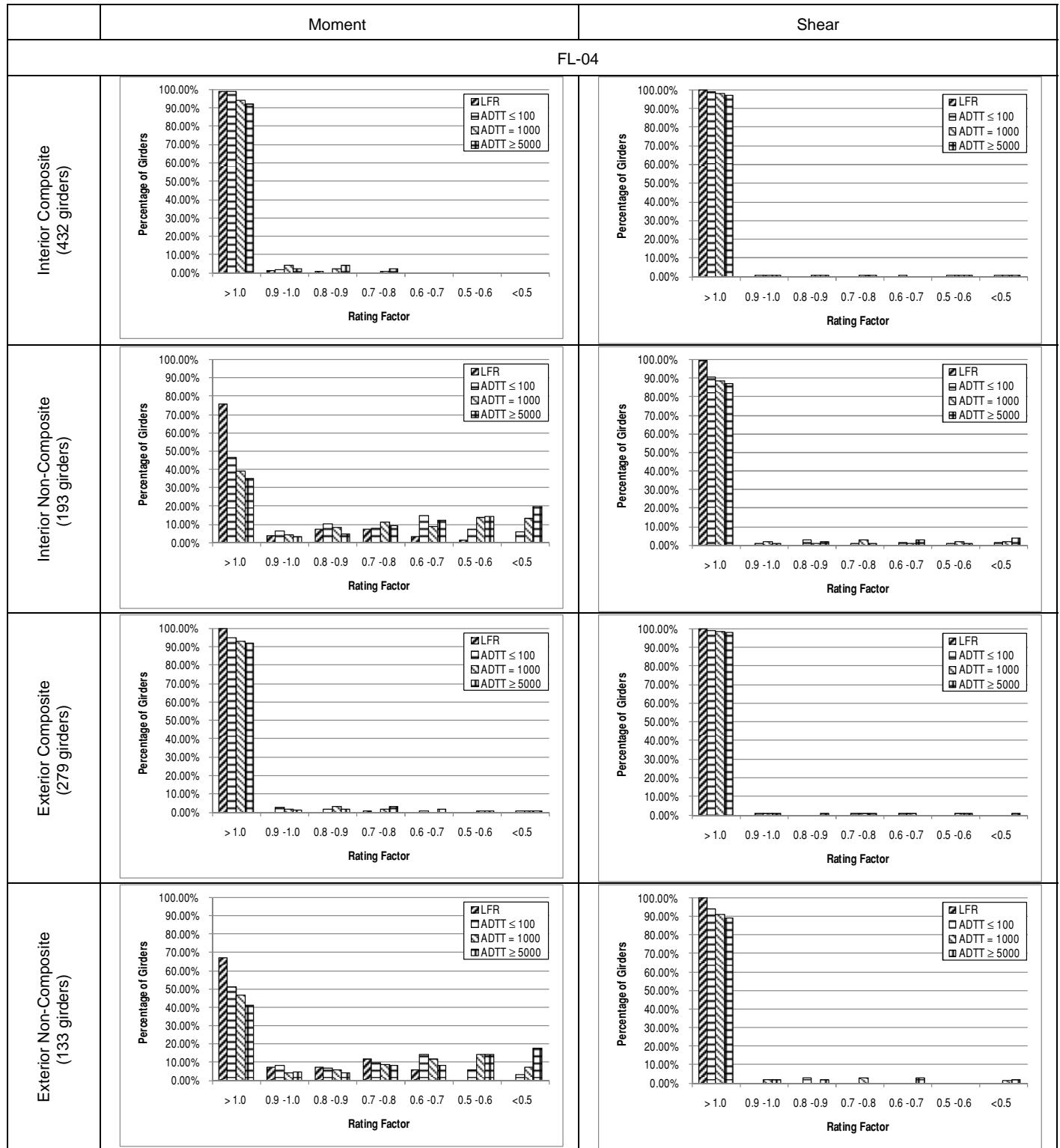


Table 23. NC-21—LRFR ratings for different ADTT.

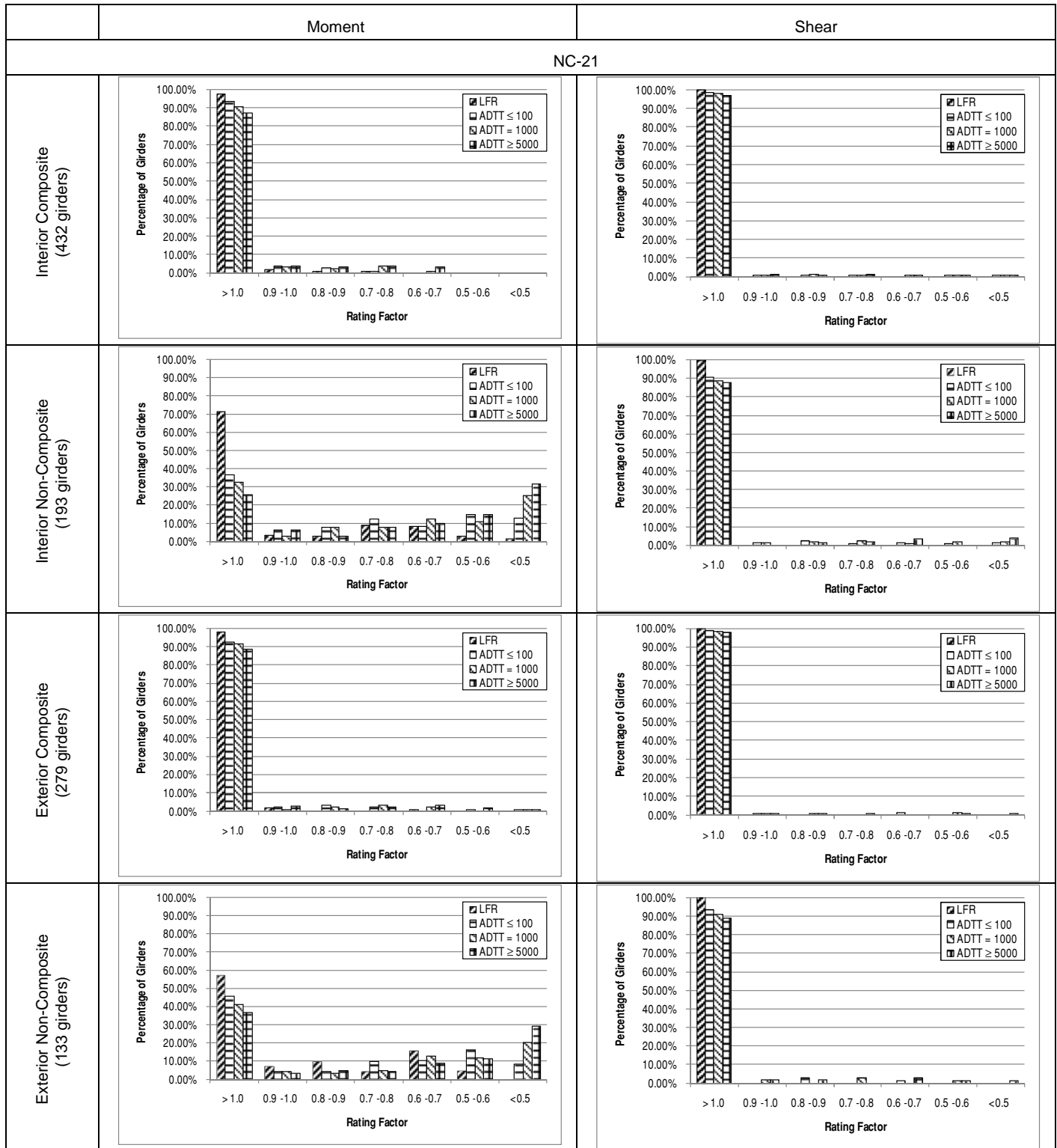


Table 24. NM-04—LRFR ratings for different ADTT.

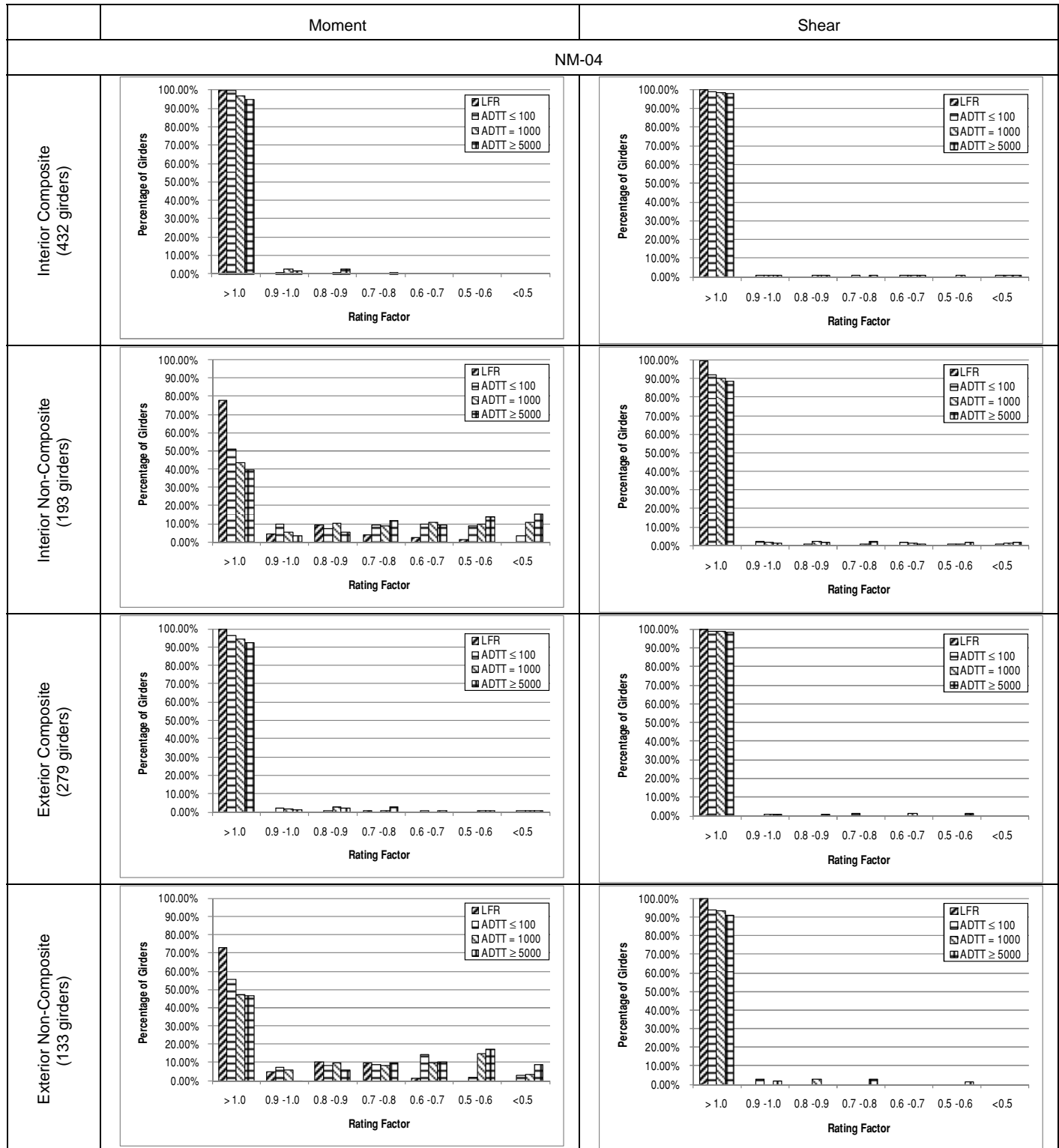


Table 25. TX-04—LFR ratings for different ADTT.

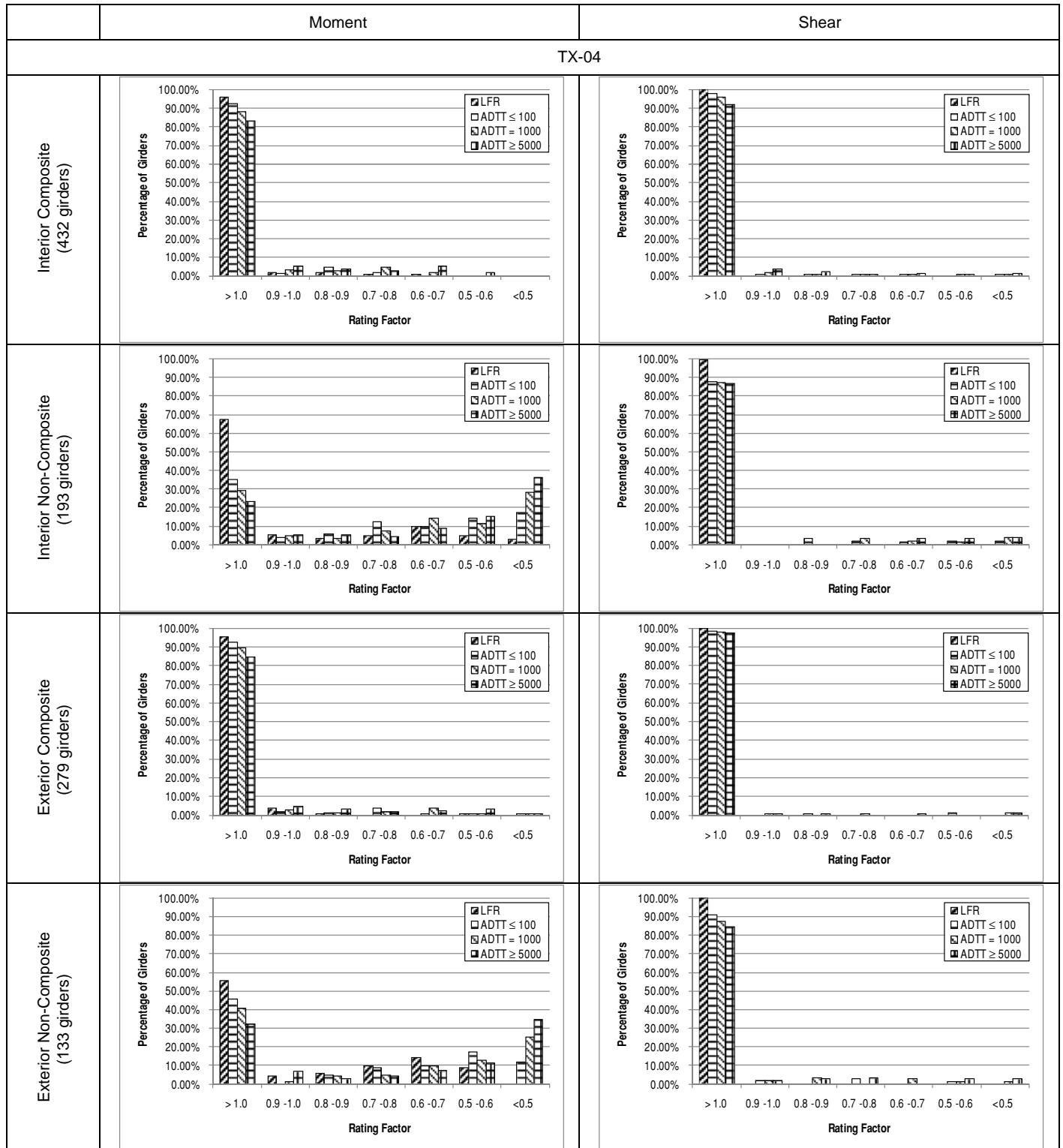


Table 26. IL-01—LRFR ratings for different ADTT.

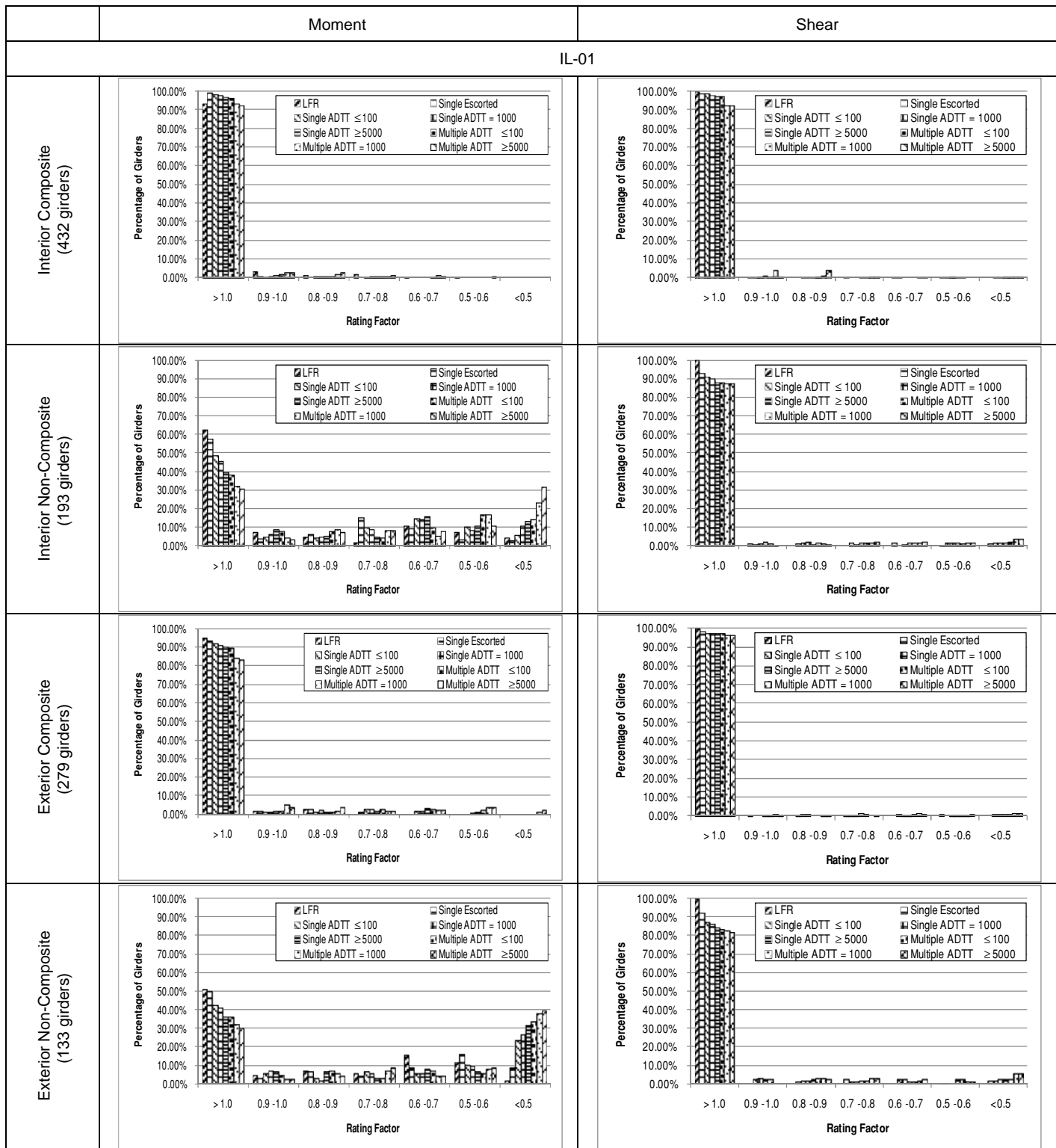


Table 27. OR-06—LRF ratings for different ADTT.

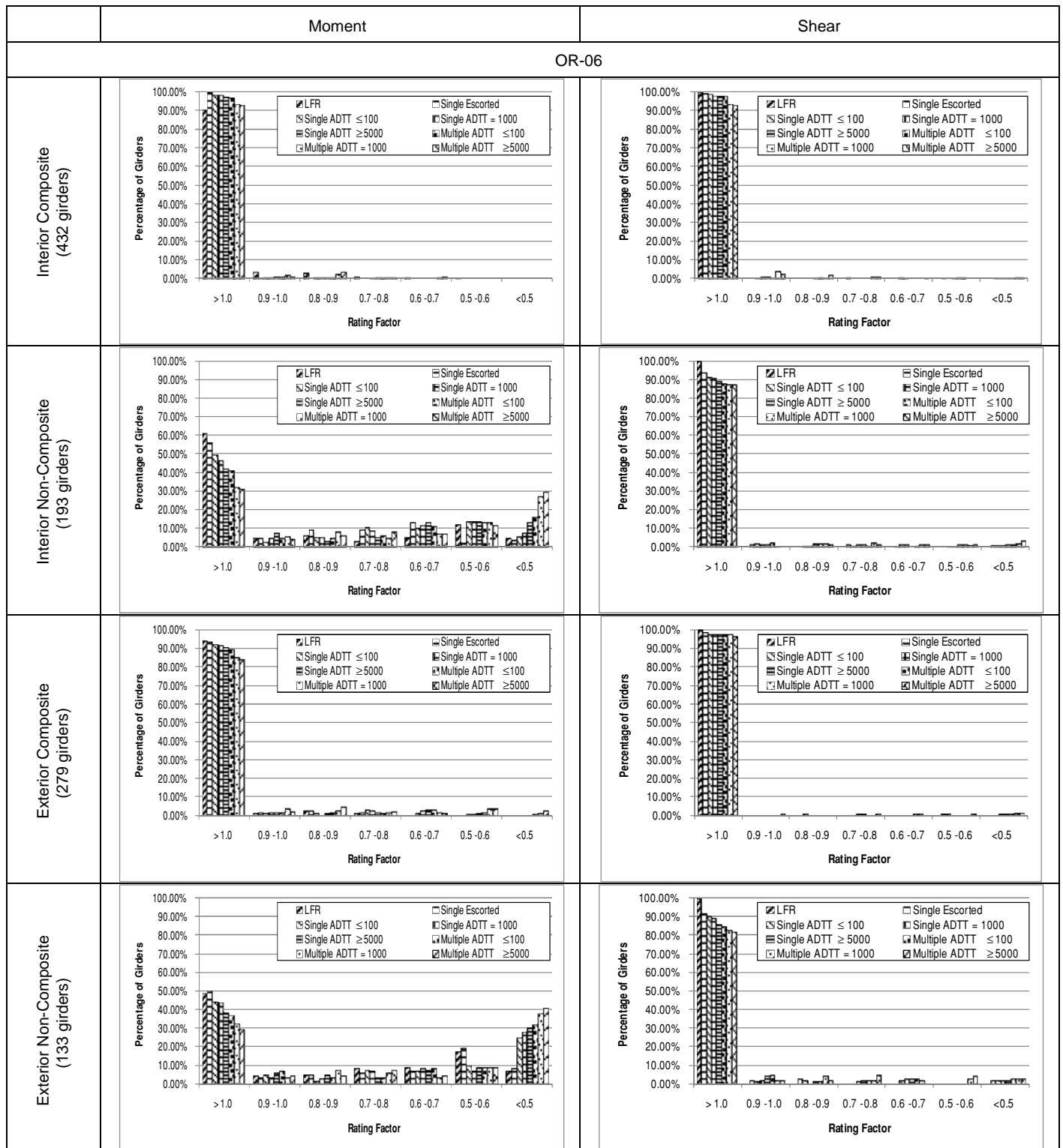
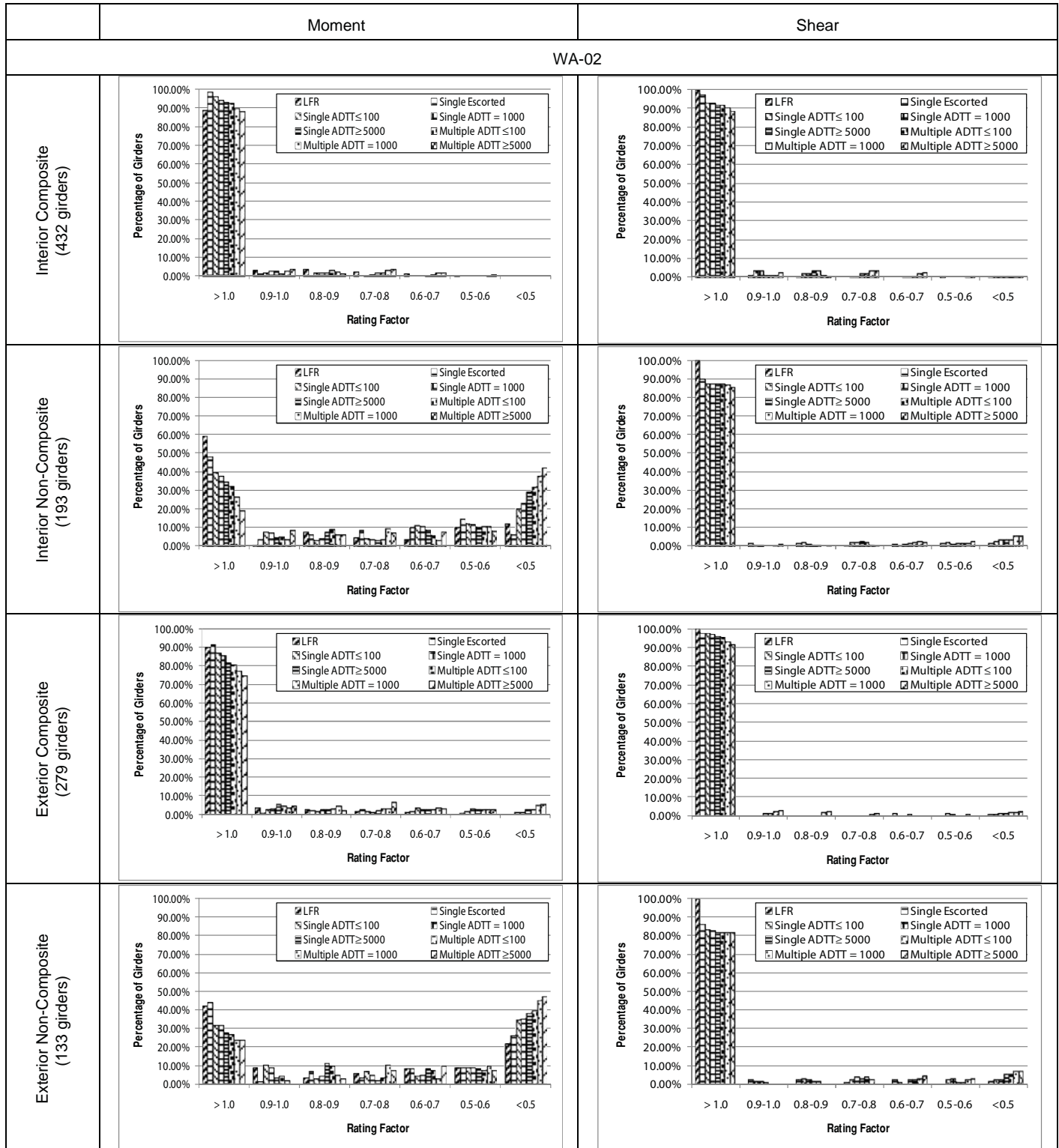


Table 28. WA-02—LFR ratings for different ADTT.



