THE GUIDE MANUAL
FOR BRIDGE EVALUATION
FOR IMPLEMENTS OF HUSBANDRY

AASHTO

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FOREWORD

The Guide Manual for Bridge Evaluation for Implements of Husbandry (MBEIoH) was first adopted by the AASHTO Highways Subcommittee on Bridges and Structures in 20???. The MBEIoH refers to *Manual for Bridge Evaluation* and *LRFD Bridge Design Specifications.*.

AASHTO Highways Subcommittee on Bridges and Structures

PREFACE

The Guide Manual for Bridge Evaluation for Implements of Husbandry (MBEIoH) offers assistance to Bridge Owners at all phases of bridge inspection and evaluation. An abbreviated table of contents follows this preface. Detailed tables of contents precede Sections 1 and 2.

Appendix A includes six illustrative examples (A1 through A4, A7, and A9), previously in the *Manual for Bridge Evaluation*. For easy comparison with and reference to the illustrative examples in *Manual for Bridge Evaluation*, the order of identification of these examples skipped A5, A6, and A8. These examples also retain the rating calculations for other vehicular loads than implements of husbandry for each comparison. All examples are rated using the LRFR method. In addition, Examples A1 and A2 are also rated using the LFR method. To clarify which rating method is being illustrated, examples using multiple methods are generally divided into Parts A through C and their articles are numbered accordingly as follows:

- Part A, LRFR;
- Part B, LFR; and
- Part C, example summary.

For ease of reference, the table of contents for Appendix A includes a summary table of the bridge types, rated members, rating live loads, limit states for evaluation, and rating methods, with the starting page number for each example and, in the case of Examples A1, A2, and A4, for each rating method. The typical detailed table of contents follows this summary table.

Appendix A includes numerous citations of other AASHTO bridge publications. To save space, the following shorthand has been adopted:

- "AASHTO Standard Specifications" refers to the current edition of the AASHTO *Standard Specifications for Highway Bridges*, 17th Edition, HB-17,
- "LRFD Design" refers to the current edition of the *AASHTO LRFD Bridge Design Specifications*, Eighth Edition, LRFD-8, and
- "MBE" refers to this publication, *The Manual for Bridge Evaluation*, Third Edition, MBE-3.

AASHTO Publications Staff

ABBREVIATED TABLE OF CONTENTS

SECTION 1: INTRODUCTION

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SECTION 1:

INTRODUCTION

1.1—PURPOSE AND IMPORTANT REFERENCES

The purpose of *The Guide Manual for Bridge Evaluation for Implements of Husbandry* (MBEIoH) is to serve as a resource for use in developing specific program, policy, and/or procedures for the inspection and evaluation of existing in-service highway bridges regarding implements of husbandry (IoH).

The MBEIoH is to be used along with other reference documents such as the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Bridge Evaluation* (MBE), which references *Manual for Bridge Element Inspection*, the Federal Highway Administration's (FHWA) *Bridge Inspector's Reference Manual* (BIRM), and the latest National Bridge Inventory (NBI) coding guidance document for the inspection and evaluation of the nation's bridges.

AASHTO MBE shall be referenced if this Guide Manual is silent.

1.2—SCOPE

The Guide Manual has been divided into two Sections:

- Section 1—Purpose and important references, scope, applicability and program establishment, inspection and evaluation quality measures, and definition of general interest terms.
- Section 2—Specifications for the load rating of bridges regarding IoH. It includes the Load and Resistance Factor method and the Load Factor method

C.1.2

ASR is not included in this Guide Manual, which is out of the scope of NCHRP Project 12-110 that developed this Guide Manual.

The successful application of this Guide Manual is directly related to the DOT and/or Bridge Owner organizational structure. Such a structure should be both effective and responsive so that the unique characteristics and special problems of individual bridges regarding IoH are considered in developing an appropriate inspection plan and load capacity determination. The involved Bridge Owner may be a local agency. Therefore, State DOT may need to provide technical and other assistance and guidance in implementing the provisions herein when needed.

1.3—APPLICABILITY AND PROGRAM ESTABLISHMENT

The provisions of this Guide Manual apply to all highway structures that qualify as bridges in accordance with the AASHTO definition and allow IoH to cross. These provisions may be applied to smaller structures which do not qualify as bridges at the discretion of the DOT and/or Bridge Owner.

Bridge Owners should carefully design and initiate the bridge evaluation program for IoH if such a program is currently not in place for the jurisdiction. This design and planning may be pursued with the following steps.

1) Identify stakeholders to work with through the remaining steps.
2) Identify vulnerable bridges to IoH load.
3) Develop contents of load rating program including consideration to results of Steps 1) and 2).
4) Develop strategies for implementation.
5) Assist in legislation if needed.
6) Implement, monitor, and further enhance.

Stakeholders are the parties that will operate, benefit from, and/or be impacted by the bridge evaluation program to accommodate IoH. Their input is important in design and planning of the program for it to be successful in ensuring safety of the bridges covered.

Vulnerable bridges in the jurisdiction's network are expected to be negatively impacted by the bridge evaluation program for IoH, such as those already load-posted.

Development of a load rating program for bridges to accommodate IoH should consider input of the stakeholders. The identified vulnerable bridges also need to receive adequate attention in such development.

Implementation of the bridge evaluation program for IoH may require a strategy, which may include stages, feedback, and update.

Legislative actions may be required when implementing the bridge evaluation program for IoH. Bridge Owners should actively assist in such activities and actions.

Continuing improvement for the bridge evaluation program for IoH is important especially in its early stage of a number of first years. Such improvement will help establish a robust program to protect the bridges in the jurisdiction and accommodate the need of IoH to access public roads.

More details of these steps are included in the recommended protocols in Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry,* Transportation Research Board, 202?

1.4—DEFINITIONS AND TERMINOLOGY

AASHTO—American Association of State Highway and Transportation Officials.

As-Built Plans—Plans that show the state of the bridge at the end of construction; usually prepared by the Contractor or the resident Engineer.

ASR—Allowable Stress Rating.

Bias—The ratio of mean to nominal value of a random variable.

Bridge—A structure including supports erected over a depression or an obstruction such as water, highway, or railway; having a track or passageway for carrying traffic or other moving loads; and having an opening measured along the center of the roadway of more than 20 ft between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes. It may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

Bridge Management System (BMS)—A system designed to optimize the use of available resources for the inspection, maintenance, rehabilitation, and replacement of bridges.

Calibration—A process of adjusting the parameters in a new standard to achieve approximately the same reliability as exists in a current standard or specification or to achieve a target reliability index.

Coefficient of Variation—The ratio of the standard deviation to the mean of a random variable.

Collapse—A major change in the geometry of the bridge rendering it unfit for use.

Complex Bridges—Movable, suspension, cable stayed, and other bridges with unusual characteristics.

Condition Rating—The result of the assessment of the functional capability and the physical condition of bridge components by considering the extent of deterioration and other defects.

Evaluation—An assessment of the performance of an existing bridge.

Exclusion Vehicle—Grandfather provisions in the federal statutes which allow states to retain higher limits than the federal weight limits if such limits were in effect when the applicable federal statutes were enacted. Exclusion vehicles are vehicles routinely permitted on highways of various states under grandfather exclusions to weight laws.

Failure—A condition where a limit state is reached or exceeded. This may or may not involve collapse or other catastrophic occurrences.

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

Implements of Husbandry (IoH) — Vehicles that are designed to be used in the field or on the farm. They typically consist of a self-propelled unit such as a tractor or of such a self-propelled unit hauling equipment or a tank. They are required to travel at a low speed by IoH design and/or by jurisdiction statutes.

Inventory Rating—Load ratings based on the inventory level allow comparisons with the capacity for new structures and, therefore, results in a live load, which can safely utilize an existing structure for an indefinite period of time.

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

LFR—Load Factor Rating.

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

Load Effect—The response (axial force, shear force, bending moment, torque) in a member or an element due to the loading.

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

Load Rating—The determination of the live-load carrying capacity of an existing bridge.

LRFD—Load and Resistance Factor Design.

LRFD Exclusion Limits—Weight and length limits of trucks operating under grandfather exclusions to federal weight laws.

LRFR—Load and Resistance Factor Rating.

Margin of Safety—Defined as *R-S*, where *S* is the maximum loading and *R* is the corresponding resistance (*R* and *S* are assumed to be independent random variables).

MUTCD—*Manual on Uniform Traffic Control Devices*.

National Bridge Inventory (NBI)—The aggregation of structure inventory and appraisal data collected to fulfill the requirements of the National Bridge Inspection Standards.

National Bridge Inspection Standards (NBIS)—Federal regulations establishing requirements for inspection procedures, frequency of inspections, a bridge inspection organization, qualifications of personnel, inspection reports, and preparation and maintenance of bridge inventory records. The NBIS apply to all structures defined as highway bridges located on or over all public roads.

NICET—National Institute for Certification in Engineering Technologies.

Nominal Resistance—Resistance of a component or connection to load effects, based on its geometry, permissible stresses, or specified strength of materials.

Operating Rating (ASR, LFR)—Load ratings based on the operating rating level generally describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at operating level may shorten the life of the bridge.

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

Owner—Agency having jurisdiction over the bridge.

Posting—Signing a bridge for load restriction.

Quality Assurance—The use of sampling and other measures to assure the adequacy of quality control procedures in order to verify or measure the quality level of the entire bridge inspection and load rating program.

Quality Control—Procedures that are intended to maintain the quality of a bridge inspection and load rating at or above a specified level.

RF—Rating Factor.

Reliability Index—A computed quantity defining the relative safety of a structural element or structure expressed as the number of standard deviations that the mean of the margin of safety falls on the safe side.

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

Safe Load Capacity—A live load that can safely utilize a bridge repeatedly over the duration of a specified inspection cycle.

Scour Critical Bridge—A bridge whose foundation (or foundations) has been determined to be unstable for the predicted scour conditions.

Service Limit State—Limit state relating to stress, deformation, and cracking.

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

Serviceability Limit States— Collective term for service and fatigue limit states.

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

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

Structure Inventory and Appraisal Sheet (SI&A)—A summary sheet of bridge data required by NBIS.

Target Reliability—A desired level of reliability (safety) in a proposed evaluation.

1.5—REFERENCES

AASHTO. *Manual for Bridge Evaluation*, 3rd Edition. American Association of State Highway and Transportation Officials, Washington, DC, 2018.

AASHTO. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17. American Association of State Highway and Transportation Officials, Washington, DC, 2002.

AASHTO. *AASHTO LRFD Bridge Design Specifications*, Eighth Edition, LRFD-8. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

Fu,G., Q.Wang, J.Chi, M.Lwin, and R.Corotis NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry,* Transportation Research Board, 202?

SECTION 1:

INTRODUCTION

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SECTION 2: LOAD RATING

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LOAD RATING

2.1—SCOPE

This Section sets forth criteria for the load rating of existing bridges and provides a choice of load rating methods regarding IoH. Part A incorporates provisions specific to the Load and Resistance Factor Rating (LRFR) method developed to provide uniform reliability in bridge load ratings and permit decisions. Part B provides safety criteria and procedures for the Load Factor Rating (LFR) method of evaluation. Its live load factors are calibrated compatible with the LRFR method. Both are also calibrated consistent with the LRFR method in *AASHTO Manual for Bridge Evaluation* for regular highway trucks. No preference is placed on any of the two rating methods herein. Any of these two methods may be used to establish live load capacities regarding IoH.

Bridge Owners should implement standardized procedures for determining the load rating of bridges based on this Guide Manual for IoH.

This Section is intended for use in evaluating the types of highway bridges commonly in use in the United States that are subjected primarily to permanent loads and vehicular loads including IoH. Methods for the evaluation of existing bridges for extreme events such as earthquake, vessel collision, wind, flood, ice, or fire are not included herein.

For load rating of bridges for IoH by nondestructive load testing, refer to Section 8 of *AASHTO Manual for Bridge Evaluation*.

C2.1

Derivation of Load and Resistance Factor Rating provisions in Part A and Load Factor Rating provisions in part B for IoH is reported in Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202? *AASHTO Manual for Bridge Evaluation* should be referenced regarding load rating in general. In accordance with the general concepts therein, this Guide Manual was developed.

Primary focus of this Section is the assessment of the safety of bridges for IoH live loads (including possible occasional overloads). Fatigue evaluation regarding IoH for steel bridges with respect to IoH is not required beyond the requirement for regular highway loads specified in *AASHTO Manual for Bridge Evaluation*, unless the Engineer decides otherwise considering specific site and bridge condition.

2.1.1—Assumptions

The load rating of a bridge is based on existing structural conditions, material properties, loads, and IoH traffic conditions at the bridge site. To maintain this capacity, the bridge is assumed to be subject to inspections at regular intervals, not to exceed the maximum interval cited in Article 4.3 of *AASHTO Manual for Bridge Evaluation*. Changes in existing structural conditions, material properties, loads, or site traffic conditions of IoH could require re-evaluation.

Every effort should be made to minimize hardships related to economic hauling without jeopardizing the safety of the public. All data used in the determination of the load rating criteria should be fully documented.

In ordinary cases, the review of a permit application should not necessitate a special inspection of the bridge, and the evaluation may be based on the results of the most recent inspection.

C2.1.1

Load rating of a bridge for IoH should not be undertaken without a recent thorough field inspection, which:

- Provides the condition data and other critical noncondition data necessary for evaluation,
- Minimizes the possibility of the evaluator making a gross error in assessing the capacity of a component or connection, and
- Improves bridge safety through early discovery of deterioration or signs of distress that could signal impending failure.

2.1.2—Condition of Bridge Members

The condition and extent of deterioration of structural components of the bridge should be considered in the computation of the dead load and live load effects when stress is chosen as the evaluation approach and for the capacity when force or moment is chosen for use in the basic rating equation.

The rating of an older bridge for its load-carrying capacity for IoH should be based on a recent thorough field investigation. All physical features of a bridge which have an effect on its structural integrity should be examined as discussed in Section 4. Note any damaged or deteriorated sections and obtain adequate data on these areas so that their effect can be properly evaluated in the analysis. Where steel is severely corroded, concrete deteriorated, or timber decayed, make a determination of the loss in a cross-sectional area as closely as reasonably possible. Determine if deep pits, nicks, or other defects exist that may cause stress concentration areas in any structural member. Lowering load capacities below those otherwise permitted or other remedial action may be necessary if such conditions exist.

C2.1.2

The effective cross-section properties used in determining the resistance or strength of the section to applied forces should be based on the gross cross-section less that portion which has deteriorated. For instance, in a steel tension member, the member should be evaluated based on the least cross-section area available to resist the applied tension force.

2.1.3—Documentation of Load Rating

The load rating for IoH should be adequately documented, including all background information such as field inspection reports, material and load test data, all supporting computations, and a clear statement of all assumptions used in calculating the load rating. If a computer model was used, the input data file should be retained for future use.

2.1.4—REFERENCES

AASHTO. *Manual for Bridge Evaluation*, 3rd Edition. American Association of State Highway and Transportation Officials, Washington, DC, 2018.

Fu,G., Q.Wang, J.Chi, M.Lwin, and R.Corotis NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202?

PART A—LOAD AND RESISTANCE FACTOR RATING

2A.1—INTRODUCTION

2A.1.1—General

The load and resistance factor rating procedures of Part A for IoH provide a methodology for load rating a bridge consistent with the load and resistance factor design philosophy of the *AASHTO LRFD Bridge Design Specifications* and the load and resistance factor rating method of the *AASHTO Manual for Bridge Evaluation*.

2A.1.2—Scope

Part A provides procedures for the rating of bridges using the load and resistance factor philosophy for IoH load. Procedures are presented for load rating bridges for IoH at three levels: Tiers 1, 2, and 3 defined by Bridge Owner as guided herein. They are respectively equivalent to the legal load, routine permit load, and special permit load as defined in *AASHTO Manual for Bridge Evaluation*. These procedures are consistent in philosophy and approach of the *AASHTO LRFD Bridge Design Specifications* and *Manual for Bridge Evaluation*. The methodology is presented in a format using load and resistance factors that have been calibrated based on structural reliability theory to achieve a minimum target reliability for the strength limit state. Guidance is provided on service limit states that are applicable to bridge load rating for IoH.

2A.1.3—Philosophy

Bridge design and rating, though similar in overall approach, differ in important aspects. Bridge ratings generally require the Engineer to consider a wider range of variables than is typical in bridge design. Design may adopt a conservative reliability index and impose checks to ensure serviceability and durability without incurring a major cost impact. In rating, the added cost of overly conservative evaluation standards can be prohibitive as load restrictions, rehabilitation, and replacement become increasingly necessary.

The rating procedures presented herein recognize a balance between safety and economics. In most cases, a lower target reliability than design has been chosen for load rating at the strength limit state. Application of serviceability limit states to rating is done on a more selective basis than is prescribed for design in the *AASHTO LRFD Bridge Design Specifications*.

C2A.1.3

The term “evaluation criteria” denotes safety and serviceability standards adopted for assessing existing bridges.

LRFD calibration reported a target LRFD reliability index β of 3.5. The LRFD design criteria based on this index were derived for a severe traffic-loading case (including the presence of 5000*ADTT*). The LRFR procedures in Part A adopt a reduced target reliability index of approximately 2.5 calibrated to past AASHTO operating level load rating. This value was chosen to reflect the reduced exposure period, consideration of site realities, and the economic considerations of rating vs. design.

2A.1.4—Application of *AASHTO LRFD Bridge Design Specifications* and *Manual for Bridge Evaluation*

This Guide Manual is consistent with the current *AASHTO LRFD Bridge Design Specifications* and

C2A.1.4

Judgment must be exercised in evaluating a structure for IoH, and in some cases the evaluation criteria may be

AASHTO Manual for Bridge Evaluation. Where this Guide Manual is silent, the current *AASHTO LRFD Bridge Design Specifications* or *Manual for Bridge Evaluation* shall govern. Where appropriate, reference is made herein to specific articles in the *AASHTO LRFD Bridge Design Specifications* or *Manual for Bridge Evaluation*.

Where the behavior of a member under traffic is not consistent with that predicted by the governing specifications, as evidenced by a lack of visible signs of distress or excessive deformation or cases where there is evidence of distress even though the specification does not predict such distress, deviation from the governing specifications based on the known behavior of the member under traffic may be used and shall be fully documented. Material sampling, instrumentation, and load tests may be helpful in establishing the load capacity for such members.

adjusted based on site conditions and/or structure conditions as recorded in the most recent inspection report. All data used in the decision to adjust the evaluation criteria shall be fully documented.

Nearly all existing bridges have been designed in accordance with the *AASHTO Standard Specifications for Highway Bridges*, most according to older editions of the specifications. The LRFD Specifications do not provide guidance on older bridge types that use materials and details no longer in common use. However, the *AASHTO LRFD Bridge Design Specifications* incorporate the state-of-the-art in design and analysis methods, loadings, and strength of materials. The *AASHTO Manual for Bridge Evaluation* extends these latest concepts to bridge evaluation permitting site- and/or bridge-specific decision or treatment.

2A.1.5—Load and Resistance Factor Rating for Implements of Husbandry

Bridge evaluations are performed for varied purposes using different live load models and evaluation criteria. This Section specifies a systematic approach to bridge load rating for IoH, using the load and resistance factor philosophy, aimed at addressing the different uses of load rating results.

The methodology for the load and resistance factor rating of bridges regarding IoH is comprised of three procedures: 1) Tier 1 IoH, 2) Tier 2 IoH and possibly 3) Tier 3 IoH, if Bridge Owner elects to have Tier 3. The results of each procedure serve specific uses and also guide the need for further evaluations to verify bridge safety or serviceability.

C2A.1.5

Evaluation live load models for regular highway truck traffic are comprised of the design live load, legal loads, and permit loads. They are covered in *AASHTO Manual for Bridge Evaluation*.

Implement of husbandry (IoH) load rating checks the safety and serviceability of bridges for the passage of IoH. State may define two or three tiers of IoH. Tier 1 is equivalent to State legal load with a consideration to wider gage widths of IoH if typically observed within the jurisdiction. Tier 2 is heavier and much less frequent than Tier 1. If used, Tier 3 is heavier and further less frequent than Tier 2. Tier 3, if adopted, shall be used for very limited bridge crossings in the jurisdiction. For one Tier 3 load, normally only a single crossing of each bridge on the permitted route should be allowed when justified.

Implements of husbandry (IoH) refer to those vehicles that are designed to be used in the field or on the farm. They typically consist of a self-propelled unit such as a tractor or of such a self-propelled unit hauling equipment or a tank. They may occasionally use public roads subject to statutes of the jurisdiction. They are usually required to travel at a low speed of 35 mph or slower by IoH design and/or by jurisdiction statutes. Provisions regarding IoH herein are intended to allow accommodation of these vehicles within the limit for bridge safety and serviceability, if there is such a demand of IoH access within the jurisdiction.

State may define the IoH tiers according to current and possibly future relevant statutes of the jurisdiction. Guidance for decision is provided in Article 1.3. Tier 1 is intended to cover most of the population of IoH in the jurisdiction. Their generally wider gage widths, if observed within the jurisdiction, distribute the vehicular load to a wider deck area and to more primary bridge members such as beams. Therefore, Tier 1 IoH load may be beyond the Federal Bridge Formula and/or current legal load of jurisdiction, which assumes a standard gage width of 6 ft but does not explicitly account for gage width.

If a demand exists and is justifiable for heavier IoH, Tier 2 may be used. Accordingly, Tier 2 is intended to cover IoH that are equivalent to routine permit loads for multiple bridge crossings.

If a demand for even heavier IoH exists and is justifiable, Tier 3 may be adopted to accommodate such loads. Accordingly, Tier 3 is intended to cover IoH that are equivalent to special or trip permit loads for a single crossing or very limited and controlled crossings. An allowance for wider gage width of IoH for this tier may also be used if observed.

NCHRP Report ??? "Proposed New AASHTO Load Rating Provisions for Implements of Husbandry" by Fu el al. 20?? contains more details and background information regarding IoH and these tiers.

2A.2—LOADS FOR EVALUATION

2A.2.1—General

Article 2A.2 describes the loads to be used in determining the load effects in the load rating equation for IoH provided in Article 2A.4.2. In general, only permanent loads and vehicular loads are considered to be of consequence in load rating for IoH. Environmental loads such as wind, ice, temperature, stream flow, and earthquake are usually not considered in rating except when unusual conditions warrant their inclusion.

2A.2.2—Permanent Loads and Load Factors

The load rating of bridges for IoH shall consider all permanent loads. Permanent loads include dead loads and locked-in force effects from the construction process.

These loads and their load factors shall be determined in accordance with *AASHTO Manual for Bridge Evaluation.*

2A.2.3—Transient Loads and Load Factors

2A.2.3.1—Vehicular Live Loads (Gravity Loads): *LL*

The nominal live loads to be used in the evaluation of bridges are selected based on the purpose and intended use of the evaluation results. Live load models for load rating for IoH include:

Implement of Husbandry Tier 1 Loads: As specified in Article 2A.4.3

Implement of Husbandry Tier 2 Load: Actual Vehicle

Implement of Husbandry Tier 3 Load: Actual Vehicle

Load factors for vehicular live loads appropriate for use in load rating are as specified in Articles 2A.4.3, 2A.4.4, and 2A.4.5, respectively for Tiers 1, 2, and 3.

C2A.2.3.1

The evaluation of bridge components to include the effects of longitudinal braking forces, specified in LRFD Design Article 3.6.4 in combination with dead- and live load effects, should be done only where the evaluator has concerns about the longitudinal stability of the structure.

The notional load in Article 2A.4.3 for Tier 1 is intended to envelope those IoH that are equivalent to legal loads by the Federal Bridge Formula (Formula B) with an allowance of 15% for their typically observed wider gage widths. The State may adopt this model if the IoH within the jurisdiction are generally consistent with this description.

A different notional load for Tier 1 IoH may be adopted by Bridge Owner. The model should envelope the intended population of IoH equivalent to legal loads in the jurisdiction. For more background information about the notional load in Article 2A.4.3, refer to Fu,G. et al., NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry,* Transportation Research Board, 202?

2A.2.3.2—Application of Vehicular Live Load

The number of traffic lanes to be loaded and the transverse placement of wheel lines shall be in conformance with the *AASHTO LRFD Bridge Design Specifications* and the following:

- Roadway widths from 18 to 20 ft shall have two traffic lanes, each equal to one half the roadway width.
- Roadway widths less than 18 ft shall carry one traffic lane only.

C2A.2.3.2

In the past, a distance as little as 1 ft between wheel load and edge of the roadway was used for rating by some agencies. This deviation from design is considered overly conservative and especially affected the rating of exterior stringers. The design of exterior stringers in many older bridges, especially those designed prior to 1957, may not have included a minimum live load distribution to the outside stringers.

- The center of any wheel load shall not be closer than 2.0 ft from the edge of a traffic lane or face of the curb.
- The distance between adjacent wheel lines of passing trucks shall not be less than 4.0 ft.
- For implements of husbandry, non-standard gage widths (i.e., other than 6.0 ft) may be used.

  For variable gage width in an implement of husbandry, the gage width *GW* value to be used in load rating should be computed as follows as a weighted average *GW*:

$$GW = \sum_{i}^{N} GW_i \left(\frac{LoadEffect_i}{\sum_{j}^{N} LoadEffect_j} \right) \tag{2A.2.3.2-1}$$

where

N = Total number of axles on the span for the maximum load effect of interest

GW_i = Gage width of axle$_i$ $i=1,2,\ldots,N$ on the span for the maximum load effect of interest. For single-tire-steering-axle IoH, the steering axle's *GW* is omitted.

$LoadEffect_i$ = Load effect of axle$_i$ $i=1,2,\ldots,N$ *for the maximum-load-effect position*

$LoadEffect_i$ = Load effect of axle$_j$ $j=1,2,\ldots,N$ for the maximum-load-effect position

All load effects herein, including those of the total vehicle ($\sum_{j}^{N} LoadEffect_j$) and individual axle ($LoaEffect_j$, $j=1,2, \ldots,N$) are calculated using the beam line theory in the longitudinal direction.

2A.2.3.3—Dynamic Load Allowance: *IM*

The dynamic load allowance for evaluation should be as specified in Articles 2A.4.3.3, 2A.4.4.3, and 2A.4.5.3.

C2A.2.3.3

In the AASHTO Standard Specifications, the dynamic load allowance was termed impact.

Part A allows the use of reduced dynamic load allowance for load rating under certain conditions as discussed in Articles C2A.4.3.3, C2A.4.4.3, and C2A.4.5.3.

Due to such a large number of parameters involved in dynamic behavior and effects to bridge members, the most reliable approach to assessing the dynamic impact effect of IoH is in-situ physical testing. Numerical modeling such as finite element analysis alone has to be based on many assumptions that are not readily verifiable.

2A.2.3.4—Other Loads

Other transient loads for load rating regarding IoH shall be determined in accordance with *AASHTO Manual for Bridge Evaluation*.

2A.3—STRUCTURAL ANALYSIS

2A.3.1—General

Methods of structural analysis suitable for the evaluation of bridges shall be as described in Section 4 of the *AASHTO LRFD Bridge Design Specifications* and in this Section.

C2A.3.1

Evaluation seeks to verify adequate performance of existing bridges with an appropriate level of effort. Within a given evaluation procedure, the evaluator has the option of using simplified methods that tend to be somewhat conservative or pursue a more refined approach for improved accuracy. It is recommended that wherever feasible, simplified evaluation procedures should be first applied before resorting to higher level evaluation methods. Refined approaches to capacity evaluation of existing bridges can be economically justified where increased capacity is required to achieve a desired safe load capacity or permit load capability.

2A.3.2—Approximate Methods of Structural Analysis

Except as specified herein, approximate methods of distribution analysis as described in LRFD Design Article 4.6.2 may be used along with the modifying factor in this Article for evaluating existing straight bridges for IoH.

The multiple presence factor of 1.2 which is included in the LRFD distribution factors for single-lane loadings should not be used when checking IoH loads for single lane loading. Adjustments in distributions to account for traffic volume provided in the *AASHTO LRFD Bridge Design Specifications* should also not be factored into the evaluation distribution factors unless justified by the Engineer considering the IoH traffic at the site.

For IoH loads, approximate methods of distribution analysis as described in LRFD Design Article 4.6.2 may be modified to account for wider or narrower gage width using the multiplicative modifying factors (*MF*) in Articles 2A.3.2.1 and 2A.3.2.2 for evaluating existing bridges.

Two types of IoH are covered in these multiplicative *MF* formulas according to the steering axle configuration. Article 2A.3.2.1 is for IoH with dual-wheel-steering-axle, and Article 2A.3.2.2 is for IoH with single-wheel-steering-axle.

The multiplicative *MF* in this Article is for one lane loading.

For variable gage width in an implement of husbandry when computing *MF* in Articles 2A.3.2.1 and 2A.3.2.2, the gage width *GW* value is should be computed in accordance with Article 2A.2.3.2.

C2A.3.2

The live load distribution formulas provided in the *AASHTO LRFD Bridge Design Specifications* were developed for common bridge types and dimensions, for the HS family of trucks. The modifying factor in this Article is to modify them to be suitable for IoH typically having different gage widths.

Available weight-in-motion data show that IoH traffic is in small volume. Therefore one lane loading is used for load rating.

Exterior girders of existing bridges may have been designed for less capacity than the interior girders. Additionally, they may also be subjected to increased deterioration due to their increased environmental exposure. Approximate methods of analysis for exterior

girders are often less reliable than interior girders due to the structural participation of curbs and parapets. The level of structural participation could vary from bridge to bridge. Field testing (load testing) procedures described in Section 8 of *AASHTO Manual for Bridge Evaluation* may be employed to verify the behavior of exterior girders.

2A.3.2.1- Dual-wheel-steering-axle Implements of Husbandry

Tables 2A.3.2.1-1 and 2A.3.2.1-2 specify *MF* for beam spans, respectively for moment and shear in beams. Tables 2A.3.2.1-3 includes *MF* for slab spans applicable to equivalent slab strip width *E*. R_1 in these tables is equal to 1.15 for $GW \leq 6$ ft and 0.85 for $GW > 6$ ft.

C2A.3.2.1

R_1 is a factor to cover variation in *MF* that is derived based on regression of finite element analysis results of IoH vehicles on typical bridge span types of interest. For more details of analysis, see Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202?

2A.3.2.2- Single-wheel-steering-axle Implements of Husbandry

Tables 2A.3.2.2-1 and 2A.3.2.2-2 contain *MF* for beam spans, respectively for moment and shear in beams. Table 2A.3.2.2-1 includes *MF* for slab spans. In this Article, R_2=1.05

C2A.3.2.2

The single-wheel-steering-axle IoH have only one wheel in the steering axle, not two wheels like other IoH. This configuration causes unique lateral live-load distribution in parallel longitudinal members such as beams and slab strips. R_2 is a factor to cover variation in *MF* that is derived based on regression of finite element analysis results of IoH vehicles on typical bridge span types of interest. For more details of analysis, see Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202?

Table 2A.3.2.1-1— Modifying Factor *MF* for Dual-Wheel-Steering-Axle IoH for Moment in Interior and Exterior Beams

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.2.1-1	For Moment in Interior Beams, One Lane Loaded (applicable to LRFD Design Table 4.6.2.2.2b-1)	For Moment in Exterior Beams, One Lane Loaded (applicable to LRFD Design Table 4.6.2.2.2d-1)	Range of Applicability
Wood Deck on Wood Beams	l	$1 - 0.340 R_1 Ln(\frac{GW}{6})$	$1 - 0.376 R_1 Ln(\frac{GW}{6})$	$0.7 \le S \le 6.0$ $20 \le L \le 45$ $3.0 \le t_s \le 10.0$ $850 \le I \le 12{,}000$ $5 \le GW \le 12$ $N_b \ge 5$
Concrete Deck on Steel Beams	a	$1 - 0.301 R_1 Ln(\frac{GW}{6})$	$1 - 0.887 R_1 Ln(\frac{GW}{6})$ $(\frac{GW}{L})^{0.870}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_s \le 14.0$ $5 \le GW \le 12$ $N_b \ge 4$
Wood Deck on Steel Beams	a	$1 - 0.499 R_1 Ln(\frac{GW}{6})$ $(\frac{GW}{L})^{0.310}$	$1 - 0.263 R_1 Ln(\frac{GW}{6})$	$1.5 \le S \le 6.0$ $20 \le L \le 140$ $3 \le t_S \le 10$ $5 \le GW \le 12$ $N_b \ge 5$
Concrete Deck on Prestressed I Beams	k	$1 - 0.650 R_1 Ln(\frac{GW}{6})(\frac{S}{L})^{0.50}$	$1 - 0.531 R_1 Ln(\frac{GW}{6})(\frac{S}{L})^{0.40}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_S \le 11.0$ $5 \le GW \le 12$ $N_b \ge 4$
Concrete deck on Adjacent Concrete Box Beams	f, g	$1 - 0.198 R_1 Ln(\frac{GW}{6})$	$1 - 0.179 R_1 Ln(\frac{GW}{6})$	$3 \le S \le 5$ $20 \le L \le 120$ $5 \le t_S \le 6$ $N_b \ge 7$ $5 \le GW \le 12$
Concrete T Beams	e	$1 - 3.281 R_1 Ln(\frac{GW}{6})(\frac{S}{L})^{1.48}$	$1 - 0.238 R_1 Ln(\frac{GW}{6})$	$3.5 \le S \le 14$ $20 \le L \le 90$ $4.5 \le t_S \le 12$ $N_b \ge 4$ $5 \le GW \le 12$

Table 2A.3.2.1-2— Modifying Factor *MF* for Dual-Wheel-Steering-Axle IoH for Shear in Interior and Exterior Beams

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.2.1-1	For Interior Beams One Lane Loaded (applicable to LRFD Design Table 4.6.2.2.3a-1)	For Exterior Beams One Lane Loaded (applicable to LRFD Design Table 4.6.2.2.3b-1)	Range of Applicability
Wood Deck on Wood Beams	l	$1-0.362R_1Ln(\frac{GW}{6})$ $(\frac{t_S}{6})^{0.51}(\frac{S}{9})^{0.17}$	$1-0.284R_1Ln(\frac{GW}{6})$ $(\frac{t_S}{6})^{0.67}(\frac{S}{9})^{0.79}$	$0.7 \le S \le 6.0$ $20 \le L \le 45$ $3.0 \le t_s \le 10.0$ $850 \le I \le 12{,}000$ $5 \le GW \le 12$ $N_b \ge 5$
Concrete Deck on Steel Beams	a	$1-0.509R_1Ln(\frac{GW}{6})(\frac{S}{14})^{0.60}$	$1-0.640R_1Ln(\frac{GW}{6})(\frac{S}{15})^{0.50}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_s \le 14.0$ $5 \le GW \le 12$ $N_b \ge 4$
Wood Deck on Steel Beams	a	$1-0.134R_1Ln(\frac{GW}{6})(\frac{L}{14})^{0.12}$ $(\frac{t_S}{6})^{1.10}(\frac{GW}{t_S})^{0.15}$	$1-0.334R_1Ln(\frac{GW}{6})$ $(\frac{t_S}{GW})^{0.76}(\frac{S}{GW})^{0.44}$	$1.5 \le S \le 6.0$ $20 \le L \le 140$ $3 \le t_S \le 10$ $5 \le GW \le 12$ $N_b \ge 5$
Concrete Deck on Prestressed I Beams	k	$1-0.863R_1Ln(\frac{GW}{6})(\frac{S}{12})^{0.25}$	$1-0.526R_1Ln(\frac{GW}{6})(\frac{S}{12})^{0.34}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_S \le 11.0$ $5 \le GW \le 12$ $N_b \ge 4$
Concrete deck on Adjacent Concrete Box Beams	f, g	$1-0.147R_1Ln(\frac{GW}{6})$	$1-0.097R_1Ln(\frac{GW}{6})$	$3 \le b \le 5$ $20 \le L \le 120$ $5 \le t_S \le 6$ $N_b \ge 7$ $5 \le GW \le 12$
Concrete T Beams	e	$1-3.097R_1Ln(\frac{GW}{6})$ $(\frac{6}{GW})^{1.87}(\frac{S}{L})^{0.93}$	$1-0.321R_1Ln(\frac{GW}{6})(\frac{S}{L})^{1.53}$	$3.5 \le S \le 14$ $20 \le L \le 90$ $4.5 \le t_S \le 12$ $N_b \ge 4$ $5 \le GW \le 12$

Table 2A.3.2.1-3— Modifying Factor *MF* for Dual-Wheel-Steering-Axle IoH for Equivalent Slab Strip Width

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.3-1	For Interior Strip One Lane Loaded (applicable to LRFD Design Eq. 4.6.2.2.3-1)	For Exterior Strip One Lane Loaded (applicable to LRFD Design Article 4.6.2.1.4b)	Range of Applicability
Wood or Concrete Slabs	a,b	$\frac{1}{1 - 0.155 R_1 Ln(\frac{GW}{6})}$	1	$20 \le L \le 60$ $25 \le t_S \le 45$ $12 \le W \le 28$ $5 \le GW \le 12$

Table 2A.3.2.2-1— Modifying Factor *MF* for Single -Wheel-Steering-Axle IoH for Moment in Interior and Exterior Beams

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.2.1-1	For Moment in Interior Beams, One Lane Loaded (applicable to LRFD Design Table 4.6.2.2.2b-1)	For Moment in Exterior Beams One Lane Loaded (applicable to LRFD Design Table 4.6.2.2.2d-1)	Range of Applicability
Wood Deck on Wood Beams	l	$1.088R_2\left(\frac{6}{GW}\right)^{0.298}\left(\frac{GW}{L}\right)^{0.022}\left(\frac{GW}{t_S}\right)^{0.012}$	$1.141R_2\left(\frac{S}{GW}\right)^{0.064}\left(\frac{Lt_S^3}{I}\right)^{0.114}\left(\frac{S}{L}\right)^{0.114}\left(\frac{9}{S}\right)^{0.105}\left(\frac{6}{t_S}\right)^{0.307}$	$0.7 \le S \le 6.0$ $20 \le L \le 45$ $3.0 \le t_s \le 10.0$ $850 \le I \le 12{,}000$ $6 \le GW \le 10$ $N_b \ge 5$
Concrete Deck on Steel Beams	a	$0.726R_2\left(\frac{L}{GW}\right)^{0.233}\left(\frac{L}{S}\right)^{0.071}\left(\frac{14}{L}\right)^{0.225}$	$R_2[1.015\left(\frac{6}{GW}\right)^{0.228} - 0.233\left(\frac{S}{L}\right) + 0.111\left(\frac{GW}{L}\right) + 0.018]$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_s \le 14.0$ $6 \le GW \le 10$ $N_b \ge 4$
Wood Deck on Steel Beams	a	$R_2[1.118\left(\frac{6}{GW}\right)^{0.151}\left(\frac{S}{L}\right)^{0.039} + 0.094 - 0.559\left(\frac{GW}{L}\right) - 0.00222L + 0.240\left(\frac{t_S}{6}\right) - 0.175\left(\frac{t_S}{GW}\right)]$	$1.736R_2\left(\frac{S}{L}\right)^{0.050}\left(\frac{1}{S}\right)^{0.235}\left(\frac{S}{GW}\right)^{0.091}\left(\frac{S}{t_S}\right)^{0.053}$	$1.5 \le S \le 6.0$ $20 \le L \le 140$ $3 \le t_S \le 10$ $6 \le GW \le 10$ $N_b \ge 5$
Concrete Deck on Prestressed I Beams	k	$1.132R_2\left(\frac{L}{GW}\right)^{0.198}\left(\frac{L}{S}\right)^{0.025}\left(\frac{1}{L}\right)^{0.150}$	$1.259R_2\left(\frac{S}{GW}\right)^{0.184}\left(\frac{L}{S}\right)^{0.204}\left(\frac{1}{L}\right)^{0.164}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_S \le 11.0$ $6 \le GW \le 10$ $N_b \ge 4$
Concrete deck on Adjacent Concrete Box Beams	f, g	$0.967R_2\left(\frac{S}{GW}\right)^{0.157}\left(\frac{L}{S}\right)^{0.0238}\left(\frac{t_S}{S}\right)^{0.176}$	$1.323R_2\left(\frac{L}{GW}\right)^{0.358}\left(\frac{1}{L}\right)^{0.514}\left(\frac{GW}{b}\right)^{0.316}\left(\frac{b}{t_S}\right)^{0.272}\left(\frac{L^2}{bd}\right)^{0.165}$	$3 \le b \le 5$ $20 \le L \le 120$ $5 \le t_S \le 6$ $N_b \ge 7$ $6 \le GW \le 10$
Concrete T Beams	e	$0.975R_2\left(\frac{L}{GW}\right)^{0.193}\left(\frac{L}{S}\right)^{0.038}\left(\frac{1}{L}\right)^{0.111}$	$0.968R_2\left(\frac{S}{GW}\right)^{0.125}\left(\frac{L}{GW}\right)^{0.023}$	$3.5 \le S \le 14$ $20 \le L \le 90$ $4.5 \le t_S \le 12$ $N_b \ge 4$ $6 \le GW \le 10$

Table 2A.3.2.2-2— Modifying Factor *MF* for Single -Wheel-Steering-Axle IoH for Shear in Interior and Exterior Beams

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.2.1-1	For Interior Beams One Lane Loaded (applicable to LRFD Design Table 4.6.2.2.3a-1)	For Exterior Beams One Lane Loaded (applicable to LRFD Design Table 4.6.2.2.3b-1)	Range of Applicability
Wood Deck on Wood Beams	l	$1.232R_2(\frac{S}{GW})^{0.035}(\frac{9}{S})^{0.074}$	$1.199R_2(\frac{S}{GW})^{0.045}(\frac{9}{S})^{0.098}$	$0.7 \le S \le 6.0$ $20 \le L \le 45$ $3.0 \le t_s \le 10.0$ $850 \le I \le 12,000$ $6 \le GW \le 10$ $N_b \ge 5$
Concrete Deck on Steel Beams	a	$1.035R_2(\frac{GW}{S})^{0.261}(\frac{L}{S})^{0.059}$ $(\frac{6}{GW})^{0.396}$	$1.045R_2(\frac{6}{GW})^{0.334}(\frac{L}{S})^{0.050}$ $(\frac{GW}{S})^{0.198}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_s \le 14.0$ $6 \le GW \le 10$ $N_b \ge 4$
Wood Deck on Steel Beams	a	$R_2[1.542 - 0.0437(\frac{GW}{6})$ $-0.337(\frac{S}{9}) - 0.0204(\frac{GW}{S})]$	$1.166R_2(\frac{9}{S})^{0.160}(\frac{S}{GW})^{0.087}$	$1.5 \le S \le 6.0$ $20 \le L \le 140$ $3 \le t_S \le 10$ $6 \le GW \le 10$ $N_b \ge 5$
Concrete Deck on Prestressed I Beams	k	$3.013R_2(\frac{S}{GW})^{0.239}(\frac{L}{S})^{0.662}$ $(\frac{1}{L})^{0.597}$	$1.297R_2(\frac{6}{GW})^{0.399}(\frac{GW}{S})^{0.264}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_S \le 11.0$ $6 \le GW \le 10$ $N_b \ge 4$
Concrete deck on Adjacent Concrete Box Beams	f, g	$1.229R_2(\frac{S}{GW})^{0.077}(\frac{L}{S})^{0.103}$	$1.193R_2(\frac{b}{GW})^{0.069}(\frac{12}{S})^{0.191}$	$3 \le S \le 5$ $20 \le L \le 120$ $5 \le t_S \le 6$ $N_b \ge 7$ $6 \le GW \le 10$
Concrete T Beams	e	$0.747R_2(\frac{L}{S})^{0.058}(\frac{14}{S})^{0.392}$ $(\frac{S}{GW})^{0.136}(\frac{t_S}{S})^{0.229}$	$0.987R_2(\frac{14}{S})^{0.307}(\frac{S}{GW})^{0.112}$	$3.5 \le S \le 14$ $20 \le L \le 90$ $4.5 \le t_S \le 12$ $N_b \ge 4$ $6 \le GW \le 10$

Table 2A.3.2.2-3— Modifying Factor *MF* for Single -Wheel-Steering-Axle IoH for Equivalent Slab Strip Width

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.3-1	For Interior Strip One Lane Loaded (applicable to LRFD Design Eq. 4.6.2.3-1)	For Exterior Strip One Lane Loaded (applicable to LRFD Design Article 4.6.2.1.4b)	Range of Applicability
Wood or Concrete Slabs	a,b	$0.618(GW)^{0.137}(\frac{L}{GW})^{0.160}\left(\frac{1}{R_2}\right)$	1	$20 \le L \le 60$ $25 \le t_S \le 45$ $12 \le W \le 28$ $6 \le GW \le 10$

2A.3.3—Refined Methods of Analysis

Bridges that exhibit insufficient load capacity when analyzed by approximate methods, and bridges or loading conditions for which accurate live load distribution formulas are not readily available may be analyzed by refined methods of analysis as described in LRFD Design Article 4.6.3.

C2A.3.3

Some cases where refined analysis methods would be considered appropriate include:

- Girder spacings and span lengths outside the range of LRFD-distribution formulas or outside the range of *MF* in Articles 2A.3.2.1 and 2A.3.2.2,
- Varying skews at supports,
- Curved bridges,
- Low-rated bridges, and
- Loads of IoH Tier 2 and Tier 3, if adopted by Bridge Owner, with nonstandard gage widths out of the range of *MF* in Articles 2A.3.2.1 or 2A.3.2.2 and/or large variations in axle configurations.

Many older bridges have parapets, railings, and curbs that are interrupted by open joints. The stiffness contribution of these elements to bridge response should be verified by load testing, if they are to be included in a refined analysis.

2A.4—LOAD-RATING PROCEDURES

2A.4.1—Introduction

For IoH, three load-rating levels that are consistent with the load and resistance factor philosophy are provided in Article 2A.4 for the load capacity evaluation of in-service bridges:

- Tier 1 load rating (first level of evaluation)
- Tier 2 load rating (second level of evaluation)
- Tier 3 load rating (third level of evaluation)

The tier definitions are provided in Article 2A.1.5.

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 2A.4.3, 2A.4.4, and 2A.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 Bridge Owner, these load factors may be used in lieu of the values given in this Guide 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 2A.4.2.

C2A.4.1

States may decide how many tiers to be included in the load rating program of bridges for IoH. Such a decision may be made by consultating with other stakeholders as discussed in Article 1.3 of this Guide Manual.

Bridges that do not pass the load rating for Tier 1 (*RF* <1) should not allow nor be evaluated for Tier 2 or Tier 3.

Weigh-In-Motion (WIM) data collected at a 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 in this Guide Manual.

2A.4.2—General Load-Rating Equation

2A.4.2.1—General

C2A.4.2.1

The following general expression should be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear):

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)} \quad (2A.4.2.1\text{-}1)$$

For the Strength Limit States:

$$C = \varphi_c \varphi_s \varphi R_n \quad (2A.4.2.1\text{-}2)$$

Where the following lower limit shall apply:

$$\varphi_c \varphi_s \geq 0.85 \quad (2A.4.2.1\text{-}3)$$

For the Service Limit States:

$$C = f_R \quad (2A.4.2.1\text{-}4)$$

where:

RF = Rating factor

C = Capacity

f_R = Allowable stress specified in the LRFD code

R_n = Nominal member resistance (as inspected)

DC = Dead load effect due to structural components and attachments

DW = Dead load effect due to wearing surface and utilities

P = Permanent loads other than dead loads

LL = Live load effect

IM = Dynamic load allowance

γ_{DC} = LRFD load factor for structural components and attachments

γ_{DW} = LRFD load factor for wearing surfaces and utilities

γ_p = LRFD load factor for permanent loads other than dead loads = 1.0

γ_{LL} = Evaluation live load factor

It should be noted that load modifiers η relating to ductility, redundancy, and operational importance contained in Article 1.3.2.1 of the *AASHTO LRFD Bridge Design Specifications* are not included in the general load-rating equation. In load rating, ductility is considered in conjunction with redundancy and incorporated in the system factor φ_s. Operational importance is not included as a factor in the LRFR load rating provisions.