As expected, as the ADTT increases and the associated load factor increases, the number of girders that will pass rating decreases. The permit type and ADTT affect flexural ratings more than they affect shear ratings. Shear is not affected as much because shear resistance is generally larger than the maximum applied shear and would require a significant increase in load to reduce the rating to a level less than 1.0. Flexure is significantly affected because the cross-section is typically designed for the maximum expected loads and the load factor used in the design is possibly smaller than that used in rating or the rating vehicle is heavier, resulting in ratings less than 1.0.

3.6 Effect of LRFR Rating on Inventory Rating

The effect of switching from LFR to LRFR on the allowable truck weight for those girders with LRFR ratings less than 1.0 was investigated. The following graphs provide an indication on how much the weight of the HL-93 design load would have to be reduced for the LRFR rating to be the same as the LFR rating. The weight of the HL-93 load would not necessarily need to be reduced by the same percentage to achieve a satisfactory rating (rating factor equals 1.0), as in some cases the LFR rating is significantly larger than the LRFR rating. All graphs shown are for the HL-93 design load and are plotted as general bridge types; the graphs are not divided up between interior and exterior or composite and non-composite. Tables and graphs for the 11 legal and permit vehicles considered are shown for each bridge type in Appendix L.

The percentages shown in Tables 29 through 36 are based upon the total number of girders for each superstructure type,

not upon the number of girders with ratings less than 1.0. Columns 4 and 5 for moment and Columns 8 and 9 for shear indicate the number of girders with LRFR ratings less than 1.0 and LFR ratings less than 1.0 (Columns 4 and 8) and LFR ratings greater than or equal to 1.0 (Columns 5 and 9). Figures 37 through 44 typically show two series; the left series is based upon the number of girders with LRFR ratings less than 1.0. The right series is based upon the number of girders with LFR ratings less than 1.0. The LFR series indicates all girders with LFR ratings less than 1.0, not just those with both LFR and LRFR ratings less than 1.0.

3.6.1 Simple Span Steel Girder Bridges

Table 29 and Figure 37 show the distribution of rating factor ratio for girders with LRFR ratings less than 1.0 and also for LFR ratings less than 1.0 in tabular and graphical format for the design vehicle and simple span steel girder bridges. Both LFR and LRFR are shown so a comparison can be made as to which method will produce lower ratings.

For simple span steel girder bridges, there are 1,037 total girders with 346 girders having LRFR flexure ratings less than 1.0 and 252 girders with LFR flexure ratings less than 1.0. Of the simple span steel girders, 5.7% have LRFR flexure rating factors less than 1.0 yet their LRFR ratings are greater than LFR ratings. Comparatively, 9.8% of the simple span steel girders have LFR flexure ratings less than 1.0, yet their LRFR rating is greater than the LFR rating. For flexure in simple span steel girder bridges, LRFR produces more low ratings than LFR. The number of girders with ratings less than 1.0 increased by 4.9% of the total inventory when switching from LFR to LRFR.

Table 29. Distribution of rating factor ratios for simple span steel girders with LRFR ratings less than 1.0 for HL-93 design load.

LRFR/LFR Range	Moment				Shear			
	LRFR		LFR		LRFR		LFR	
	No. of Girders	% of Total Inventory (1037)	No. of Girders < 1.0	No. of Girders > 1.0	No. of Girders	% of Total Inventory (1037)	No. of Girders < 1.0	No. of Girders > 1.0
0.0-0.09	0	0.0%	0	0	9	0.9%	0	9
0.1-0.19	2	0.2%	2	0	42	4.1%	0	42
0.2-0.29	15	1.4%	1	14	25	2.4%	0	25
0.3-0.39	10	1.0%	2	8	17	1.6%	0	17
0.4-0.49	35	3.4%	9	26	7	0.7%	0	7
0.5-0.59	12	1.2%	6	6	8	0.8%	0	8
0.6-0.69	17	1.6%	10	7	5	0.5%	0	5
0.7-0.79	49	4.7%	38	11	4	0.4%	0	4
0.8-0.89	96	9.3%	77	19	0	0.0%	0	0
0.9-0.99	50	4.8%	48	2	0	0.0%	0	0
> 1.00	59	5.7%	59	0	0	0.0%	0	0
Total	346		252	93	117		0	117
Average LRFR/LFR	0.98				0.65			

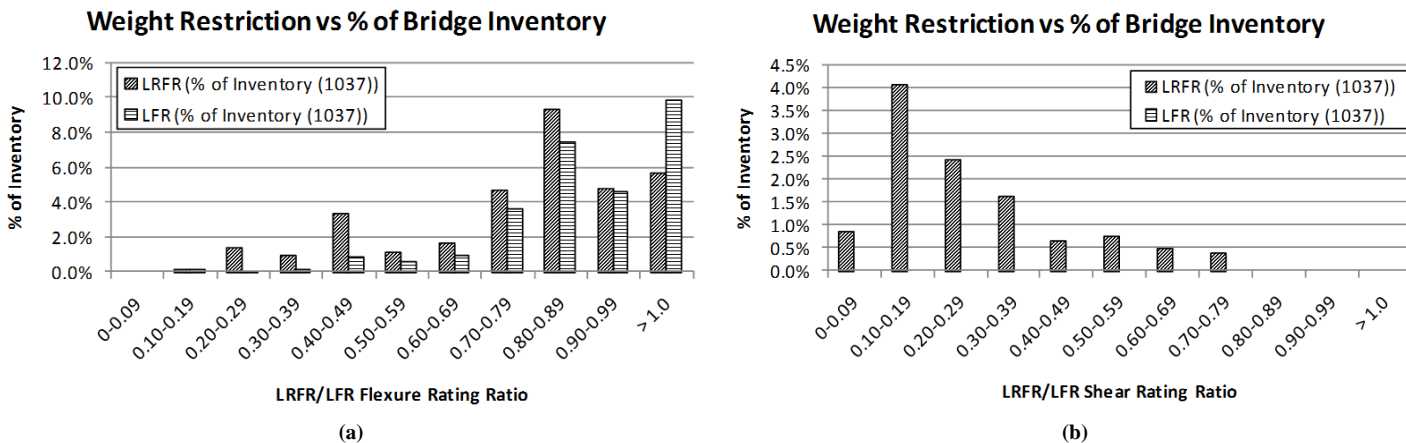


Figure 37. Distribution of rating factor ratio for simple span steel girders with LRFR rating less than 1.0 (a) moment and (b) shear for HL-93 design load.

Seventy percent of the girders with LRFR ratings less than 1.0 also have LFR ratings less than 1.0.

For shear in simple span steel girders, 117 of 1,037 girders have LRFR shear ratings less than 1.0 and no girders have LFR shear ratings less than 1.0. Ninety-three of the 100 girders with LRFR rating less than 1.0 and ratio of LRFR to LFR rating less than 0.5 are controlled by the rating for the bearing stiffener. The number of girders with ratings less than 1.0 increased by 11.3% of the total inventory when switching from LFR to LRFR.

Girders with ratio of LRFR flexure rating to LFR flexure rating less than 0.5 and LRFR ratings less than 1.0 and LFR ratings greater than 1.0 are typically spaced less than 3.5 feet apart. As mentioned earlier, it was determined that close girder spacing is one reason that the LFR and LRFR ratings are sig-

nificantly different. The difference in the rating factors is caused by the difference in distribution factor. As the ratio increases, both ratings are less than 1.0 or the LFR rating is greater than 1.0 and the LRFR rating is near 1.0

Girders with ratio of LRFR shear rating to LFR shear rating less than 0.5, with LRFR ratings less than 1.0 are controlled by the bearing stiffener rating. There are 100 girders with an LRFR to LFR ratio less than 0.5; 94 of these are controlled by the bearing stiffener rating. The other six girders are controlled by the web shear rating.

3.6.2 Continuous Span Steel Girder Bridges

Table 30 and Figure 38 show the distribution of rating factor ratio for girders with LRFR ratings less than 1.0 and also

Table 30. Distribution of rating factor ratios for continuous span steel girders with LRFR ratings less than 1.0.

LRFR/LFR Range	Moment				Shear			
	LRFR		LFR		LRFR		LFR	
	No. of Girders < 1.0	% of Total Inventory (418)	No. of Girders < 1.0	No. of Girders > 1.0	No. of Girders < 1.0	% of Total Inventory (418)	No. of Girders < 1.0	No. of Girders > 1.0
0.0-0.09	0	0.0%	0	0	7	1.7%	0	7
0.1-0.19	0	0.0%	0	0	16	3.8%	0	16
0.2-0.29	2	0.5%	1	1	20	4.8%	0	20
0.3-0.39	6	1.4%	2	4	21	5.0%	2	19
0.4-0.49	11	2.6%	3	8	6	1.4%	2	4
0.5-0.59	10	2.4%	1	9	7	1.7%	5	2
0.6-0.69	20	4.8%	7	13	5	1.2%	1	4
0.7-0.79	33	7.9%	15	18	5	1.2%	2	3
0.8-0.89	25	6.0%	9	16	7	1.7%	3	4
0.9-0.99	23	5.5%	19	4	4	1.0%	4	0
> 1.00	15	3.6%	15	0	2	0.5%	2	0
Total	145		72	73	100		21	79
Average LRFR/LFR	0.92				0.75			

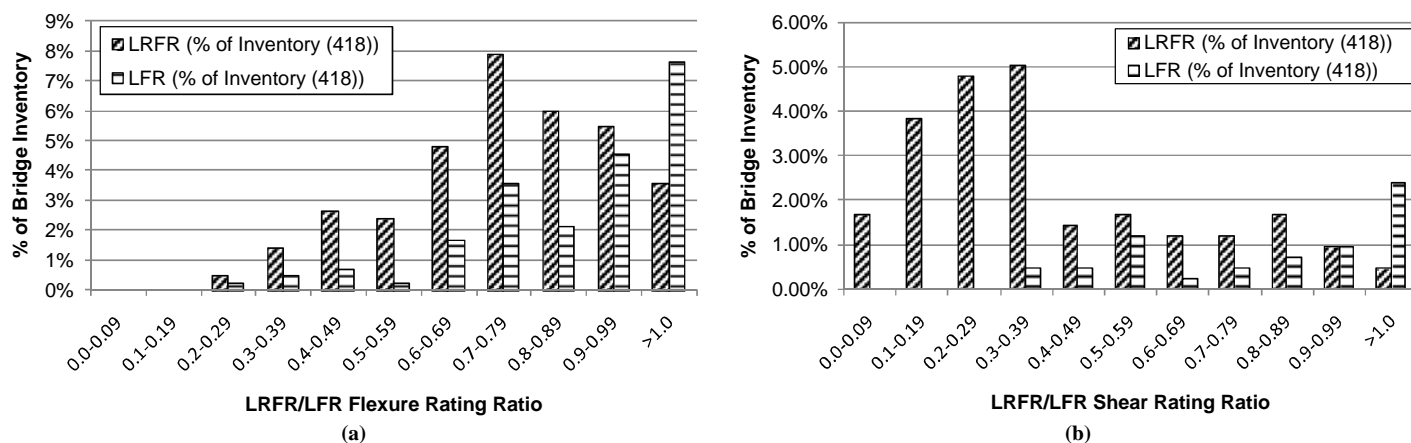


Figure 38. Distribution of rating factor ratio for continuous span steel girders with LRFR rating less than 1.0 (a) moment and (b) shear.

for LFR ratings less than 1.0 in tabular and graphical format for the design vehicle and continuous span steel girder bridges. Both LFR and LRFR are shown so a comparison can be made as to which method will produce lower ratings.

For continuous span steel girder bridges, there are 418 total girders and 145 girders with a LRFR flexure rating less than 1.0 and 72 girders with a LFR flexure rating less than 1.0. Ninety-six girders, 66% with LRFR ratings less than 1.0, have a LRFR rating that is at least 70% of the LFR rating. The number of girders with ratings less than 1.0 increased by 13.4% of the total inventory when switching from LFR to LRFR.

There are 100 girders with a LRFR shear rating less than 1.0 and 21 girders with an LFR shear rating less than 1.0. For shear, 64% of the girders have an LRFR rating less than 1.0 and have an LRFR rating that is less than 40% of the LFR rating. The number of girders with ratings less than 1.0 increased by 16.9% of the total inventory when switching from LFR to LRFR.

Girders with LRFR flexure ratings less than 1.0 and with a ratio of ratings less than 0.8 typically have factored LRFR design live loads that are 50% greater than the factored LFR design live loads when the ratings are at the same critical location. The LRFR resistance is also approximately 90% of the LFR resistance for the same set of girders.

Girders with LRFR shear ratings less than 1.0 and a ratio of ratings less than 0.6 are controlled by the bearing stiffener rating for 66 of the 77 girders. The remaining 11 girders are controlled by the web shear rating. The LRFR and LFR resistances are generally similar when the critical locations are the same.

3.6.3 Simple Span Prestressed I-Girder Bridges

Table 31 and Figure 39 show the distribution of rating factor ratio for girders with LRFR ratings less than 1.0 and

also for LFR ratings less than 1.0 in tabular and graphical format for the design vehicle and simple span prestressed I-girder bridges. Both LFR and LRFR are shown so a comparison can be made as to which method will produce lower ratings.

For simple span prestressed concrete I-girder bridges, there are 467 total girders; 289 girders have an LRFR flexure rating less than 1.0 and 25 girders have an LFR flexure rating less than 1.0. This is an increase of 57% of the total inventory having a rating of less than 1.0. This is due to the requirement to check the stress in the longitudinal steel near the support (AASHTO LRFD Article 5.8.3.5); 215 of the 289 girders with LRFR ratings less than 1.0 are controlled by the longitudinal steel rating. The remaining girders are controlled by the stress at the bottom of the beam under service loads.

There are 193 girders with LRFR shear ratings less than 1.0 and 33 girders with LFR shear ratings less than 1.0. Most girders have an LRFR rating in the range of 30–80% of the LFR rating. This is an increase of 32.9% of the total inventory not passing rating when switching from LFR to LRFR. Seventy-five percent of the girders with LRFR shear ratings less than 1.0 for the design vehicle are controlled by the shear friction rating; this is rating of the stress at the interface between the girder and the slab.

For flexure, the factored live load moment increased by approximately 40% for the routine permit and legal vehicles, 25% for the design vehicle, and decreased by 10% for the special permit vehicles. The change in live load moment for the legal, routine permit, and special permit vehicles was accompanied by a decrease of approximately 5% in moment resistance. The factored LRFR live load shear increased by 25 to 95% over the factored LFR live load shear. The increase in live load was accompanied by a similar increase in shear resistance.

Table 31. Distribution of rating factor ratio for simple span prestressed concrete I-girders with LRFR rating less than 1.0.

LRFR/LFR Range	Moment				Shear			
	LRFR		LFR		LRFR		LFR	
	No. of Girders < 1.0	% of Total Inventory (467)	No. of Girders < 1.0	No. of Girders > 1.0	No. of Girders < 1.0	% of Total Inventory (467)	No. of Girders < 1.0	No. of Girders > 1.0
0.0-0.09	2	0.4%	0	2	1	0.2%	1	0
0.1-0.19	2	0.4%	0	2	7	1.5%	0	7
0.2-0.29	8	1.7%	2	6	8	1.7%	0	8
0.3-0.39	33	7.1%	4	29	24	5.1%	0	24
0.4-0.49	53	11.4%	1	52	29	6.2%	2	27
0.5-0.59	54	11.6%	2	52	39	8.4%	4	35
0.6-0.69	81	17.3%	6	75	34	7.3%	9	25
0.7-0.79	42	9.0%	7	35	34	7.3%	9	25
0.8-0.89	12	2.6%	2	10	13	2.8%	5	8
0.9-0.99	1	0.2%	0	1	2	0.4%	1	1
> 1.00	1	0.2%	1	0	2	0.4%	2	0
Total	289		25	264	193		33	160
Average LRFR/LFR	0.62				0.78			

3.6.4 Simple Span Prestressed Box Girder Bridges

Table 32 and Figure 40 show the distribution of rating factor ratio for girders with LRFR ratings less than 1.0 and also for LFR ratings less than 1.0 in tabular and graphical format for the design vehicle and simple span prestressed box girder bridges. Both LFR and LRFR are shown so a comparison can be made as to which method will produce lower ratings.

For simple span prestressed concrete box-girder bridges, there are 377 total girders with 156 girders having LRFR flexure ratings less than 1.0 and 48 girders with LFR flexure ratings less than 1.0. This is an increase of 27.3% of the total inventory that will not pass the rating when switching from LFR to

LRFR. Fifty-five of the 156 girders with LRFR flexure ratings less than 1.0 are controlled by the longitudinal steel rating. Ninety-eight girders are controlled by the stress at the bottom of beam under service loads.

There are 72 girders with LRFR shear ratings less than 1.0 and 15 girders with LFR shear ratings less than 1.0. This is an increase of 13.8% of the total inventory that will not pass the rating when switching from LFR to LRFR. Ten of the 72 girders with LRFR ratings less than 1.0 are controlled by the shear friction rating; this is the stress at the interface between the cast-in-place slab and girder for those with composite decks.

The increase in the number of girders with LRFR flexure ratings less than 1.0 is due to increased live loads. For girders with LRFR ratings less than 1.0 and ratio of LRFR rating

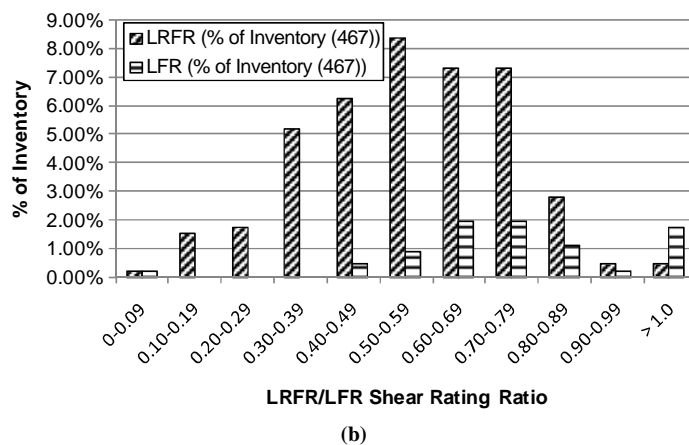
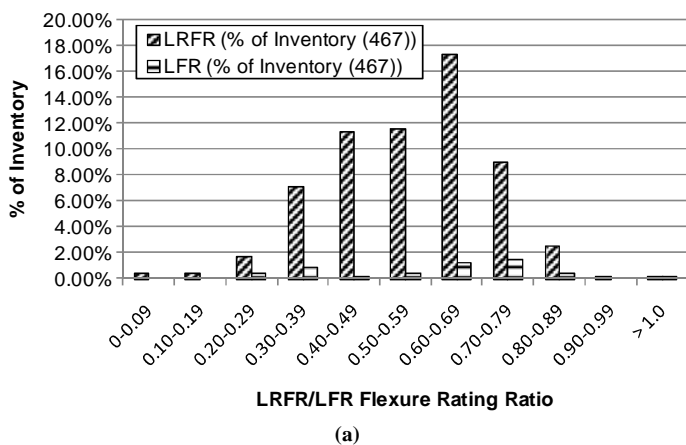


Figure 39. Distribution of rating factor ratio for simple span prestressed concrete I-Girders with LRFR rating less than 1.0 (a) moment and (b) shear.

Table 32. Distribution of rating factor ratio for simple span prestressed concrete box girders with LRFR rating less than 1.0.

LRFR/LFR Range	Moment				Shear			
	LRFR		LFR		LRFR		LFR	
	No. of Girders <1.0	% of Total Inventory (377)	No. of Girders <1.0	No. of Girders >1.0	No. of Girders <1.0	% of Total Inventory (377)	No. of Girders <1.0	No. of Girders >1.0
0.0-0.09	3	0.8%	0	3	2	0.5%	1	1
0.1-0.19	2	0.5%	2	0	1	0.3%	0	1
0.2-0.29	4	1.1%	1	3	8	2.1%	0	8
0.3-0.39	9	2.4%	0	9	10	2.7%	1	9
0.4-0.49	13	3.4%	2	11	16	4.2%	0	16
0.5-0.59	36	9.5%	7	29	13	3.4%	5	8
0.6-0.69	29	7.7%	5	24	8	2.1%	1	7
0.7-0.79	21	5.6%	4	17	9	2.4%	4	5
0.8-0.89	19	5.0%	8	11	3	0.8%	1	2
0.9-0.99	12	3.2%	11	1	2	0.5%	2	0
> 1.00	8	0.8%	8	0	0	0.0%	0	0
Total	156		48	108	72		15	57
Average LRFR/LFR	0.76				0.93			

to LFR rating less than 0.8, the factored LRFR live loads are approximately 50% higher than the factored LFR live loads. The flexural resistance increased by approximately 10%. There are a large number of girders with LRFR shear ratings less than 1.0 for the design load, but for most other vehicles there are very few.

3.6.5 Simple Span Reinforced Concrete T-Beam Bridges

Table 33 and Figure 41 show the distribution of rating factor ratio for girders with LRFR ratings less than 1.0 and for LFR ratings less than 1.0 in tabular and graphical format for the design vehicle and simple span reinforced concrete T-beam bridges.

Both LFR and LRFR are shown such that a comparison can be made as to which method will produce lower ratings.

For simple span reinforced concrete T-beam bridges, there are 295 total girders with 269 girders having LRFR flexure ratings less than 1.0 and 131 girders having LFR flexure ratings less than 1.0. This is an increase of 46.8% of the total inventory not passing rating when switching from LFR to LRFR. Two hundred fifty-eight of the 269 girders with LRFR flexure ratings less than 1.0 are controlled by the stress in the longitudinal steel near the ends of the girders. Most girders have LRFR ratings that are between 50 and 90% of the LFR rating.