The load rating of a deteriorated bridge should be based on a recent thorough field inspection. Only sound material should be considered in determining the nominal resistance of the deteriorated section. Load ratings may also be calculated using as-built member properties to serve as a baseline for comparative purposes.

φ_c = Condition factor

φ_s = System factor

φ = LRFD resistance factor

Live load effect *LL* in Eq.2A.4.2.1-1 should be determined in accordance with Article 2A.4.3.2 for Tier 1, Article 2A.4.4.3 for Tier 2, and 2A.4.5.3 for Tier 3.

Dynamic load allowance *IM* Eq.2A.4.2.1-1 should be determined in accordance with Articles 2A.4.3.3 for Tier 1, 2A.4.4.3 for Tier 2, and 2A.4.5.3 for Tier 3.

The evaluation live load factors γ_{LL} in Eq.2A.4.2.1-1 is provided in Article 2A.4.3.2 for Tier 1, Article 2A.4.4.3 for Tier 2, and 2A.4.5.3 for Tier 3.

Other quantities in Eq.2A.4.2.1-1 shall be determined as specified in Article 6A.4.2.1 of *AASHTO Manual for Bridge Evaluation.*

The load rating shall be carried out at each applicable limit state and load effect with the lowest value determining the controlling rating factor. Limit states and load factors for load rating shall be selected from Table 2A.4.2.2-1.

2A.4.2.2—Limit States

Strength is the primary limit state for load rating for IoH; service limit state is selectively applied in accordance with the provisions of this Guide Manual. Fatigue limit state for steel members does not need to be checked for IoH. Applicable limit states are summarized in Table 2A.4.2.2-1.

C2A.4.2.2

Low IoH volumes observed in available WIM data sets used in calibration for this Guide Manual do not justify check for fatigue limit state for steel members with regard to IoH.

Table 2A.4.2.2-1—Limit States and Load Factors for Load Rating

Bridge Type	Limit State*	Dead Load γ_{DC}	Dead Load γ_{DW}	Design Load Inventory γ_{LL}	Design Load Operating γ_{LL}	IoH Tier 1** γ_{LL}	Tier 2 γ_{LL}	IoH Tier 3 γ_{LL}
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 2A.4.3.2.2-1	—	—
	Strength II	1.25	1.50	—	—	—	Table 2A.4.4.3.2-1	Table 2A.4.5.3.2-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00	1.00
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 2A.4.3.2.2-1	—	—
	Strength II	1.25	1.50	—	—	—	Table 2A.4.4.3.2-1	Table 2A.4.5.3.2-1
	Service I	1.00	1.00	—	—	—		1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 2A.4.3.2.2-1	—	—
	Strength II	1.25	1.50	—	—	—	Table 2A.4.4.3.2-1	Table 2A.4.5.3.2-1
	Service III	1.00	1.00	0.80	—	1.00	—	—
	Service I	1.00	1.00	—	—	—		1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Tables 2A.4.3.2.2-1	—	—
	Strength II	1.25	1.50	—	—	—	Table 2A.4.4.3.2-1	Table 2A.4.5.3.2-1

* Defined in the *AASHTO LRFD Bridge Design Specifications*.

** Use one-lane loading with built in multiple presence factor divided out.

Notes:

- Shaded cells of the table indicate optional checks.
- Service I is used to check the 0.9 F_y stress limit in reinforcing steel.
- Load factor for DW at the strength limit state may be taken as 1.25 where thickness has been field measured.
- Fatigue limit state does not need to be checked for IoH, unless the Engineer decides otherwise considering the site's IoH traffic.

2A.4.3—IoH Tier 1 Load Rating

2A.4.3.1—Purpose

The performance of existing bridges is addressed in this Article utilizing IoH Tier 1 load in Article 2A.4.3.2.1 or a Tier 1 model specified by State equivalent to the jurisdiction's legal load similar to the AASHTO legal load as defined in *Manual for Bridge Evaluation* with an allowance for wider gage width.

The live load factor for IoH Tier 1 is given in Article 2A.4.3.2.2. Tier 1 represents the largest portion of IoH in the jurisdiction, compared with Tier 2 and Tier 3, accessing to public roads and bridges if practiced in the jurisdiction. It also represents the lowest load-demand among all the tiers.

Bridges that pass Design Load Rating (HL93) at the Inventory level as defined in *AASHTO Manual for Bridge Evaluation* will have adequate capacity for IoH Tier 1 load.

The results of IoH Tier 1 load rating are also suitable for use in bridge management and bridge administration, at a local or national level. The rating results could also guide future inspections by identifying vulnerable limit states for each bridge.

C2A.4.3.1

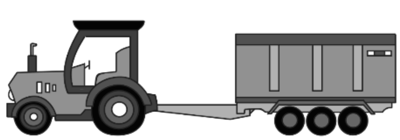
Article 1.3 discusses development of IoH load rating program for a jurisdiction. Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry,* Transportation Research Board, 202? provides more guidance for this purpose.

No evaluation for IoH Tier 1 is necessary for bridges that have adequate capacity (RF > 1) at the Inventory level reliability for HL-93. Bridges that pass HL-93 screening only at the Operating level reliability may not have adequate capacity for IoH Tier 1 load. They should be evaluated by load rating for IoH Tier 1 load using procedures provided in Article 2A.4.3.1.

2A.4.3.2—Live Loads and Load Factors

2A.4.3.2.1—Live Load

The IoH Tier 1 notional model in this Article or a Tier 1 model defined by State, which is equivalent to about 115% jurisdiction's legal load, shall be used for Tier 1 load rating.



C2A.4.3.2.1

The Tier 1 notional model in this Article is at 115% of Federal Bridge Formula. The 15% allowance is to account for typical wider gage widths and low operating speeds of IoH. They lead to lower stresses in bridge members than the standard 6 ft gage width and normal highway speed. A notional IoH load for Tier 1 adopted by Bridge Owner as an option is recommended to be equivalent to about 115% of the legal load of the jurisdiction. This equivalence includes a load allowance for wider gage widths and lower speeds observed with IoH to be enveloped.

If the State's legal load is significantly heavier than the AASHTO legal loads defined in *Manual for Bridge Evaluation*, the State-specific Tier 1 notional model should be reasonably close to the AASHTO legal loads with about 15% load allowance in order to use the live load factors recommended in Article 2A.4.3.2.2. Otherwise, State-

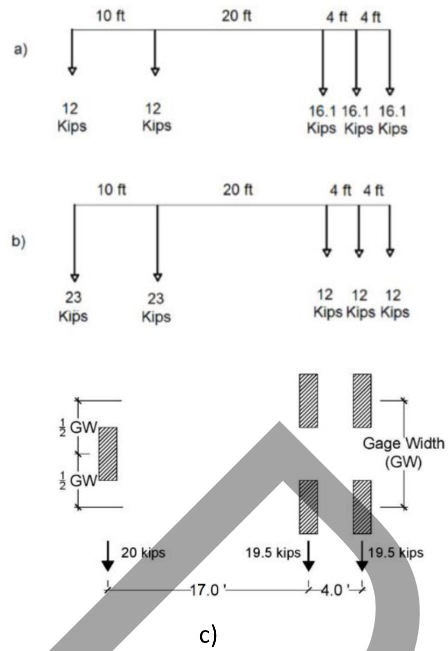


Figure 2A.4.3.2.1-1 Notional Load for Implement of Husbandry Tier 1: a), b), or c) whichever induces maximum load effect, a) and b) for dual-wheel-steering-axle IoH, c) for single-wheel-steering-axle IoH. Maximum axle load = 23 kips. Maximum gross vehicle weight = 92 kips.

specific live load factors need to be developed corresponding to the adopted Tier 1 notional model.

2A.4.3.2.2—Live load Factors

The evaluation live load factors for the Strength I limit state for Tier 1 shall be taken as shown in Table 2A.4.3.2.2-1. One lane load should be used.

Table 2A.4.3.2.2-1**—Load Factors for Design Load for Tier 1: γ_{LL}**

Traffic Volume (One direction)	Load Factor
Unknown	1.45
ADTT ≥ 5000	1.45
ADTT ≤ 1000	1.30

Linear interpolation is permitted for *ADTT* values between 5000 and 1000. One lane load distribution should be used.

C2A.4.3.2.2

These live load factors are calibrated using weigh-in-motion records that include IoH, as reported in Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202? They are intended to be applied to the notional Tier 1 model in Article 2A.4.3.2.1. That model is at 115% of the Federal Bridge Formula.

The live load factors in this Article may also be used for State-selected Tier 1 notional models that have only minor variations from 115% of AASHTO legal loads defined in *Manual for Bridge Evaluation*. For States that use legal load significantly heavier than AASHTO legal loads or IoH Tier 1 model in Article 2A.4.3.2.1 and would like to use their own Tier 1 model varying significantly from the AASHTO legal loads, the live load factors for their Tier 1 need to be developed considering IoH traffic condition of the jurisdiction.

FHWA requires an *ADTT* to be recorded on the Structural Inventory and Appraisal (SI&A) form for all bridges. In cases where site traffic conditions are unavailable or unknown, worst-case traffic conditions should be assumed.

Live load varies from site to site. More refined site-specific load factors appropriate for a specific bridge site may be estimated if more detailed traffic and truck load data are available for the site. *ADTT* and truck loads through weigh-in-motion measurements recorded over a period of time allow the estimation of site-specific load factors that are characteristic of a particular bridge site.

2A.4.3.3—Dynamic Load Allowance: *IM*

The dynamic load allowance for IoH shall be 20 percent for the strength and service limit states to account for the dynamic effects of moving vehicles. The dynamic load allowance shall be applied to the axle loads.

The dynamic load allowance to be applied for IoH load effects in wood components shall be 20% if the components are older than 15 years. For new wood components, dynamic allowance need not be applied. For wood components younger than 15 years, the dynamic load allowance may be linearly interpolated.

C2A.4.3.3

The factor to be applied to the static load effects shall be taken as 1 + dynamic load allowance. This factor is applicable to simple and continuous span configurations.

The 20 percent dynamic load allowance is mainly due to expected lower travel speed of IoH at 35 mph or lower.

Dynamic test results have shown that old timber components experience noticeable dynamic amplification of static load effect, although sometimes slightly lower than steel counterparts (Fu et al. NCHRP Report ??? "Proposed New AASHTO Load Rating Provisions for Implements of Husbandry" 20??)

The Engineer may decide to increase or decrease *IM* according to the surface condition of the approaches, bridge deck, and/or deck joints. Smoother surfaces reduce *IM* and bumpy ones increase *IM*.

The dynamic load allowance for components determined by field testing may be used in lieu of values specified herein. The use of full-scale dynamic testing under controlled or normal traffic conditions remains the most reliable way of obtaining the dynamic load allowance for a specific bridge.

2A.4.3.4—Exterior Beams

Permit load factor given in Table 2A.4.5.3.2-1 is applicable to both interior and exterior beam ratings. Distribution of live load to exterior beams as defined in LRFD Design Article 4.6.2.2.2d for one-lane load along with the modifying factor *MF* in Articles 2A.3.2.1 and 2A.3.2.2 shall apply.

2A.4.4—IoH Tier 2 Load Rating

2A.4.4.1 –Background

Bridge owners usually have established procedures and regulations which allow the passage of vehicles

C2A.4.4.1

above the legally established limitations on the highway system. These procedures involve the issuance of a permit which describes the features of the vehicles and/or its load and, in most jurisdictions, which specifies the allowable routes of travel.

Permits are issued by Bridge Owners on a single trip, multiple trips, or annual basis. Routine or annual permits are usually for unlimited trips over a period of time, not to exceed one year, for vehicles of a given configuration within specified gross and axle weight limits. Bridge owner may decide the upper limit of Tier 2 for the jurisdiction. IoH Tier 2 is recommended to be equivalent to annual permit of the jurisdiction with an appropriate allowance for wider gage width.

For States having significantly heavier legal load above AASHTO legal loads as specified in *AASHTO Manual for Bridge Evaluation*, IoH Tier 2 should include those equivalent to the jurisdiction’s legal load with an appropriate allowance for wider gage width.

2A.4.4.2—Purpose

Tier 2 represents those IoH heavier than Tier 1 and demanding access to public roads and bridges, if such demand is present and justified. Accommodating this demand requires higher load capacity of the bridges. Tier 2 load rating is to verify this capacity.

Bridges that do not have sufficient capacity under the Tier 1 load rating shall not be load rated for Tier 2 load and shall not allow such vehicles to cross. Load rating for IoH Tier 2 determines the safe load capacity of a bridge for IoH equivalent to annual permit trucks in the jurisdiction, using safety and serviceability criteria considered appropriate for evaluation. A single safe load capacity is obtained for a Tier 2 load.

C2A.4.4.2

Evaluation procedures are presented herein to establish a safe load capacity for an existing bridge that recognizes a balance between safety and economics.

The single safe load capacity produced by the procedures presented in this Guide Manual considers redundancy and bridge condition in the load-rating process. The load and resistance factors have been calibrated to provide uniform levels of reliability and permit the introduction of bridge- and site- specific data in a rational and consistent format. It provides a level of reliability, corresponding to the Operating level reliability for redundant bridges in good condition. The capacity of nonredundant bridges and deteriorated bridges should be reduced during the load-rating process by using system factors and condition factors. The safe load capacity may approach or exceed the equivalent of Operating rating for redundant bridges in good condition on low traffic routes, and may fall to the equivalent of Inventory levels or below for heavily deteriorated, nonredundant bridges on high traffic routes.

Tier 2 IoH loads should be justifiable and controlled for their travel, not to impose undue pressure on current infrastructure of bridge structures in the jurisdiction. There should be fewer Tier 2 loads than Tier 1 in the jurisdiction. . They should be managed similarly as for the semi-annual, annual, or routine permits currently practiced in the jurisdiction.

2A.4.4.3—Live Loads and Load Factors

2A.4.4.3.1—Live Loads

IoH Tier 2 load is the actual vehicle belonging to the tier and heavier than Tier 1.

C2A.4.4.3.1

Tier 2 represents those IoH beyond 115% of Federal Bridge Formula. This tier is equivalent to the jurisdiction's annual permit load for most States. For those States having legal load significantly heavier than the AASHTO legal loads as defined in *Manual for Bridge Evaluation*, Tier 2 should be equivalent to the heavier portion of their legal loads beyond the lighter portion that is equivalent to AASHTO legal loads.

2A.4.4.3.2—Live Load Factors

The LRFR provisions provide live load factors for load rating that have been calibrated to provide uniform and acceptable level of reliability. Load factors appropriate for use with IoH Tier 2 vehicles are defined based on the traffic data available.

Traffic conditions at bridge sites are usually characterized by traffic volume. The *ADTT* at a site is usually known or can be estimated.

The live load factors for IoH Tier 2 are listed in Table 2A.4.4.3.2-1. They are intended to be used for one lane of IoH load. Therefore, one lane distribution factor should be applied. Article 2A.3.2.1 specifies a modifying factor *MF* as multiplicative factor to the live load distribution factors in LRFD Design. The built in 1.2 factor in the AASHTO one-lane distribution factors should be divided out.

C2A.4.4.3.2

FHWA requires an *ADTT* to be recorded on the Structural Inventory and Appraisal (SI&A) form for all bridges. In cases where site traffic conditions are unavailable or unknown, worst-case traffic conditions should be assumed.

Live load varies from site to site. More refined site-specific load factors appropriate for a specific bridge site may be estimated if more detailed traffic and truck load data are available for the site. *ADTT* and truck loads through weigh-in-motion measurements recorded over a period of time allow the estimation of site-specific load factors that are characteristic of a particular bridge site.

The load factors listed in Table 2A.4.4.3.2-1 were developed under NCHRP 12-110 project and are based on the same target reliability as assured in *AASHTO Manual for Bridge Evaluation*.

Available weigh-in-motion records for sites experiencing IoH show their low volume. Thus, there is no need to include other vehicles in another lane for Tier 2 load rating. Reference: Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202?

Table 2A.4.4.3.2-1 Live Load Factor γ_{LL} for IoH Tier 2

Frequency	Loading Condition	$DF_{one\text{-}lane}$[a]	*ADTT* (one direction)	Load Factor by Implement of Husbandry Weight Ratio[b]		
				GVW / AL < 2.0 (kip/ft)	2.0 < GVW/AL < 3.0 (kip/ft)	GVW/AL > 3.0 (kip/ft)
Limited crossings (less than 100 crossings per year)	Mix with traffic	One lane	=3000	1.30	1.30	1.20
			=1000	1.30	1.20	1.10
			$\leq$100	1.20	1.10	1.10

[a] $DF_{one\ lane}$ = LRFD one-lane distribution factor with the built-in multiple presence factor divided out and the modifying factor in Articles 2A.3.2.1 and 2A.3.2.2 applied to account for wider or narrower gage width.

[b] Implement of Husbandry Weight Ratio = GVW/AL;. GVW = Gross Vehicle Weight; AL = Front axle to rear axle length; use only axles on the bridge.

2A.4.4.4—Dynamic Load Allowance: *IM*

The dynamic load allowance for Tier 2 implements of husbandry shall be as specified in Article 2A.4.3.3. The dynamic load allowance shall be applied to the axle loads.

2A.4.5—IoH Tier 3 Load Rating

2A.4.5.1—Background

Bridge Owners usually have established procedures and regulations which allow the passage of vehicles above the legally established weight limitations on the highway system. These procedures involve the issuance of a permit which describes the features of the vehicle and/or its load and, in most jurisdictions, which specifies the allowable route or routes of travel.

Bridge Owner may define a Tier 3 for its jurisdiction. IoH Tier 3 should be equivalent to special or trip permit loads in the jurisdiction with a possible and justifiable allowance for wider gage width and lower travel speed. Tier 3 loads are heavier than Tier 2. It should be limited to an extremely small number of IoH trips under special circumstances for the jurisdiction. Bridge Owner should carefully evaluate to have this tier or not and to design such a tier based on the need of farming industry as well as the impact to the bridges of the jurisdiction. Article 1.3 of this Guide Manual discusses factors that should be taken into account when designing IoH load rating program, which may include a Tier 3.

Routing Tier 3 loads to prevent overloading certain concerned bridges and adequate weight limit enforcing shall be practiced for Tier 3 IoH travel. Escorting may be needed.

C2A.4.5.1

To assure that permit restrictions and conditions are met and to warn the other traffic, special escort vehicles may be needed or required by State law. Traffic safety needs should always be considered.

Tier 3 should only be considered when Bridge Owner faces a justified demand for such heavy IoH to use public roads and bridges. This tier may impose significant pressure on current bridge infrastructure of the jurisdiction. Consequences and/or impacts of this tier to bridges need to be considered and evaluated as possible scenarios when adopting Tier 3. If Tier 3 is adopted, normally only one crossing of the bridges on the selected route(s) should be allowed and considered for one vehicle for a trop. Each trip of such load needs to be rigorously examined for approval.

Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202? provides more guidance on developing IoH load rating program for bridge safety.

2A.4.5.2—Purpose

Article 2A.4.5 provides procedures for checking bridges to determine the load effects induced by IoH Tier 3 and their capacity to safely carry the loads. Tier 3 load rating shall be performed only if the bridge has a rating factor greater than 1.0 when evaluated for Tier 1 loads. A specific route shall be identified that ensures safety of the bridges to be crossed.

C2A.4.5.2

IoH Tier 3 vehicles should be rated by using load-rating procedures given in Article 2A.4.5. The live load to be used in the load-rating equation for approval decisions shall be the actual IoH vehicle weight and axle configuration.

The live load factor recommended in Article 2A.4.5.3 for evaluating Tier 3 loads is calibrated with the assumptions that the bridge, as a minimum, can safely carry IoH Tier 1 loads. This requirement is especially evident when using reduced live load factor for Tier 3 based on an extremely small likelihood that there will be multiple presence of more than one heavy vehicle on the span at one time.

2A.4.5.3—Live Load and Load Factors

2A.4.5.3.1—Live Load

The live load to be used in the evaluation for Tier 3 decisions shall be the actual Tier 3 vehicle or the vehicle producing the highest load effect in a class of Tier 3

C2A.4.5.3.1

vehicles operating under a single approval. The loading shall consider the gross vehicle weight, its axle configuration and distribution of loads to the axles, designated lane position, and any speed restrictions associated with the issuance of the approval.

Only the Tier 3 vehicle shall be considered present in one lane.

Available weigh-in-motion records for sites experiencing IoH show their low volume. Thus, there is no need to include other vehicles in load rating for Tier 3. Reference: Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202?

2A.4.5.3.2—Load Factors

Table 2A.4.5.3.2-1 specifies live load factor for Tier 3 load rating that is calibrated to provide the same uniform and acceptable level of reliability as in *AASHTO Manual for Bridge Evaluation*. Load factor is defined with consideration to loading condition and traffic data.

Tier 3 load factor given in Table 2A.4.5.3.2-1 for the Strength II limit state is intended for spans having a rating factor greater than 1.0 when evaluated for Tier 1 loads. The Tier 3 load factor is not intended for use in loadrating bridges for Tier 1 or Tier 2 loads.

Table 2A.4.5.3.2-1—Live Load Factor γ_{LL} for IoH Tier 3

Frequency	Loading Condition	$DF_{one\text{-}lane}$[a]	All ADTTs All weights
Single trip	Mix with traffic	One Lane	1.10

[a]Use one-lane load distribution factor of LRFD Design with built-in 1.2 divided out along with modifying factor *MF* in Articles 2A.3.2.1 and 2A.3.2.2.

2A.4.5.4—Dynamic Load Allowance: *IM*

The dynamic load allowance to be applied for Tier 3 load rating shall be as specified in Article 2A.4.3.3 for Tier 1 loads, except that for slow moving (≤10 mph) Tier 3 vehicles the dynamic load allowance may be eliminated when the required moving speed is fully complied with.

APPENDIX A2A—LIVE LOAD MOMENTS ON LONGITUDINAL STRINGERS OR GIRDERS (SIMPLE SPAN)

Table A2A-1—Live Load Moments in kip ft per Lane with 20 percent *IM*

Span,	AASHTO Legal Loads				Design Load	IoH
ft	3	3-S2	3-Mar	Lane	HL-93	Tier 1
20	165.4	150.7	136.1		270.2	212.5
21	175.4	159.9	144.5		288.3	227.0
22	185.5	169.2	152.7		306.6	241.5
23	195.6	180.5	161.1		325.0	256.1
24	205.7	192.7	169.4		343.6	270.5
25	215.7	204.9	177.8		362.4	285.0
26	226.1	217.2	186.0		381.3	299.5
27	236.1	229.4	194.4		400.3	314.0
28	246.2	241.7	202.8		419.5	328.4
29	256.3	254.2	211.2		438.9	342.9
30	270.9	266.4	219.6		458.4	357.4
32	300.7	290.9	243.6		498.0	386.4
34	330.2	315.6	269.5		538.2	415.4
36	360.0	340.1	295.4		579.0	444.4
38	389.8	364.6	321.6		620.5	473.3
40	419.5	389.3	347.5		663.9	502.3
42	449.3	413.8	373.7		718.6	531.3
44	479.3	438.5	399.9		776.2	560.3
46	509.1	462.9	425.8		834.4	590.8
48	538.8	487.7	451.9		893.2	626.6
50	568.8	529.9	478.3		952.5	662.4
52	598.6	572.1	514.3		1012.7	698.3
54	628.3	614.6	555.2		1073.0	734.3
56	658.3	657.1	595.9		1134.3	770.2
58	688.3	699.3	636.7		1196.5	806.1
60	718.1	742.1	677.5		1258.8	842.0
70	867.6	955.2	893.3		1582.1	1081.7
80	1017.4	1169.1	1132.6		1920.4	1325.7
90	1167.2	1383.3	1372.1		2274.8	1570.1
100	1316.8	1598.2	1611.6		2646.0	1814.9
120	1616.7	2028.3	2091.2		3434.7	2305.0

140	1916.4	2458.8	2570.6		4287.4	2795.6
160	2216.4	2889.8	3050.2		5204.1	3286.6
180	2516.1	3320.8	3529.9		6184.8	3777.8
200	2816.1	3752.4	4009.9	4007.4	7228.6	4269.2
250	3565.9	4831.2	5209.4	5469.5	10117.6	5498.1
300	4315.7	5910.2	6409.2	7056.9	13404.8	6727.4

Table A2A-2—Live Load Moments in kip ft per Lane with 20 percent *IM*

Span, ft	Specialized Hauling Vehicles				NRL	Design Load
	SU4	SU5	SU6	SU7		HL93
20	192.5	201.6	211.4	211.4	211.4	270.2
21	205.0	216.7	228.7	228.7	228.7	288.3
22	217.7	231.6	246.2	246.2	246.2	306.6
23	230.2	246.7	263.5	264.5	264.5	325.0
24	242.9	261.6	281.1	284.1	284.1	343.6
25	255.3	276.7	298.3	303.7	303.7	362.4
26	267.9	291.6	315.9	323.5	325.2	381.3
27	280.6	306.7	333.1	343.2	347.5	400.3
28	296.1	321.6	350.6	362.9	369.6	419.5
29	312.3	336.7	367.9	382.6	391.9	438.9
30	328.3	351.6	385.4	402.2	414.0	458.4
32	360.5	381.6	420.3	441.6	458.4	498.0
34	392.7	412.8	455.0	481.2	502.8	538.2
36	425.1	449.5	493.0	520.6	549.4	579.0
38	457.2	486.5	534.5	565.4	597.1	620.5
40	489.5	523.2	576.0	612.0	645.1	663.9
42	521.8	560.1	617.5	658.3	693.1	718.6
44	554.2	596.8	659.3	704.8	740.8	776.2
46	586.3	633.8	700.8	751.4	788.8	834.4
48	618.7	670.8	742.3	797.8	836.8	893.2
50	651.2	707.7	783.9	844.3	884.8	952.5
52	683.5	744.7	825.6	890.9	932.7	1012.7
54	715.7	781.7	867.2	937.2	980.7	1073.0
56	748.1	818.6	908.8	983.7	1028.7	1134.3
58	780.5	855.9	950.4	1030.3	1076.7	1196.5
60	812.8	892.8	991.9	1076.8	1124.4	1258.8
70	974.5	1078.1	1200.3	1309.2	1364.3	1582.1
80	1136.4	1263.6	1408.6	1541.7	1604.1	1920.4
90	1298.2	1449.1	1616.8	1774.1	1844.0	2274.8
100	1460.1	1634.9	1825.2	2006.6	2083.9	2646.0
120	1783.9	2006.4	2241.9	2471.5	2563.8	3434.7
140	2107.9	2378.1	2658.9	2936.5	3043.7	4287.4
160	2431.7	2749.8	3075.7	3401.5	3523.7	5204.1
180	2755.7	3121.5	3492.7	3866.5	4003.7	6184.8
200	3079.7	3493.4	3909.6	4331.5	4483.6	7228.6
250	3889.7	4423.2	4951.9	5494.1	5683.4	10117.6

300	4699.4	5353.0	5994.5	6656.4	6883.4	13404.8

PART B— LOAD FACTOR RATING

2B.1—GENERAL

Section 2, Part B of this Guide Manual provides load factor rating as a choice of load rating method. Load ratings at Operating and Inventory levels using the load factor method can be calculated and may be especially useful for comparison with past practices.

C2B.1

In recent years, methods have been developed to provide more uniform safety margins for structures in terms of a reliability index. For bridge evaluation, the load and resistance factor rating (LRFR) method contained in this Guide Manual provides uniform reliability in bridge load ratings and load postings. See Section 2, Part A, for more information on LRFR for IoH.

The operating rating factor is determined by multiplying the inventory rating by the ratio of the inventory live load factor to the operating level live load factor (1.67). These live load factors are provided in Article 2B.5.3.

2B.1.1—Application of Standard Design Specifications

For all matters not covered by this Guide Manual, the current applicable AASHTO *Standard Specifications for Highway Bridges* (AASHTO Standard Specifications) should be used as a guide. However, there may be instances in which the behavior of a member under traffic is not consistent with that predicted by the controlling specification. In this situation, deviations from the controlling specifications based on the known behavior of the member under traffic may be used and should be fully documented. Diagnostic load tests may be helpful in establishing the safe load capacity for such members.

For ease of use and where appropriate, reference is made to specific articles in the AASHTO *Standard Specifications for Highway Bridges.*

2B.2—RATING LEVELS

Each highway bridge should be load rated at two levels, Inventory and Operating levels.

2B.2.1—Inventory Rating Level

The Inventory rating level generally corresponds to the customary design level of stresses but reflects the existing bridge and material conditions with regard to deterioration and loss of section. Load ratings based on the Inventory level allow comparisons with the capacity for new structures and, therefore, results in a live load, which can safely utilize an existing structure for an indefinite period of time.

2B.2.2—Operating Rating Level

Load ratings based on the Operating rating level generally describe the maximum permissible live load to

which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at Operating level may shorten the life of the bridge.

2B.3—RATING METHOD

In the IoH load rating of bridge members, one method for checking the capacity of the members is provided in Section 2, Part B—the Load Factor method.

The Load Factor method is based on analyzing a structure subject to multiples of the actual loads (factored loads). Different factors are applied to each type of load, which reflect the uncertainty inherent in the load calculations. The rating is determined such that the effect of the factored loads does not exceed the strength of the member.

C2B.3

In addition to the one method described in Part B, the LRFR method for IoH may be used. See Section 2, Part A, for more information on LRFR.

2B.4—RATING EQUATION

2B.4.1—General

The following general expression should be used in determining the load rating of the structure:

$$RF = \frac{C - A_1 D}{A_2 L(1+I)} \tag{2B.4.1-1}$$

where:

RF = The rating factor for the live load carrying capacity.

C = The capacity of the member (see Article 2B.5). The determination of C should be in accordance with Article 6B.4.1 of *AASHTO Manual for Bridge Evaluation.*

D = The dead load effect on the member. The determination of D is provided in Article 6B.6.1 of *AASHTO Manual for Bridge Evaluation.*

L = The live load effect on the member (see Article 2B.6.2)

I = The impact factor to be used with the live load effect (see Article 2B.6.4)

A_1 = Factor for dead loads. The determination of A_1 is provided in Article 6B.4.1 of *AASHTO Manual for Bridge Evaluation.*

A_2 = Factor for live load (see Articles 2B.4.2 and 2B.4.3)

C2B.4.1

The rating equation is used for the Load Factor method to evaluate a member capacity. The application of the basic rating equation to steel, concrete, and timber bridges is illustrated in Appendix A (load rating examples).For comparison, a number of standard loads other than IoH are used in these examples using load and resistance factor method, load factor method, and allowable stress method. However, the allowable stress method is not covered in this Guide Manual for IoH. This method was out of the scope of NCHRP Project 12-110 that developed this Guide Manual.

For example, at the maximum moment section of a girder, the bending moment is the “load effect” to be evaluated. The capacity of the girder will be the moment capacity which the girder cross-section could safely carry at the rating level desired. The dead load effect is the theoretical bending moment due to dead loads at the section being evaluated. The live load bending moment is computed based on the truck configuration or lane load selected for the rating and AASHTO impact and distribution factors. Appropriate factors (A_1 and A_2) need be selected and RF determined.

Fatigue evaluation for IoH is not required due to low traffic volume observed in available weigh-in-motion records. Reference: Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202?

2B.4.2—Load Factor

For implement of husbandry Tier 1 load rating, A_2 = 2.17 for Inventory level and A_2=1.3 for Operating level. For implements of husbandry Tier 2 and 3 load rating, A_2 =2.06 for Inventory level and A_2 =1.24 for Operating level.

2B.5—NOMINAL CAPACITY: *C*

2B.5.1—General

The nominal capacity to be used in the rating equation depends on the structural materials, the rating method, and rating level used. Nominal capacities based on the Load Factor method are discussed in Article 2B.5.3.

2B.5.2—Allowable Stress Method

The allowable stress method is not covered in this Guide Manual.

C2B.5.2

The allowable stress method was out of the scope of NCHRP Project 12-110 that developed this Guide Manual.

2B.5.3—Load Factor Method

Nominal capacity of structural steel, reinforced concrete and prestressed concrete should be the same as specified in Article 6B.4.1 of *AASHTO Manual for Bridge Evaluation.*

2B.6—LOADINGS

Article 2B.6 discusses the loads to be used in determining the load effects in the basic rating Eq. 2B.4.1-1.

2B.6.1—Dead Load: *D*

The dead load effects of the structure should be computed in accordance with Article 6B.6.1 of *AASHTO Manual for Bridge Evaluation*.

2B.6.2—Rating Live Load

Three levels of IoH load rating should be considered for IoH load rating: Tiers 1 to 3. Tier 3 shall be limited to extreme cases if the need is justifiable.

The three tiers are respectively equivalent to legal load rating, annual/semiannual or routine permit load

C2B.6.2

Bridge Owner may define tiers for accommodating and managing implements of husbandry (IoH) to meet the need in the jurisdiction. It is recommended that Tier 1 load rating be equivalent to legal load rating; Tier 2 be equivalent to overweight routine or annual/semiannual

rating, and special or trip permit load rating. See more details in Article 2B.6.2.2.

permit load rating, and Tier 3 be equivalent to overweight special/trip permit load rating if these demands exist and are justifiable. All three tiers may have an allowance for wider gage widths of IoH if observed in the jurisdiction. Wider gage widths distribute the vehicular load to a wider deck area and to more primary members. As a result effectively, for example, Tier 1 IoH load may be heavier than the Federal Bridge Formula (FBF) limit, while the internal forces or stresses will be at similar levels as FBF.

2B.6.2.1—Wheel Loads (Deck)

In general, stresses in the deck do not control the load rating except in special cases. The calculation of bending moments in the deck should be in accordance with AASHTO Standard Specifications. Wheel loads should be in accordance with the current AASHTO Standard Specifications.

2B.6.2.2—Truck Loads

The live or moving loads to be applied on the deck for determining the rating depends on the rating tier.

For IoH Tier 1 equivalent to the jurisdiction's legal load that does not vary significantly from the AASHTO legal load defined in *Manual for Bridge Evaluation*, the notional model in Figure 2B.6.2-1 is recommended to be used for Tier 1 load rating.

For IoH Tier 2 equivalent to the jurisdiction's annual or semiannual permit load, the actual IoH vehicle should be used for Tire 2 load rating.

For IoH Tier 3 equivalent to the jurisdiction's special or trip permit load, the actual IoH vehicle should be used for Tier 3 load rating.

Respectively for these Tiers, the live load factors are recommended in Article 2B.4.2.

The transverse placement of wheel lines should be in conformance with the current AASHTO Standard Specifications and the following:

Roadway widths from 18 to 20 ft should have two design lanes, each equal to one-half the roadway width. Live loadings should be centered in these lanes. Roadway widths less than 18 ft should carry one traffic lane only.

When conditions of traffic movements and volume would warrant it, fewer traffic lanes than specified by AASHTO may be considered.

One-lane loading should be applied for all three tiers.

C2B.6.2.2

For States having legal load much heavier than the AASHTO legal load, the notional model in Figure 2B.6.2-1 or another similar model selected by Bridge Owner is recommended to be used for Tier 1 load rating. If a different model much heavier is selected, the live load factors for Tier 1 need to be developed for the jurisdiction.

Article 1.3 discusses development of IoH load rating program with participation of stakeholders for the jurisdiction. Fu et al. NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202? provides more guidance for ensuring bridge safety for load rating IoH.

Available weigh-in-motion data have shown low volumes of IoH, which does not justify additional lane(s) of load. Refer to Fu et al. NCHRP Report ???, 202?

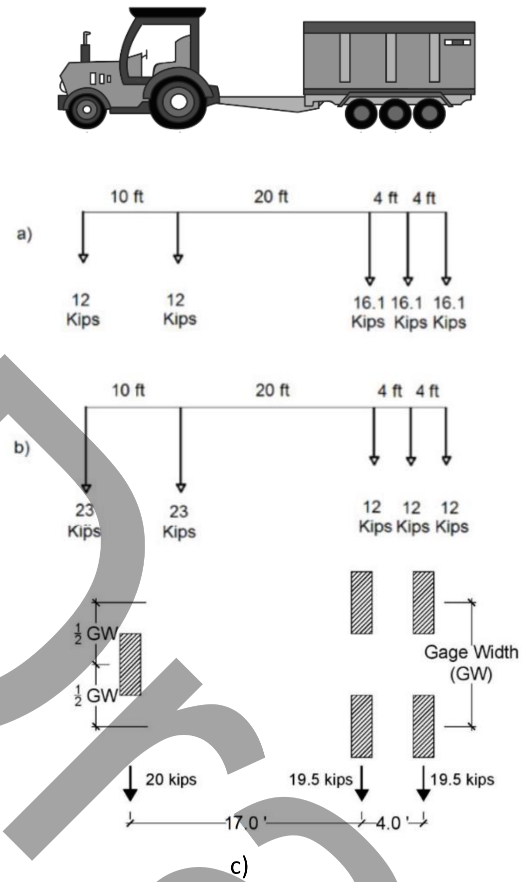


Figure 2B.6.2-1—Notional Load for Implement of Husbandry Tier 1: a), b), or c) whichever inducing maximum load effect; a) and b) for dual-wheel-steering –axle IoH, c) for single-wheel-steering-axle IoH; Maximum axle load = 23 kips. Maximum gross vehicle weight = 92 kips.

2B.6.2.3—Lane Loads

No lane load is required in IoH load rating.

C2B.6.2.3

Observed low volumes of IoH in available weigh-in-motion records justify no lane load to be used for load rating.

2B.6.2.4—Live Load Effects: *L*

Live load moments in longitudinal stringers and girders may be calculated using Table A2B-3 for live load moments produced by implement of husbandry Tier 1 load in Figure 2B.6.2-1.

Live load moments of IoH Tier 1 in Figure 2B.6.2-1 in the intermediate and end floor beams of trusses and through girders may be calculated by using Tables B2B-3 and C2B-3. The tables provide a convenient means of computing the live load moments based on the IoH Tier 1 load.

C2B.6.2.4

For comparison, Tables A2B-1 and A2B-2 include live load moments produced by the HS-20 and other standard trucks for comparison.

For comparison, Tables B2B-1, B2B-2, C2B-1, and C2B-2 include intermediate and end floor beams of trusses and through girders for HS-20 and other standard trucks for comparison.

2B.6.3—Distribution of Loads

For IoH load rating, the modifying factors *MF* in Articles 2B.6.3.1 and 2B.6.3.2 are recommended as multiplicative factors to the distribution factors in the current AASHTO Standard Specifications.

Two types of implements of husbandry are covered in these multiplicative modifying factors. Article 2B.6.3.1 specifies *MF* for dual-wheel-steering-axle implements of husbandry, and Article 2B.6.3.2 for single-wheel-steering-axle implements of husbandry.

When the vehicle's gage width GW varies among the axles, the gage width *GW* is to be computed in accordance with Article 2A.2.3.2.

2B.6.3.1- Dual-wheel-steering-axle Implements of Husbandry

Tables 2B.6.3.1-1 and 2B.6.3.1-2 contain modifying factor *MF* respectively for moment and shear in beam spans. Table 2B.6.3.1-3 includes *MF* for equivalent slab strip width in slab spans.

R_1 in this Article is equal to 1.15 for $GW \leq 6$ ft and 0.85 for $GW > 6$ ft.

2B.6.3.2- Single-wheel-steering-axle Implements of Husbandry

Tables 2B.6.3.2-1 and 2B.6.3.2-2 contain modifying factor *MF* respectively for moment and shear in beam spans. Table 6B.6.3.2-3 includes *MF* for equivalent slab strip width in slab spans.

R_2 in this Article is equal to 1.05.

C2B.6.3

The recommended multiplicative modifying factors are to account for the effect of wider or narrower gage widths than standard 6 ft and/or single-wheel-steering-axle of implements of husbandry.

Table 2B.6.3.1-1—Modifying Factor *MF* for Moment for Dual -Wheel-Steering-Axle IoH in Interior and Exterior BeamsType of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.2.1-1	For Moment in Interior Beams, One Lane Loaded (applicable to Standard Specifications Article 3.23.2)	For Moment in Exterior Beams, One Lane Loaded (applicable to Standard Specifications Article 3.23.2)	Range of Applicability
Wood Deck on Wood Beams	l	$1 - 0.340 R_1 Ln(\frac{GW}{6})$	$1 - 0.376 R_1 Ln(\frac{GW}{6})$	$0.7 \le S \le 6.0$ $20 \le L \le 45$ $3.0 \le t_s \le 10.0$ $850 \le I \le 12,000$ $5 \le GW \le 12$ $N_b \ge 5$
Concrete Deck on Steel Beams	a	$1 - 0.301 R_1 Ln(\frac{GW}{6})$	$1 - 0.887 R_1 Ln(\frac{GW}{6})$ $(\frac{GW}{L})^{0.870}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_s \le 14.0$ $5 \le GW \le 12$ $N_b \ge 4$
Wood Deck on Steel Beams	a	$1 - 0.499 R_1 Ln(\frac{GW}{6})$ $(\frac{GW}{L})^{0.310}$	$1 - 0.263 R_1 Ln(\frac{GW}{6})$	$1.5 \le S \le 6.0$ $20 \le L \le 140$ $3 \le t_S \le 10$ $5 \le GW \le 12$ $N_b \ge 5$
Concrete Deck on Prestressed I Beams	k	$1 - 0.650 R_1 Ln(\frac{GW}{6})(\frac{S}{L})^{0.50}$	$1 - 0.531 R_1 Ln(\frac{GW}{6})(\frac{S}{L})^{0.40}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_S \le 11.0$ $5 \le GW \le 12$ $N_b \ge 4$
Concrete deck on Adjacent Concrete Box Beams	f, g	$1 - 0.198 R_1 Ln(\frac{GW}{6})$	$1 - 0.179 R_1 Ln(\frac{GW}{6})$	$3 \le S \le 5$ $20 \le L \le 120$ $5 \le t_S \le 6$ $N_b \ge 7$ $5 \le GW \le 12$
Concrete T Beams	e	$1 - 3.281 R_1 Ln(\frac{GW}{6})(\frac{S}{L})^{1.48}$	$1 - 0.238 R_1 Ln(\frac{GW}{6})$	$3.5 \le S \le 14$ $20 \le L \le 90$ $4.5 \le t_S \le 12$ $N_b \ge 4$ $5 \le GW \le 12$

Table 2B.6.3.1-2— Modifying Factor *MF* for Shear for Dual -Wheel-Steering-Axle IoH in Interior and Exterior Beams

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.2.1-1	For Interior Beams One Lane Loaded (applicable to Standard Specifications Article 3.23.2)	For Exterior Beams One Lane Loaded (applicable to Standard Specifications Article 3.23.2)	Range of Applicability
Wood Deck on Wood Beams	l	$1-0.362R_1Ln(\frac{GW}{6})$ $(\frac{t_s}{6})^{0.51}(\frac{S}{9})^{0.17}$	$1-0.284R_1Ln(\frac{GW}{6})$ $(\frac{t_s}{6})^{0.67}(\frac{S}{9})^{0.79}$	$0.7 \le S \le 6.0$ $20 \le L \le 45$ $3.0 \le t_s \le 10.0$ $850 \le I \le 12,000$ $5 \le GW \le 12$ $N_b \ge 5$
Concrete Deck on Steel Beams	a	$1-0.509R_1Ln(\frac{GW}{6})(\frac{S}{14})^{0.60}$	$1-0.640R_1Ln(\frac{GW}{6})(\frac{S}{15})^{0.50}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_s \le 14.0$ $5 \le GW \le 12$ $N_b \ge 4$
Wood Deck on Steel Beams	a	$1-0.134R_1Ln(\frac{GW}{6})(\frac{L}{14})^{0.12}$ $(\frac{t_s}{6})^{1.10}(\frac{GW}{t_s})^{0.15}$	$1-0.334R_1Ln(\frac{GW}{6})$ $(\frac{t_s}{GW})^{0.76}(\frac{S}{GW})^{0.44}$	$1.5 \le S \le 6.0$ $20 \le L \le 140$ $3 \le t_s \le 10$ $5 \le GW \le 12$ $N_b \ge 5$
Concrete Deck on Prestressed I Beams	k	$1-0.863R_1Ln(\frac{GW}{6})(\frac{S}{12})^{0.25}$	$1-0.526R_1Ln(\frac{GW}{6})(\frac{S}{12})^{0.34}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_s \le 11.0$ $5 \le GW \le 12$ $N_b \ge 4$
Concrete deck on Adjacent Concrete Box Beams	f, g	$1-0.147R_1Ln(\frac{GW}{6})$	$1-0.097R_1Ln(\frac{GW}{6})$	$3 \le S \le 5$ $20 \le L \le 120$ $5 \le t_s \le 6$ $N_b \ge 7$ $5 \le GW \le 12$
Concrete T Beams	e	$1-3.097R_1Ln(\frac{GW}{6})$ $(\frac{6}{GW})^{1.87}(\frac{S}{L})^{0.93}$	$1-0.321R_1Ln(\frac{GW}{6})(\frac{S}{L})^{1.53}$	$3.5 \le S \le 14$ $20 \le L \le 90$ $4.5 \le t_s \le 12$ $N_b \ge 4$ $5 \le GW \le 12$

Table 2B.6.3.1-3— Modifying Factor *MF* for Dual -Wheel-Steering-Axle IoH for Equivalent Slab Width

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.3-1	For Interior Strip One Lane Loaded (applicable to Standard Specifications Article 3.24.3.2)	For Exterior Strip One Lane Loaded (applicable to Standard Specifications Article 3.24.8)	Range of Applicability
Wood or Concrete Slabs	a,b	$\dfrac{1}{1-0.155R_1 Ln(\frac{GW}{6})}$	1	$20 \le L \le 60$ $25 \le t_S \le 45$ $12 \le W \le 28$ $5 \le GW \le 12$

Table 2B.6.3.2-1— Modifying Factor MF for Moment for Single -Wheel-Steering-Axle IoH in Interior and Exterior Beams