There are 194 girders with LRFR shear ratings less than 1.0 and 148 girders with LFR shear ratings less than 1.0. This is an increase of 8.4% of the total inventory not passing rating

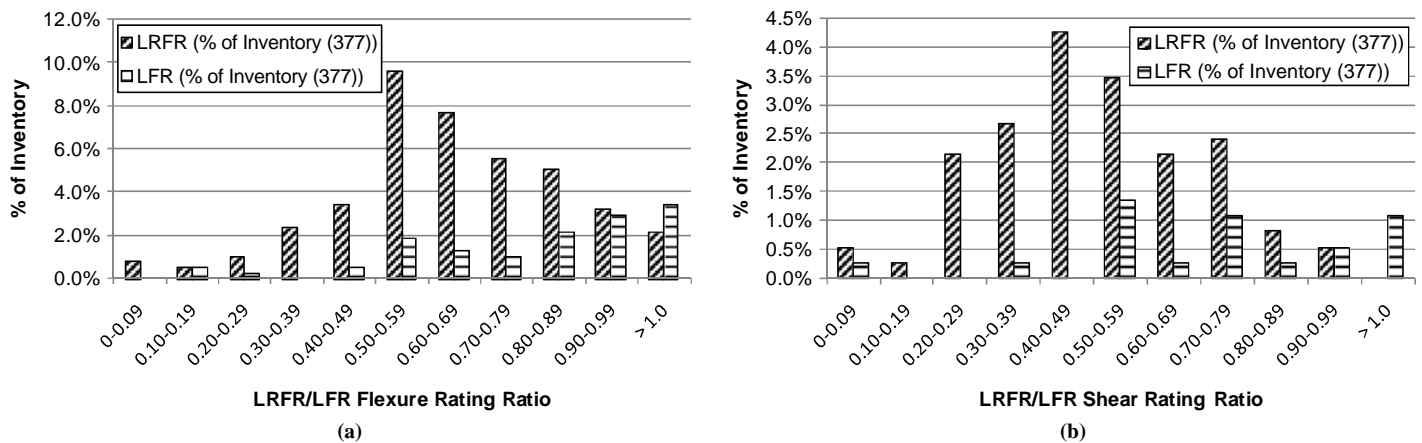


Figure 40. Distribution of rating factor ratio for simple span prestressed concrete box girders with LRFR rating less than 1.0 (a) moment and (b) shear for HL-93.

Table 33. Distribution of rating factor ratio for simple span reinforced concrete T-beams with LRFR rating less than 1.0.

LRFR/LFR Range	Moment				Shear			
	LRFR		LFR		LRFR		LFR	
	No. of Girders < 1.0	% of Total Inventory (295)	No. of Girders < 1.0	No. of Girders > 1.0	No. of Girders < 1.0	% of Total Inventory (295)	No. of Girders < 1.0	No. of Girders > 1.0
0.0-0.09	2	0.7%	1	1	3	0.5%	3	0
0.1-0.19	5	1.7%	1	4	2	0.3%	1	1
0.2-0.29	11	3.7%	2	9	2	2.1%	1	1
0.3-0.39	10	3.4%	3	7	11	2.7%	4	7
0.4-0.49	10	3.4%	2	8	18	4.2%	11	7
0.5-0.59	51	17.3%	17	34	24	3.4%	20	4
0.6-0.69	75	25.4%	36	39	32	2.1%	26	6
0.7-0.79	70	23.7%	40	30	25	2.4%	14	11
0.8-0.89	29	9.8%	23	6	22	0.8%	15	7
0.9-0.99	5	1.7%	5	0	21	0.5%	19	2
> 1.00	1	0.3%	1	0	34	0.0%	34	0
Total	269		131	138	194		148	46
Average LRFR/LFR	0.64				0.83			

when switching from LFR to LRFR. Most girders have a ratio of LRFR rating to LFR rating greater than 40%.

3.6.6 Simple Span Reinforced Concrete Slab Bridges

Table 34 and Figure 42 show the distribution of rating factor ratio for girders with LRFR ratings less than 1.0 and also for LFR ratings less than 1.0 in tabular and graphical format for the design vehicle and simple span reinforced concrete slab bridges. Both LFR and LRFR are shown so a comparison can be made as to which method will produce lower ratings.

For simple span reinforced concrete slab bridges, there are 99 total bridges with 64 bridges having LRFR flexure ratings

less than 1.0 and 27 bridges with LFR flexure ratings less than 1.0. This is an increase of 37% of total inventory that will not pass the rating when switching from LFR to LRFR. Sixty of the 64 bridges with LRFR ratings less than 1.0 are controlled by the stress in the longitudinal steel near the end of the girders; the remaining four are controlled by the strength limit state. Most bridges with LRFR ratings less than 1.0 have LRFR ratings in the range of 50–70% of the LFR rating.

There are five bridges with LRFR shear ratings less than 1.0 and no bridges with LFR shear ratings less than 1.0. This is an increase of 3% of total inventory that will not pass the rating when switching from LFR to LRFR. This also shows that for reinforced concrete slab bridges, checking shear is most likely not necessary.

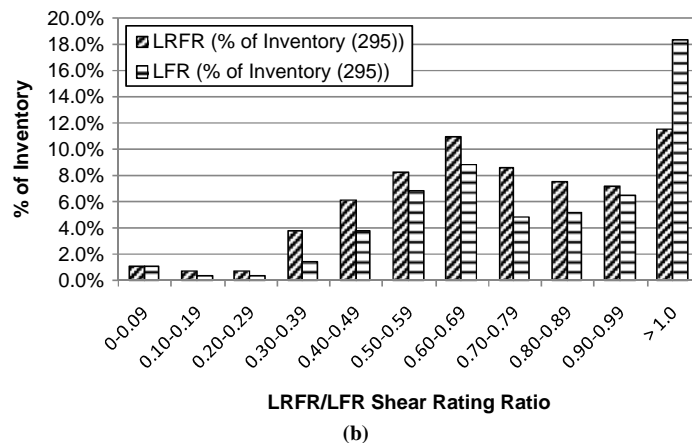
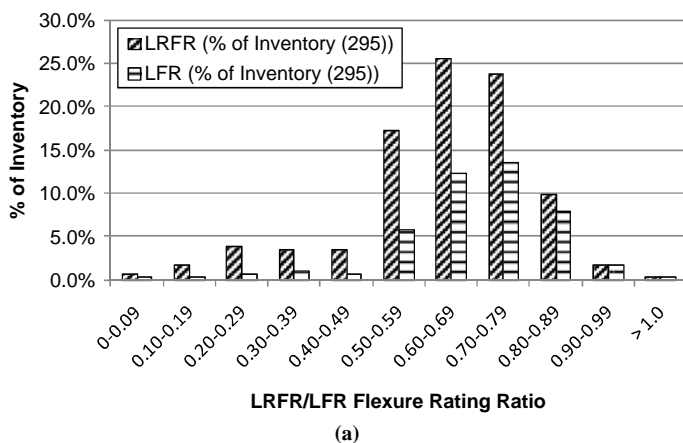


Figure 41. Distribution of rating factor ratio for simple span reinforced concrete T-beams with LRFR rating less than 1.0 (a) moment and (b) shear.

Table 34. Distribution of rating factor ratio for simple span reinforced concrete slabs with LRFR rating less than 1.0.

LRFR/LFR Range	Moment				Shear			
	LRFR		LFR		LRFR		LFR	
	No. of Girders < 1.0	% of Total Inventory (99)	No. of Girders < 1.0	No. of Girders > 1.0	No. of Girders < 1.0	% of Total Inventory (99)	No. of Girders < 1.0	No. of Girders > 1.0
0.0-0.09	2	2.0%	1	1	0	0.0%	0	0
0.1-0.19	1	1.0%	0	1	0	0.0%	0	0
0.2-0.29	0	0.0%	0	0	0	0.0%	0	0
0.3-0.39	2	2.0%	0	2	2	2.0%	0	2
0.4-0.49	4	4.0%	0	4	0	0.0%	0	0
0.5-0.59	3	3.0%	1	2	3	3.0%	0	3
0.6-0.69	15	15.2%	3	12	0	0.0%	0	0
0.7-0.79	24	24.2%	14	10	0	0.0%	0	0
0.8-0.89	8	8.1%	3	5	0	0.0%	0	0
0.9-0.99	5	5.1%	5	0	0	0.0%	0	0
> 1.00	0	0.0%	0	0	0	0.0%	0	0
Total	64		27	37	5		0	5
Average LRFR/LFR	0.73				1.27			

3.6.7 Continuous Span Reinforced Concrete Slab Bridges

Table 35 and Figure 43 show the distribution of rating factor ratio for girders with LRFR ratings less than 1.0 and also for LFR ratings less than 1.0 in tabular and graphical format for the design vehicle and continuous reinforced concrete slab bridges. Both LFR and LRFR are shown so a comparison can be made as to which method will produce lower ratings.

For continuous span reinforced concrete slab bridges, there are 105 total bridges with 96 bridges having LRFR flexure ratings less than 1.0 and 66 bridges with LFR flexure ratings less than 1.0. This is an increase of 28.6% of the continuous RC slab bridge inventory. Ninety-five of the 96 slabs with LRFR

ratings less than 1.0 are controlled by the stress in the longitudinal steel near the end of the span. Most bridges have a LRFR flexure rating in the range of 50–80% of the LFR rating.

There are 74 bridges with LRFR shear ratings less than 1.0 and 41 bridges with LFR shear rating less than 1.0. This is an increase of 29.5% of the continuous RC slab bridge inventory. Most bridges have a LRFR rating in the range of 50–80% of the LFR rating.

3.6.8 Continuous Prestressed Concrete I-Girder Bridges

Table 36 and Figure 44 show the distribution of rating factor ratio for girders with LRFR ratings less than 1.0

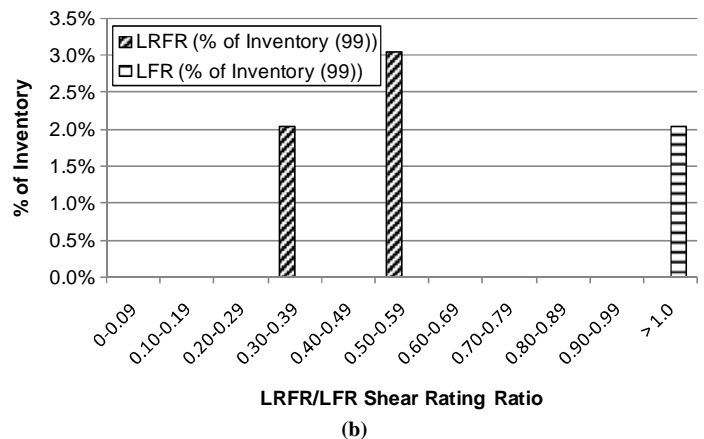
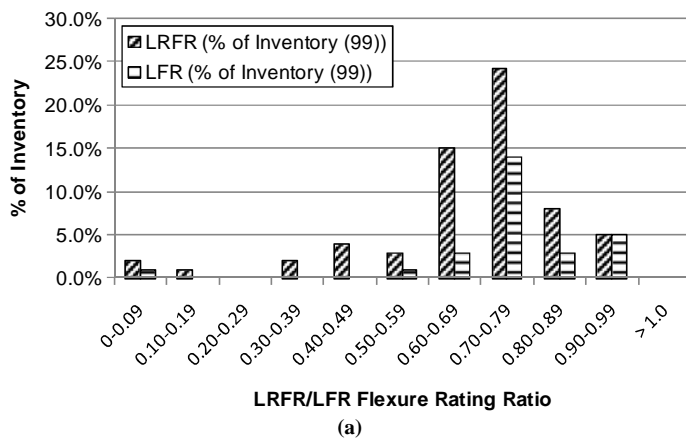


Figure 42. Distribution of rating factor ratio for simple span reinforced concrete slabs with LRFR rating less than 1.0 (a) moment and (b) shear.

Table 35. Distribution of rating factor ratio for continuous span reinforced concrete slabs with LRFR rating less than 1.0.

LRFR/LFR Range	Moment				Shear			
	LRFR		LFR		LRFR		LFR	
	No. of Girders < 1.0	% of Total Inventory (105)	No. of Girders < 1.0	No. of Girders > 1.0	No. of Girders < 1.0	% of Total Inventory (105)	No. of Girders < 1.0	No. of Girders > 1.0
0.0-0.09	6	5.7%	6	0	0	0.0%	0	0
0.1-0.19	0	0.0%	0	0	0	0.0%	0	0
0.2-0.29	0	0.0%	0	0	4	3.8%	0	4
0.3-0.39	1	1.0%	1	0	0	0.0%	0	0
0.4-0.49	2	1.9%	2	0	2	1.9%	1	1
0.5-0.59	23	21.9%	15	8	7	6.7%	3	4
0.6-0.69	36	34.3%	28	8	16	15.2%	7	9
0.7-0.79	23	21.9%	10	13	19	18.1%	9	10
0.8-0.89	5	4.8%	4	1	15	14.3%	11	4
0.9-0.99	0	0.0%	0	0	8	7.6%	7	1
> 1.00	0	0.0%	0	0	3	2.9%	3	0
Total	96		66	30	74		41	33
Average LRFR/LFR	0.62				0.79			

and also for LFR ratings less than 1.0 in tabular and graphical format for the design vehicle and simple span steel I-girder bridges. Both LFR and LRFR are shown so a comparison can be made as to which method will produce lower ratings.

For continuous, prestressed concrete I-girder bridges, there are 238 total girders with 226 girders having LRFR flexure ratings less than 1.0 and 19 girders with LFR flexure ratings less than 1.0. This is an increase of 86.9% of the continuous prestressed concrete girders inventory that will have ratings less than 1.0 when switching from LFR to LRFR. Two hundred twenty-five of the 226 girders with LRFR ratings of less than

1.0 are controlled by the stress in the longitudinal steel near the end of the girders. The LRFR ratings are generally less than 80% of the LFR rating.

There are 113 girders with LRFR shear ratings less than 1.0 and 31 girders with LFR shear ratings less than 1.0. This is an increase of 32.4% of the number of prestressed concrete girders inventory that will have insufficient ratings when switching from LFR to LRFR. Most girders have LRFR ratings that are 30–90% of the LFR rating. Ninety of the 113 girders with LRFR shear ratings of less than 1.0 are controlled by the rating of the stress at the interface between the girder and the composite deck.

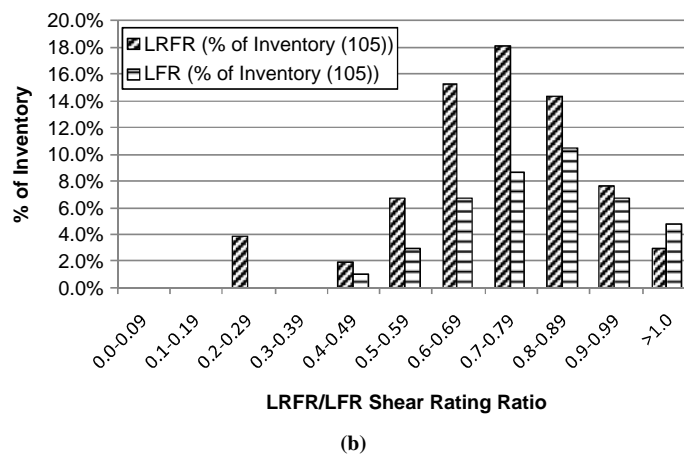
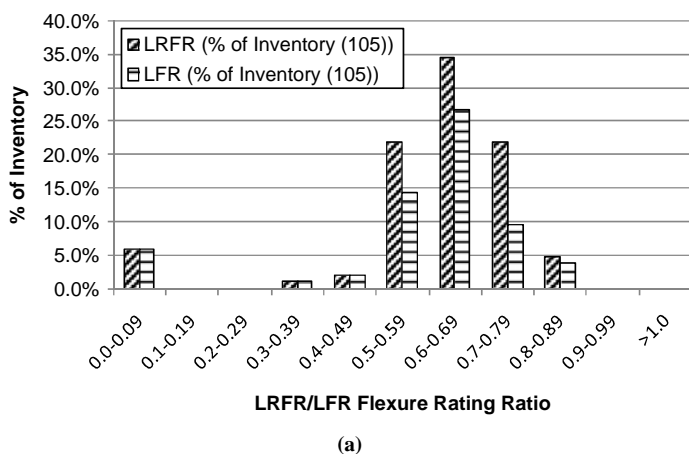


Figure 43. Distribution of rating factor ratio for continuous span reinforced concrete slabs with LRFR rating less than 1.0 (a) moment and (b) shear.

Table 36. Distribution of rating factor ratio for continuous span prestressed concrete I-girders with LRFR rating less than 1.0.

LRFR/LFR Range	Moment				Shear			
	LRFR		LFR		LRFR		LFR	
	No. of Girders < 1.0	% of Total Inventory (238)	No. of Girders < 1.0	No. of Girders > 1.0	No. of Girders < 1.0	% of Total Inventory (238)	No. of Girders < 1.0	No. of Girders > 1.0
0.0-0.09	34	14.3%	5	29	0	0.0%	0	0
0.1-0.19	14	5.9%	3	11	0	0.0%	0	0
0.2-0.29	24	10.1%	0	24	13	5.5%	0	13
0.3-0.39	27	11.3%	4	23	33	13.9%	12	21
0.4-0.49	37	15.5%	1	36	10	4.2%	2	8
0.5-0.59	41	17.2%	2	39	8	3.4%	0	8
0.6-0.69	29	12.2%	2	27	16	6.7%	3	13
0.7-0.79	18	7.6%	1	17	11	4.6%	3	8
0.8-0.89	2	0.8%	1	1	13	5.5%	3	10
0.9-0.99	0	0.0%	0	0	4	1.7%	3	1
> 1.00	0	0.0%	0	0	5	2.1%	5	0
Total	226		19	207	113		31	82
Average LRFR/LFR	0.42				0.81			

3.7 Comparison of Reliability Index to Live Load Factors

The determination of the reliability index corresponding to the live load factor needed to make the factored loads equal to the factored resistance provided insight into what level of reliability the current load factors are providing. For each vehicle, a scatter plot is created comparing the reliability index to the required live load factor for an ADTT = 1,000. The figures show the results for all types of structures used in this project. The following graphs also provide an indication into the number of girders that will have a rating less than 1.0 when the value of a certain load factor is changed. All points

to the right of a specific load factor will have a rating greater than 1.0 when this load factor is specified.

Each point in the following figures was determined using the following process:

1. The factored dead and live load force effects and factored resistance were determined using NCHRP Process 12-50 output.
2. The load factor for live load that will make the girder have a rating of 1.0 was determined.
3. The section reliability index was determined.
4. The graph was plotted using the values of the load factor and reliability index for each girder.

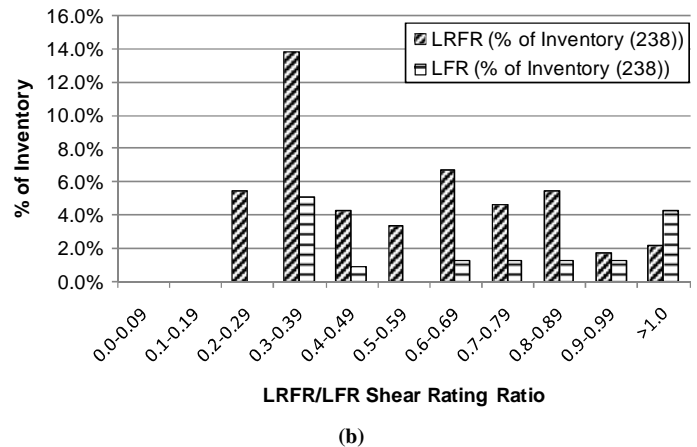
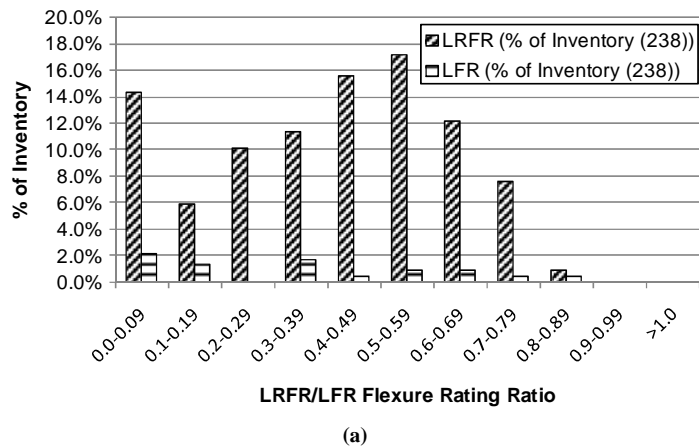


Figure 44. Distribution of rating factor ratio for continuous span prestressed concrete I-girders with LRFR rating less than 1.0 (a) moment and (b) shear.

5. For any girder, specifying a load factor equal to or less than the one calculated for this girder (Step 2) will mean that the girder will have a rating factor greater than or equal to 1.0.

3.7.1 Design Vehicle

For the HL-93 Design Load, the graphs show that a live load factor of 1.75 provides the “Inventory” or “Design” level of reliability corresponding to a reliability index of 3.5. For flexure, most types of bridges follow the same trend; the exception is the precast, prestressed box beam bridges. The prestressed box beams tend to have higher levels of reliability for low live load factors than for other types. In the graph depicting the level of reliability for shear, depending on the type of girder, the steel bridges tend to be either above or below the concrete bridges. The rolled shapes (and built-up shapes, since they were assumed to have the same statistical parameters for shear) have higher reliability because there is less variation in the web section, thus reducing the number of locations where the section is highly utilized in shear, while plate girders have more variation as shown in Figure 45.

3.7.2 Routine Permit Vehicles

From the MBE, it was determined that routine permit vehicles are to have the same level of reliability as the traditional AASHTO Operating rating. This level corresponds to a reliability index of 2.5. In the following figures, an approximate load factor is selected by following the assumed reliability index horizontally to the intersection with the data points. These values may be used to modify the load factors present in the upper portion of Table 6A.4.5.4.2a-1 in the MBE. The proposed load factors are indicated by solid lines; the current load factors and associated reliability index are indicated by a dashed line. Table 37 shows the target reliability index assumed in the development of the MBE in the third column and the current live load factor for an ADTT of 1,000 in the fourth column. Using Figure 46 through Figure 50, the reliability index corresponding to the current live load factor of 1.6 is estimated and shown in the fifth column. The sixth column shows the live load factor estimated from Figure 46 through Figure 50 to correspond to the target reliability index of 2.5.

The proposed live load factors shown in Table 37 are for an ADTT = 1,000. The current MBE assumes that multiple lanes are loaded for routine permit vehicles; the calculated live load factors assume that multiple lanes are loaded. Multiple lanes loaded implies that other traffic exists on the bridge, therefore the proposed live load factors can be used in direct comparison with those currently in the MBE.

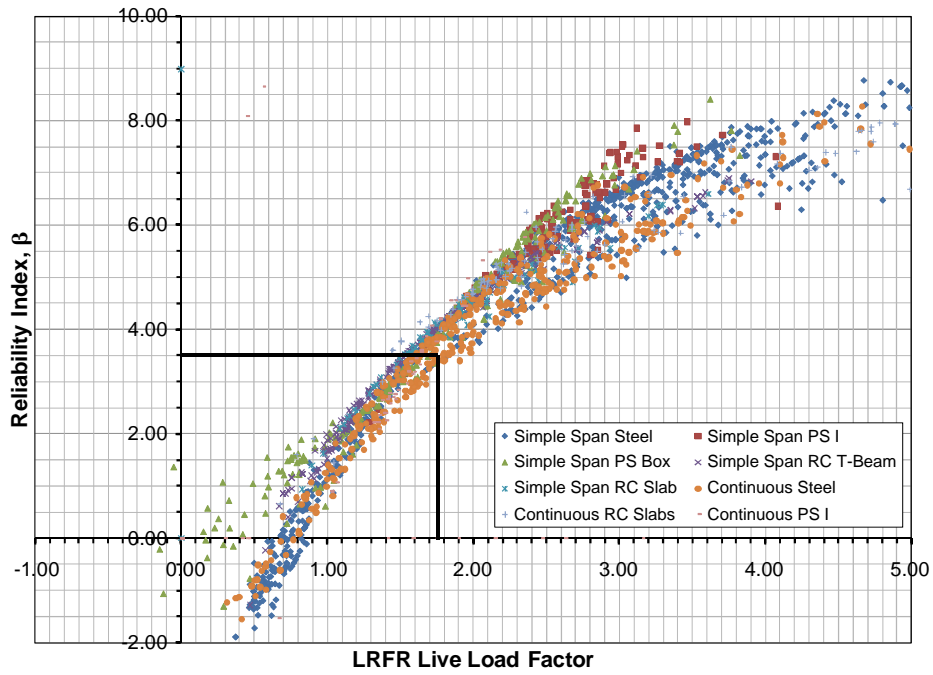
3.7.3 Special or Limited Crossing Permit Vehicles

From the MBE, it was determined that special or limited crossing permit vehicles, with the exception of the single-trip permit with an escort, are to have the same level of reliability as the design load, the traditional AASHTO Inventory rating. This level corresponds to a reliability index of 3.5. In the following graphs, a load factor is selected by following the reliability index needed horizontally to the intersection with the data points. These values may be used to modify the load factors present in the lower portion of Table 6A.4.5.4.2a-1 in the MBE. The proposed load factors are indicated by solid lines; the current load factors and associated reliability index are indicated by a dashed line. Figures 51 to 53 also show the load factor needed to obtain the operating level (reliability index of 2.5) of reliability.

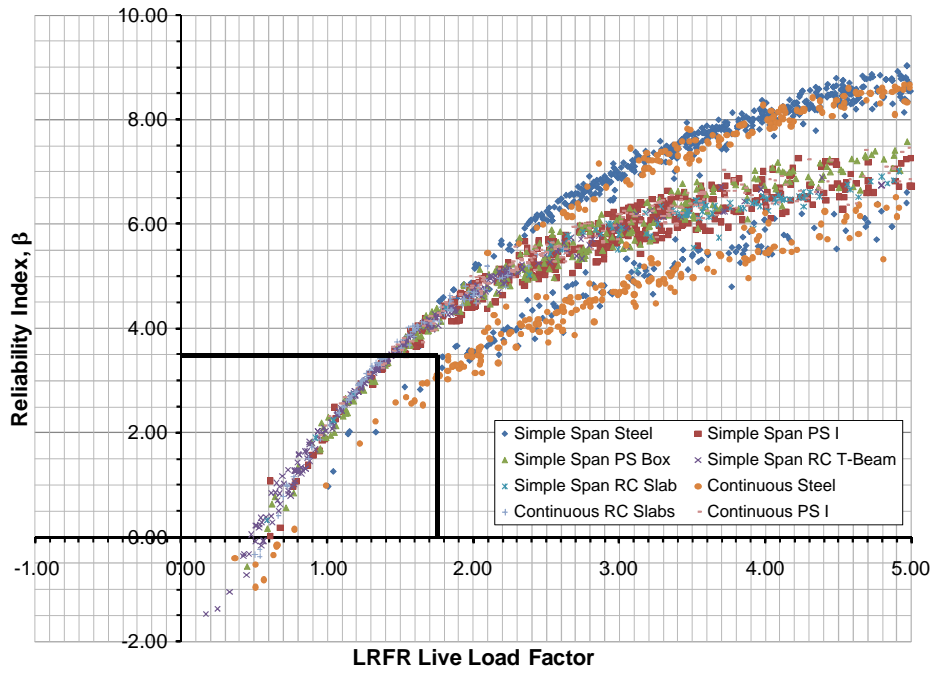
Table 38 shows the reliability index corresponding to the current live load factor of 1.4. For two vehicles, it was determined that the current live load factor does not provide the target reliability level of 3.5 for moment. To obtain a target reliability of 3.5 for all vehicles, a live load factor of at least 1.45 should be used. If the target reliability was reduced to the AASHTO Operating level, i.e., a reliability index of 2.5, the proposed live load factor would be 1.10. The rating for special permit vehicles is based upon a single lane loaded and the load factor accounts for the traffic in the second lane. As the calculated live load factors assume no other heavy vehicles significantly contribute to the maximum load effect when the permit vehicle is also positioned to produce maximum load, it is proposed that the values shown in Table 38 be used for ADTT = 100. The live load factors for ADTT = 1,000 and ADTT ≥ 5,000 are determined by increasing the live load factor for ADTT ≤ 100 by the same amount as currently shown in the MBE. This would result in proposed live load factors of 1.5 for a reliability index of 3.5 and 1.15 for a reliability index of 2.5 for ADTT = 1,000.

3.7.4 AASHTO Legal Vehicles

From the MBE, it was determined that legal vehicles are to have the same level of reliability as associated with the traditional AASHTO Operating rating. This level corresponds to a reliability index of 2.5. In the following figures, an approximate load factor is selected by following the target reliability index horizontally to the intersection with the data points. These values may be used to modify the load factors present in Table 6A.4.4.2.3a-1 in the MBE. The proposed load factors are indicated by solid lines; the current load factors and associated reliability index are indicated by a dashed line. The following figures also show the load factor needed to obtain the operating level of reliability.



(a)



(b)

Figure 45. Reliability index versus live load factor for HL-93 vehicle (a) moment and (b) shear.

Table 37. Proposed live load factors for routine permit vehicles.

Truck	Force Effect	Target β in MBE	γ_L in MBE (ADTT = 1,000)	β corresponding to current γ_L	γ_L corresponding to target β	Figure
DE-07	Moment	2.5	1.6	3.6	1.15	Figure (a)
	Shear			3.5	1.15	Figure (b)
FL-04	Moment			3.5	1.15	Figure (a)
	Shear			3.5	1.15	Figure (b)
NC-21	Moment			3.6	1.15	Figure (a)
	Shear			3.25	1.15	Figure (b)
NM-04	Moment			3.4	1.15	Figure (a)
	Shear			3.3	1.25	Figure (b)
TX-04	Moment			3.6	1.15	Figure (a)
	Shear			3.5	1.15	Figure (b)

Table 39 shows the reliability index corresponding to the current live load factor for the AASHTO legal trucks when an ADTT of 1,000 is assumed. The reliability index corresponding to a live load factor of 1.65 is at least 3.5, while the target reliability is 2.5. Considering the target reliability and using Figures 54 through 56, the live load factor can be reduced to approximately 1.3.

3.7.5 Proposed Live Load Factors

To be adequate for all types of structures, the proposed live load factors must encompass the largest values needed for moment and shear. The required values for each vehicle included in the study are presented in Table 40. As the load factors listed for “special and limited crossing permits” in Table 35 are based on analysis utilizing the single lane distribution factors with no other vehicles on the structure, the values for the “special or limited crossing permits” shown in Table 37 have been increased by 0.05 from those shown in Table 38 to account for the possibility of other vehicles being present and contributing to the maximum load effect (the same increase in live load factor between ADTT = 100 and ADTT = 1,000 currently in the MBE). The table indicates the live load factor for routine permits has been decreased from 1.60 to 1.25 and for the AASHTO legal vehicles from 1.65 to 1.30. The live load factor for the special permit vehicles may increase, if the target reliability index remains at 3.5, from 1.4 to 1.5 or it may decrease, if the target reliability index is reduced to 2.5, from 1.4 to 1.15.

It is proposed that for an ADTT = 1,000 that the following live load factors be used:

- Routine Permit vehicles = 1.25,
- Special Single trip allowed to mix with other vehicles = 1.5 for $\beta = 3.5$ or 1.15 for $\beta = 2.5$
- AASHTO Legal Vehicles = 1.30

As indicated earlier, using a reliability index of 3.5 for single and limited crossing permit vehicles was intended to ensure

that the LRFR will not allow permit vehicles significantly heavier than those allowed under LFR. The analysis of the large bridge sample used in this research indicates that ensuring a reliability index of 3.5 for the trucks included in this study will require a higher load factor than currently shown in the MBE. This would result in more bridges not passing the rating under LRFR and was thought to be too restrictive. In consultation with the project panel, it was decided that a reliability index of 2.5, which is used for legal vehicles and routine permit vehicles, should be used for single and limited crossing permit vehicles.

Only results associated with the load factors determined assuming a target reliability index of 2.5 are shown in the following sections.

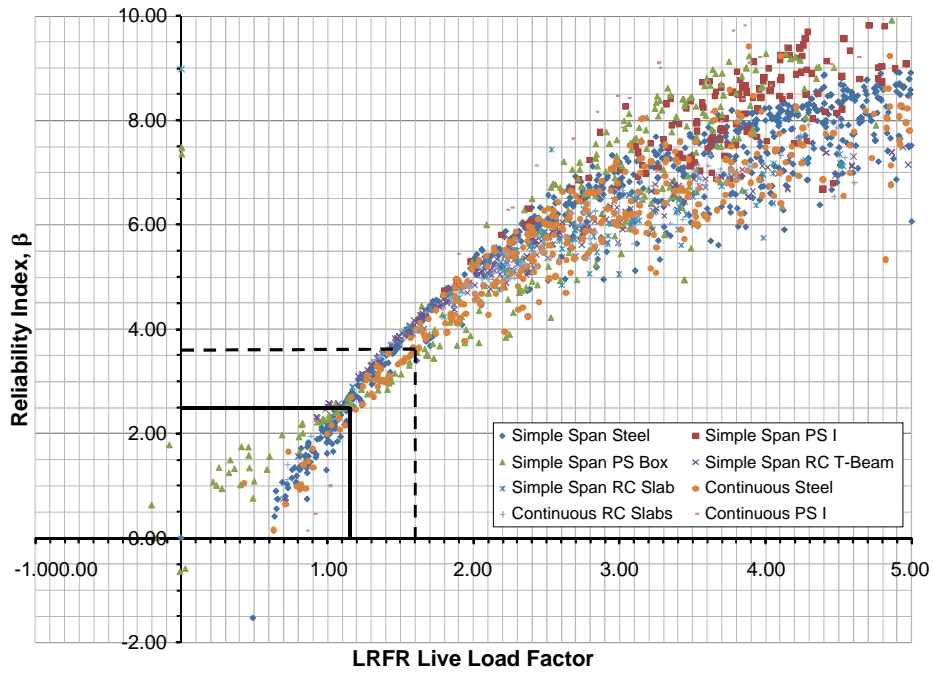
3.8 Selection of Load Factors for Implementation in the MBE

For any particular load factor, the highest value among those determined for different types of structures should be selected. The controlling values are listed in Tables 41 through 43. Tables 38, 39, and 40 are similar to Table 6A.4.4.2.3a-1, Table 6A.4.4.2.3b-1, and Table 6A.4.5.4.2a-1 of the MBE, respectively.

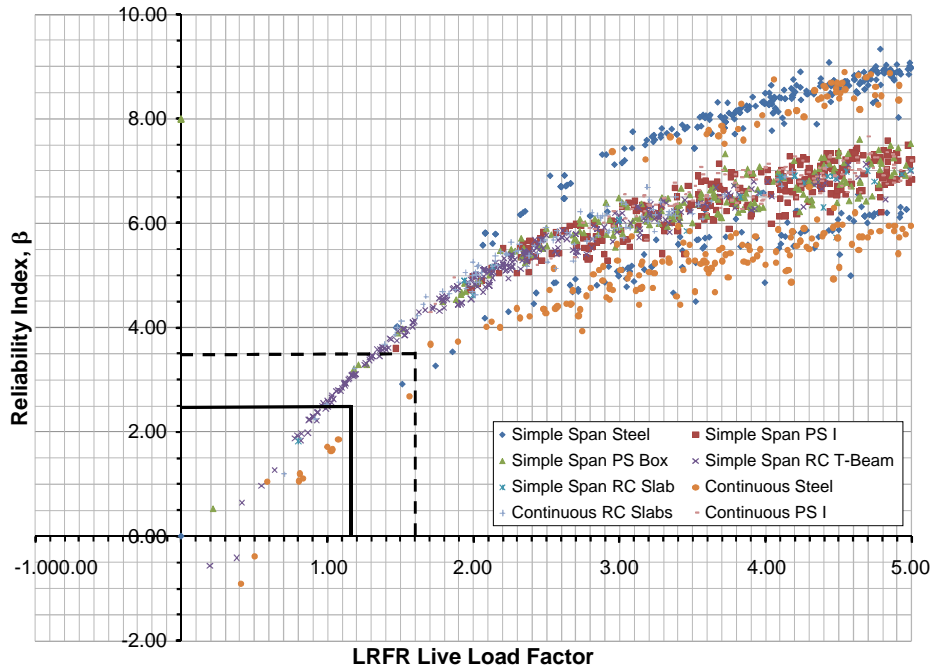
In some cases, this research did not specifically develop a value for the load factor. In such cases, engineering judgment is used to develop the proposed values. For example, when the value determined in this research is thought to correspond to a certain ADTT, the values corresponding to other ADTTs are determined to have the same difference between the values as in the current MBE. In case of the specialized hauling vehicles, the values were selected such that they are not higher than those determined for the routine commercial traffic.

The values shown with strikethrough in Tables 41 through 43 represent the values in the current MBE.

- In Table 42 and Table 43: No analyses were conducted under NCHRP Project 12-78 to support the values shown

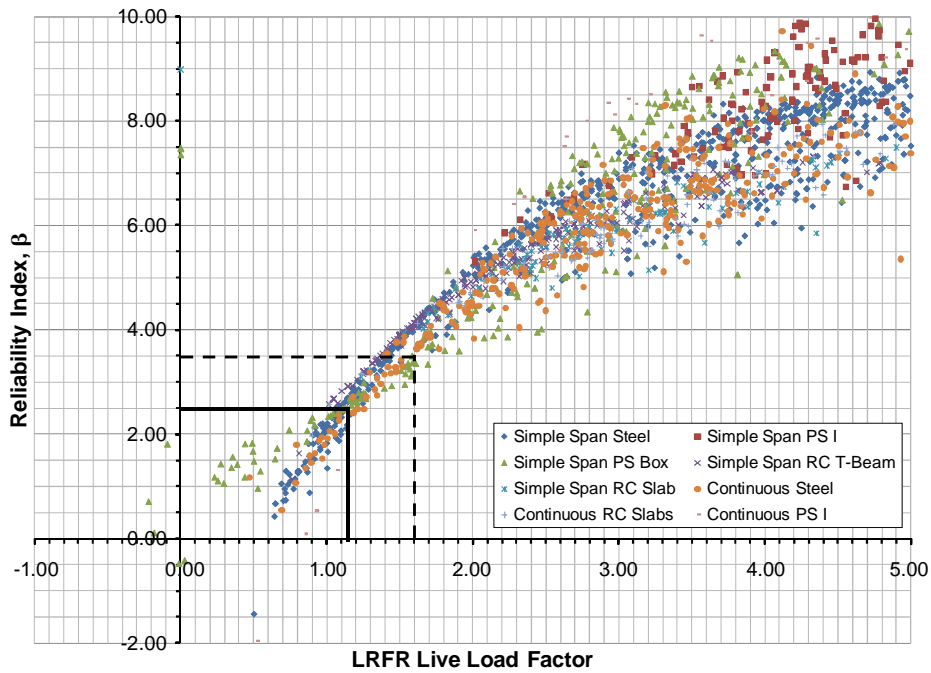


(a)

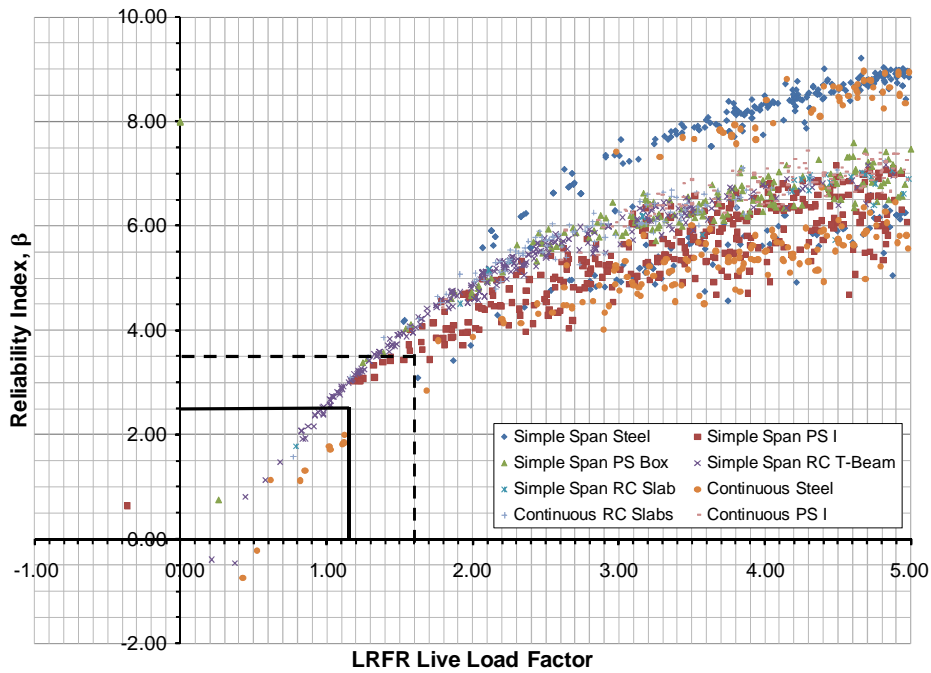


(b)

Figure 46. Reliability index versus live load factor for DE-07 vehicle (a) moment and (b) shear.

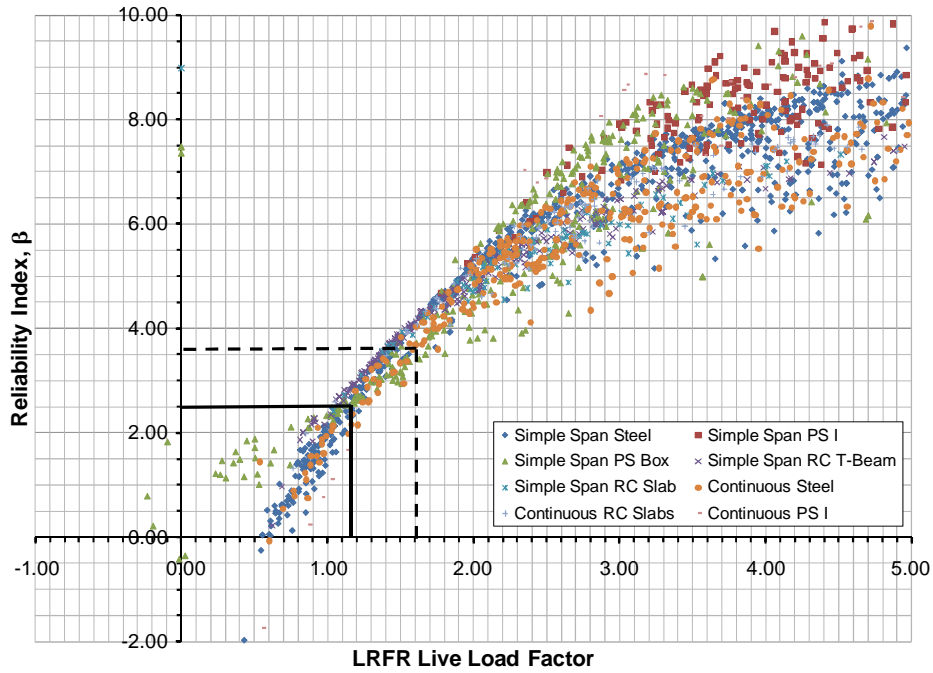


(a)

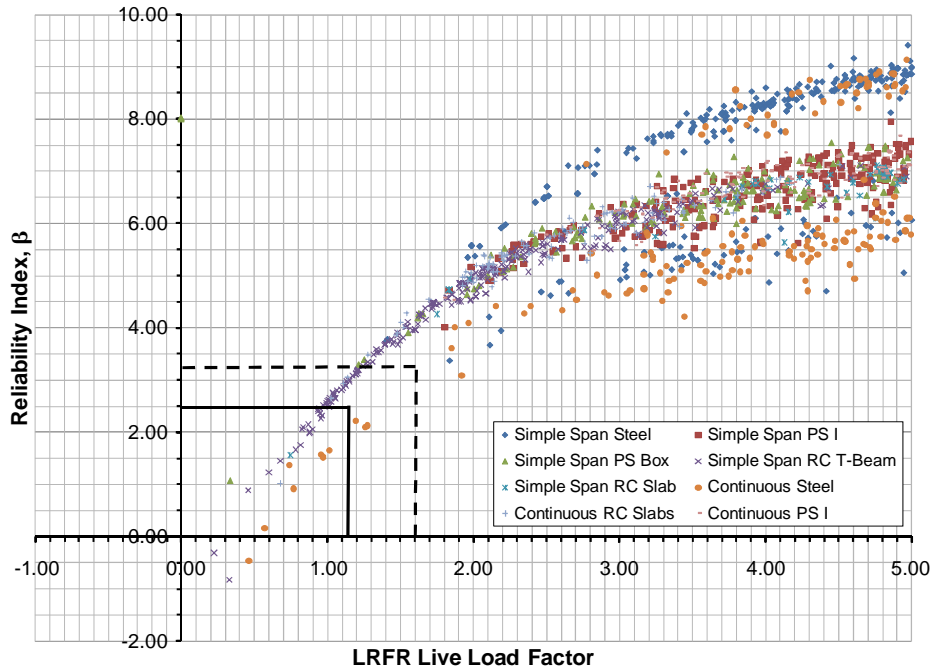


(b)

Figure 47. Reliability index versus live load factor for FL-04 vehicle (a) moment and (b) shear.

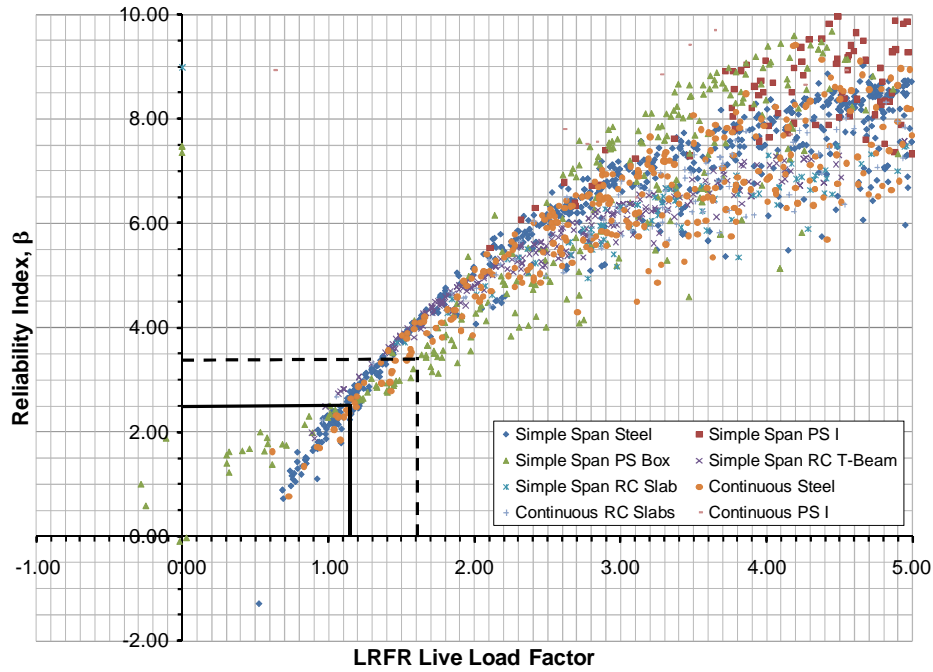


(a)

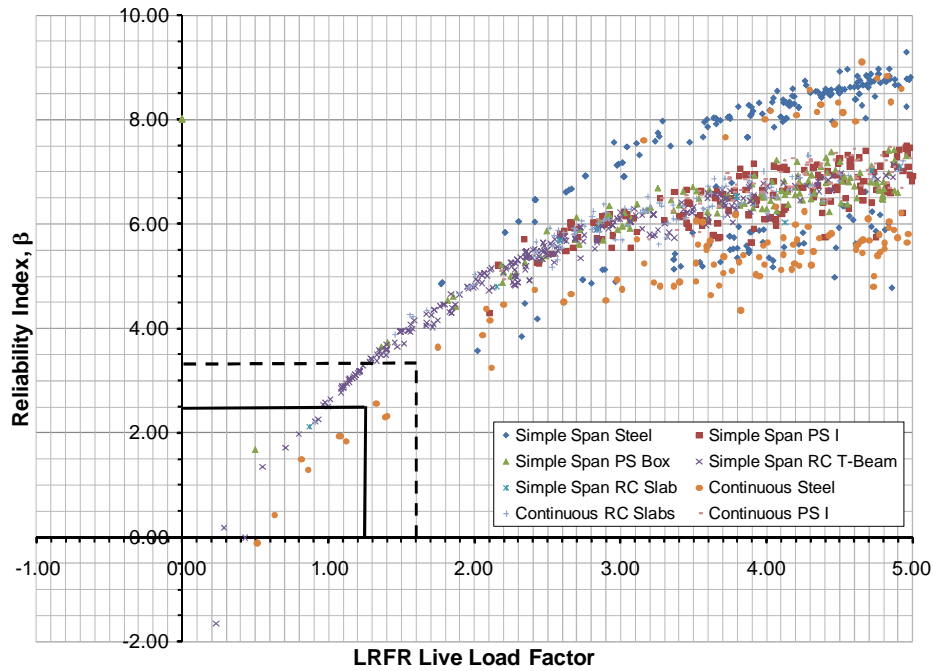


(b)

Figure 48. Reliability index versus live load factor for NC-21 vehicle (a) moment and (b) shear.