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.2.1-1	For Moment in Interior Beams, One Lane Loaded (applicable to LRFD Design Table 4.6.2.2.2b-1)	For Moment in Exterior Beams One Lane Loaded (applicable to LRFD Design Table 4.6.2.2.2d-1)	Range of Applicability
Wood Deck on Wood Beams	l	$1.088R_2\left(\frac{6}{GW}\right)^{0.298}\left(\frac{GW}{L}\right)^{0.022}\left(\frac{GW}{t_s}\right)^{0.012}$	$1.141R_2\left(\frac{S}{GW}\right)^{0.064}\left(\frac{Lt_S^3}{I}\right)^{0.114}\left(\frac{S}{L}\right)^{0.114}\left(\frac{9}{S}\right)^{0.105}\left(\frac{6}{t_S}\right)^{0.307}$	$0.7 \le S \le 6.0$ $20 \le L \le 45$ $3.0 \le t_s \le 10.0$ $850 \le I \le 12,000$ $6 \le GW \le 10$ $N_b \ge 5$
Concrete Deck on Steel Beams	a	$0.726R_2\left(\frac{L}{W}\right)^{0.233}\left(\frac{L}{S}\right)^{0.071}\left(\frac{14}{L}\right)^{0.225}$	$R_2[1.015\left(\frac{6}{GW}\right)^{0.228} - 0.233\left(\frac{S}{L}\right) + 0.111\left(\frac{GW}{L}\right) + 0.018]$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_s \le 14.0$ $6 \le GW \le 10$ $N_b \ge 4$
Wood Deck on Steel Beams	a	$R_2[1.118\left(\frac{6}{GW}\right)^{0.151}\left(\frac{S}{L}\right)^{0.039} + 0.094 - 0.559\left(\frac{GW}{L}\right) - 0.00222L + 0.240\left(\frac{t_S}{6}\right) - 0.175\left(\frac{t_S}{GW}\right)]$	$1.736R_2\left(\frac{S}{L}\right)^{0.050}\left(\frac{1}{S}\right)^{0.235}\left(\frac{S}{GW}\right)^{0.091}\left(\frac{S}{t_S}\right)^{0.053}$	$1.5 \le S \le 6.0$ $20 \le L \le 140$ $3 \le t_S \le 10$ $6 \le GW \le 10$ $N_b \ge 5$
Concrete Deck on Prestressed I Beams	k	$1.132R_2\left(\frac{L}{GW}\right)^{0.198}\left(\frac{L}{S}\right)^{0.025}\left(\frac{1}{L}\right)^{0.150}$	$1.259R_2\left(\frac{S}{GW}\right)^{0.184}\left(\frac{L}{S}\right)^{0.204}\left(\frac{1}{L}\right)^{0.164}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_S \le 11.0$ $6 \le GW \le 10$ $N_b \ge 4$
Concrete deck on Adjacent Concrete Box Beams	f, g	$0.967R_2\left(\frac{S}{GW}\right)^{0.157}\left(\frac{L}{S}\right)^{0.0238}\left(\frac{t_S}{S}\right)^{0.176}$	$1.323R_2\left(\frac{L}{GW}\right)^{0.358}\left(\frac{1}{L}\right)^{0.514}\left(\frac{GW}{b}\right)^{0.316}\left(\frac{b}{t_S}\right)^{0.272}\left(\frac{L^2}{bd}\right)^{0.165}$	$3 \le S \le 5$ $20 \le L \le 120$ $5 \le t_S \le 6$ $N_b \ge 7$ $6 \le GW \le 10$
Concrete T Beams	e	$0.975R_2\left(\frac{L}{GW}\right)^{0.193}\left(\frac{L}{S}\right)^{0.038}\left(\frac{1}{L}\right)^{0.111}$	$0.968R_2\left(\frac{S}{GW}\right)^{0.125}\left(\frac{L}{GW}\right)^{0.023}$	$3.5 \le S \le 14$ $20 \le L \le 90$ $4.5 \le t_S \le 12$ $N_b \ge 4$ $6 \le GW \le 10$

Table 2B.6.3.2-2— Modifying Factor MF for Shear for Single -Wheel-Steering-Axle IoH in Interior and Exterior Beams

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.2.1-1	For Interior Beams One Lane Loaded (applicable to Standard Specifications Article 3.23.2)	For Exterior Beams One Lane Loaded (applicable to Standard Specifications Article 3.23.2)	Range of Applicability
Wood Deck on Wood Beams	l	$1.232R_2(\frac{S}{GW})^{0.035}(\frac{9}{S})^{0.074}$	$1.199R_2(\frac{S}{GW})^{0.045}(\frac{9}{S})^{0.098}$	$0.7 \le S \le 6.0$ $20 \le L \le 45$ $3.0 \le t_s \le 10.0$ $850 \le I \le 12,000$ $6 \le GW \le 10$ $N_b \ge 5$
Concrete Deck on Steel Beams	a	$1.035R_2(\frac{GW}{S})^{0.261}(\frac{L}{S})^{0.059}(\frac{6}{GW})^{0.396}$	$1.045R_2(\frac{6}{GW})^{0.334}(\frac{L}{S})^{0.050}(\frac{GW}{S})^{0.198}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_s \le 14.0$ $6 \le GW \le 10$ $N_b \ge 4$
Wood Deck on Steel Beams	a	$R_2[1.542 - 0.0437(\frac{GW}{6}) - 0.337(\frac{S}{9}) - 0.0204(\frac{GW}{S})]$	$1.166R_2(\frac{9}{S})^{0.160}(\frac{S}{GW})^{0.087}$	$1.5 \le S \le 6.0$ $20 \le L \le 140$ $3 \le t_S \le 10$ $6 \le GW \le 10$ $N_b \ge 5$
Concrete Deck on Prestressed I Beams	k	$3.013R_2(\frac{S}{GW})^{0.239}(\frac{L}{S})^{0.662}(\frac{1}{L})^{0.597}$	$1.297R_2(\frac{6}{GW})^{0.399}(\frac{GW}{S})^{0.264}$	$3.5 \le S \le 14.0$ $20 \le L \le 150$ $5.5 \le t_S \le 11.0$ $6 \le GW \le 10$ $N_b \ge 4$
Concrete deck on Adjacent Concrete Box Beams	f, g	$1.229R_2(\frac{S}{GW})^{0.077}(\frac{L}{S})^{0.103}$	$1.193R_2(\frac{b}{GW})^{0.069}(\frac{12}{S})^{0.191}$	$3 \le S \le 5$ $20 \le L \le 120$ $5 \le t_S \le 6$ $N_b \ge 7$ $6 \le GW \le 10$
Concrete T Beams	e	$0.747R_2(\frac{L}{S})^{0.058}(\frac{14}{S})^{0.392}(\frac{S}{GW})^{0.136}(\frac{t_S}{S})^{0.229}$	$0.987R_2(\frac{14}{S})^{0.307}(\frac{S}{GW})^{0.112}$	$3.5 \le S \le 14$ $20 \le L \le 90$ $4.5 \le t_S \le 12$ $N_b \ge 4$ $6 \le GW \le 10$

Table 2B.6.3.2-3— Modifying Factor *MF* for Single -Wheel-Steering-Axle IoH for Equivalent Slab Width

Type of Superstructure	Applicable Cross-Section from LRFD Design Table 4.6.2.3-1	For Interior Strip One Lane Loaded (applicable to Standard Specifications Article 3.24.3.2)	For Exterior Strip One Lane Loaded (applicable to Standard Specifications Article 3.24.8)	Range of Applicability
Wood or Concrete Slabs	a,b	$0.618(GW)^{0.137}(\frac{L}{GW})^{0.160}\left(\frac{1}{R_2}\right)$	1	$20 \le L \le 60$ $25 \le t_S \le 45$ $12 \le W \le 28$ $6 \le GW \le 10$

2B.6.4—Impact: *I*

Impact should be added to the live load used for rating in accordance with the current AASHTO Standard Specifications up to 20%.

Impact to be applied for IoH load effects in superstructure wood components shall be 20% if the components are older than 15 years. For new superstructure wood components, dynamic allowance need~~s~~ not be applied. For superstructure wood components younger than 15 years, impact may be linearly interpolated.

The Engineer may adjust the dynamic load allowance considering the field condition including, but not limited to, pavement smoothness, deck surface condition, enforced speed, and other conditions that may require a vehicle to substantially reduce speed to cross the bridge.

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C2B.6.4

The condition of the approach roadway and deck joints may also influence the selection of an appropriate impact factor. Some guidelines are provided in Article C2A.4.4.3.

The 20 percent impact is mainly due to expected lower travel speed of IoH at 35 mph or lower.

Dynamic load test results have shown that old superstructure timber components experience noticeable dynamic amplification of static load effect, although relatively sometimes lower than steel counterparts (Fu et al. ???? NCHRP Report ??? "*Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*" 202?)

Due to such a large number of parameters involved in dynamic behavior and effects to bridge members, the most reliable approach to assessing the dynamic impact effect of IoH is in-situ physical testing. Numerical modeling such as finite element analysis alone has to be based on many assumptions that are not readily verifiable.

APPENDIX A2B—LIVE LOAD MOMENTS ON LONGITUDINAL STRINGERS OR GIRDERS

Table A2B-1—Live Load Moments on Longitudinal Stringers or Girders for Routine Commercial Traffic

Live Load Moments in ft-kips per Wheel Line										
Type of Loading (without Impact)					Span, ft c/c	Type of Loading (with Impact)				
H-15	**HS-20**	3	3S2	3-3		H-15	**HS-20**	3	3S2	3-3
15.0	20.0	10.6	9.7	10.0	5	19.5	26.0	13.8	12.6	13.0
18.0	24.0	12.8	11.6	12.0	6	23.4	31.2	16.6	15.1	15.6
21.0	28.0	15.2	13.8	14.0	7	27.3	36.4	19.7	18.0	18.2
24.0	32.0	19.1	17.4	16.0	8	31.2	41.6	24.9	22.7	20.8
27.0	36.0	23.1	21.1	19.1	9	35.1	46.8	30.1	27.4	24.8
30.0	40.0	27.2	24.8	22.4	10	39.0	52.0	35.4	32.2	29.1
33.0	44.0	31.3	28.5	25.8	11	42.9	57.2	40.7	37.1	33.5
36.0	48.0	35.4	32.2	29.2	12	46.8	62.4	46.0	42.0	37.9
39.0	52.0	39.6	36.1	32.6	13	50.7	67.6	51.4	46.9	42.3
42.0	56.0	43.7	39.9	36.0	14	54.6	72.8	56.8	51.8	46.8
45.0	60.0	47.9	43.7	39.4	15	58.5	78.0	62.2	56.8	51.3
48.0	64.0	52.1	47.5	42.9	16	62.4	83.2	67.7	61.7	55.7
51.0	68.0	56.3	51.3	46.3	17	66.3	88.4	73.1	66.7	60.2
54.0	72.0	60.4	55.1	49.8	18	70.2	93.6	78.6	71.6	64.7
57.0	76.0	64.6	58.9	53.2	19	74.1	98.8	84.0	76.6	69.2
60.0	80.0	68.9	62.8	56.7	20	78.0	104.0	89.5	81.6	73.7
63.0	84.0	73.1	66.6	60.2	21	81.9	109.2	95.0	86.6	78.2
66.0	88.0	77.3	70.5	63.6	22	85.8	114.4	100.5	91.6	82.7
69.0	92.0	81.5	75.2	67.1	23	89.7	119.6	105.9	97.7	87.2
72.0	96.3	85.7	80.3	70.6	24	93.6	125.2	111.4	104.4	91.8
75.0	103.7	89.9	85.4	74.1	25	97.5	134.8	116.9	111.0	96.3
78.0	111.1	94.2	90.5	77.5	26	101.4	144.4	122.4	117.7	100.8
81.3	118.5	98.4	95.6	81.0	27	105.7	154.1	127.9	124.3	105.3
85.1	126.0	102.6	100.7	84.5	28	110.6	163.8	133.4	131.0	109.8
88.8	133.5	106.8	105.9	88.0	29	115.4	173.6	138.9	137.6	114.4
92.5	141.0	112.9	111.0	91.5	30	120.2	183.3	146.8	144.3	118.9
99.8	156.2	125.3	121.2	101.5	32	130.0	203.1	162.9	157.6	132.0
107.4	171.8	137.6	131.5	112.3	34	139.6	223.3	178.9	170.9	146.0
114.8	189.4	150.0	141.7	123.1	36	149.2	246.2	195.0	184.2	160.1
122.3	207.1	162.4	151.9	134.0	38	159.0	269.2	211.1	197.5	174.1
129.7	224.9	174.8	162.2	144.8	40	168.6	292.4	227.3	210.8	188.3
137.2	242.7	187.2	172.4	155.7	42	178.3	315.3	243.3	224.0	202.3
144.7	260.4	199.7	182.7	166.6	44	187.5	337.5	258.7	236.7	215.8
152.1	278.3	212.1	192.9	177.4	46	196.6	359.6	274.1	249.3	229.3
159.6	296.1	224.5	203.2	188.3	48	205.7	381.7	289.4	261.9	242.8
167.1	314.0	237.0	220.8	199.3	50	214.8	403.8	304.7	283.9	256.2
174.6	331.8	249.4	238.4	214.3	52	223.9	425.5	319.9	305.8	274.8
182.0	349.7	261.8	256.1	231.3	54	232.8	447.3	335.0	327.6	295.9
189.5	367.6	274.3	273.8	248.3	56	241.8	469.1	350.1	349.4	316.9
198.8	385.4	286.8	291.4	265.3	58	253.1	490.6	365.1	371.1	337.7
209.2*	403.3	299.2	309.2	282.3	60	265.8*	512.2	380.1	392.7	358.5
265.1*	492.8	361.5	398.0	372.2	70	333.1*	619.0	454.2	500.1	467.6
327.0*	582.4	423.9	487.1	471.9	80	406.8*	724.5	527.3	605.9	587.0
394.9*	672.2	486.3	576.4	571.7	90	486.7*	828.8	599.4	710.5	704.6
468.8*	762.0	548.7	665.9	671.5	100	572.9*	931.2	670.7	813.9	820.7
634.5*	941.6	673.6	845.1	871.3	120	764.0*	1133.7	811.1	1017.5	1049.1
824.2*	1121.4	798.5	1024.5	1071.1	140	979.8*	1333.3	949.2	1217.8	1273.2
1038.0*	1384.0*	923.5	1204.1	1270.9	160	1220.1*	1626.2*	1085.5	1415.3	1493.9
1275.8*	1701.0*	1048.4	1383.7	1470.8	180	1484.9*	1980.0*	1222.3	1610.6	1712.0
1537.5*	2050.0*	1173.4	1563.5	1670.8	200	1774.0*	2365.7*	1353.9	1804.0	1927.8
2296.9*	3062.5*	1485.8	2013.0	2170.6	250	2603.1*	3469.8*	1683.9	2281.4	2460.0
3206.2*	4275.0*	1798.2	2462.6	2670.5	300	3583.5*	4779.4*	2009.8	2752.4	2984.7

* Based on standard lane loading. All other values are based on standard truck loading.

Table A2B-2—Live Load Moments on Longitudinal Stringers or Girders for Specialized Hauling Vehicles

Live Load Moments in ft-kips per Wheel Line												
Type of Loading (without Impact)						Span, ft c/c	Type of Loading (with Impact)					
HS-20	NRL	SU4	SU5	SU6	SU7		HS-20	NRL	SU4	SU5	SU6	SU7
20.0	10.6	10.6	10.6	10.6	10.6	5	26.0	13.8	13.8	13.8	13.8	13.8
24.0	12.8	12.8	12.8	12.8	12.8	6	31.2	16.6	16.6	16.6	16.6	16.6
28.0	15.2	15.2	15.2	15.2	15.2	7	36.4	19.8	19.8	19.8	19.8	19.8
32.0	19.1	19.1	19.1	19.1	19.1	8	41.6	24.8	24.8	24.8	24.8	24.8
36.0	23.1	23.1	23.1	23.1	23.1	9	46.8	30.0	30.0	30.0	30.0	30.0
40.0	27.9	27.9	27.9	27.9	27.9	10	52.0	36.3	36.3	36.3	36.3	36.3
44.0	33.1	33.1	33.1	33.1	33.1	11	57.2	43.0	43.0	43.0	43.0	43.0
48.0	38.3	38.3	38.3	38.3	38.3	12	62.4	49.8	49.8	49.8	49.8	49.8
52.0	43.5	43.5	43.5	43.5	43.5	13	67.6	56.6	56.6	56.6	56.6	56.6
56.0	48.8	48.8	48.8	48.8	48.8	14	72.8	63.4	63.4	63.4	63.4	63.4
60.0	54.4	54.0	54.0	54.4	54.4	15	78.0	70.7	70.2	70.2	70.7	70.7
64.0	60.6	59.2	59.2	60.6	60.6	16	83.2	78.8	77.0	77.0	78.8	78.8
68.0	66.7	64.5	65.3	66.7	66.7	17	88.4	86.7	83.9	84.9	86.7	86.7
72.0	73.6	69.7	71.5	73.6	73.6	18	93.6	95.7	90.6	93.0	95.7	95.7
76.0	80.8	74.9	77.8	80.8	80.8	19	98.8	105.0	97.4	101.1	105.0	105.0
80.0	88.1	80.2	84.0	88.1	88.1	20	104.0	114.5	104.3	109.2	114.5	114.5
84.0	95.3	85.4	90.3	95.3	95.3	21	109.2	123.9	111.0	117.4	123.9	123.9
88.0	102.6	90.7	96.5	102.6	102.6	22	114.4	133.4	117.9	125.5	133.4	133.4
92.0	110.2	95.9	102.8	109.8	110.2	23	119.6	143.3	124.7	133.6	142.7	143.3
96.3	118.4	101.2	109.0	117.1	118.4	24	125.2	153.9	131.6	141.7	152.2	153.9
103.7	126.6	106.4	115.3	124.3	126.6	25	134.8	164.5	138.3	149.9	161.6	164.5
111.1	135.5	111.6	121.5	131.6	134.8	26	144.4	176.2	145.1	158.0	171.1	175.2
118.5	144.8	116.9	127.8	138.8	143.0	27	154.1	188.2	152.0	166.1	180.4	185.9
126.0	154.0	123.4	134.0	146.1	151.2	28	163.8	200.2	160.4	174.2	189.9	196.6
133.5	163.3	130.1	140.3	153.3	159.4	29	173.6	212.3	169.1	182.4	199.3	207.2
141.0	172.5	136.8	146.5	160.6	167.6	30	183.3	224.3	177.8	190.5	208.7	217.9
156.2	191.0	150.2	159.0	175.1	184.0	32	203.1	248.3	195.3	206.7	227.6	239.2
171.8	209.5	163.6	172.0	189.6	200.5	34	223.3	272.4	212.7	223.6	246.5	260.7
189.4	228.9	177.1	187.3	205.4	216.9	36	246.2	297.6	230.2	243.5	267.0	282.0
207.1	248.8	190.5	202.7	222.7	235.6	38	269.2	323.4	247.7	263.5	289.5	306.3
224.9	268.8	204.0	218.0	240.0	255.0	40	292.4	349.4	265.1	283.4	312.0	331.5
242.7	288.8	217.4	233.4	257.3	274.3	42	315.4	375.3	282.5	303.3	334.3	356.4
260.4	308.7	230.9	248.7	274.7	293.7	44	337.4	400.0	299.2	322.3	356.0	380.6
278.3	328.7	244.3	264.1	292.0	313.1	46	359.7	424.8	315.7	341.3	377.4	404.6
296.1	348.7	257.8	279.5	309.3	332.4	48	381.7	449.5	332.3	360.3	398.7	428.5
314.0	368.7	271.3	294.9	326.6	351.8	50	403.7	474.0	348.8	379.2	419.9	452.3
331.8	388.6	284.8	310.3	344.0	371.2	52	425.5	498.4	365.3	398.0	441.2	476.1
349.7	408.6	298.2	325.7	361.3	390.5	54	447.4	522.7	381.5	416.7	462.2	499.6
367.6	428.6	311.7	341.1	378.7	409.9	56	469.1	547.0	397.8	435.3	483.3	523.1
385.4	448.6	325.2	356.6	396.0	429.3	58	490.7	571.2	414.1	454.0	504.2	546.6
403.3	468.5	338.7	372.0	413.3	448.7	60	512.2	595.1	430.2	472.5	525.0	569.9
492.8	568.5	406.1	449.2	500.1	545.5	70	619.2	714.2	510.2	564.4	628.3	685.4
582.5	668.4	473.5	526.5	586.9	642.4	80	724.5	831.4	589.0	654.9	730.0	799.0
672.2	768.4	540.9	603.8	673.7	739.2	90	828.5	947.0	666.7	744.2	830.4	911.1
762.0	868.3	608.4	681.2	760.5	836.1	100	931.3	1061.3	743.6	832.6	929.5	1021.9
941.6	1068.3	743.3	836.0	934.2	1029.8	120	1133.8	1286.3	895.0	1006.6	1124.8	1240.0
1121.4	1268.2	878.3	990.9	1107.9	1223.6	140	1333.0	1507.5	1044.0	1177.8	1316.9	1454.4
1384.0*	1468.2	1013.2	1145.8	1281.6	1417.3	160	1626.8*	1725.8	1191.0	1346.8	1506.4	1665.9
1701.0*	1668.2	1148.2	1300.7	1455.3	1611.1	180	1979.9*	1941.7	1336.4	1513.9	1693.9	1875.2
2050.0*	1868.2	1283.2	1455.6	1629.0	1804.8	200	2365.4*	2155.6	1480.6	1679.5	1879.6	2082.5
3062.5*	2368.1	1620.7	1843.0	2063.3	2289.2	250	3470.8*	2683.8	1836.8	2088.7	2338.4	2594.4
4275.0*	2868.1	1958.1	2230.4	2497.7	2773.5	300	4777.9*	3205.5	2188.5	2492.8	2791.5	3099.8

* Based on standard loading. All other values based on standard truck loading.

Table A2B-3—Live Load Moments on Longitudinal Stringers or Girders for Implement of Husbandry Tier 1

Live Load Moments in ft-kips per Wheel Line				
Type of Loading (without Impact)				Span, ft c/c
H-15	**HS-20**	NRL	IoH Tier 1	
15.0	20.0	10.6	14.4	5
18.0	24.0	12.8	17.3	6
21.0	28.0	15.2	20.2	7
24.0	32.0	19.1	23.0	8
27.0	36.0	23.1	25.9	9
30.0	40.0	27.9	29.3	10
33.0	44.0	33.1	34.3	11
36.0	48.0	38.3	40.3	12
39.0	52.0	43.5	46.3	13
42.0	56.0	48.8	52.3	14
45.0	60.0	54.4	58.4	15
48.0	64.0	60.6	64.4	16
51.0	68.0	66.7	70.5	17
54.0	72.0	73.6	76.5	18
57.0	76.0	80.8	82.6	19
60.0	80.0	88.1	88.6	20
63.0	84.0	95.3	94.6	21
66.0	88.0	102.6	100.6	22
69.0	92.0	110.2	106.7	23
72.0	96.3	118.4	112.7	24
75.0	103.7	126.2	118.8	25
78.0	111.1	135.5	124.8	26
81.3	118.5	144.8	130.9	27
85.1	126.0	154.0	136.9	28
88.8	133.5	163.3	142.9	29
92.5	141.0	172.5	148.9	30
99.8	156.2	191.0	161.0	32
107.4	171.8	209.5	173.1	34
114.8	189.4	228.9	185.2	36
122.3	207.1	248.8	197.2	38
129.7	224.9	268.8	209.3	40
137.2	242.7	288.8	221.4	42
144.7	260.4	308.7	233.5	44
152.1	278.3	328.7	246.2	46
159.6	296.1	348.7	261.1	48
167.1	314.0	368.7	276.0	50
174.6	331.8	388.6	291.0	52
182.0	349.7	408.6	305.9	54
189.5	367.6	428.6	320.9	56
198.8	385.4	448.6	335.9	58
209.2*	403.3	468.5	350.8	60
265.1*	492.8	568.5	450.7	70
327.0*	582.4	668.4	552.4	80
394.9*	672.2	768.4	654.2	90
468.8*	762.0	868.3	756.2	100
634.5*	941.6	1068.3	960.4	120
824.2*	1121.4	1268.2	1164.9	140
1038.0*	1384.0*	1468.2	1369.4	160
1275.8*	1701.0*	1668.2	1574.1	180
1537.5*	2050.0*	1868.2	1778.8	200
2296.9*	3062.5*	2368.1	2290.9	250
3206.2*	4275.0*	2868.1	2803.1	300

* Based on standard lane loading. All other values are based on standard truck loading.

APPENDIX B2B—STRINGER LIVE LOAD REACTIONS ON TRANSVERSE FLOOR BEAMS AND CAPS (INTERMEDIATE TRANSVERSE BEAMS) (SIMPLE SPAN ONLY)

Table B2B-1—Live Load Reactions *R* in kips per Wheel Line, No Impact, for Routine Commercial Traffic

	Live Load Reactions *R* in kips per Wheel Line, No Impact				
	Type of Loading				
Stringer Span, ft	Type 3	Type 3S2	Type 3-3	H-15	**HS-20**
10	13.6	12.4	11.2	12.0	16.0
11	13.9	12.7	11.5	12.0	16.0
12	14.2	13.1	11.7	12.0	16.0
13	14.4	13.7	11.9	12.0	16.0
14	14.6	14.2	12.0	12.0	16.0
15	14.8	14.6	12.2	12.2	17.3
16	15.3	15.0	12.3	12.4	18.5
17	15.8	15.4	12.7	12.5	19.5
18	16.4	15.6	13.3	12.7	20.4
19	16.8	15.9	13.7	12.8	21.3
20	17.2	16.1	14.2	12.9	22.0
21	17.6	16.3	14.5	13.0	22.7
22	18.0	16.5	14.9	13.1	23.3
23	18.3	16.7	15.2	13.2	23.8
24	18.5	16.9	15.5	13.3	24.3
25	18.8	17.0	15.7	13.4	24.8
26	19.0	17.5	16.2	13.4	25.2
27	19.3	18.2	16.8	13.5	25.6
28	19.5	18.8	17.5	13.5	26.0
29	19.7	19.4	18.0	13.6	26.3
30	19.9	20.1	18.8	13.6	26.7

One-Lane Loading $M = \dfrac{(L-3)^2 R}{2L}$

*Two-Lane Roadway over 18 ft $M = \left(L - 9 + \dfrac{2.25}{L}\right)R$

*Wheel Line/Truss: $\begin{cases} \text{One-Lane Loading } = \left(1 + \dfrac{W-9}{C}\right) \\ \text{Two-Lane Loading } = \left(1 + \dfrac{W-18}{C}\right)2 \end{cases}$

where:

M = Moment in transverse beam

R = Reaction (tabular value)

L = Span of transverse beam

W = Width of roadway

C = Spacing, center-to-center of trusses

All values based on standard truck loadings.

* Based on 9-ft lane width.

Table B2B-2—Live Load Reactions *R* in kips per Wheel Line, No Impact, for Specialized Hauling Vehicles

Stringer Span, ft	Live Load Reactions *R* in kips per Wheel Line, No Impact — Type of Loading					
	SU4	SU5	SU6	SU7	NRL	HS-20
10	16.0	16.8	17.6	17.6	17.6	16.0
11	16.5	17.5	18.6	18.6	18.6	16.0
12	16.8	18.2	19.5	19.5	19.5	16.0
13	17.2	18.7	20.2	20.5	20.2	16.0
14	17.4	19.1	20.9	21.4	20.9	16.0
15	18.1	19.5	21.4	22.2	21.4	17.3
16	18.6	19.9	21.9	22.9	21.9	18.5
17	19.1	20.2	22.3	23.5	22.3	19.5
18	19.6	20.4	22.7	24.0	23.0	20.4
19	19.9	21.0	23.3	24.8	23.7	21.3
20	20.3	21.5	23.9	25.5	24.3	22.0
21	20.6	22.0	24.4	26.1	24.8	22.7
22	20.9	22.4	24.9	26.7	25.3	23.3
23	21.2	22.7	25.3	27.2	26.0	23.8
24	21.4	23.1	25.7	27.7	26.6	24.3
25	21.6	23.4	26.1	28.1	27.1	24.8
26	21.8	23.7	26.4	28.5	27.6	25.2
27	22.0	24.0	26.7	28.9	28.1	25.6
28	22.2	24.2	27.0	29.3	28.5	26.0
29	22.4	24.4	27.3	29.6	28.9	26.3
30	22.5	24.7	27.5	29.9	29.3	26.7

All values based on standard truck loadings.

Table B2B-3—Live Load Reactions *R* in kips per Wheel Line, No Impact, for Implement of Husbandry (IoH) Tier 1 (Intermediate Transverse Beams)

Stringer Span, ft	Live Load Reactions *R* in kips per Wheel Line, No Impact — Type of Loading: IoH Tier 1	NRL	HS-20
10	17.7	17.6	16.0
11	18.3	18.6	16.0
12	18.8	19.5	16.0
13	19.2	20.2	16.0
14	19.6	20.9	16.0
15	19.9	21.4	17.3
16	20.1	21.9	18.5
17	20.4	22.3	19.5
18	20.6	23.0	20.4
19	20.8	23.7	21.3
20	20.9	24.3	22.0
21	21.1	24.8	22.7
22	21.2	25.3	23.3
23	21.4	26.0	23.8
24	21.5	26.6	24.3
25	21.8	27.1	24.8
26	22.1	27.6	25.2
27	22.4	28.1	25.6
28	22.7	28.5	26.0
29	23.0	28.9	26.3
30	23.2	29.3	26.7

All values based on standard truck loadings.

APPENDIX C2B—STRINGER LIVE LOAD REACTIONS ON TRANSVERSE FLOOR BEAMS AND CAPS (END TRANSVERSE BEAMS) (SIMPLE SPAN ONLY)

Table C2B-1—Live Load Reactions *R* in kips per Wheel Line, No Impact, for Routine Commercial Traffic

	Live Load Reactions *R* in kips per Wheel Line, No Impact				
	Type of Loading				
Stringer Span, ft	Type 3	Type 3S2	Type 3-3	H-15	**HS-20**
10	13.6	12.4	11.2	12.0	16.0
11	13.9	12.7	11.5	12.0	16.0
12	14.2	12.9	11.7	12.0	16.0
13	14.4	13.1	11.9	12.0	16.0
14	14.6	13.3	12.0	12.0	16.0
15	14.7	13.4	12.1	12.2	17.1
16	14.9	13.9	12.3	12.4	18.0
17	15.0	14.3	12.4	12.5	18.9
18	15.1	14.6	12.4	12.7	19.6
19	15.2	14.9	12.5	12.8	20.2
20	15.7	15.2	12.6	12.9	20.8
21	16.1	15.5	13.1	13.0	21.3
22	16.6	15.7	13.5	13.1	21.8
23	16.9	15.9	13.8	13.2	22.2
24	17.3	16.1	14.2	13.3	22.6
25	17.6	16.3	14.5	13.4	23.0
26	17.9	16.4	14.8	13.4	23.4
27	18.1	16.6	15.0	13.5	23.7
28	18.4	16.7	15.3	13.5	24.0
29	18.6	16.8	15.5	13.6	24.4
30	18.8	17.0	15.7	13.6	24.8

All values based on standard truck loadings.

Table C2B-2—Live Load Reactions *R* in kips per Wheel Line, No Impact, for Specialized Hauling Vehicles

Stringer Span, ft	Live Load Reactions *R* in kips per Wheel Line, No Impact — Type of Loading: SU4	SU5	SU6	SU7	NRL	HS-20
10	14.4	14.4	14.4	14.4	14.4	16.0
11	14.9	15.0	15.0	15.0	14.9	16.0
12	15.5	15.4	15.5	15.5	15.5	16.0
13	15.8	16.0	16.1	16.1	16.1	16.0
14	16.2	16.6	16.7	16.7	16.9	16.0
15	16.6	17.3	17.3	17.4	17.4	17.1
16	16.8	17.8	17.8	17.8	17.8	18.0
17	17.1	18.1	18.2	18.2	18.4	18.9
18	17.1	18.5	18.5	18.7	18.9	19.6
19	17.8	18.9	19.0	19.0	19.4	20.2
20	18.0	19.3	19.3	19.3	20.1	20.8
21	18.6	19.5	19.5	19.5	20.3	21.3
22	19.0	19.6	19.6	19.6	20.7	21.8
23	19.2	20.2	20.2	20.2	21.0	22.2
24	19.8	20.6	20.7	20.7	21.5	22.6
25	20.0	21.2	21.2	21.2	21.9	23.0
26	20.1	21.4	21.4	21.4	21.9	23.4
27	17.2	21.7	21.7	21.7	22.2	23.7
28	20.6	22.1	22.1	22.2	22.6	24.0
29	20.7	22.5	22.4	22.4	22.8	24.4
30	21.2	22.6	22.6	22.6	23.1	24.8

All values based on standard truck loadings.

Table C2B-3—Live Load Reactions *R* in kips per Wheel Line, No Impact, for Implement of Husbandry (IoH) Tier 1 (End Transvers Beams)

	Live Load Reactions *R* in kips per Wheel Line, No Impact		
	Type of Loading		
Stringer Span, ft	IoH Tier 1	NRL	HS-20
10	15.6	14.4	16.0
11	16.0	14.9	16.0
12	16.3	15.5	16.0
13	16.7	16.1	16.0
14	17.3	16.9	16.0
15	17.7	17.4	17.1
16	18.1	17.8	18.0
17	18.5	18.4	18.9
18	18.8	18.9	19.6
19	19.1	19.4	20.2
20	19.3	20.1	20.8
21	19.6	20.3	21.3
22	19.8	20.7	21.8
23	20.0	21.0	22.2
24	20.1	21.5	22.6
25	20.3	21.9	23.0
26	20.4	21.9	23.4
27	20.6	22.2	23.7
28	20.7	22.6	24.0
29	21.0	22.8	24.4
30	21.3	23.1	24.8

All values based on standard truck loadings.

APPENDIX A: ILLUSTRATIVE EXAMPLES

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Specifications items identified to the right in the following examples refer to LRFD Design for “AASHTO LRFD Specifications for Bridge Design”, AASHTO for “AASHTO Standard Specifications for Highway Bridges”, or MBEIoH for this Guide Manual “AASHTO Guide Manual for Bridge Evaluation for Implements of Husbandry”. Those items without the specifications explicitly identified refer to AASHTO Manual for Bridge Evaluation. Load ratings of various standard loads are included herein for comparison with IoH load ratings.

APPENDIX A:

ILLUSTRATIVE EXAMPLES

A1—SIMPLE SPAN COMPOSITE STEEL STRINGER BRIDGE

PART A—LOAD AND RESISTANCE FACTOR RATING METHOD

A1A.1—Evaluation of an Interior Stringer

A1A.1.1—Bridge Data

Span:	65 ft
Year Built:	1964
Material:	A36 Steel F_y = 36 ksi f'_c = 3 ksi
Condition:	No deterioration (NBI Item 59 = 7) Member is in good condition
Riding Surface:	Minor surface deviations (Field verified and documented)
ADTT (one direction):	700
$[ADTT_{SL}]_0$	200 (ADTT at year 0)
$[ADTT_{SL}]_{LIMIT}$	1,200 (roadway limit ADTT)
Skew:	0°
Additional Information:	Diaphragms spaced at 16 ft 3 in.

A1A.1.2—Section Properties

In unshored construction, the noncomposite steel stringer must support its own weight plus the weight of the concrete slab. For the composite section, the concrete is transformed into an equivalent area of steel by dividing the area of the slab by the modular ratio. Live load plus impact stresses are carried by the composite section using a modular ratio of *n*. To account for the effect of creep, superimposed dead-load stresses are carried by the composite section using a modular ratio of 3*n* (LRFD Design 6.10.1.1.1.b). The as-built section properties are used in this analysis as there is no deterioration.

A1A.1.2.1—Noncomposite Section Properties

Section properties of rolled shapes are subject to change with changes in rolling practices of the steel industry. Identify steel components from available records, construction date, and field measurements. The section properties for this beam were determined from *AISC Manual of Steel Construction*, Sixth Edition, printed during the period from July 1963 to March 1967, which is consistent with the "Year Built" date for this bridge.

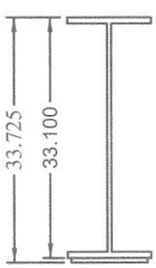
W 33 × 130

t_f = 0.855 in.
b_f = 11.51 in.
t_w = 0.58 in.
A = 38.26 in.^{2}
I = 6,699 in.^{4}

PL $^5/_8$ in. × 10 $^1/_2$ in.

t = 0.625 in.
b = 10.5 in.

$A = t \times b$ = 6.56 in.^{2}
I ~0 in.^{4} (negligible)



$$\bar{y} = \frac{\left(\frac{D_{W33\times130}}{2} + t_{PL}\right)\left(A_{W33\times130}\right) + \left(\frac{t_{PL}}{2}\right)\left(t_{PL} \times b_{PL}\right)}{A_{W33\times130} + \left(t_{PL} \times b_{PL}\right)}$$

$$\bar{y} = \frac{(17.175)(38.26) + (0.313)(6.56)}{38.26 + 6.56} \quad \text{Distance to } C.G.$$

$\bar{y} = 14.71$ in. from bottom of section to centroid

$$I_x = 6{,}699 + 38.26(2.47)^2 + 6.56(14.40)^2$$

$$I_x = 8{,}293 \text{ in.}^4$$

$$S_t = \frac{8{,}293}{19.02} = 436.0 \text{ in.}^3 \quad \text{Section Modulus at top of steel}$$

$$S_b = \frac{8{,}293}{14.71} = 563.7 \text{ in.}^3 \quad \text{Section Modulus at bottom of steel}$$

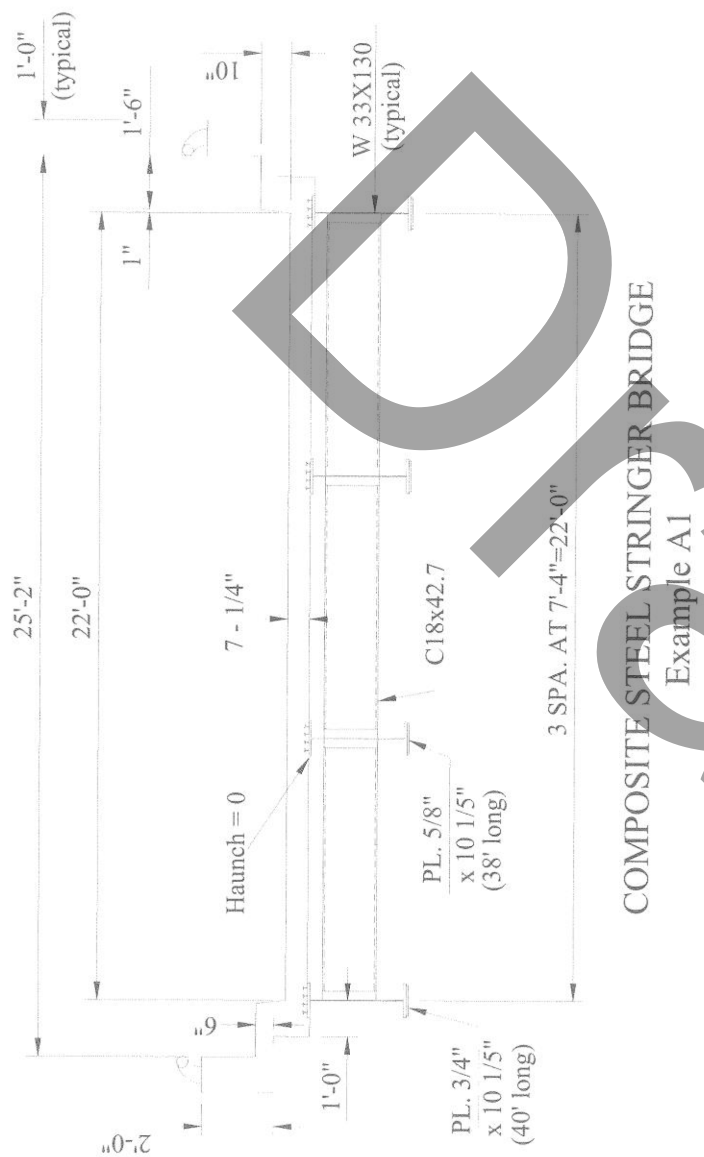


Figure A1A.1.2.1-1—Composite Steel Stringer Bridge

A1A.1.2.2—Composite Section Properties (LRFD Design 4.6.2.6.1)

Effective Flange Width, b_e, may be taken as the tributary width perpendicular to the axis of the member.

b_e = 7’–4” = 88 in.

Modular Ratio, n LRFD Design 6.10.1.1.1b

f'_c = 3 ksi

For $2.9 < f'_c < 3.6$, $n = 9$ LRFD Design C6.10.1.1.1b

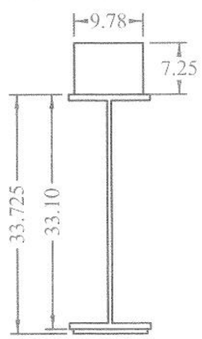
Typical Interior Stringer:

Short-Term Composite, (n):

$W33 \times 130$, $PL\ {}^5/_8$ in. × $10^1/_2$ in. and *Conc.* $7^1/_4$ in. × 88 in.

Effective Flange Width, $b_e = \dfrac{88}{n} = 9.78$ in.

Transformed Slab



$$\bar{y} = \frac{(17.175)(38.26) + (0.313)(6.56) + \left(\frac{88}{9} \times 7.25\right)(37.35)}{38.26 + 6.56 + \left(\frac{88}{9} \times 7.25\right)}$$

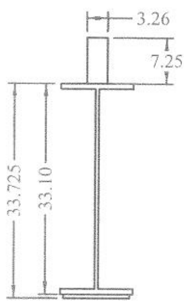
$\bar{y} = 28.58$ in. from bottom of section to centroid

$$I_x = (6,699) + (38.26)(11.40)^2 + (6.56)(28.27)^2 + \frac{\left(\frac{88}{9}\right)(7.25)^3}{12} + \left(\frac{88}{9} \times 7.25\right) \times (8.77)^2$$

$$I_x = 22,677 \text{ in.}^4$$

$$S_t = \frac{22,677}{5.14} = 4,412 \text{ in.}^3$$ Section Modulus at top of steel

$$S_b = \frac{22,677}{28.58} = 793 \text{ in.}^3$$ Section Modulus at bottom of steel



Long-Term Composite, $3n$:

$W33 \times 130$, $PL\ ^5/_8$ in. × 10 $^1/_2$ in. and *Conc.* 7 $^1/_4$ in. × 88 in.

Effective Flange Width, $b_e = \dfrac{88}{(3\times 9)} = 3.26$ in.

$$\bar{y} = \frac{(17.175)(38.26) + (0.313)(6.56) + \left(\frac{88}{27}\times 7.25\right)(37.35)}{38.26 + 6.56 + \left(\frac{88}{27}\times 7.25\right)}$$

$\bar{y} = 22.52$ in. from bottom of section to centroid

$$I_x = (6,699) + (38.26)(5.34)^2 + (6.56)(22.21)^2 + \frac{\left(\frac{88}{27}\right)(7.25)^3}{12} + \left(\frac{88\times 7.25}{27}\right)\times(14.83)^2$$

$$I_x = 16,326 \text{ in.}^4$$

$S_t = \dfrac{16,326}{11.20} = 1,458 \text{ in.}^3$ Section Modulus at top of steel

$S_b = \dfrac{16,326}{22.52} = 725 \text{ in.}^3$ Section Modulus at bottom of steel

A1A.1.2.3—Summary of Section Properties at Midspan

A1A.1.2.3a—Steel Section Only

S_{TOP} = 436 in.^{3}

S_{BOT} = 563.7 in.^{3}

A1A.1.2.3b—Composite Section—Short Term, n = 9

$S_{TOPsteel}$ = 4,412 in.^{3}

S_{BOT} = 793 in.^{3}

A1A.1.2.3c—Composite Section—Long Term, 3n = 27

$S_{TOPsteel}$ = 1,458 in.^{3}

S_{BOT} = 725 in.^{3}

A1A.1.3—Dead-Load Analysis—Interior Stringer

A1A.1.3.1—Components and Attachments, DC

In general, attachments may include connection plates, stiffeners, diaphragms, bracing, and other miscellaneous components. A refined rating calculation accounts for major weight components; alternatively, a percentage of stringer weight can be used as an estimate. For this example, three interior diaphragms were taken into account and end diaphragms that are directly over the supports were neglected when estimating uniform span loads.

A1A.1.3.1a—Noncomposite Dead Loads, DC_1

Deck: $(7.33\text{ ft})\dfrac{(7.25\text{ in.})}{12}(0.150\text{ kcf})$ = 0.664 kip/ft

Stringer: (0.130 kip/ft) (1.06) = 0.138 kip/ft
(six percent increase for connections)

Cover Plate:

$$\frac{(0.625\text{ in.})(10.5\text{ in.})\left(\dfrac{0.490\text{ kcf}}{144}\right)(1.06)(38\text{ ft})}{65\text{ ft}}$$ = 0.014 kip/ft

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

Total per stringer = 0.831 kip/ft

$$M_{DC1} = \frac{0.831(65)^2}{8} = 439\text{ kip-ft at midspan}$$

$$V_{DC1} = 0.831\left(\frac{65}{2}\right) = 27\text{ kips at bearing}$$

A1A.1.3.1b—Composite Dead Loads, DC_2

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

The unit weight of reinforced concrete is generally taken as .005 kcf greater than the unit weight of plain concrete, hence for estimating concrete loads 0.150 kcf was assumed. LRFD Design C3.5.1

Curb: $(1\text{ ft})\left(\dfrac{10\text{ in.}}{12}\right)(0.150\text{ kcf})\left(\dfrac{2\text{ curbs}}{4\text{ beams}}\right)$ = 0.062 kip/ft

Parapet:

$$\left[\left(\frac{6\text{ in.}\times 19\text{ in.}}{144}\right)+\left(\frac{18\text{ in.}\times 12\text{ in.}}{144}\right)\right](0.150\text{ kcf})\left(\frac{2\text{ parapets}}{4\text{ beams}}\right)$$ = 0.172 kip/ft

Railing: Assume $0.020\text{ kip/ft}\left(\dfrac{2\text{ railings}}{4\text{ beams}}\right)$ = 0.010 kip/ft

Total per stringer = 0.244 kip/ft

$$M_{DC2} = \frac{0.244(65)^2}{8} = 129\text{ kip-ft at midspan}$$

$$V_{DC2} = 0.244\left(\frac{65}{2}\right) = 8 \text{ kips at bearing}$$

A1A.1.3.2—Wearing Surface

$DW = 0$

A1A.1.4—Live Load Analysis—Interior Stringer (LRFD Design Table 4.6.2.2.1-1)

A1A.1.4.1—Compute Live Load Distribution Factors (Type (a) cross section)

Longitudinal Stiffness Parameter, K_g — LRFD Design 4.6.2.2.1

$$K_g = n\left(I + Ae_g^2\right)$$ LRFD Design Eq. 4.6.2.2.1-1

in which $n = \frac{E_B}{E_D}$ — LRFD Design Eq. 4.6.2.2.1-2

$$E_D = 33,000\left(w_c\right)^{1.5}\sqrt{f_c'}$$ LRFD Design Eq. C5.4.2.4-2

$$= 33,000\left(0.145\right)^{1.5}\sqrt{3}$$

$$= 3,155.9 \text{ ksi}$$

$$E_B = 29,000 \text{ ksi}$$

Beam + Cov. *PL*

I = 8,293 in.^{4}

A = 44.82 in.^{2}

e_g = $\frac{1}{2}$ (7.25) + 19.02 = 22.65 in.

$$K_g = \frac{29,000}{3,155.9}\left(8,293 + 44.82 \times 22.65^2\right)$$

K_g = 287,498 in.^{4}

A1A.1.4.1a—Distribution Factor for Moment, g_m (LRFD Design Table 4.6.2.2.2b-1)

$$\frac{K_g}{12.0Lt_s^3} = \frac{287,498}{12 \times 65 \times 7.25^3} = 0.967$$

One Lane Loaded:

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

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

$$= 0.460$$

Two or More Lanes Loaded:

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

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

$$= 0.626 > 0.460$$

$\therefore$ use $g_m = 0.626$

A1A.1.4.1b—Distribution Factor for Shear, g_v (LRFD Design 4.6.2.2.3a)

One Lane Loaded:

$$g_{v_1} = 0.36 + \frac{S}{25.0}$$ LRFD Design Table 4.6.2.2.3a-1

$$= 0.36 + \frac{7.33}{25.0}$$

$$= 0.653$$

Two or More Lanes Loaded:

$$g_{v_2} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$ LRFD Design Table 4.6.2.2.3a-1

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

$$= 0.767 > 0.653$$

$\therefore$ use $g_v = 0.767$

A1A.1.4.2—Compute Maximum Live Load Effects

A1A.1.4.2a—Maximum Design Live Load (HL-93) Moment at Midspan

The maximum moment effects are estimated to occur with the design live load centered on the span. Calculate moments by statics.