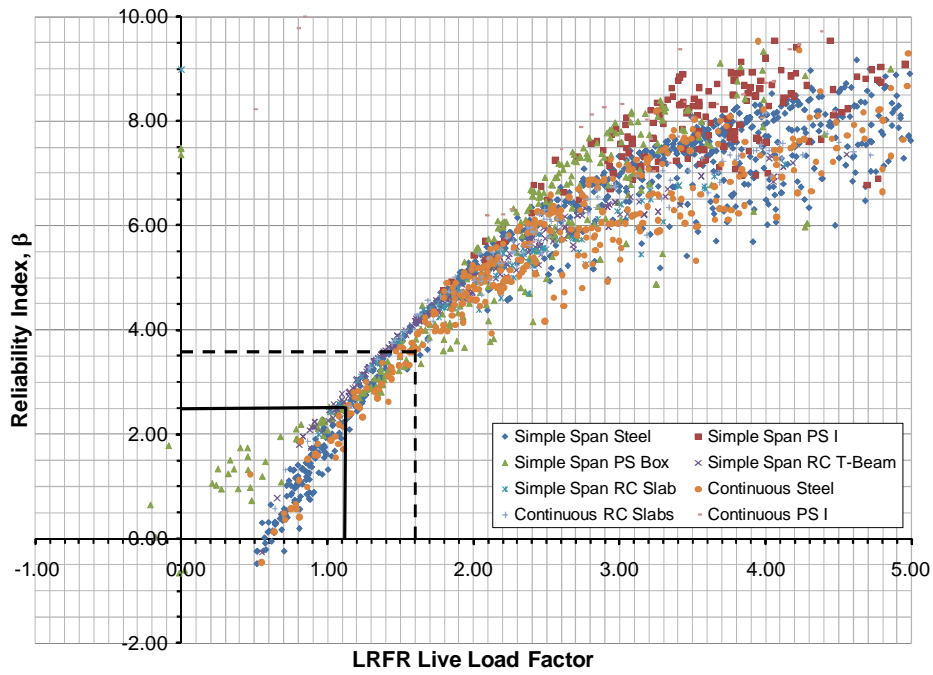


(a)

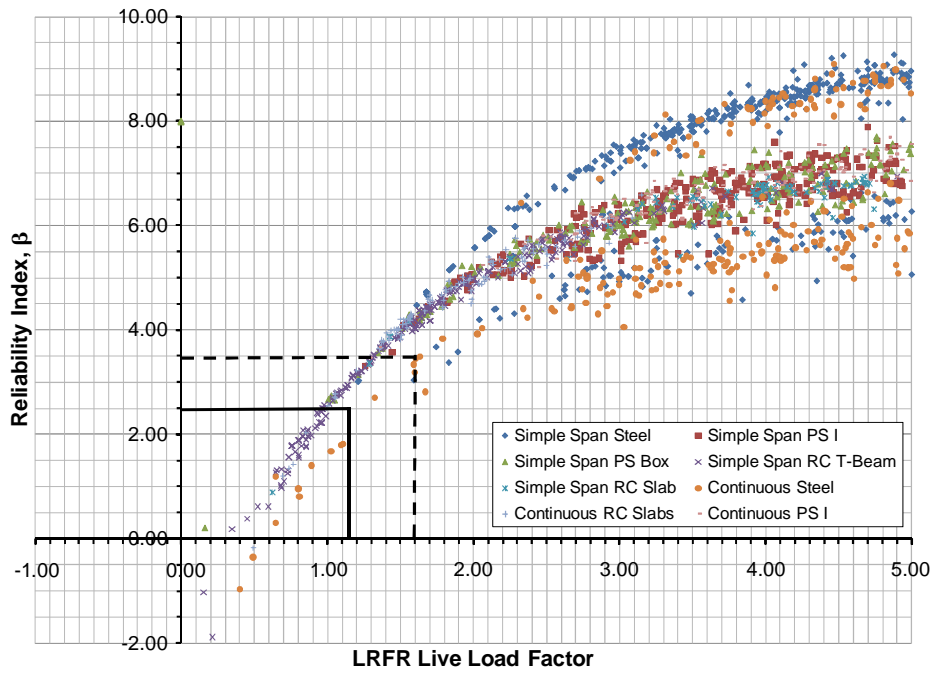


(b)

Figure 49. Reliability index versus live load factor for NM-04 vehicle (a) moment and (b) shear.

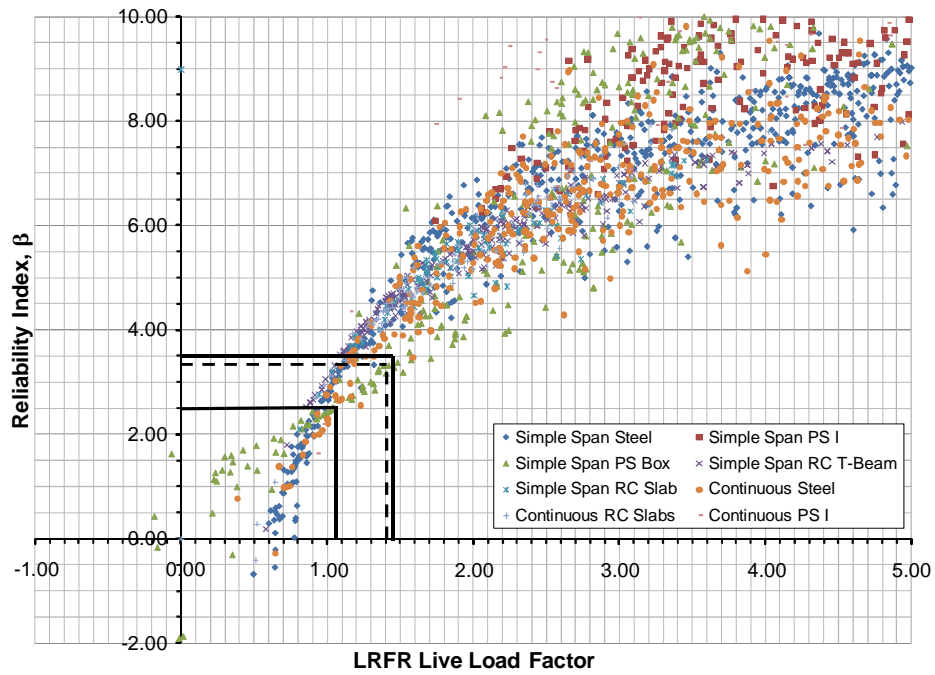


(a)

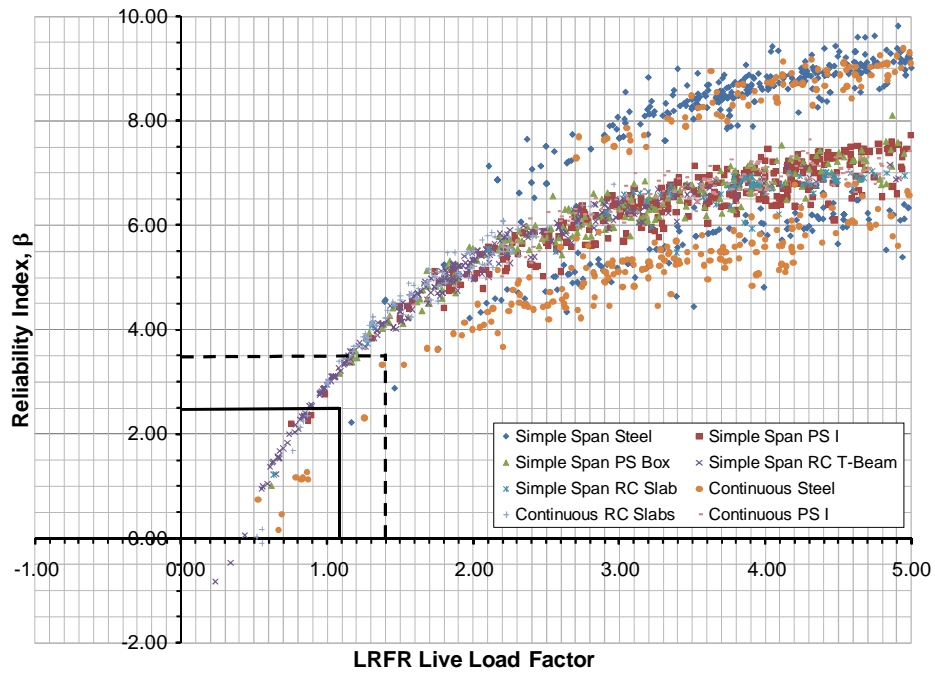


(b)

Figure 50. Reliability index versus live load factor for TX-04 vehicle (a) moment and (b) shear.

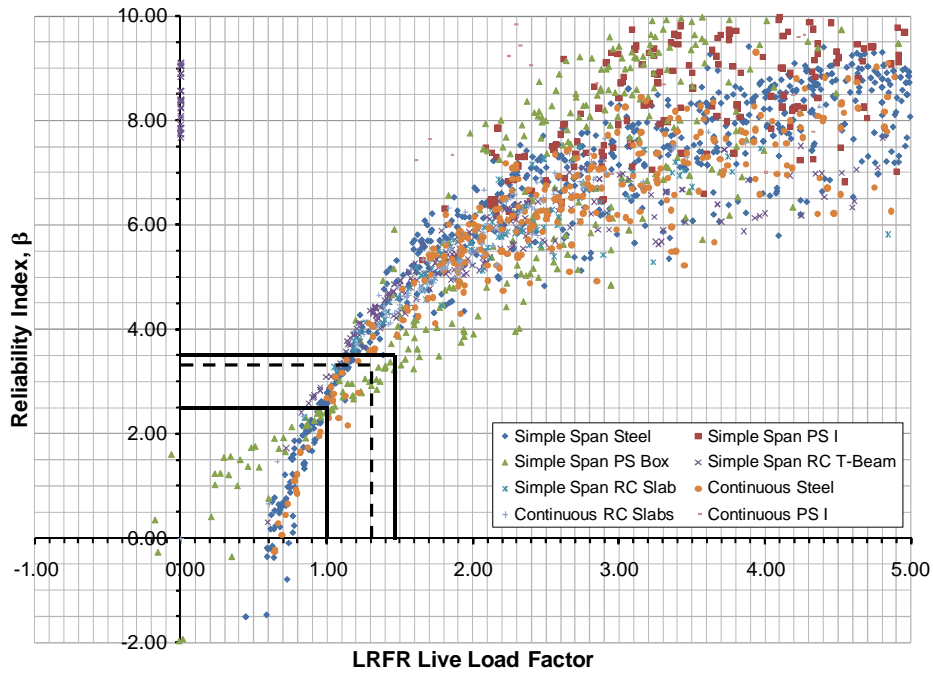


(a)

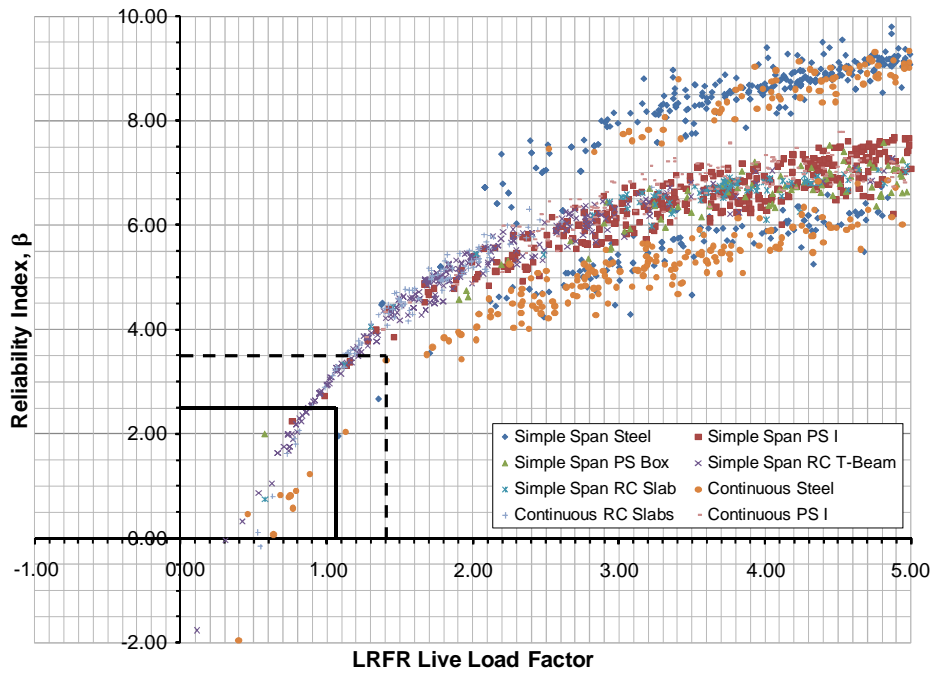


(b)

Figure 51. Reliability index versus live load factor for IL-01 vehicle (a) moment and (b) shear.

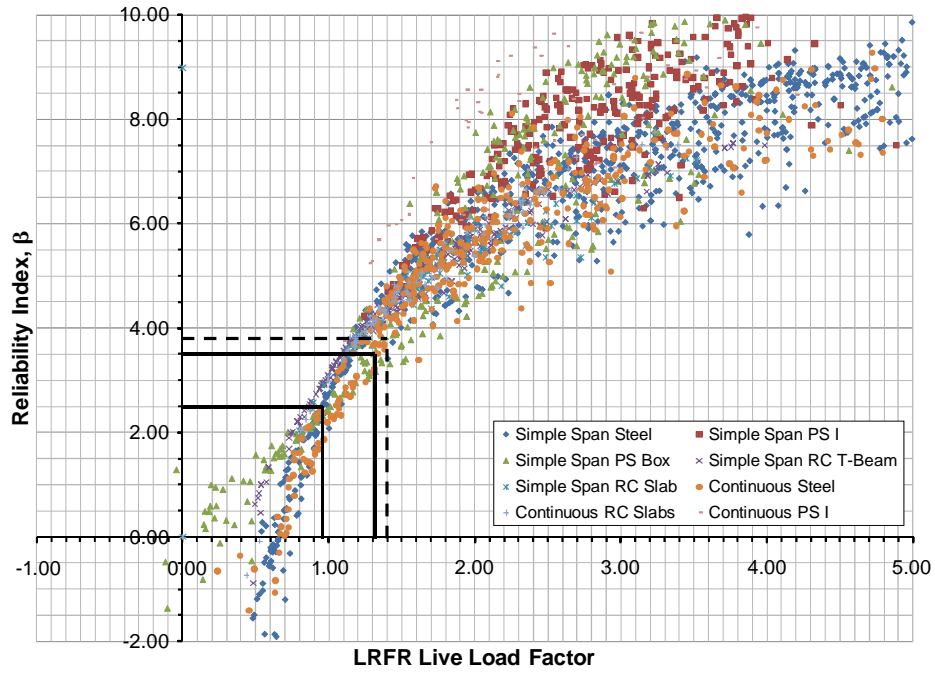


(a)

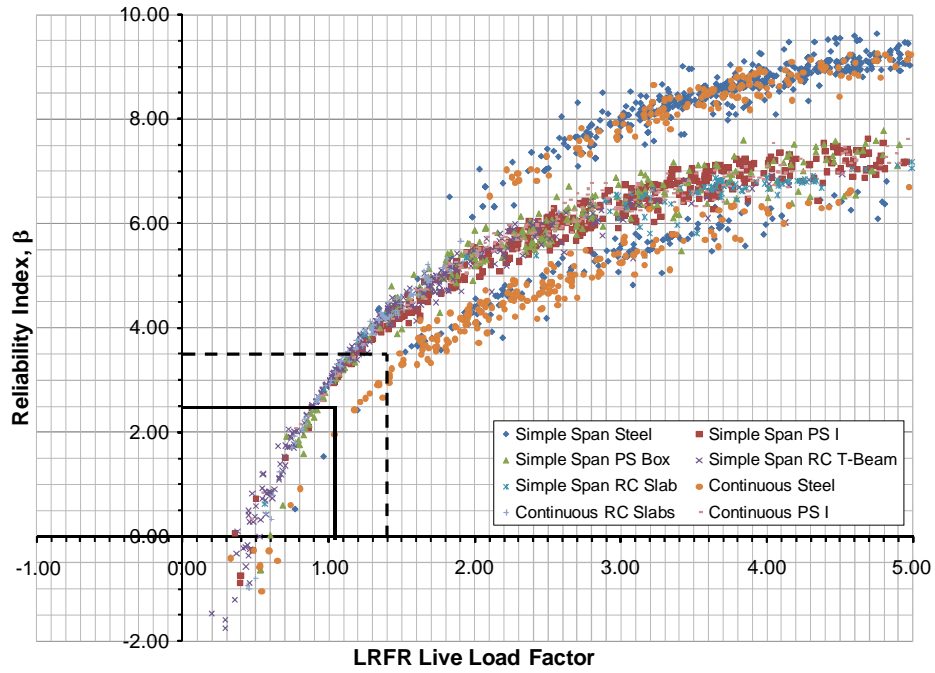


(b)

Figure 52. Reliability index versus live load factor for OR-06 vehicle (a) moment and (b) shear.



(a)



(b)

Figure 53. Reliability index versus live load factor for WA-02 vehicle (a) moment and (b) shear.

Table 38. Proposed live load factors for special permit vehicles.

Truck	Force Effect	Target β in MBE	γ_L in MBE	β corresponding to current γ_L	γ_L corresponding to target β	% of girders with β less than β_{Target}	Figure			
IL-01	Moment	3.5	ADTT = 100: 1.35	3.35			Figure (a)			
	($\beta = 3.5$)				1.45	8.40%				
	($\beta = 2.5$)				1.05	4.58%				
	Shear			3.5				Figure (b)		
	($\beta = 3.5$)				1.40	3.49%				
	($\beta = 2.5$)				1.10	1.98%				
OR-06	Moment			3.5	ADTT = 100: 1.35	3.3			Figure (a)	
	($\beta = 3.5$)						1.45	8.80%		
	($\beta = 2.5$)						1.00	5.27%		
	Shear					3.5				Figure (b)
	($\beta = 3.5$)						1.40	3.26%		
	($\beta = 2.5$)						1.10	1.68%		
WA-02	Moment	3.5	ADTT = 1,000: 1.4			3.8			Figure (a)	
	($\beta = 3.5$)						1.30	12.85%		
	($\beta = 2.5$)						1.00	8.90%		
	Shear					3.5				Figure (b)
	($\beta = 3.5$)						1.40	8.21%		
	($\beta = 2.5$)						1.05	4.35%		

in bold/italic typeface. The shown values were rationally determined considering the currently existing values in the MBE and the difference between the existing and proposed load factors for the same ADTT where analyses were conducted to support the proposed values.

- In Table 43: Values listed in the current MBE for Special or Limited Crossing (single or multiple trips mixed with other traffic) are based on a target reliability index of 3.5. Values listed in Table 43 for these permits are based on a target reliability index of 2.5.

The process of determining reliability indices and the associated load factors involves several sources of approximations. As shown, some of the load factors determined based on the analyses appear to be lower than what the engineering community is accustomed to. It is recommended that these low factors be increased to account for the approximations in the analyses. The following minimum values are recommended for the lowest ADTT category (<100) based on engi-

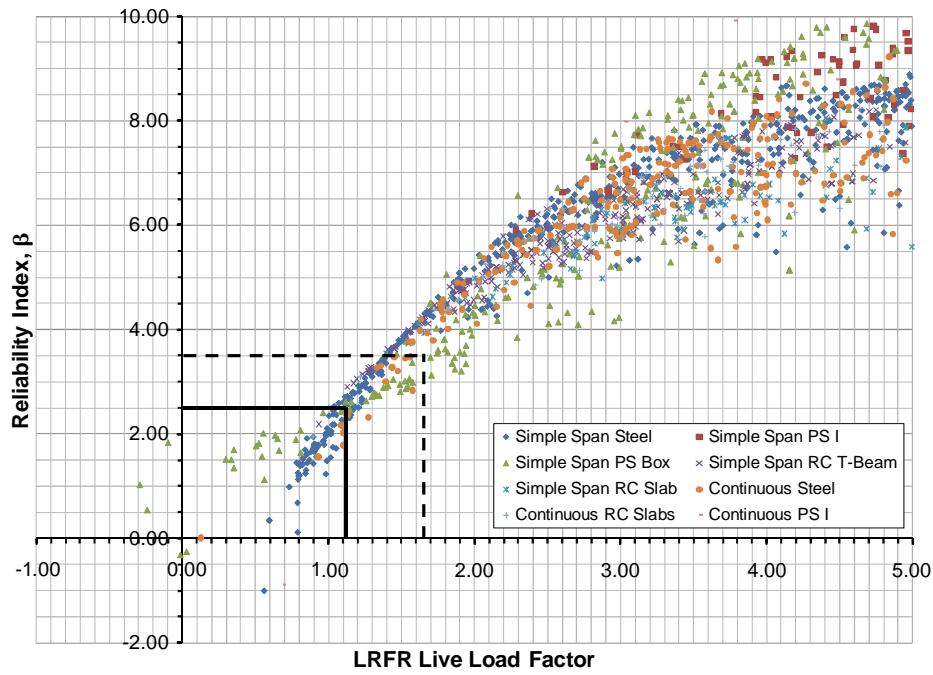
neering judgment and the minimum values accepted in the past:

- For Generalized Live Load Factors, γ_L for Routine Commercial Traffic: 1.20
- Generalized Live Load Factors, γ_L for Specialized Hauling Vehicles: 1.15
- For routine and annual permits:
 - For vehicles up to 100 kips gross weight 1.15
 - For vehicles > 150 kips gross weight (no revision) 1.10
- For Single-Trip Escorted permit (no revision): 1.15
- For Single-Trip mixed with traffic permit: 1.20

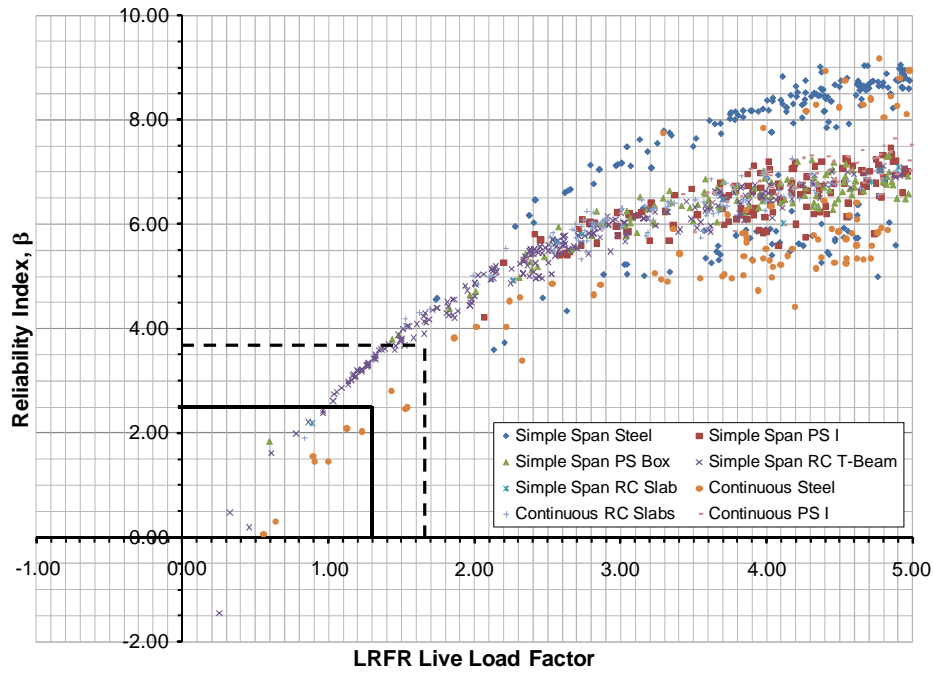
When these recommended values are incorporated and values for other ADTTs are adjusted to maintain the difference between the calculated values for different ADTTs and the adjusted values for the different ADTTs, Tables 44 through 46 are developed.

Table 39. Proposed live load factors for AASHTO legal trucks.

Truck	Force Effect	Target β in MBE	γ_L in MBE (ADTT = 1,000)	β corresponding to current γ_L	γ_L corresponding to target β	Figure
AASHTO Type 3	Moment	2.5	1.65	3.5	1.15	Figure (a)
	Shear			3.7	1.30	Figure (b)
AASHTO Type 3-3	Moment			3.5	1.15	Figure (a)
	Shear			3.5	1.10	Figure (b)
AASHTO Type 3S2	Moment			3.7	1.10	Figure (a)
	Shear			3.5	1.10	Figure (b)

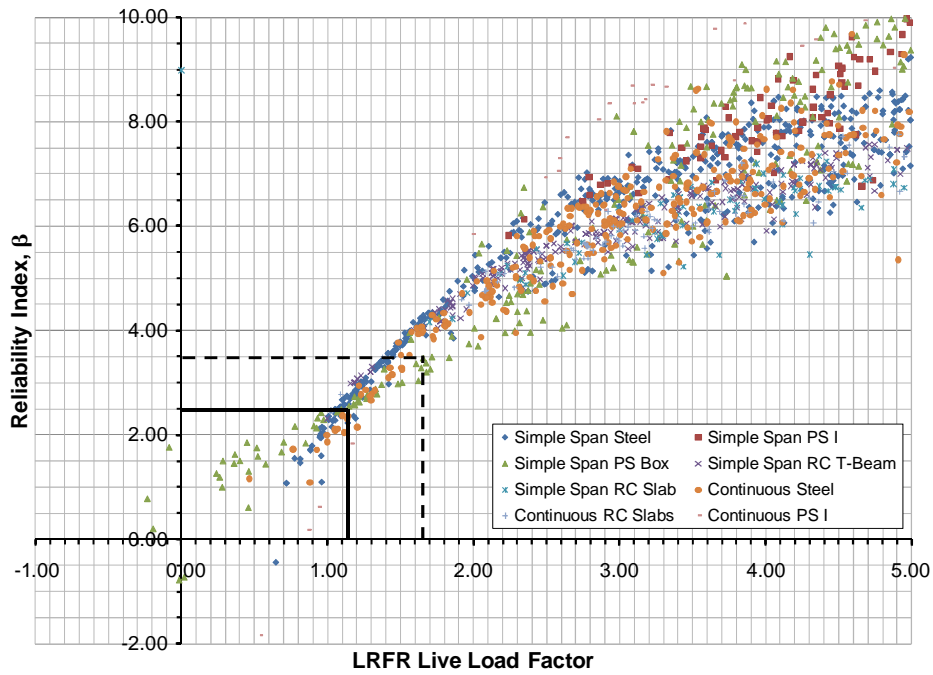


(a)

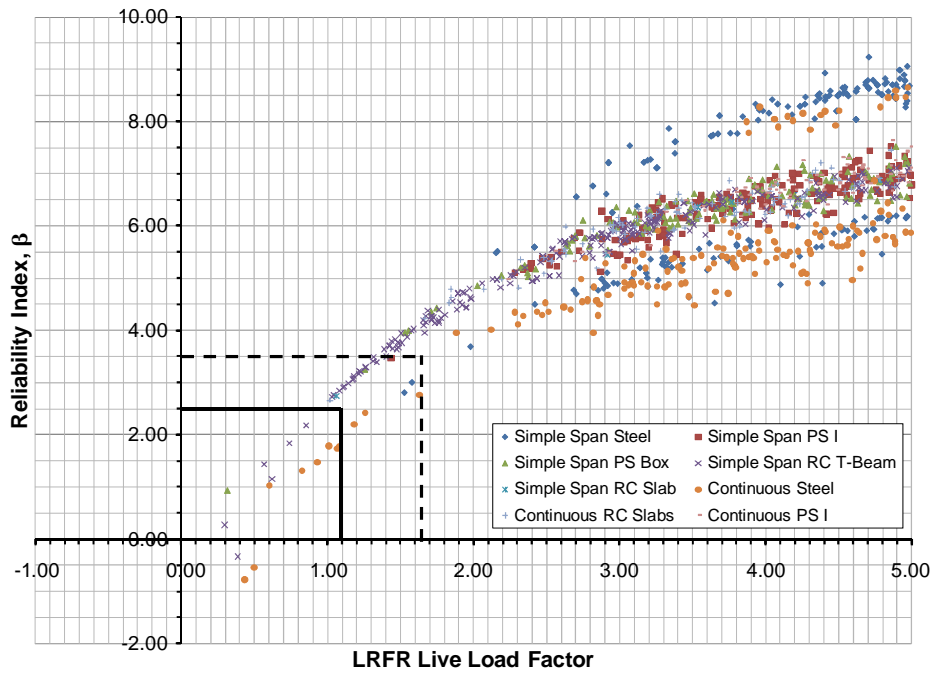


(b)

Figure 54. Reliability index versus live load factor for Type 3 vehicle (a) moment and (b) shear.

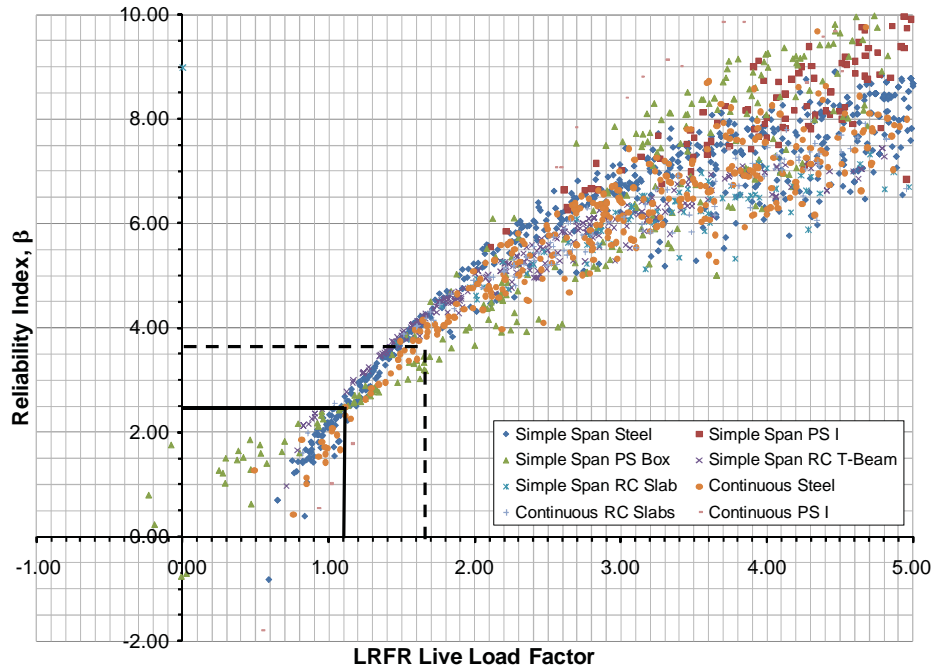


(a)

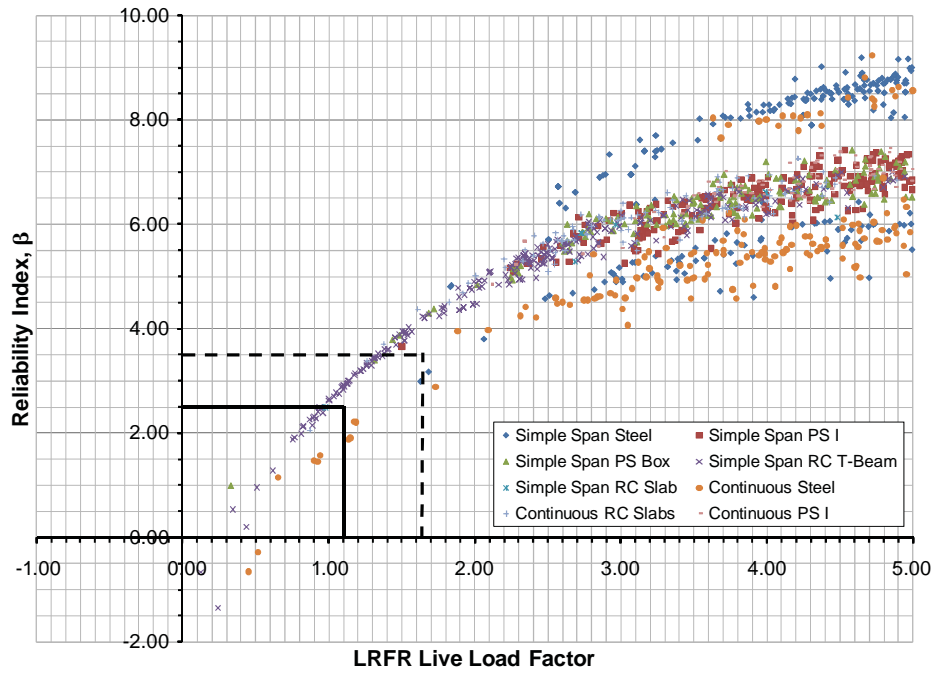


(b)

Figure 55. Reliability index versus live load factor for Type 3-3 vehicle (a) moment and (b) shear.



(a)



(b)

Figure 56. Reliability index versus live load factor for Type 3S2 vehicle (a) moment and (b) shear.

Table 40. Proposed live load factors for ADTT = 1,000.

Vehicle		Moment	Shear	Critical
Routine Permit Vehicles				
DE-07	$\beta = 2.5$	1.15	1.15	1.15
FL-04	$\beta = 2.5$	1.15	1.15	1.15
NC-21	$\beta = 2.5$	1.15	1.15	1.15
NM-04	$\beta = 2.5$	1.15	1.25	1.25
TX-04	$\beta = 2.5$	1.15	1.15	1.15
Special or Limited Crossing Permits				
IL-01	$\beta = 2.5$	1.10	1.15	1.15
	$\beta = 3.5$	1.50	1.45	1.50
OR-06	$\beta = 2.5$	1.05	1.15	1.10
	$\beta = 3.5$	1.50	1.45	1.50
WA-02	$\beta = 2.5$	1.05	1.10	1.10
	$\beta = 3.5$	1.35	1.45	1.45
Legal Loads				
Type 3	$\beta = 2.5$	1.15	1.30	1.30
Type 3S2		1.10	1.10	1.10
Type 3-3		1.15	1.10	1.15

Due to the generally small differences between the load factors for ADTT less than 100 and those for ADTT = 1000 and due to the difficulty in enforcing the routes used by permit and commercial vehicles, it is recommended that the values corresponding to ADTT less than 100 be removed from the load factor tables. Tables 47 through 49 are the same as Table 44 through Table 46, respectively, with the rows corresponding to ADTT less than 100 removed. In addition, for the routine or annual permits, the distinction between vehicles up to 100 kips and vehicles above 150 kips was also eliminated. This distinction resulted in difficulty in determining the load factors as the current MBE required that only the axles on the bridge be used in determining the category (up to 100 kips or above 150 kips). For a truck moving across a short bridge the load factor varied as the weight of the axles on the bridge changed.

Table 41. Calculated generalized live load factors, γ_L for routine commercial traffic.

Traffic Volume (one direction)	Load Factor
Unknown	4.80 1.45
ADTT \geq 5000	4.80 1.45
ADTT = 1000	4.65 1.30
ADTT \leq 100	4.40 1.05

Table 42. Calculated generalized live load factors, γ_L for specialized hauling vehicles.

Traffic Volume (one direction)	Load Factor
Unknown	4.60 1.45
ADTT \geq 5000	4.60 1.45
ADTT = 1000	4.40 1.30
ADTT \leq 100	4.15 1.05

3.9 Effect of Using Proposed Live Load Factors on Rating Factors

The effect of changing the LRFR live load factors to the proposed values is shown in the following tables for each type of bridge that was used in this study. The proposed load factors used in this section are 1.3 for the AASHTO Legal Vehicles, 1.25 for routine permit vehicles, and 1.25 for the special permit vehicles for a reliability index of 2.5. The first eight columns of each table are the same and are described below:

- Column 1—Bridge Type
- Column 2—Total number of girders from this bridge type

Table 43. Calculated permit load factors, γ_L .

Permit Type	Frequency	Loading Condition	DF	ADTT (one direction)	Load Factor by Permit Weight			
					Up to 100 kips	> 150 kips		
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	4.80 1.45	1.30		
				=1000	4.60 1.25	1.20		
				<100	4.40 1.05	1.10		
All Weights								
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15			
				Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	4.60 1.25
							=1000	4.40 1.15
	<100	4.35 1.10						
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	4.85 1.60			
				=1000	4.75 1.50			
<100				4.55 1.45				

Table 44. Calculated generalized live load factors, γ_L for routine commercial traffic.

Traffic Volume (one direction)	Load Factor
Unknown	4.80 1.45
ADTT \geq 5000	4.80 1.45
ADTT = 1000	4.65 1.30
ADTT \leq 100	4.40 1.20

Table 45. Calculated generalized live load factors, γ_L for specialized hauling vehicles.

Traffic Volume (one direction)	Load Factor
Unknown	4.60 1.45
ADTT \geq 5000	4.60 1.45
ADTT = 1000	4.40 1.30
ADTT \leq 100	4.15 1.15

Table 47. Calculated generalized live load factors, γ_L for routine commercial traffic.

Traffic Volume (one direction)	Load Factor
Unknown	4.80 1.45
ADTT \geq 5000	4.80 1.45
ADTT = 1000	4.65 1.30

Table 48. Calculated generalized live load factors, γ_L for specialized hauling vehicles.

Traffic Volume (one direction)	Load Factor
Unknown	4.60 1.45
ADTT \geq 5000	4.60 1.45
ADTT = 1000	4.40 1.30

Column 3—Vehicle type (design, legal, routine permit, special permit)

Column 4—Vehicle Name

Column 5—Load Effect (Moment/Shear)

Column 6—Number of girders with LFR ratings less than 1.0, inventory ratings for design vehicle, operating ratings for all others

Column 7—Number of girders with LRFR ratings less than 1.0 assuming that no criteria are ignored in the LRFR rating.

Column 8—Percent increase in number of girders not passing rating as a percentage of the total number of girders, calculated using the following formula:

$$\% \text{ change} = \frac{LRFR < 1.0 - LFR < 1.0}{Total} \times 100$$

The remaining columns will be described for each table below.

3.9.1 Simple Span Steel

Table 50 shows the effect of changing the load factors to those proposed earlier for the 1037 simple span steel girders included

Table 46. Calculated permit load factors, γ_L .

Permit Type	Frequency	Loading Condition	DF	ADTT (one direction)	Load Factor by Permit Weight	
					Up to 100 kips	> 150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	4.80 1.45	1.30
				=1000	4.60 1.25	1.20
				<100	4.40 1.15	1.10
All Weights						
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	4.50 1.35	
				=1000	4.40 1.25	
				<100	4.35 1.20	
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	4.85 1.60	
=1000				4.75 1.50		
<100				4.55 1.45		

Table 49. Calculated permit load factors, γ_L .

Permit Type	Frequency	Loading Condition	DF	ADTT (one direction)	Load Factor
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.80 <u>1.45</u>
				=1000	1.60 <u>1.25</u>
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.50 <u>1.35</u>
				=1000	1.40 <u>1.25</u>
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.85 <u>1.60</u>
=1000				1.75 <u>1.50</u>	

Table 50. Simple span steel—effect of proposed load factors.

Type	# of Girders	Vehicle Type	Vehicle	Effect	# of girders w/ LFR < 1.0	Using Existing Load Factors				Using Proposed Load Factors		
						# of girders w/ LRFR < 1.0				# of girders w/ LRFR < 1.0	# of girders w/ LFR < 1.0	% Change
						Total	% Change	Ignoring 80007	% Change			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Simple Span Steel	1037	Design	HL-93	Moment	295	346	4.92%			n/a	n/a	n/a
				Shear	0	117	11.28%	21	2.03%	n/a	n/a	n/a
		Legal	AASHTO Type 3	Moment	72	211	13.40%			151	72	7.62%
				Shear	0	41	3.95%	0	0.00%	24	0	2.31%
			AASHTO Type 3-3	Moment	37	175	13.31%			110	37	7.04%
				Shear	0	41	3.95%	2	0.19%	27	0	2.60%
			AASHTO Type 3S2	Moment	65	195	12.54%			142	65	7.43%
				Shear	0	45	4.34%	1	0.10%	18	0	1.74%
		Routine Permit	DE-07	Moment	100	240	13.50%			146	100	4.44%
				Shear	0	52	5.01%	3	0.29%	33	0	3.18%
			FL-04	Moment	97	235	13.31%			149	97	5.01%
				Shear	0	48	4.63%	2	0.19%	30	0	2.89%
			NC-21	Moment	130	273	13.79%			202	130	6.94%
				Shear	0	49	4.73%	2	0.19%	30	0	2.89%
			NM-04	Moment	80	209	12.44%			126	80	4.44%
				Shear	0	39	3.76%	0	0.00%	21	0	2.03%
			TX-04	Moment	153	296	13.79%			229	153	7.33%
				Shear	0	66	6.36%	3	0.29%	45	0	4.34%
		Special Permit	IL-01	Moment	180	217	3.57%			186	180	0.58%
				Shear	0	54	5.21%	3	0.29%	43	0	4.15%
			OR-06	Moment	201	210	0.87%			186	201	-1.45%
				Shear	0	48	4.63%	4	0.39%	34	0	3.28%
			WA-02	Moment	231	276	4.34%			228	231	-0.29%
				Shear	1	87	8.29%	8	0.68%	66	1	6.27%

in the study. Column 9 shows the number of girders with shear ratings less than 1.0 when the bearing stiffener rating is ignored. Column 10 shows the percent change in girders with shear ratings less than 1.0 when the bearing stiffener rating (80007) is ignored. Column 11 is the number of girders with LRFR ratings less than 1.0 using the proposed live load factors, while column 12 is the number of girders with LFR ratings less than 1.0 (same as column 6). Column 13 shows the percent change between the number of girders with ratings less than 1.0 for LRFR using the proposed live load factors and LFR. The increase in number of girders with LRFR flexure ratings less than 1.0 is mostly due to an increase in live load and a decrease of approximately 5% in resistance. The number of girders not passing the moment ratings is larger than those not passing the shear rating. Most girders with ratings less than 1.0 are non-composite, for example, for the AASHTO Type 3 truck 180 of the 211 girders with an LRFR moment rating less than 1.0 are non-composite.

When the bearing stiffener rating is ignored, almost all girders pass the LRFR shear rating for all loads considered even when the existing, higher live load factors are applied. When the proposed live load factors are applied, all girders

passed the LRFR shear rating (same as for LFR) and the number of girders not passing the LRFR moment rating dropped but is still higher than the number of girders not passing the LFR rating.

3.9.2 Simple Span Prestressed I-Beams

Table 51 shows the effect of changing the LRFR live load factors to those proposed for the 467 simple span prestressed I-beams included in the study. Column 9 shows the number of girders with LRFR ratings less than 1.0 when the longitudinal steel rating (Report ID 85004) is ignored. Column 10 shows the percent change in number of girders with LRFR ratings less than 1.0, ignoring the longitudinal steel rating, compared to the number with LFR ratings less than 1.0. Column 11 shows the number of girders controlled by two additional criteria found to have a significant effect. Report ID 85107 corresponds to the service limit state for tension in the bottom of the beam (which is optional or not required for some vehicles but was found to be applied by some state DOTs) and Report ID 85003 corresponds to the shear fric-

Table 51. Simple span prestressed I beams—effect of proposed live load factors.

Type	# of Girders	Vehicle Type	Vehicle	Effect	# of girders with LFR < 1.0	Using Existing Load Factors					Using Proposed Load Factors			
						# of girders w/ LRFR < 1.0					# of girders w/ LRFR < 1.0	# of girders with LFR < 1.0	% Change	
						Total	% Change	Ignoring 85004	% Change	85107 (M) 85003 (V)				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	
Simple Span Prestressed Concrete I Beams	467	Design	HL-93	Moment	25	289	56.53%	95	14.99%	95	n/a	n/a	n/a	
				Shear	39	193	32.98%			148	n/a	n/a	n/a	
		Legal	AASHTO Type 3	Moment	0	188	40.26%	27	5.78%	27	115	0	24.63%	
				Shear	1	35	7.28%			34	19	1	3.85%	
			AASHTO Type 3-3	Moment	0	222	47.54%	32	6.85%	321	33	0	28.48%	
				Shear	1	43	8.99%			41	20	1	4.07%	
			AASHTO Type 3S2	Moment	0	223	47.75%	35	7.49%	351	26	0	26.98%	
				Shear	1	43	8.99%			41	19	1	3.85%	
		Routine Permit	DE-07	Moment	0	231	49.46%	53	11.35%	531	43	0	30.62%	
				Shear	2	51	10.49%			49	29	2	5.78%	
			FL-04	Moment	0	212	45.40%	39	8.35%	39	124	0	26.55%	
				Shear	2	49	10.06%			47	29	2	5.78%	
			NC-21	Moment	0	214	45.82%	51	10.92%	51	134	0	28.69%	
				Shear	2	49	10.06%			48	29	2	5.78%	
			NM-04	Moment	0	185	39.61%	28	6.00%	28	113	0	24.20%	
				Shear	1	37	7.71%			36	20	1	4.07%	
			TX-04	Moment	0	246	52.68%	73	15.63%	73	158	0	33.83%	
				Shear	3	80	16.49%			76	41	3	8.14%	
			Special Permit	IL-01	Moment	0	120	25.70%	68	14.56%	68	90	0	19.27%
					Shear	11	55	9.42%			48	40	11	6.21%
		OR-06		Moment	0	125	26.77%	73	15.63%	73	102	0	21.84%	
				Shear	9	54	9.64%			44	37	9	6.00%	
		WA-02		Moment	6	216	44.97%	1723	5.55%	172	194	6	40.26%	
				Shear	44	119	16.06%			89	82	44	8.14%	

tion rating between the girder and composite slab. Column 12 contains the number of girders with LRFR ratings less than 1.0 when the proposed load factors are utilized and no rating criteria are ignored; column 13 is the same as Column 6 and contains the number of girders with LFR ratings less than 1.0. Column 14 is the percent change between Column 12 and Column 13. For example, for the WA-02 vehicle, it was determined that if both the longitudinal steel rating and the service limit state are ignored, seven girders will have ratings less than 1.0 (very similar to the LFR number of six). Thirty-four girders have LRFR shear ratings that are less than 1.0 for the WA-02 vehicle when the shear friction rating is ignored.

3.9.3 Simple Span Prestressed Box Beams

Table 52 shows the effect of changing the LRFR live load factors to those proposed for the 377 prestressed box beams included in the study. The description of Columns 9 through 14 is similar to that described for Table 51. The service limit state controls a significant portion of the girders with LRFR

ratings less than 1.0 when the longitudinal steel rating is ignored. Thus, changing the live load factors for the strength limit state will not have a significant effect on the number of girders with ratings less than 1.0, when the Service III limit state is checked.

3.9.4 Simple Span Reinforced Concrete T-Beams

Table 53 shows the effect of changing the LRFR live load factors to those proposed for the 295 reinforced concrete T-beams included in the study. Column 9 shows the number of girders with LRFR ratings less than 1.0 when the longitudinal steel rating (Report ID 85004) is ignored. Column 10 shows the percent change in number of girders with LRFR ratings less than 1.0, ignoring the longitudinal steel rating, compared to the number with LFR ratings less than 1.0. Column 11 contains the number of girders with LRFR ratings less than 1.0 when the proposed load factors are utilized and no rating criteria are ignored; Column 12 is the same as Column 6 and contains the number of girders with LFR ratings less

Table 52. Simple span prestressed box beams—effect of proposed live load factors.