Design Lane Load Moment $= \dfrac{wl^2}{8} = \dfrac{0.640\,\text{klf}\,(65\,\text{ft})^2}{8} = 338\,\text{kip-ft}$

Design Truck Moment with the middle axle located at midspan:

Design Truck Moment $= \dfrac{P_{32}\ell}{4} + \dfrac{(P_8 + P_{32})xb}{\ell}$

$$= \frac{32^k \times 65\,\text{ft}}{4} + \frac{(8^k + 32^k)32.5\,\text{ft} \times 18.5\,\text{ft}}{65\,\text{ft}}$$

Design Truck Moment = 890 kip-ft Governs

Tandem Axles Moment with tandem axles located equidistant from midspan:

Tandem Axles Moment $= P_{25}a = 25^k \times 30.5\,\text{ft} = 762.5$ kip-ft

IM = 33 percent LRFD Design Table 3.6.2.1-1

M_{LL+IM} = 338 + 890 × 1.33

= 1,521.7 kip-ft

A1A.1.4.2b—Maximum Design Live Load Shear at Beam Ends

The maximum shear effects occur with the heaviest axle located to create the maximum end reaction. Calculate shears by statics.

Design Lane Load Shear $= \dfrac{w\ell}{2} = \dfrac{0.640\,\text{klf}\,(65\,\text{ft})}{2} = 20.8\,\text{kips}$

Design Truck Shear $= P_{32} + P_{32}\left(\dfrac{\ell - x_{32}}{\ell}\right) + P_8\left(\dfrac{\ell - x_8}{\ell}\right)$

$= 32^k + 32^k\left(\dfrac{65\,\text{ft} - 14\,\text{ft}}{65\,\text{ft}}\right) + 8^k\left(\dfrac{65\,\text{ft} - 28\,\text{ft}}{65\,\text{ft}}\right)$

Design Truck Shear = 61.7 kips Governs

Tandem Axles Shear $= P_{25} + P_{25}\left(\dfrac{\ell - x_{25}}{\ell}\right) = 25^k + 25^k\left(\dfrac{65\text{ ft} - 4\text{ ft}}{65\text{ ft}}\right) = 48.5$ kips

V_{LL+IM} = 20.8 kips + 61.7 kips × 1.33

= 102.9 kips

A1A.1.4.2c—Distributed Live Load Moments and Shears

Design Live-Load HL-93:

M_{LL+IM} = 1,521.7 × g_m

= 1,521.7 × 0.626

= 952.6 kip-ft

V_{LL+IM} = 102.9 × g_v

= 102.9 × 0.767

= 78.9 kips

A1A.1.5—Compute Nominal Resistance of Section at Midspan

Locate Plastic Neutral Axis PNA:

t_f = 0.855 in.

t_w = 0.58 in.

b_f = 11.51 in.

Cov. *PL* Area A_p

= 6.56 in.^{2}

(*PL* $^5/_8$ in. × $10^1/_2$ in.)

Web Depth:

D = 33.10 in. – 2 (0.855 in.)

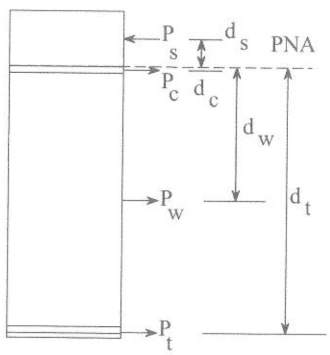
= 31.39 in.

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

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

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

= 0.724 in. (from top of tension flange to centroid of flange and cover plate)



Plastic Forces LRFD Design Appendix D6.1

Note the forces in longitudinal reinforcement may be conservatively neglected.

Set P_{rb} and P_{rt} = 0

P_s = $0.85 f'_c b_{eff} t_s$

= 0.85 × 3.0 × 88 × 7.25

= 1626.9 kips

$$\frac{c_{rb}}{t_s} = \frac{5.25}{7.25}$$

where c_{rb} is the distance from the top of the concrete slab to the center of the bottom layer of the longitudinal concrete deck reinforcement and t_s is the thickness of the concrete deck. Asssume cover + $^1/_2$ bar diameter = 2 in., then c_{rb} equals 5.25 in.

P_c = F_yA_c where $A_c = b_ft_f$

= 36 × 11.51 × 0.855

= 354.3 kips

P_w = F_yDt_w

D = 33.10 – 2 x 8.55 = 31.39

= 36 × 31.39 × 0.58

= 655.4 kips

P_t = F_yA_t *where* $A_t = b_ft_f + A_p$

= 36(11.51 × 0.855 + 6.56)

= 590.4 kips

$$P_t + P_w + P_c = 590.4 + 655.4 + 354.3 = 1,600.1 \text{ kips}$$

$$\frac{c_{rb}}{t_s} P_s + P_{rb} + P_{rt} = \frac{5.25}{7.25} 1,626.9 + 0.0 + 0.0 \text{ kips} = 1,178.1 \text{ kips}$$

$$P_c + P_w + P_t \geq \frac{c_{rb}}{t_s} P_s + P_{rb} + P_{rt}$$

$$1,600.1 \geq 1,178.1$$

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

$$\bar{Y} = (t_s)\left[\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right]$$

LRFD Design Appendix D6.1

$$\bar{Y} = (7.25)\frac{1,600.1}{1,626.9}$$

$\bar{Y}$ = 7.13 in. from the top of the concrete deck slab

A1A.1.5.1—Classify Section (LRFD Design 6.10.7 and Figure C6.4.5-1)

Following the I-Sections in Positive Flexure Flowchart
(Section is considered to be Constant Depth)

A1A.1.5.1a—Check Web Slenderness (LRFD Design 6.10.6.2.2)

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

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

A1A.1.5.1b—Check Ductility Requirement (LRFD Design 6.10.7.1.2)

$D_p = \overline{Y} = 7.13$ in.

D_t = Depth of Composite Section
$= d + t_s = 33.725 + 7.25$
$= 40.98$ in.

If $D_p \leq 0.1D_t$, then $M_n = M_p$ LRFD Design Eq. 6.10.7.1.2-1

Otherwise, $M_n = M_p\left(1.07 - 0.7\dfrac{D_p}{D_t}\right)$ LRFD Design Eq. 6.10.7.1.2-2

$0.1D_t = 0.1 \times 40.98 = 4.098$ in.
7.13 in. $\not\leq$ 4.098 in. therefore calculate $M_n < M_p$

A1A.1.5.2—Plastic Moment, M_p

Moment arms about the PNA:

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

$= \left(7.25 - 7.13\right) + \dfrac{0.855}{2}$

$= 0.55$ in.

$d_w = \left(t_s - \overline{Y}\right) + t_c + \dfrac{D}{2}$

$= \left(7.25 - 7.13\right) + 0.855 + \dfrac{31.39}{2}$

$= 16.67$ in.

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

$= \left(7.25 - 7.13\right) + 0.855 + 31.39 + 0.724$

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

$= 33.09$ in.

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

$$M_p = \left(\frac{\overline{Y}^2 P_s}{2t_s}\right) + [P_{rt}d_{rt} + P_{rb}d_{rb} + P_c d_c + P_w d_w + P_t d_t]$$ LRFD Design Table D6.1-1

$$= \left(\frac{7.13^2 \times 1,626.9}{2 \times 7.25}\right) + \left[0 + 0 + 354.3 \times 0.55 + 655.4 \times 16.67 + 590.4 \times 33.09\right]$$

Commented [REH1]: 1.This equation did not come through in print

= 36,361 kip-in. or 3,030 kip-ft

A1A.1.5.3—Nominal Flexural Resistance, M_n (LRFD Design 6.10.7.1.2)

$D_p \not\leq 0.1 D_t$ LRFD Design Eq. 6.10.7.1.2-1

Therefore, $M_n = M_p\left(1.07 - 0.7\dfrac{D_p}{D_t}\right)$ LRFD Design Eq. 6.10.7.1.2-2

Flange lateral bending stress: $f_l = 0$.

A1A.1.5.4—Nominal Shear Resistance, V_n (LRFD Design 6.10.9.2)

*W*33 × 130 Rolled section, no stiffeners.

D = $d - 2tf$ (Clear distance between flanges)
= 33.1 – 2 × 0.855
= 31.39 in.

If $\dfrac{D}{t_w} \leq 1.12\sqrt{\dfrac{Ek}{F_{yw}}}$ then $C = 1.0$

where k = 5 for unstiffened web

$$\frac{D}{t_w} = \frac{31.39}{0.580} = 54.12$$

$1.12\sqrt{\dfrac{Ek}{F_{yw}}} = 1.12\sqrt{\dfrac{29,000 \times 5}{36}} = 71.1$ LRFD Design Eq. 6.10.9.3.2-4

$54.12 \leq 71.1$, therefore $C = 1.0$

then:

$V_n = V_{cr} = CV_p$ LRFD Design Eq. 6.10.9.2-1

where $V_p = 0.58 F_{yw} D t_w$ LRFD Design Eq. 6.10.9.2-2

= 1.0 x 0.58 × 36 × 31.39 × 0.580

= 380.15 kips

A1A.1.5.5—Summary for Interior Stringer

	Dead Load DC_1	Dead Load DC_2	Live Load Distribution Factor	Dist. Live Load + Impact	Nominal Capacity
Moment, kip-ft	439.0	129.0	gm = 0.626	952.6	2,873.0
Shear, kips	27.0	8.0	gv = 78.9	78.9	380.15

A1A.1.6—General Load-Rating Equation

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DN})(DW)\pm(\gamma_{P})(P)}{(\gamma_{L})(LL+IM)}$$ Eq. 6A.4.2.1-1

A1A.1.7—Evaluation Factors (for Strength Limit States)

1. Resistance Factor, φ — LRFD Design 6.5.4.2
   $\varphi = 1.0$ for flexure and shear

2. Condition Factor, φ_c — 6A4.2.3
   $\varphi_c = 1.0$ Member is in good condition. NBI Item 59 = 7.

3. System Factor, φ_s — 6A.4.2.4
   $\varphi_s = 1.0$ 4-girder bridge, spacing > 4 ft (for flexure and shear).

A1A.1.8—Design Load Rating (6A.4.3)

A1A.1.8.1—Strength I Limit State (6A.6.4.1)

Capacity $C = (\varphi_c)(\varphi_s)(\varphi)R_n$

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

A1A.1.8.1a—Inventory Level

Load Factors — Table 6A.4.2.2-1

γ_{DC}	1.25
γ_{DW}	1.50
γ_{LL}	1.75

The dead load demands established for load cases DC_1 and DC_2 are permanent loads and therefore the load factor for these loads will be taken from the load case DC.

$$\frac{RF = (1.0)(1.0)(1.0)(2{,}873)-(1.25)(439+129)-(1.50)(0)}{(1.75)(952.6)}$$

Flexure: $$RF = \frac{(1.0)(1.0)(1.0)(2{,}873)-(1.25)(439)-(1.25)(129)}{(1.75)(952.6)}$$

$= 1.2975$

Note: The general rule for simple spans carrying moving concentrated loads states: the maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support. In a refined analysis with the HL-93 truck located in such a manner, the resulting rating factor for flexure is RF = 1.2922 for this stringer. It should be understood that locating the precise critical section and load position for rating depends on the combined influence of dead load, live load, member capacity, and load factors that make up the general rating factor equation.

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

$= 2.29$

A1A.1.8.1b—Operating Level

Load	Load Factor γ	Table 6A.4.2.2-1
DC	1.25	
LL	1.35	

For Strength I Operating Level, only the live-load factor changes; therefore, the rating factor can be calculated by direct proportions.

Flexure: $RF = 1.29 \times \dfrac{1.75}{1.35}$

$= 1.67$

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

$= 2.97$

A1A.1.8.2—Service II Limit State (6A.6.4.1)

Capacity $C = f_R$

$$RF = \frac{f_R - (\gamma_{DC})(f_{DC}) - (\gamma_{DW})(f_{DW}) \pm (\gamma_P)(f_P)}{(\gamma_{LL})(f_{LL+IM})}$$ Eq. 6A.6.4.2.1-1

For this example, the terms:

$(\gamma_{DW})(f_{DW}) \pm (\gamma_P)(f_P)$

do not contribute and the general equation reduces to:

$$RF = \frac{f_R - (\gamma_{DC})(f_{DC})}{(\gamma_{LL})(f_{LL+IM})}$$

A1A.1.8.2a—Inventory Level

Allowable Flange Stress for tension flange $f_R = 0.95 R_h F_{yf}$ ($f_\ell = 0$) LRFD Design Eq. 6.10.4.2.2-2

Checking the tension flange as compression flanges typically do not govern for composite sections.

R_h = 1.0 for non-hybrid sections LRFD Design 6.10.1.10.1

f_R = $0.95 \times 1.0 \times 36$

= 34.2 ksi

f_D = $f_{DC_1} + f_{DC_2}$

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

= 9.35 + 2.14 = 11.49 ksi

f_{LL+IM} = $\dfrac{952.6 \times 12}{793} = 14.42$ ksi

γ_{LL} = 1.30 γ_{DC} = 1.0 Table 6A.4.2.2-1

$$RF = \frac{34.2-(1.0)(11.49)}{(1.3)(14.42)}$$

$$= 1.21$$

A1A.1.8.2b—Operating Level

$\gamma_{LL} = 1.0 \qquad \gamma_{DC} = 1.0$ Table 6A.4.2.2-1

$$RF = \frac{34.2-(1.0)(11.49)}{(1.0)(14.42)}$$

$$= 1.57$$

A1A.1.8.3—Fatigue Limit State (6A.6.4.1)

Determine if the bridge has any fatigue-prone details (Category C or lower).

The transverse welds detail connecting the ends of cover plates to the flange are fatigue-prone details. Use Category E' details because the flange thicknesss = 0.855 in. is greater than 0.8 in. LRFD Design Table 6.6.1.2.3-1

If $2.2(\Delta f)_{tension} > f_{dead\text{-}load\ compression}$, the detail may be prone to fatigue. Eq. 7.2.3-1

$f_{dead\text{-}load\ compression}$ 7.2.3

= 0 at cover plate at all locations because beam is a simple span and cover plate is located in the tension zone

∴ must consider fatigue; determine if the detail possesses infinite life.

Composite section properties without cover plate:

$$\bar{y} = \frac{\sum A \times \bar{y}}{\sum A} = \frac{(38.26)(16.55)+\left(\frac{88}{9}\times 7.25\right)(36.725)}{(38.26)+\left(\frac{88}{9}\times 7.25\right)}$$

$$= 29.65 \text{ in. from bottom of flange}$$

$$I_x = 6,699+(38.26)(13.10)^2+\frac{\left(\frac{88}{9}\right)(7.25)^3}{12}+\frac{88}{9}(7.25)(7.07)^2$$

$$= 17,119 \text{ in.}^4$$

$$S_b = \frac{17,119}{29.65} = 577 \text{ in.}^3$$

Live Load at Cover Plate Cut-Off (13.5 ft. from centerline of bearing)

Fatigue Load: Design truck with a spacing of 30 ft between 32 kip axles. LRFD Design 3.6.1.4.1 and Figure 3.6.1.2.2-1

M_{LL} = (32 kips) (10.69 ft) + (32 kips) (4.46 ft) + (8 kips) (1.56 ft)

= 497 kip-ft = 5,967 kip-in. Using influence lines.

IM = 15 percent LRFD Design Table 3.6.2.1-1

M_{LL+IM} = (1.15) (5,967) = 6,862 kip-in.

A1A.1.8.3a—Load Distribution for Fatigue LRFD Design 3.6.1.4.3b

The single-lane distribution factor will be used for fatigue. LRFD Design 3.6.1.1.2

Remove multiple presence factor from the single-lane distribution. LRFD Design C3.6.1.1.2

$$g_{Fatigue} = \frac{1}{1.2}(g_{m1})$$

$$= \frac{1}{1.2}(0.46)$$

$$= 0.383$$

Distributed Live-Load Moment:

$$gM_{LL+IM} = (0.383)(6862)$$

$$= 2{,}628 \text{ kip-in.}$$

Fatigue Load Stress Range:

$$\Delta f_{LL+IM} = \frac{2{,}628}{577}$$

$= 4.56$ ksi at the cover plate weld

Nominal fatigue resistance for infinite life:

$(\Delta F)_{TH}$ = 2.6 ksi for Detail Category E′ LRFD Design Table 6.6.1.2.5-3

Infinite-Life Fatigue Check: 7.2.4

$(ADTT)_{PRESENT} = 700$
Span Length = 65 ft
Number of lanes = 2

R_p = 0.988 + 6.87x10^{-5} Span Length + 4.01x10^{-6} $(ADTT)_{PRESENT}$ + 0.0107 / Number of lanes 7.2.2.1
= 0.988 + 6.87x10^{-5}*65 + 4.01x10^{-6}*700+0.0107/2
= 1.00

$(\Delta f)_{max} = (R_p)(\Delta f_{FATIGUE\ I}) = (1.00)(1.75)(4.56)$ 7.2.4
= 7.99 ksi > 2.6 ksi
Thus, $(\Delta f)_{max} > (\Delta F)_{TH}$.

The detail does not possess infinite fatigue life.
Evaluate the estimated remaining fatigue life using procedures given in Section 7.

A1A.1.8.3b—Calculation of Estimated Remaining Fatigue Life

Fatigue life determination will be based upon the finite fatigue life.

ADTT (One Direction) = 700 (present value) LRFD Design
$[ADTT_{SL}]_{PRESENT}$ = 0.85(700) = 595 (call 600) Table 3.6.1.4.2-1

Traffic Growth Rate *g*: 1.0 percent is applied over the life of the bridge (input as 0.010)
Bridge Age *a*: 48 years

Assume Evaluation 1 Life to be used for bridge assessment.
Hence, R_R = 1.3 Table 7.2.5.1-1

Calculate effective stress range:

R_p = 1.00
R_{sa} = 1.0 Table 7.2.2.1-1

$R_{st} = 1.0$
$R_s = R_{sa} \times R_{st} = 1.0$
$\Delta f_{eff} = (R_p)(R_s)(\Delta f_{FATIGUE\ II}) = (1.00)(1.0)(0.80)(4.56) = 3.65$ ksi 7.2.2
$A = 3.9 \times 10^8$ ksi^3 LRFD Design Table 6.6.1.2.5-1

$n = 1.0$ simple span girders LRFD Design Table 6.6.1.2.5-2

Check that there is remaining fatigue life at the present age. Noting that $(ADTT_{SL})_{PRESENT} \neq (ADTT_{SL})_o$), that is, 7.2.5.1

$$N_{av} > N_1$$

$$N_{av} = \frac{R_R A}{(\Delta f_{eff})^3} = \frac{1.3(3.9\times 10^8)}{(3.65)^3} = 10426280\,cycles$$

$$N_1 = 365n\,(ADTT_{SL})_{PRESENT}\left[\frac{1-\dfrac{(ADTT_{SL})_o}{(ADTT_{SL})_{PRESENT}}}{\left(\dfrac{(ADTT_{SL})_{PRESENT}}{(ADTT_{SL})_o}\right)^{\frac{1}{a}}-1}+1\right]$$

$$N_1 = 365(1.0)(600)\left[\frac{1-\dfrac{200}{600}}{\left(\dfrac{600}{200}\right)^{\frac{1}{48}}-1}+1\right] = 6525235\,cycles < N_{av}\ ok$$

Calculate the estimated remaining fatigue life, Y_{REM}, of the fatigue-prone detail as follows:

$$Y_{REM} = \frac{\log\left[\left(\dfrac{g}{g+1}\right)\left(\dfrac{N_{av}-N_1}{365n(ADTT_{SL})_{PRESENT}}\right)+1\right]}{\log(1+g)}$$

$$= \frac{\log\left[\left(\dfrac{0.01}{1+0.01}\right)\left(\dfrac{10426280-6525235}{365*1*600}\right)+1\right]}{\log(1+0.01)} = 16.3\ years$$

Check the following:

$$(ADTT_{SL})_{FUTURE} \le (ADTT_{SL})_{LIMIT}$$

$$(ADTT_{SL})_{FUTURE} = \left[(ADDT_{SL})_{PRESENT}\right](1+g)^{Y_{REM}}$$

$$= (600)(1+0.01)^{16.3} = 706 < (ADTT_{SL})_{LIMIT} = 1,200\ ok$$

A1A.1.8.3c—Calculation of Fatigue Serviceability Index

Fatigue Serviceability Index $Q = \left(\dfrac{Y-a}{N}\right)GRI$ 7.2.6.1

No. of load paths (in this case, girders) = 4

G = 1.0 Table 7.2.6.1-1
No. of Spans = 1 (Simple Span)
R = 0.90 Table 7.2.6.1-2
N = (larger of 100 or Y) = 100
Assuming that the bridge is on an Interstate Highway, I = 0.9 Table 7.2.6.1-3

$$Q = \left(\frac{64.3 - 48}{100}\right)(1.0)(0.9)(0.9) = 0.13$$

Based on the value of the Fatigue Serviceability Index, the bridge owner will need to define the inspection frequency based upon the importance of the structure. Note, however, that the Fatigue Serviceability Index value could be increased if the owner decided to accept a greater risk of fatigue cracking and use an Evaluation 2 Life estimate instead of the Evaluation 1 Life estimate. This is illustrated below for the same example.

Assume that Evaluation 2 Life is used for the bridge fatigue assessment.
Hence, R_R = 1.6 Table 7.2.5.1-1
$(\Delta f)_{eff}$ = 3.65 ksi
$A = 3.9 \times 10^8$ ksi^3 LRFD Design Table 6.6.1.2.5-1
n = 1.0 simple span girders LRFD Design Table 6.6.1.2.5-2

$$N_{av} = \frac{R_R A}{\left[\left(\Delta f_{eff}\right)^3\right]} = \frac{1.6\left(3.9\times10^8\right)}{(3.65)^3} = 12{,}832{,}344 \textit{ cycles}$$

$$Y = \frac{\log\left[\left(\frac{0.01}{1+0.01}\right)\left(\frac{12{,}832{,}344 - 6{,}525{,}235}{365(1.0)(600)}\right)+1\right]}{\log(1+0.01)} = 25.2 \textit{ years}$$

CALCULATION OF FATIGUE SERVICEABILITY INDEX

Fatigue Serviceability Index $Q = \left(\frac{Y-a}{N}\right)GRI$ 7.2.6.1

No. of load paths (in this case, girders) = 4
G = 1.0 Table 7.2.6.1-1
No. of Spans = 1 (Simple Span)
R = 0.90 Table 7.2.6.1-2
N = (larger of 100 or Y) = 100
$Y = Y_{REM}$ = 25.2 + 48 = 73.2
Assuming that the bridge is on an Interstate Highway, I = 0.9 Table 7.2.6.1-3

$$Q = \left(\frac{73.2 - 48}{100}\right)(1.0)(0.9)(0.9) = 0.20$$

Note that the Fatigue Serviceability Index, Q, has increased from 0.13 to 0.20 as a result of accepting a greater risk of fatigue cracking at the critical detail.