Type	# of Girders	Vehicle Type	Vehicle	Effect	# of girders with LFR < 1.0	Using Existing Load Factors					Using Proposed Load Factors		
						# of girders w/ LRFR < 1.0					# of girders w/ LRFR < 1.0	# of girders with LFR < 1.0	% Change
						Total	% Change	Ignoring 85004	% Change	85107 (M) 85003 (V)			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Simple Span Prestressed Concrete Box Beams	377	Design	HL-93	Moment	53	156	27.32%	128	19.89%	94	n/a	n/a	n/a
				Shear	19	72	14.06%			10	n/a	n/a	n/a
		Legal	AASHTO Type 3	Moment	0	54	14.32%	41	10.88%	36	45	0	11.94%
				Shear	1	3	0.53%			0	1	1	0.00%
			AASHTO Type 3-3	Moment	0	55	14.59%	44	11.67%	42	47	0	12.47%
				Shear	1	3	0.53%			0	1	1	0.00%
			AASHTO Type 3S2	Moment	0	65	17.24%	52	13.79%	47	54	0	14.32%
				Shear	1	3	0.53%			0	1	1	0.00%
		Routine Permit	DE-07	Moment	0	87	23.08%	71	18.83%	65	76	0	20.16%
				Shear	1	6	1.33%			0	3	1	0.53%
			FL-04	Moment	0	73	19.36%	59	15.65%	54	63	0	16.71%
				Shear	1	6	1.33%			0	2	1	0.27%
			NC-21	Moment	1	102	26.79%	91	23.87%	72	96	1	25.20%
				Shear	1	4	0.80%			0	2	1	0.27%
			NM-04	Moment	0	56	14.85%	44	11.67%	42	48	0	12.73%
				Shear	1	3	0.53%			0	1	1	0.00%
			TX-04	Moment	1	132	34.75%	117	30.77%	87	122	1	32.10%
				Shear	3	16	3.45%			2	5	3	0.53%
		Special Permit	IL-01	Moment	1	89	23.34%	83	21.75%	69	85	1	22.28%
				Shear	8	16	2.12%			1	11	8	0.80%
			OR-06	Moment	2	92	23.87%	88	22.81%	71	90	2	23.34%
				Shear	1	1	0.00%			1	1	1	0.00%
			WA-02	Moment	19	166	38.99%	162	37.93%	125	161	19	37.67%
				Shear	16	46	7.96%			4	37	16	5.57%

Table 53. Simple span reinforced concrete T-beams—effect of proposed live load factors.

Type	# of Girders	Vehicle Type	Vehicle	Effect	# of girders w/ LFR < 1.0	Using Existing Load Factors				Using Proposed Load Factors			
						# of girders w/ LRFR < 1.0				# of girders w/ LRFR < 1.0	# of girders w/ LFR < 1.0	% Change**	
						Total	% Change	Ignoring 85004	% Change				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	
Simple Span Reinforced Concrete T-Beams	295	Design	HL-93	Moment	1312	69	46.78%	220	30.17%	n/a	n/a	n/a	
				Shear	1691	94	8.47%			n/a	n/a	n/a	
		Legal	AASHTO Type 3	Moment	9	124	38.98%	56	15.93%	73	9	21.69%	
				Shear	21	68	15.93%			40	21	6.44%	
			AASHTO Type 3-3	Moment	7	88	27.46%	32	8.47%	48	7	13.90%	
				Shear	17	51	11.53%			31	17	4.75%	
			AASHTO Type 3S2	Moment	8	106	33.22%	49	13.90%	68	8	20.34%	
				Shear	21	62	13.90%			34	21	4.41%	
		Routine Permit	DE-07	Moment	22	150	43.39%	93	24.07%	91	22	23.39%	
				Shear	27	79	17.63%			59	27	10.85%	
			FL-04	Moment	21	139	40.00%	80	20.00%	84	21	21.36%	
				Shear	26	76	16.95%			56	26	10.17%	
			NC-21	Moment	38	178	47.46%	122	28.47%	103	38	22.03%	
				Shear	27	81	18.31%			58	27	10.51%	
			NM-04	Moment	19	120	34.24%	59	13.56%	68	19	16.61%	
				Shear	21	68	15.93%			41	21	6.78%	
			TX-04	Moment	45	216	57.97%	139	31.86%	133	45	29.83%	
				Shear	49	138	30.17%			79	49	10.17%	
			Special Permit	IL-01	Moment	56	190	45.42%	123	22.71%	153	56	32.88%
					Shear	56	118	21.02%			94	56	12.88%
		OR-06		Moment	61	200	47.12%	139	26.44%	159	61	33.22%	
				Shear	63	126	21.36%			99	63	12.20%	
		WA-02		Moment	80	221	47.80%	160	27.12%	188	80	36.61%	
				Shear	86	166	27.12%			127	86	13.90%	

than 1.0. Column 13 is the percent change between Column 11 and Column 12.

3.9.5 Simple Span Reinforced Concrete Slabs

Table 54 shows the effect of changing the LRFR live load factors to those proposed for the 99 simple span reinforced concrete slabs included in the study. The information in Columns 9 through 13 is similar to that described for Table 53. The table shows that ignoring the longitudinal steel rating significantly reduces the number of slabs with LRFR ratings less than 1.0.

3.9.6 Continuous Span Steel Girders

Table 55 shows the effect of changing the load factors to those proposed for the 418 continuous span steel girders included in

the study. The information in Columns 9 through 13 is similar to that described in Table 50.

3.9.7 Continuous Span Reinforced Concrete Slabs

Table 56 shows the effect of changing the LRFR live load factors to those proposed for the 105 continuous reinforced concrete slabs included in the study. The information in Columns 9 through 13 is similar to that described in Table 53.

3.9.8 Continuous Span Prestressed Concrete I-Beams

Table 57 shows the effect of changing the LRFR live load factors to those proposed for the 238 continuous span prestressed I beams included in the study. The information in Columns 9 through 14 is similar to that described for Table 51.

Table 54. Simple span reinforced concrete slabs—effect of proposed live load factors.

Type	# of Girders	Vehicle Type	Vehicle	Effect	# of girders w/ LFR < 1.0	Using Existing Load Factors				Using Proposed Load Factors			
						# of girders w/ LRFR < 1.0				# of girders w/ LRFR < 1.0	# of girders w/ LFR < 1.0	% Change	
						Total	% Change	Ignoring 85004	% Change				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	
Simple Span Reinforced Concrete Slab Bridges	99	Design	HL-93	Moment	27	64	37.37%	46	19.19%	n/a	n/a	n/a	
				Shear	2	5	3.03%			n/a	n/a	n/a	
		Legal	AASHTO Type 3	Moment	0	23	23.23%	4	4.04%	9	0	9.09%	
				Shear	0	1	1.01%			1	0	1.01%	
			AASHTO Type 3-3	Moment	0	10	10.10%	2	2.02%	7	0	7.07%	
				Shear	0	1	1.01%			1	0	1.01%	
			AASHTO Type 3S2	Moment	0	14	14.14%	3	3.03%	9	0	9.09%	
				Shear	0	1	1.01%			1	0	1.01%	
		Routine Permit	DE-07	Moment	2	27	25.25%	11	9.09%	13	2	11.11%	
				Shear	0	1	1.01%			1	0	1.01%	
			FL-04	Moment	1	24	23.23%	7	6.06%	10	1	9.09%	
				Shear	0	1	1.01%			1	0	1.01%	
			NC-21	Moment	5	32	27.27%	20	15.15%	19	51	4.14%	
				Shear	0	1	1.01%			1	0	1.01%	
			NM-04	Moment	0	13	13.13%	2	2.02%	8	0	8.08%	
				Shear	0	1	1.01%			1	0	1.01%	
			TX-04	Moment	8	43	35.35%	22	14.14%	27	8	19.19%	
				Shear	0	5	5.05%			2	0	2.02%	
			Special Permit	IL-01	Moment	14	31	17.17%	21	7.07%	28	14	14.14%
					Shear	0	5	5.05%			4	0	4.04%
		OR-06		Moment	14	33	19.19%	21	7.07%	28	14	14.14%	
				Shear	0	5	5.05%			4	0	4.04%	
		WA-02		Moment	16	37	21.21%	23	7.07%	29	16	13.13%	
				Shear	0	5	5.05%			4	0	4.04%	

Generally, Tables 50 through 57 indicate that a significant percentage of girders that passed rating under the LFR and produced rating factors below 1.0 for the LRFR were controlled by criteria that were not checked under the LFR method of rating. Some of these criteria are not known to have caused problems in the past. When these criteria are ignored in the LRFR rating, the difference in the number of bridges not passing the LRFR rating, as compared to the number of those not passing the LFR rating, decreases significantly. In addition, when the LRFR load factors for live load are changed from those currently in the MBE to the proposed lower load factors associated with the target reliability index assumed in the development of the MBE, the number of bridges not passing the LRFR rating is further reduced.

The load factor for live load in the MBE for legal and permit loads is dependent on the ADTT while the load factor for live load in the LFR method is independent of ADTT. The MBE includes three ADTT categories: 100, 1,000, and 5,000.

Tables 50 through 57 are based on assuming an ADTT of 1,000. The numbers of bridges not passing the LRFR rating in these tables will increase if the load factors corresponding to ADTT of 5,000 are assumed while these numbers will decrease when the load factors corresponding to an ADTT of 100 are assumed.

3.9.9 Average Ratio of Rating Factors for Existing and Proposed Live Load Factors

The following sections show the average reduction in rating factor using the existing live load factors in the MBE and those proposed as a result of this research. Each section contains a table showing the effects of the proposed load factors for each vehicle (no revisions are proposed to the live load factor for the design load). The effects of using the proposed load factor on the distribution of ratings (similar to those

Table 55. Continuous span steel—effect of proposed live load factors.

Type	# of Girders	Vehicle Type	Vehicle	Effect	# of girders w/ LFR < 1.0	Using Existing Load Factors				Using Proposed Load Factors		
						# of girders w/ LRFR < 1.0				# of girders w/ LRFR < 1.0	# of girders w/ LFR < 1.0	% Change
						Total	% Change	Ignoring 80007	% Change			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Continuous Span Steel	418	Design	HL-93	Moment	89	145	13.40%			n/a	n/a	n/a
				Shear	29	100	16.99%	26	-0.72%	n/a	n/a	n/a
		Legal	AASHTO Type 3	Moment	2	24	5.26%			14	2	2.87%
				Shear	0	27	6.46%	10	2.39%	20	0	4.78%
			AASHTO Type 3-3	Moment	2	27	5.98%			14	2	2.87%
				Shear	5	45	9.57%	11	1.44%	31	5	6.22%
			AASHTO Type 3S2	Moment	3	30	6.46%			18	3	3.59%
				Shear	5	40	8.37%	18	3.11%	28	5	5.50%
		Routine Permit	DE-07	Moment	10	37	6.46%			23	10	3.11%
				Shear	5	45	9.57%	13	1.91%	32	5	6.46%
			FL-04	Moment	7	32	5.98%			17	7	2.39%
				Shear	5	39	8.13%	11	1.44%	28	5	5.50%
			NC-21	Moment	9	33	5.74%			24	9	3.59%
				Shear	3	36	7.89%	11	1.91%	23	3	4.78%
			NM-04	Moment	3	25	5.26%			12	3	2.15%
				Shear	1	28	6.46%	10	2.15%	20	1	4.55%
			TX-04	Moment	24	40	3.83%			26	24	0.48%
				Shear	5	47	10.05%	13	1.91%	27	5	5.26%
		Special Permit	IL-01	Moment	44	35	-2.15%			28	44	-3.83%
				Shear	14	50	8.61%	17	0.72%	41	14	6.46%
			OR-06	Moment	45	40	-1.20%			33	45	-2.87%
				Shear	17	54	8.85%	16	-0.24%	49	17	7.66%
			WA-02	Moment	141	122	-4.55%			75	141	-15.79%
				Shear	45	97	12.44%	59	3.35%	79	45	8.13%

shown in Section 3.6) are presented in Appendix M. For the special permit vehicles, only the values corresponding to a target reliability index of 2.5 are shown.

3.9.9.1 Simple Span Steel Girders

Table 58 shows the average of the ratio between the LRFR rating and the LFR rating for the existing live load factors in the MBE and the live load factors proposed. The proposed live load factors increase the ratio from the range of 0.71 to 0.76 to between 0.90 and 0.97 for moment for legal and routine permit vehicles. For special permit vehicles, the range increases from between 1.12 and 1.20 to between 1.26 and 1.35. The proposed live load factors increase the ratio from the range of 0.50 to 0.56 to between 0.64 and 0.76 for shear for legal and routine permit vehicles. For special permit vehicles, the range increases from between 0.78 and 0.81 to between 0.88 and 0.90; the average ratio is reduced due to the effect of the bearing stiffener rating.

3.9.9.2 Simple Span Prestressed Concrete I-Girders

Table 59 shows the average of the ratio between the LRFR rating and the LFR rating for the existing live load factors in the MBE and the live load factors proposed. The proposed live load factors result in a small increase in the average ratio for moment. This is due to either the effect of shear on the longitudinal steel causing extremely low ratings or the controlling rating being a service limit state which is unaffected by the proposed changes. The average ratio for the routine permit and legal vehicles is between 0.30 and 0.43, while for the special permit vehicles it is between 0.55 and 0.58. The average ratio of LRFR shear rating to LFR shear rating increased and is above 0.92 for all vehicles.

3.9.9.3 Simple Span Prestressed Concrete Box-Girders

Table 60 shows the average of the ratio between the LRFR rating and the LFR rating for the existing live load factors in

Table 56. Continuous reinforced concrete slabs—effect of proposed live load factors.

Type	# of Girders	Vehicle Type	Vehicle	Effect	# of girders with LFR < 1.0	Using Existing Load Factors				Using Proposed Load Factors		
						# of girders w/ LRFR < 1.0				# of girders w/ LRFR < 1.0	# of girders with LFR < 1.0	% Change**
						Total	% Change	Ignoring 85004	% Change			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Continuous Span Reinforced Concrete Slab Bridges	105	Design	HL-93	Moment	66	96	28.57%	80	13.33%	n/a	n/a	n/a
				Shear	43	74	29.52%			n/a	n/a	n/a
		Legal	AASHTO Type 3	Moment	8	51	40.95%	18	9.52%	24	8	15.24%
				Shear	0	5	4.76%			1	0	0.95%
			AASHTO Type 3-3	Moment	8	42	32.38%	10	1.90%	26	8	17.14%
				Shear	0	1	0.95%			1	0	0.95%
			AASHTO Type 3S2	Moment	9	71	59.05%	22	12.38%	36	9	25.71%
				Shear	0	5	4.76%			2	0	1.90%
		Routine Permit	DE-07	Moment	14	70	53.33%	21	6.67%	66	14	49.52%
				Shear	0	8	7.62%			5	0	4.76%
			FL-04	Moment	9	45	34.29%	11	1.90%	42	9	31.43%
				Shear	0	6	5.71%			2	0	1.90%
			NC-21	Moment	13	61	45.71%	23	9.52%	58	13	42.86%
				Shear	0	10	9.52%			5	0	4.76%
			NM-04	Moment	9	38	27.62%	12	2.86%	36	9	25.71%
				Shear	0	5	4.76%			1	0	0.95%
			TX-04	Moment	20	71	48.57%	35	14.29%	69	20	46.67%
				Shear	3	22	18.10%			10	3	6.67%
		Special Permit	IL-01	Moment	40	88	45.71%	59	18.10%	87	40	44.76%
				Shear	3	34	29.52%			23	3	19.05%
OR-06	Moment		34	85	48.57%	53	18.10%	84	34	47.62%		
	Shear		3	36	31.43%			27	3	22.86%		
WA-02	Moment		58	97	37.14%	80	20.95%	97	58	37.14%		
	Shear		9	68	56.19%			53	9	41.90%		

the MBE and the proposed live load factors. The proposed live load factors result in a small increase in the average ratio for moment. This is due to either the effect of shear on the longitudinal steel causing extremely low ratings or the controlling rating being a service limit state which is unaffected by the proposed changes. The average ratio for the routine permit and legal vehicles is between 0.47 and 0.51 while the ratio for all special permit vehicles is 0.66. The average ratio of LRFR shear rating to LFR shear rating increased and is above 1.27 for all vehicles.

3.9.9.4 Simple Span Reinforced Concrete T-Beams

Table 61 shows the average of the ratio between the LRFR rating and the LFR rating for the existing live load factors in the MBE and the proposed live load factors. The proposed live load factors result in a small increase in the average ratio for moment from 0.49–0.56 to 0.62–0.71 for the routine permit and legal vehicles. For the special permit vehicles the ratio

ranged from 0.62 to 0.64 and using the proposed live load factors has increased to 0.70–0.71. For shear, the average ratio increased from 0.72–0.81 to 0.86–1.0 for all vehicle types.

3.9.9.5 Simple Span Reinforced Concrete Slab Bridges

Table 62 shows the average of the ratio between the LRFR rating and the LFR rating for the existing live load factors in the MBE and the proposed live load factors. The proposed live load factors result in an increase in the average ratio for moment from 0.63–0.67 to 0.80–0.86 for the routine permit and legal vehicles. The average ratio of flexure ratings for the special permit vehicles increased from 0.79–0.81 to 0.89–0.91. For shear, the average ratio using the current load factors was between 1.05 and 1.11 for the routine permit and legal vehicles. This has increased to between 1.34 and 1.42 using the proposed load factors. For special permit vehicles, the ratio increased from 1.17–1.19 to 1.31–1.34.

Table 57. Continuous prestressed concrete I-beams—effect of proposed live load factors.

Type	# of Girders	Vehicle Type	Vehicle	Effect	# of girders with LFR < 1.0	Using Existing Load Factors					Using Proposed Load Factors		
						# of girders w/ LRFR < 1.0					# of girders w/ LRFR < 1.0	# of girders with LFR < 1.0	% Change**
						Total	% Change	Ignoring 85004	% Change	85107 (M) 85003 (V)			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Continuous Span Prestressed Concrete I-Beam	238	Design	HL-93	Moment	19	226	86.97%	75	23.53%	15.00	n/a	n/a	n/a
				Shear	36	113	32.35%			90.00	n/a	n/a	n/a
		Legal	AASHTO Type 3	Moment	0	172	72.27%	2	0.84%	2.00	133	0	55.88%
				Shear	0	45	18.91%			45.00	40	0	16.81%
			AASHTO Type 3-3	Moment	0	178	74.79%	2	0.84%	2.00	143	0	60.08%
				Shear	0	49	20.59%			49.00	36	0	15.13%
			AASHTO Type 3S2	Moment	0	181	76.05%	3	1.26%	3.001	42	0	59.66%
				Shear	0	50	21.01%			50.00	39	0	16.39%
		Routine Permit	DE-07	Moment	0	190	79.83%	6	2.52%	6.00	150	0	63.03%
				Shear	1	51	21.01%			51.00	44	1	18.07%
			FL-04	Moment	0	180	75.63%	5	2.10%	5.00	141	0	59.24%
				Shear	0	50	21.01%			50.00	43	0	18.07%
			NC-21	Moment	0	185	77.73%	11	4.62%	11.00	145	0	60.92%
				Shear	0	51	21.43%			51.00	44	0	18.49%
			NM-04	Moment	0	171	71.85%	3	1.26%	3.00	136	0	57.14%
				Shear	0	46	19.33%			46.00	39	0	16.39%
			TX-04	Moment	0	191	80.25%	14	5.88%	14.00	150	0	63.03%
				Shear	0	58	24.37%			56.00	46	0	19.33%
		Special Permit	IL-01	Moment	0	150	63.03%	14	5.88%	14.00	134	0	56.30%
				Shear	4	53	20.59%			49	47	4	18.07%
			OR-06	Moment	3	158	65.13%	14	4.62%	12.00	140	3	57.56%
				Shear	3	51	20.17%			48.00	43	3	16.81%
			WA-02	Moment	13	186	72.69%	64	21.43%	55.00	167	13	64.71%
Shear	28			80	21.85%			67.00	70	28	17.65%		

Table 58. Comparison of average LRFR/LFR ratio for simple span steel girders for existing and proposed load factors.

Vehicle	Moment (LRFR/LFR)		Shear (LRFR/LFR)	
	Existing	Proposed	Existing	Proposed
HL-93	0.98	N/A	0.65	N/A
DE-07	0.76	0.97	0.55	0.71
FL-04	0.76	0.97	0.55	0.71
IL-01	1.12	1.26	0.79	0.88
NC-21	0.75	0.97	0.56	0.72
NM-04	0.76	0.97	0.56	0.72
OR-06	1.20	1.35	0.81	0.90
TX-04	0.75	0.97	0.56	0.72
WA-02	1.12	1.26	0.78	0.88
Type 3	0.71	0.90	0.52	0.67
Type 3-3	0.71	0.90	0.50	0.64
Type 3S2	0.71	0.92	0.51	0.76

Table 59. Comparison of average LRFR/LFR rating ratio for simple span prestressed concrete I-girders for existing and proposed live load factors.

Vehicle	Moment (LRFR/LFR)		Shear (LRFR/LFR)	
	Existing	Proposed	Existing	Proposed
HL-93	0.62	N/A	0.78	N/A
DE-07	0.30	0.35	0.79	1.01
FL-04	0.29	0.34	0.81	1.04
IL-01	0.52	0.55	1.08	1.24
NC-21	0.31	0.43	0.82	0.92
NM-04	0.28	0.30	0.83	0.96
OR-06	0.52	0.56	1.07	1.20
TX-04	0.31	0.43	0.80	1.32
WA-02	0.56	0.58	1.00	1.12
Type 3	0.26	0.31	0.81	1.02
Type 3-3	0.25	0.30	0.78	1.00
Type 3S2	0.26	0.31	0.77	0.97

Table 60. Comparison of average LRFR/LFR ratio for simple span prestressed concrete box-girders for existing and proposed load factors.

Vehicle	Moment (LRFR/LFR)		Shear (LRFR/LFR)	
	Existing	Proposed	Existing	Proposed
HL-93	0.76	N/A	0.93	N/A
DE-07	0.48	0.50	1.04	1.33
FL-04	0.48	0.51	1.06	1.36
IL-01	0.64	0.66	1.30	1.46
NC-21	0.49	0.51	1.07	1.37
NM-04	0.48	0.50	1.11	1.41
OR-06	0.65	0.66	1.58	1.77
TX-04	0.49	0.51	0.99	1.27
WA-02	0.65	0.66	1.18	1.32
Type 3	0.46	0.48	1.06	1.34
Type 3-3	0.44	0.47	1.05	1.33
Type 3S2	0.45	0.48	1.03	1.31

Table 61. Comparison of average LRFR/LFR ratios for simple span reinforced concrete T-beams for existing and proposed load factors.

Vehicle	Moment (LRFR/LFR)		Shear (LRFR/LFR)	
	Existing	Proposed	Existing	Proposed
HL-93	0.64	N/A	0.83	N/A
DE-07	0.53	0.67	0.76	0.97
FL-04	0.53	0.67	0.76	0.98
IL-01	0.62	0.70	0.80	0.89
NC-21	0.56	0.71	0.76	0.97
NM-04	0.53	0.68	0.78	1.00
OR-06	0.64	0.71	0.81	0.90
TX-04	0.53	0.68	0.72	0.93
WA-02	0.63	0.71	0.76	0.86
Type 3	0.50	0.64	0.76	0.97
Type 3-3	0.49	0.62	0.78	0.99
Type 3S2	0.50	0.64	0.77	0.97

Table 62. Comparison of average LRFR/LFR ratios for simple span reinforced concrete slab bridges for existing and proposed load factors.

Vehicle	Moment (LRFR/LFR)		Shear (LRFR/LFR)	
	Existing	Proposed	Existing	Proposed
HL-93	0.73	N/A	1.27	N/A
DE-07	0.66	0.85	1.10	1.41
FL-04	0.66	0.85	1.10	1.40
IL-01	0.81	0.90	1.19	1.34
NC-21	0.67	0.86	1.08	1.39
NM-04	0.64	0.82	1.11	1.42
OR-06	0.81	0.91	1.18	1.32
TX-04	0.65	0.84	1.05	1.34
WA-02	0.79	0.89	1.17	1.31
Type 3	0.63	0.80	1.08	1.37
Type 3-3	0.63	0.80	1.10	1.40
Type 3S2	0.64	0.82	1.10	1.40

3.9.9.6 Continuous Span Steel Girder Bridges

Table 63 shows the average of the ratio between the LRFR rating and the LFR rating for the existing live load factors in the MBE and the proposed live load factors. The proposed live load factors result in an increase in the average ratio for moment from 0.81–0.88 to 1.02–1.12 for the AASHTO legal and routine permit vehicles. For shear, the average ratio increased from 0.63–0.69 to 0.79–0.89 for the legal and routine permit vehicles. For the special permit vehicles, the average moment ratio increased from 1.25–1.28 to 1.40–1.44 and for shear increased from 0.94–0.96 to 1.06–1.08.

3.9.9.7 Continuous Span Reinforced Concrete Slab Bridges

Table 64 shows the average of the ratio between the LRFR rating and the LFR rating for the existing live load factors in the MBE and the proposed live load factors. The proposed live load factors result in a very small increase in the average ratio for moment for the routine and special permit vehicles. For the AASHTO Legal Trucks, the average ratio increased from 0.46–0.50 to 0.59–0.63. For shear, the average ratio

increased from 0.74–0.85 to 0.95–1.12 for the routine permit and AASHTO Legal vehicles. The average ratio of shear ratings for the special permit vehicles increased from 0.71–0.78 to 0.80–0.87.