A1A.1.9—Legal Load Rating 6A.6.4.2

Note: The Inventory Design Load Rating produced rating factors greater than 1.0 (with the exception of fatigue). This indicates that the bridge has adequate load capacity to carry all legal loads within LRFD exclusion limits and need not be subject to legal load ratings. Appendix A6A

The load rating computations that follow have been done for illustrative purposes. Shear ratings have not been illustrated.

Live Load: AASHTO Legal Loads—Type 3, 3S2, 3-3 (rate for all three)

g_m = 0.626

IM = 20 percent The standard dynamic load allowance of 33 percent is decreased based on a field evaluation verifying that the approach and bridge riding surfaces have only minor surface deviations or depressions. Table C6A.4.4.3-1

Table E6A-1

The following table compares interpolating to determine M_{LL} without impact for 65 ft span with exact values determined by statics. Note that for the Type 3-3, interpolating M_{LL} results in a value that is 1.5 percent greater than the true value. Judgement should be exercised whether to interpolate tabulated values.

	Type 3	Type 3S2	Type 3-3	
M_{LL} interpolated	660.7	707.2	654.5	kip-ft
M_{LL} statics	660.77	707.03	644.68	kip-ft
gM_{LL+IM}	496.3	531.2	484.3	kip-ft

Live Load: AASHTO Legal Loads—Specialized Hauling Units and Notional Rating Load—SU4, SU5, SU6, SU7, and NRL

Interpolated values shall be used for the Specialized Hauling Units in this example for illustrative purposes and to familiarize the reader with the Appendix A tables. Table E6A-2

Interpolating to determine M_{LL} without impact for 65 ft span

	SU4	SU5	SU6	SU7	NRL	
M_{LL} interpolated	744.7	821.2	913.5	994.1	1037.0	kip-ft
gM_{LL+IM}	559.4	616.9	686.2	746.8	779.0	kip-ft

6A.6.4.2.1

A1A.1.9.1—Strength I Limit State

For Types 3, 3S2, and 3-3

Dead Load DC: γ_{DC} = 1.25 Table 6A.4.2.2-1

$ADTT$ = 1,000

Generalized Live-Load Factor for Legal Loads, γ_{LL} = 1.30 Table 6A.4.4.2.3a-1

$$\text{Flexure: } RF = \frac{(1.0)(1.0)(1.0)(2873)-(1.25)(439)-(1.25)(129)}{(1.30)(M_{LL+IM})}$$

	Type 3	Type 3S2	Type 3-3
RF	3.35	3.12	3.43

For Specialized Hauling Units and NRL

Dead Load DC: γ_{DC} = 1.25 Table 6A.4.2.2-1

$ADTT$ = 1,000 Assumed

Generalized Live Load Factor for Legal Loads γ_{LL} = 1.30 Table 6A.4.4.2.3b-1

$$\text{Flexure: } RF = \frac{(1.0)(1.0)(1.0)(2{,}873)-(1.25)(439)-(1.25)(129)}{(1.30)(M_{LL+IM})}$$

	SU4	SU5	SU6	SU7	NRL
RF	2.97	2.69	2.42	2.23	2.13

A1A.1.9.2—Service II Limit State 6A.6.4.2.2

For Types 3, 3S2, and 3-3, and for Specialized Hauling Units and NRL

γ_{LL} = 1.3 γ_D = 1.0 Table 6A.4.2.2-1

f_R = 34.2 ksi

$$f_D = f_{DC_1} + f_{DC_2}$$

$$= \frac{439\times12}{563.7} + \frac{129\times12}{725} = 11.49 \text{ ksi}$$

$$f_{LL+IM} = \frac{M_{LL+IM}\times12}{793}$$

$$RF = \frac{34.2-11.49}{1.3(f_{LL+IM})}$$

	Type 3	Type 3S2	Type 3-3	
f_{LL+IM}	7.51	8.04	7.33	ksi
RF	2.33	2.17	2.38	

	SU4	SU5	SU6	SU7	NRL	
f_{LL+IM}	8.47	9.34	10.38	11.30	11.79	ksi
RF	2.06	1.87	1.68	1.55	1.48	

No posting required as *RF* > 1.0.

A1A.1.9.3—Summary

Truck	Type 3	Type 3S2	Type 3-3
Weight (tons)	25	36	40
RF (Service II Controlling)	2.33	2.17	2.38
Safe Load Capacity (tons)	58	78	95

Truck	SU4	SU5	SU6	SU7	NRL
Weight (tons)	27	31	34.8	38.8	40
RF (Service II Controlling)	2.06	1.87	1.68	1.55	1.48
Safe Load Capacity (tons)	55	58	58	60	59

The NRL rating demonstrates Article C6A.4.4.2.1b: “Bridges that rate for the NRL loading will have adequate load capacity for all legal Formula B truck configurations up to 80 kips.”

Example A1 shows this holding true NRL *RF* > 1 and all SU *RF* > 1, while Example A2 shows when NRL *RF* < 1, *RF* for the SUs may or may not be >1 and need to be checked on an individual basis.

A1A.1.10—Permit Load Rating 6A.6.4.2

Permit Type: Special (Single-Trip, Escorted)
Permit Weight: 220 kips
Permit Vehicle: Shown in Figure A1A.1.10-1
ADTT (one direction): 1,000

From Live Load Analysis by Computer Program:

Undistributed Maximum M_{LL} = 2,127.9 kip-ft

Undistributed Maximum V_{LL} = 143.5 kips

A1A.1.10.1—Strength II Limit State 6A.6.4.2.1

γ_{LL} = 1.10 (Single-Trip, Escorted) Table 6A.4.5.4.2a-1

Use one-lane distribution factor and divide out the 1.2 multiple presence factor. 6A.4.5.4.2b

$$g_{m1} = \frac{0.46}{1.2} = 0.383$$

$$g_{v1} = \frac{0.653}{1.2} = 0.544$$

IM = 20 percent (no speed control, minor surface deviations) 6A.4.5.5

Distributed Live-Load Effects:

$$M_{LL+IM} = (2,127.9)(0.383)(1.20)$$
$$= 978.0 \text{ kip-ft}$$

$$V_{LL+IM} = (143.5)(0.544)(1.20)$$
$$= 93.7 \text{ kips}$$

Flexure: $$RF = \frac{(1.0)(1.0)(1.0)(2873)-(1.25)(439)-(1.25)(129)}{(1.10)(978.0)}$$

$$= 2.01 > 1.0 \quad \text{OK}$$

Shear: $$RF = \frac{(1.0)(1.0)(1.0)(360.3)-(1.25)(27+8)}{(1.10)(93.7)}$$

$$= 3.07 > 1.0 \quad \text{OK}$$

A1A.1.10.2—Service II Limit State (Optional) 6A.6.4.2.2

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

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

γ_L = 1.0 γ_D = 1.0 Table 6A.4.2.2-1

f_R = 34.2 ksi

f_D = 11.49 ksi

Live-load effects for the Service II permit rating of vehicles that mix with traffic are calculated using the LRFD distribution analysis methods. This check is based on past practice and does not use the one-lane distribution with permit load factors that have been calibrated for the Strength II permit rating. For escorted permits, a one-lane distribution factor can be used as the permit crosses the bridge with no other vehicles allowed on the bridge at the same time. C6A.6.4.2.2

g_m = 0.383 (m = 1.2 has been divided out)

M_{LL+IM} = (2,127.9) (0.383) (1.2) = 978.0 kip-ft. = 11,736 kip-in.

$$f_{LL+IM} = \frac{M_{LL+IM}}{S_b} = \frac{11,736}{793} = 14.8 \text{ ksi}$$

$$RF = \frac{34.2 - (1.0)(11.49)}{(1.0)(14.8)} = 1.53$$

A1A.1.11—IoH Tier 1 Load Rating (Interior Beam)

Vehicle Weight: 82 kips
Gage Width GW: 8 ft
From Live Load Analysis by Computer Program:

Undistributed Maximum M_{LL} = 801.6 kip-ft (IoH Tier 1a controls)

Undistributed Maximum V_{LL} = 59.5 kips (IoH Tier 1a controls)

A1A.1.11.1—Strength I Limit State

γ_{LL} = 1.30 — MBEIoH Table 2A.4.3.2.2-1

Use one-lane distribution factor and divide out the 1.2 multiple presence factor.

$$g_{m1,HL93\ truck} = \frac{0.46}{1.2} = 0.383$$

$$g_{v1,HL93\ truck} = \frac{0.653}{1.2} = 0.544$$

$$MF_{moment} = 1\text{-}0.301\ R_1\ Ln\left(\frac{GW}{6}\right) = 1\text{-}0.301\ (0.85)\ Ln\left(\frac{8}{6}\right) = 0.926;$$ MBEIoH Table 2A.3.2.1-1

$$MF_{shear} = 1\text{-}0.509\ R_1\ Ln\left(\frac{GW}{6}\right)\left(\frac{S}{14}\right)^{0.60} = 1\text{-}0.509(0.85)\ Ln\left(\frac{8}{6}\right)\left(\frac{7.33}{14}\right)^{0.60} = 0.916$$ MBEIoH Table 2A.3.2.1-2

$g_{m1} = g_{m1,HL93\ truck}\ MF_{moment}$ = 0.383 (0.926) = 0.355

$g_{v1} = g_{v1,HL93\ truck}\ MF_{shear}$ = 0.544 (0.916) = 0.498

IM = 20 percent — MBEIoH 2A.4.3.3

Distributed Live-Load Effects:

M_{LL+IM} = (801.6) (0.355) (1.20)
= 341.5 kip-ft

V_{LL+IM} = (59.5) (0.498) (1.20)
= 35.6 kips

Flexure: $$RF = \frac{(1.0)(1.0)(1.0)(2873)-(1.25)(439)-(1.25)(129)}{(1.30)(341.5)}$$

= 4.87 > 1.0 OK

Shear: $$RF = \frac{(1.0)(1.0)(1.0)(360.3)-(1.25)(27+8)}{(1.30)(35.6)}$$

= 6.84 > 1.0 OK

A1A.1.11.2—Service II Limit State (Optional) — MBEIoH Table 2A.4.2.2-1

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

IM = 20 percent

γ_L = 1.0 γ_D = 1.0 MBEIoH Table 2A.4.2.2-1

f_R = 34.2 ksi

f_D = 11.49 ksi

g_m = 0.350 (m = 1.2 has been divided out)

M_{LL+IM} = (801.6) (0.355) (1.2) = 341.5 kip-ft. = 4,098 kip-in.

$$f_{LL+IM} = \frac{M_{LL+IM}}{S_b} = \frac{4,098}{793} = 5.2 \text{ ksi}$$

$$RF = \frac{34.2-(1.0)(11.49)}{(1.0)(5.2)} = 4.37$$

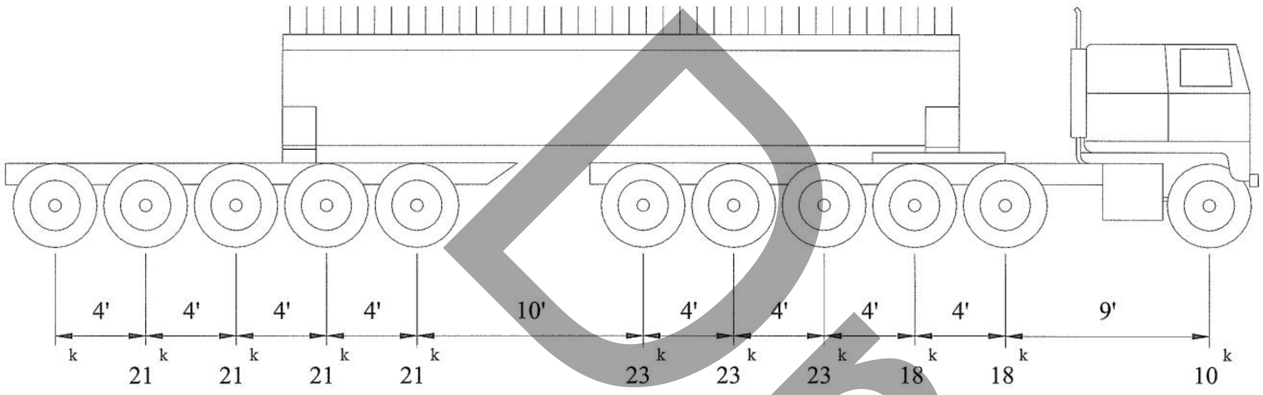


Figure A1A.1.10-1—Permit Truck Loading Configuration

A1A.2—Evaluation of an Exterior Stringer

The same given bridge data as for interior stringers applies.

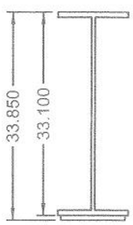
A1A.2.1—Section Properties

A1A.2.1.1—Noncomposite Section Properties

W 33 × 130 and PL $^3/_4$ in. × 10 $^1/_2$ in.

The section properties for this beam were determined from the *AISC Manual of Steel Construction*, Sixth Edition, printed during the period from July 1963 to March 1967, which is consistent with the "Year Built" date for this bridge.

W 33 × 130	PL $^3/_4$ in. × 10 $^1/_2$ in.
t_f = 0.855 in.	t = 0.750 in.
b_f = 11.51 in.	b = 10.5
t_w = 0.58 in.	
A = 38.26 in.^{2}	$A = t \times b$ = 7.875 in.^{2}
I = 6,699 in.^{4}	I ~0 in.^{4} (negligible)



$$\bar{y} = \frac{(17.30)(38.26)+(0.375)(7.875)}{38.26+7.875}$$ Distance to *C.G.*

$\bar{y} = 14.41$ in. from bottom of section to centroid

$$I_x = 6,699 + 38.26(2.89)^2 + 7.875(14.04)^2$$

$$I_x = 8,570.9 \text{ in.}^4$$

$$S_t = \frac{8,570.9}{19.44} = 440.8 \text{ in.}^3$$ Section Modulus at top of steel

$$S_b = \frac{8,570.9}{14.41} = 594.7 \text{ in.}^3$$ Section Modulus at bottom of steel

A1A.2.1.2—Composite Section Properties

Barrier is not known to be structurally continuous.

Effective Flange Width, b_e, may be taken as one-half the distance to the adjacent stringer or girder plus the full overhang width. LRFD Design 4.6.2.6.1

Effective Flange Width $b_e = \frac{1}{2}(88 \text{ in.}) + 12 \text{ in.} = 56 \text{ in.}$

Modular Ratio, n LRFD Design 6.10.1.1.1b

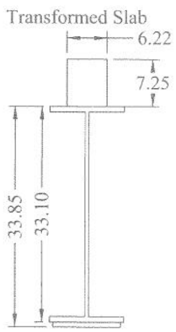
f'_c = 3 ksi

For 2.9 $< f'_c <$ 3.6, n = 9 LRFD Design C6.10.1.1.1b

Short-Term Composite, n:

W 33 × 130, PL $^3/_4$ in. × 10$^1/_2$ in. and *Conc.* 7$^1/_4$ in. × 56 in.

$$\frac{56}{9} = 6.22 \text{ in.}$$



$$\bar{y} = \frac{(17.30)(38.26)+(0.375)(7.875)+\left(\frac{56}{9}\right)(7.25)(37.475)}{38.26+7.875+\left(\frac{56}{9}\right)(7.25)}$$

$$\bar{y} = 25.81 \text{ in. from bottom of section to centroid}$$

$$I_x = 6{,}699+38.26(8.51)^2+(7.875)(25.43)^2 + \frac{\left(\frac{56}{9}\right)(7.25)^3}{12}+\left(\frac{56}{9}\right)(7.25)(11.66)^2$$

$$I_x = 20{,}893 \text{ in.}^4$$

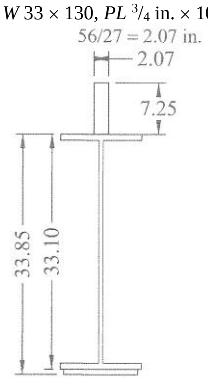
$$S_t = \frac{20{,}893}{8.04} = 2{,}599 \text{ in.}^3$$ Section Modulus at top of steel

$$S_b = \frac{20{,}893}{25.81} = 809 \text{ in.}^3$$ Section Modulus at bottom of steel

Long-Term Composite, 3*n*:

3*n* = 3 × 9 = 27

W 33 × 130, *PL* $^3/_4$ in. × $10^1/_2$ in. and *Conc.* $7^1/_4$ in. × 56 in.



$$\bar{y} = \frac{(17.30)(38.26)+(0.375)(7.875)+\left(\frac{56}{27}\times 7.25\right)(37.475)}{38.26+7.875+\left(\frac{56}{27}\times 7.25\right)}$$

$$\bar{y} = 20.08 \text{ in. from bottom of section to centroid}$$

$$I_x = 6{,}699+38.26(2.78)^2+(7.875)(19.70)^2 + \frac{\left(\frac{56}{27}\right)(7.25)^3}{12}+\left(\frac{56}{27}\right)(7.25)(17.39)^2$$

$$I_x = 14{,}664 \text{ in.}^4$$

$$S_t = \frac{14{,}664}{13.77} = 1{,}065 \text{ in.}^3$$ Section Modulus at top of steel

$$S_b = \frac{14{,}664}{20.08} = 730 \text{ in.}^3$$ Section Modulus at bottom of steel

A1A.2.1.3—Summary of Section Properties at Midspan

1. Steel Section Only

S_{TOP} = 440.8 in.^{3}

S_{BOT} = 594.7 in.^{3}

2. Composite Section—Short Term, *n* = 9

$S_{TOP\ steel}$ = 2,599 in.^{3}

S_{BOT} = 809 in.^{3}

3. Composite Section—Long Term, $3n = 27$

$S_{TOP\ steel}$ = 1,065 in.^{3}

S_{BOT} = 730 in.^{3}

A1A.2.2—Dead Load Analysis—Exterior Stringer

A1A.2.2.1—Components and Attachments, DC

A1A.2.2.1a—Noncomposite Dead Loads, DC_1

Deck: $\left(1+\frac{7.33}{2}\right)\left(\frac{7.25}{12}\right)(0.150 \text{ kip/ft})$ = 0.423 kip/ft

Stringer: (same as interior) = 0.138 kip/ft

Cover Plate: $\frac{0.75\times10.5}{144\,\text{in.}^2/\text{ft}^2}\times0.490\,\text{klf}\times1.06\times\frac{40\text{ ft}}{65\text{ ft}}$ = 0.017 kip/ft

Diaphragms: $\frac{(3)(0.0427)\left(\frac{7.33}{2}\right)(1.06)}{65\text{ ft}}$ = 0.008 kip/ft

Total per stringer = 0.586 kip/ft

$M_{DC_1} = \frac{(0.586)(65)^2}{8}$ = 309.5 kip-ft at midspan

$V_{DC_1} = (0.586)\left(\frac{65}{2}\right)$ = 19.0 kips at bearing

A1A.2.2.1b—Composite Dead Loads, DC_2 (same as interior)

M_{DC_2} = 129 kip-ft

V_{DC_2} = 8 kips

A1A.2.2.2—Wearing Surface

$DW = 0$

A1A.2.3—Live Load Analysis—Exterior Stringer

A1A.2.3.1—Compute Live Load Distribution Factors

A1A.2.3.1a—Distribution Factor for Moment, g_m (LRFD Design Table 4.6.2.2.2d-1)

One Lane Loaded:

Lever Rule

For one lane loaded, the multiple presence factor, $m = 1.20$ — LRFD Design Table 3.6.1.1.2-1

For:

$S + d_e$ = 7.33 ft + 0 ft < 8 ft one wheel acting upon the girder

$$g_{m1} = m\left(\frac{S+d-2\text{ ft}}{2S}\right)=1.2\left(\frac{7.33+0-2}{2(7.33)}\right)=0.436$$

Two or More Lanes Loaded:

$g_{m2} = eg_{interior}$ $\quad e = 0.77+\frac{d_e}{9.1}=0.77$

$g_{m2} = (0.77)\,(0.626) = 0.482 > 0.436$

A1A.2.3.1b—Distribution Factor for Shear, g_v (LRFD Design Table 4.6.2.2.3b-1)

One Lane Loaded:

Lever Rule

$g_{v1} = g_{m1} = 0.436$

Two or More Lanes Loaded:

$g = eg_{interior}$ $\quad e = 0.6+\frac{d_e}{10}=0.6$

$g_{v2} = (0.6)\,(0.767) = 0.460 > 0.436$

A1A.2.3.1c—Special Analysis for Exterior Girders with Diaphragms or Cross-Frames (LRFD Design 4.6.2.2.2d)

Roadway Layout: two 11-ft wide lanes

$$R = \frac{N_L}{N_b}+\frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2}$$

LRFD Design Eq. C4.6.2.2.2d-1

$g_{special} = (m)\,(R)$

One Lane Loaded:

$$R = \frac{1}{4}+\frac{(11)(6)}{\left[11^2+3.67^2+(-3.67)^2+(-11)^2\right]}=0.495$$

$g_{special1} = 1.2\,(0.495) = 0.595$

Two Lanes Loaded:

$$R = \frac{2}{4}+\frac{(11)\left[6+(-5)\right]}{\left[11^2+3.67^2+(-3.67)^2+(-11)^2\right]}=0.541$$

$g_{special2} = 1.0\,(0.541) = 0.541$

A1A.2.3.1d—Summary of Distribution Factors for the Exterior Girders

Moment, g_m

1 Lane	=	0.436	
2 or More Lanes	=	0.482	
Special Analysis (1 Lane)	=	0.595	Governs
Special Analysis (2 Lanes)	=	0.541	
g_m	=	0.595	

Shear, g_v

1 Lane	=	0.436	
2 or More Lanes	=	0.460	
Special Analysis (1 Lane)	=	0.595	Governs
Special Analysis (2 Lanes)	=	0.541	
g_v	=	0.595	

A1A.2.3.2—Compute Maximum Live Load Effects for HL-93

Same as for interior girder

Midspan: M_{LL+IM} = 1,521.7 kip-ft

Bearing: V_{LL+IM} = 102.9 kips

A1A.2.3.2a—Distributed Live Load Moments and Shears

Design Live Load HL-93

$$M_{LL+IM} = 1{,}521.7 \times g_m = (1{,}521.7)(0.595) = 905.4 \text{ kip-ft}$$

$$V_{LL+IM} = 102.9 \text{ kips} \times g_v = (102.9)(0.595) = 61.2 \text{ kips}$$

A1A.2.4—Compute Nominal Resistance of Section at Midspan

Locate PNA:

D = 31.39 in.

t_f = 0.855 in.

t_w = 0.58 in.

b_f = 11.51 in.

Cov. *PL* A_p = 7.875 in.^{2}

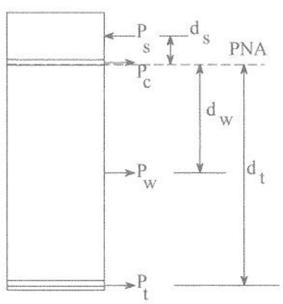
(*PL* $^3/_4$ in. × $10^1/_2$ in.)

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

$$A_t = (11.51)(0.855) + (10.5)(0.75) = 17.72 \text{ in.}^2$$

$$y = \frac{(11.51)(0.855)\dfrac{(0.855)}{2} + (10.5)(0.75)\left(0.855 + \dfrac{0.75}{2}\right)}{(11.51)(0.855) + (10.5)(0.75)}$$

= 0.784 in. (from top of tension flange to centroid of flange and cover plate)



Plastic Forces

LRFD Design Article D6.1

Note the forces in longitudinal reinforcement may be conservatively neglected.

Set P_{rb} and $P_{rt} = 0$

P_s = $0.85 f'_c b_{eff} t_s$

= 0.85 (3.0) (56) (7.25)

= 1035.3 kips

P_c = $F_y b_f t_f$

= (36) (11.51) (0.855)

= 354.3 kips

P_w