3.9.9.8 Continuous Span Prestressed Concrete I-Girder Bridges

Table 65 shows the average of the ratio between the LRFR rating and the LFR rating for the existing live load factors in the MBE and the proposed live load factors. The proposed live load factors result in a small increase in the average ratio for moment for the AASHTO legal and routine permit vehicles. The ratio for moment generally increased from 0.2–0.25 to 0.25–0.32. For shear, the average ratio increased from 0.76–0.80 to 0.96–1.02 for the routine permit and AASHTO legal vehicles. For the special permit vehicles, the average moment rating increased from 0.43–0.56 to 0.47–0.61 and the average shear rating increased from 1.02–1.12 to 1.14–1.25. The small increase in moment ratios is a result of including the ratings for service limit states (which is unaffected by changing the strength load factor) and the effect of shear on longitudinal steel.

Table 63. Comparison of average LRFR/LFR ratios for continuous span steel girder bridges for existing and proposed load factors.

Vehicle	Moment (LRFR/LFR)		Shear (LRFR/LFR)	
	Existing	Proposed	Existing	Proposed
HL-93	0.92	N/A	0.75	N/A
DE-07	0.86	1.10	0.67	0.86
FL-04	0.86	1.10	0.67	0.86
IL-01	1.28	1.44	0.96	1.07
NC-21	0.88	1.12	0.69	0.89
NM-04	0.87	1.12	0.68	0.88
OR-06	1.27	1.42	0.96	1.08
TX-04	0.88	1.12	0.69	0.89
WA-02	1.25	1.40	0.94	1.06
Type 3	0.85	1.07	0.66	0.84
Type 3-3	0.81	1.02	0.63	0.79
Type 3S2	0.82	1.04	0.64	0.81

Table 64. Comparison of average LRFR/LFR ratios for continuous span reinforced concrete slab bridges for existing and proposed load factors.

Vehicle	Moment (LRFR/LFR)		Shear (LRFR/LFR)	
	Existing	Proposed	Existing	Proposed
HL-93	0.62	N/A	0.79	N/A
DE-07	0.56	0.57	0.79	1.01
FL-04	0.56	0.57	0.81	1.04
IL-01	0.57	0.57	0.78	0.87
NC-21	0.59	0.60	0.80	1.02
NM-04	0.58	0.59	0.87	1.12
OR-06	0.58	0.58	0.77	0.86
TX-04	0.59	0.59	0.74	0.95
WA-02	0.56	0.57	0.71	0.80
Type 3	0.50	0.63	0.84	1.07
Type 3-3	0.46	0.59	0.85	1.09
Type 3S2	0.48	0.60	0.82	1.04

Table 65. Comparison of average LRFR/LFR ratios for continuous span prestressed concrete I-girder bridges for existing and proposed load factors.

Vehicle	Moment (LRFR/LFR)		Shear (LRFR/LFR)	
	Existing	Proposed	Existing	Proposed
HL-93	0.42	N/A	0.81	N/A
DE-07	0.24	0.31	0.78	1.00
FL-04	0.23	0.29	0.80	1.02
IL-01	0.43	0.47	1.12	1.25
NC-21	0.24	0.30	0.78	0.99
NM-04	0.21	0.27	0.80	1.02
OR-06	0.49	0.54	1.11	1.24
TX-04	0.25	0.32	0.79	1.01
WA-02	0.56	0.61	1.02	1.14
Type 3	0.20	0.25	0.79	1.00
Type 3-3	0.21	0.26	0.76	0.97
Type 3S2	0.21	0.27	0.76	0.96

CHAPTER 4

Conclusions

Following are conclusions and suggestions that can be drawn based on the review of the analysis results shown in this report and in the appendices:

- No clear trends based on the sorting criteria checked, i.e., year of construction, girder spacing, skew angle and span length, could be identified.
- A significant percentage of the girders that passed rating under the LFR and produced rating factors below 1.0 for the LRFR were controlled by criteria that were not checked under the LFR method of rating. Some of these criteria are not known to have caused problems in the past. It is proposed that these criteria be checked only when visible signs of distress related to these criteria are observed or make checking these criteria optional. This is similar to the current provisions in the MBE that allow shear rating of concrete components to be ignored under legal loads when no visible signs of distress exist and the provisions that make checking the Service III limit state under legal loads optional. These criteria include bearing stiffeners on steel bridges, the effect of shear on the stress in longitudinal reinforcement near the ends of concrete girders, interface shear between girders and cast-in-place decks, and service limit states.
- When the additional design criteria checked under the LRFR rating are ignored, the difference in the number of bridges not passing the LRFR rating, as compared to the number of those not passing the LFR rating, decreases significantly
- For AASHTO Legal Loads and routine permit vehicles, the load factors for live load included in the MBE appear to produce a higher reliability index higher than that assumed in the development of the MBE. Reducing the load factors to correspond to the target reliability index will result in reducing the number of bridges not passing the LRFR rating for these loads and still satisfy the target level of reliability.
- The target reliability index for special permits originally used in the development of the MBE was 2.5. Toward the end of the MBE development, it was decided to increase this target reliability index for these cases to 3.5. The rationale behind this increase was the belief that using the target reliability index of 2.5 will allow heavier trucks on the system and will lead to additional maintenance problems. Based on the analysis conducted herein, the load factors currently in the MBE for unescorted special permit vehicles are showing that a large percentage of bridges do not pass the LRFR rating even though these bridges passed the LFR rating and no signs of distress that can be related to the special permit vehicle could be detected. This indicates that using a target reliability index of 3.5 for these cases may be over-conservative. It is proposed that the target reliability index for these cases be lowered to 2.5 as assumed for legal vehicles and as was originally assumed throughout most of the development of the MBE.
- Due to the difficulty in enforcing the routes used by permit vehicles, it is proposed that the values corresponding to ADTT less than 100 be removed from the load factor tables.

4.1 Proposed Revisions to the MBE

Based on the conclusions and proposals presented, the following revisions to the MBE are suggestions. Deletions to the existing MBE provisions are shown with a strike through while additions are shown underlined.

Note: The revisions to Appendix A of the MBE related to these proposed revisions are provided in Appendix N of this document.

The proposed revisions provided on the following pages are modified from the *AASHTO Manual for Bridge Evaluation*, Copyright 2008, by the American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.

6A.4—LOAD-RATING PROCEDURES

6A.4.1—Introduction

Three load-rating procedures that are consistent with the load and resistance factor philosophy have been provided in Article 6A.4 for the load capacity evaluation of in-service bridges:

- Design load rating (first level evaluation)
- Legal load rating (second level evaluation)
- Permit load rating (third level evaluation)

Each procedure is geared to a specific live load model with specially calibrated load factors aimed at maintaining a uniform and acceptable level of reliability in all evaluations.

Load factors for evaluation may be taken from Articles A6.4.3, A6.4.4 and A6.4.5, as applicable. Where adequate information on the traffic is available, site, route or region-specific load factors may be developed. If accepted by the Owner, these load factors may be used in lieu of the values given in this manual.

The load rating is generally expressed as a rating factor for a particular live load model, using the general load-rating equation provided in Article 6A.4.2.

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6A.4.4.2.3—Generalized Live Load Factors: γ_L

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

Generalized live load factors for the STRENGTH I limit state are specified in Table 1 for routine commercial traffic. If in the Engineer’s judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in this Manual when determining the safe legal load, the Engineer may increase the factors in Table 1, not to exceed the value of the factor multiplied by 1.3.

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

Traffic Volume (one direction)	Load Factor
Unknown	1.80 1.45
ADTT 5000	1.80 1.45

C6A.4.1

The load-rating procedures are structured to be performed in a sequential manner, as needed, starting with the design load rating (see flowchart in Appendix A6A). Load rating for AASHTO Legal loads is required only when a bridge fails ($RF < 1$) the Design load rating at the Operating level. Similarly, only bridges that pass the load rating for AASHTO legal loads should be evaluated for overweight permits.

Weigh-In-Motion (WIM) data collected at specific site, along a specific route or around a specific region may be used to perform a load calibration to determine site, route, or region-specific load factors. Depending on the traffic pattern and truck counts, these load factors may be higher or lower than those listed in this manual.

C6A.4.4.2.3

C6A.4.4.2.3a

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

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

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

ADTT = 1000	1.65 1.30
ADTT = 100	1.40

Linear interpolation is permitted for other ADTT.

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

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

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

Traffic Volume (one direction)	Load Factor
Unknown	1.60 1.45
ADTT \geq 5000	1.60 1.45
ADTT = 1000	1.40 1.30
ADTT \leq 100	1.15

Linear interpolation is permitted for other ADTT.

6A.4.5.4.2a—Routine (Annual) Permits

The live load factors given in Table 1 for evaluating routine permits shall be applied to a given permit vehicle or to the maximum load effects of all permit vehicles allowed to operate under a single-routine permit. A multi-lane loaded distribution factor shall be used to account for the likelihood of the permits being present alongside other heavy vehicles while crossing a bridge.

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

C6A.4.4.2.3b

The live load factors provided in these specifications account for the multiple-presence of two heavy trucks side-by-side on a multi-lane bridge as well as the probability that trucks may be loaded in such a manner that they exceed the corresponding legal limits. Using the reliability analysis and data applied in AASHTO LRFD and LRFR Specifications show that the live load factor should increase as the ADTT increases. The increase in γ_L with ADTT is provided in Table 1 for routine commercial traffic. The same consideration for SHVs using field data and assumptions for the percent of SHVs in the traffic stream led to the γ_L factors in Table 1 for SHVs. Since there are typically fewer SHVs than routine commercial trucks in the traffic stream the live load factor in Table 1 are appreciably smaller than the corresponding factors in Table 6A.4.4.2.3a-1. A description of the development of the γ_L values is given in NCHRP Report 454 and the NCHRP Project 12-63 Final Report.

C6A.4.5.4.2a

The target reliability level for routine permit crossings is established as the same level as for legal loads given in Article 6A.4.4, namely, consistent with traditional AASHTO Operating ratings.

The live load factors for routine permits given in Table 1 depend on both the ADTT of the site ~~and the magnitude of the permit load~~. In the case of routine permits, the expected number of such permit-crossings is unknown so a conservative approach to dealing with the possibility of multiple presence is adopted.

The load factors listed in earlier editions of this manual for special or limited crossings for vehicles mixed with other traffic were based on a target reliability index of 3.5.

The load factors listed below for these permits were developed under the NCHRP 12-78 project and are based on a target reliability index of 2.5.

If a bridge is located on a low volume route, say ADTT <100, it is unlikely that more than one truck will exist on the bridge at the same time. In such cases, a single lane loaded distribution factors may be appropriate when evaluating routine or annual permits.

Table 6A.4.5.4.2a-1—Permit Load Factors: γ_L

Permit Type	Frequency	Loading Condition	DF ^a	ADTT (one direction)	Load Factor by Permit Weight ^b	
					Up to 100 kips	>150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.80 <u>1.45</u>	1.30
				=1000	1.60 <u>1.25</u>	1.20
				<100	1.40	1.10
					All Weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	<u>1.15</u>	
				Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane
	=1000	1.40 <u>1.25</u>				
	<100	1.35				
	Multiple-Trips (less than 100 crossings) ^b	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.85 <u>1.60</u>	
				=1000	1.75 <u>1.50</u>	
<100				1.55		

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

^b ~~For routine permits between 100 kips and 150 kips, interpolate the load factor by weight and ADTT value. Use only axle weights on the bridge.~~

^b The limit of 100 crossings is meant to be the number of crossings allowed under the particular permit being evaluated. It is not meant to be the number of crossings made by all permit vehicles that may cross the bridge. If a limited crossing permit that allows more than 100 trips is issued, this permit should be evaluated as a routine permit.

6A.5—CONCRETE STRUCTURES

6A.5.1—Scope

The provisions of Article 6A.5 apply to the evaluation of concrete bridge components Provisions for the rating of

6A.5.4—Limit States

The applicable limit states and their load combinations for the evaluation

6A.5.4.1—Design-Load Rating

The Strength I load combinations shall be checked for reinforced concrete

6A.5.4.2—Legal Load Rating and Permit Load Rating

Load ratings for legal loads and permit loads shall be based on satisfying the requirements for the strength limit and service limit states, guided by considerations presented in these articles.

6A.5.4.2.1—Strength Limit State

Concrete bridge components shall be load rated for the Strength I load combination for legal loads, and for Strength II load combination for permit loads.

*6A.5.4.2.2—Service Limit State**6A.5.4.2.2a—Legal Load Rating*

Load rating of prestressed concrete bridges based on satisfying limiting concrete tensile stresses under service loads at the Service III limit state is considered optional, except for segmentally constructed bridges. A live load factor of 1.0 is recommended for legal loads when using this check for rating purposes.

C6A.5.4.1

Service III need not be checked for HL-93 at the Operating level as Service III is

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C6A.5.4.2.2a

Prestressed concrete components are expected to crack in flexure under the first heavy load causing the concrete tensile stresses to exceed the modulus of rupture of the concrete. After cracking, the cracks will open when the decompression state is reached, i.e. when the concrete is under tensile stress. Infrequent opening of the cracks is not considered detrimental to the performance of the prestressed concrete components. However, frequent crack opening, particularly if the width of the crack is relatively large, is undesirable as it may allow contaminants to enter the crack and cause corrosion of the prestressing strands. Unless the legal load is relatively heavy and is expected to frequently cross the bridge, the crack opening under the legal load may be considered as part of the expected crack opening under routine traffic. At the time of writing of these provisions (2010), research is being conducted under the NCHRP Project 12-83 to quantify the frequency of crack opening under routine traffic.

These provisions for evaluation of prestressed concrete bridges permit, but do not encourage, the past practice of limiting concrete tensile stresses at service load. In design, limiting the tensile stresses of fully prestressed concrete members based on uncracked section properties is considered appropriate. This check of the Service III load combination may be appropriate for prestressed concrete bridges that exhibit visible flexural cracking under normal traffic.

6A.5.4.2.2b—Permit Load Rating

The provisions of this Article are considered optional and apply to the Service I load combination for reinforced concrete components and prestressed concrete components.

During permit load rating, the stresses in the reinforcing bars and/or prestressing steel nearest the extreme tension fiber of the member should not exceed 0.90 of the yield point stress for unfactored loads.

In the absence of a well-defined yield stress for prestressing steels, the following values of f_{py} are defined:

Table 6A.5.4.2.2b-1—Yield Strength of Prestressing Steel

Type of Tendon	f_{py}
Low-Relaxation Strand	$0.9f_{pu}$
Stress-Relieved Strand and Type 1 High-Strength Bar	$0.85f_{pu}$
Type 2 High-Strength Bar	$0.80f_{pu}$

6A.5.9—Evaluation for Shear

The shear capacity of existing reinforced and prestressed concrete bridge members should be evaluated for permit loads when the factored load effects from the permit load exceed the factored load effects from the design load. In-service concrete bridges that show no

Service limit states are mandatory for the rating of segmental concrete bridges, as specified in Article 6A.5.13.5.

C6A.5.4.2.2b

This check is carried out using the Service I combination where all loads are taken at their nominal values. It should be noted that in design, Service I is not used to investigate tensile steel stresses in concrete components. In this regard, it constitutes a departure from the *AASHTO LRFD Bridge Design Specifications*.

Limiting steel stress to $0.9F_y$ is intended to ensure that cracks that develop during the passage of overweight vehicles will close once the vehicle is removed. It also ensures that there is reserve ductility in the member.

LRFD distribution analysis methods specified in LRFD Design Article 4.6.2 should be used when checking Service I for permit loads. (Whereas, Strength II analysis is done using distribution analysis methods prescribed in this Manual.) In other words, a one- or two-lane distribution factor, whichever applies or governs, should be used for both routine and special permits when checking Service I. Escorted special permits operating with no other vehicles on the bridge may be analyzed using one-lane distribution factors.

For concrete members with standard designs and closely clustered tension reinforcement, the Engineer may, as an alternate to limiting the steel stress, choose to limit unfactored moments to 75 percent of nominal flexural capacity. Where computations are performed in terms of moments rather than stresses, it is often easier to check limiting moments than it is to check limiting stresses. This is especially true for prestressed components where stress checks usually require the consideration of loading stages.

Uncracked prestressed concrete components are expected to develop flexural cracks, and the flexural cracks previously developed in prestressed components are expected to open, under heavy permit loads. Therefore, rating for Service III limit state is not required under permit loading. However, past practice indicates that some owners with routes known to be used by a high number of permit vehicles still choose to rate bridges on such routes for the Service III limit state.

C6A.5.9

Evaluation for shear was not required under earlier load factor design rating. Not evaluating for shear when rating for design and legal loads when no visible signs of shear distress exist is meant to balance the need to ensure the safety of bridges and the need not to be overly conservative.

visible signs of shear distress need not be checked for shear when rating for the design load or legal loads.

In order to eliminate the possibility of being more conservative when rating for permit vehicles than when rating for design loads, the evaluation for shear under permit vehicles is limited to the cases where the factored load effects from the permit vehicles exceed that from the design load.

Design provisions based on the Modified Compression Field Theory (MCFT) are incorporated in the LRFD Design specifications. The MCFT is capable of giving more accurate predictions of the shear response of existing reinforced and prestressed concrete bridge members, with and without web reinforcement. In lieu of the more detailed analysis outlined in the LRFD Design specifications, a simplified analysis that assumes $\beta = 2.0$ and $\theta = 45^\circ$ may be first attempted for reinforced concrete sections and standard prestressed concrete sections with transverse reinforcement. The expressions for shear strength then become essentially identical to those traditionally used for evaluating shear resistance. Where necessary, a more accurate evaluation using MCFT may be performed.

Live load shear for existing bridge girders using the LRFD Design specifications could be higher than the shear obtained from the AASHTO Standard Specifications due to higher live load, higher live load distribution factors for shear, and the higher dynamic load allowance. On the other hand, LRFD Design specifications may yield higher shear resistance for prestressed concrete sections at high-shear locations. MCFT uses the variable angle (θ) truss model to determine shear resistance. Higher prestress levels give flatter θ angles. Flatter θ angles could give higher shear resistances except at regions with high moment and shear.

Prestressed concrete shear capacities are load dependent, which means computing the shear capacity involves an iterative process when using the current AASHTO MCFT. Multiple locations, preferably at 0.05 points, need to be checked for shear. Location where shear is highest may not be critical because the corresponding moment may be quite low. Typically, locations near the 0.25 point could be critical because of relatively high levels of both shear and moment. Also contributing to the need for checking multiple locations along the beam is the fact that the stirrup spacings are typically not constant, but vary.

~~This check is required~~

When using the Modified Compression Field Theory (MCFT) for the evaluation of concrete shear resistance, the longitudinal reinforcement should be checked for the increased tension caused by shear, in accordance with LRFD Design Article 5.8.3.5. This check need not be applied to in-service concrete bridges that show no visible signs of shear distress when rating for the design load or legal loads. For permit vehicles, this check is required for permit loads only when the factored load effects from the permit load exceed the factored load effects from the design load.

Rating for the shear friction provisions of the AASHTO LRFD at the interface between a concrete beam and the concrete deck is not required.

The literature does not include cases of distress due to shear friction failure along the interface between a concrete beam and the deck.

6A.6—STEEL STRUCTURES

6A.6.1—Scope

The provisions of Article 6A.6 shall apply to the evaluation of steel and wrought-iron

C6A.6.1

LRFD Design Article 6.10 provides a unified approach for consideration of combined

6A.6.4—Limit States

The applicable limit states and their load combinations for the evaluation of structural steel and wrought iron members are specified for the various rating procedures. The load combinations, and the load factors which comprise them, are specified in Table 6A.4.2.2-1 and in these Articles.

6A.6.4.1—Design-Load Rating

Strength I and Service II load combinations shall be checked for the design loading. Live load factors shall be taken as tabulated in Table 6A.4.2.2-1.

C6A.6.4.1

Rating factors for applicable strength, service, and fatigue limit states computed during the design load rating will aid in identifying vulnerable limit states for further evaluation and future inspections.

At bearing locations on rolled shapes and at other locations on built-up sections or rolled shapes subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, either bearing stiffeners shall be provided or the web shall satisfy the provisions of LRFD Article D6.5.

Load rating of bearing stiffeners is not required.

The design method of bearing stiffeners did not change over the years. The design equations controlling the design of these components originated in the AASHTO Standard Specifications for Highway Bridges and were based on designing for service loads. For the range of dead load to live load ratios associated with typical bridges, the equations in AASHTO LRFD Bridge Design Specifications give essentially the same resistance as in AASHTO Standard Specifications after accounting for the load factors. The rating calculations under past rating manuals did not require load rating based on the load and resistance of bearing stiffeners. There have been no reports of distress in bearing stiffeners of in-service bridges. This lack of history of bearing stiffener distress is the rationale behind not considering them in determining the controlling load rating.

In situations where fatigue-prone details are present

(category C or lower) a rating factor for infinite fatigue life should be computed. Members that do not satisfy the infinite fatigue life check may be evaluated for remaining fatigue life using procedures given in Section 7. This is an optional requirement.

6A.6.4.2—Legal Load Rating and Permit Load Rating

Ratings for legal loads and permit loads shall be based on satisfying the requirements for the strength and service limit states, guided by the considerations discussed in this Article.

6A.6.4.2.1—Strength Limit State

Steel bridge components shall be load rated for the Strength I load combination for legal loads, and for Strength II load combination for permit loads.

At bearing locations on rolled shapes and at other locations on built-up sections or rolled shapes subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, either bearing stiffeners shall be provided or the web shall satisfy the provisions of LRFD Article D6.5.

Load rating of bearing stiffeners is not required for legal load rating. For permit vehicles, evaluation of the bearing stiffeners is required for permit loads only when the factored reaction from the permit load exceeds the factored reaction from the design load.

6A.6.4.2.2—Service Limit State

Service II load combination check, in conjunction with the service limit state control of permanent deflection of LRFD Design Article 6.10.4.2 and

C6A.6.4.2.1

Load factors for the Strength Limit state are given in Table 6A.4.4.2.3a-1 and Table 6A.4.5.4.2a-1.

The rationale behind ignoring bearing stiffeners in determining the controlling load ratings is presented in Article C6A.4.6.4.1.

C6A.6.4.2.2

The reduced load factors for Service II, compared to load-factor design and rating, reflect a more liberalized approach to applying Service II checks for evaluation versus design.

4.2 Data Archiving

It is expected that the data results produced by the final bridge domain will be of interest for future specification refinement and thus will be made available in a format that permits regression testing.

The data at the end of this research provides a snapshot of the bridges included in the research for the LRFR speci-

cations available at that time. The data will be delivered in Microsoft Access format in tables as described in *NCHRP Report 485: Bridge Software—Validation Guidelines and Examples*.

Since Process 12-50 primarily documents analysis results related data, the input for the bridge database will be stored using the XML export option in Virtis. The format for the file naming will be as described earlier in this report.

APPENDICES A THROUGH R

Appendices A through R are not published herein, but are available on the TRB website at <http://www.trb.org/Main/Blurbs/165576.aspx>. The appendix titles are:

Appendix A	Final Bridge/Girder List
Appendix B	Simple Span Steel Girder Bridges
Appendix C	Simple Span Prestressed I-Girder Bridges
Appendix D	Simple Span Prestressed Box Girder Bridges
Appendix E	Simple Span Reinforced Concrete T-Beam Bridges
Appendix F	Simple Span Reinforced Concrete Slab Bridges
Appendix G	Steel I-Girder Continuous Span Bridges
Appendix H	Continuous Reinforced Concrete Slab Bridges
Appendix I	Continuous Prestressed I-Girder Bridges
Appendix J	Calculated Reliability Indices
Appendix K	Effect of Permit Type and ADTT on LRFR Ratings
Appendix L	Effect of LRFR Rating on Operating Rating
Appendix M	Effect of LRFR on Rating Using Proposed Load Factors
Appendix N	MBE Examples
Appendix O	Review of the NBI/Virtis Databases
Appendix P	Final Survey
Appendix Q	Changes required for NCHRP 12-50/Software Documentation
Appendix R	Format of CSV output produced by RIO software

Abbreviations and acronyms used without definitions in TRB publications:

AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI-NA	Airports Council International-North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	Air Transport Association
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation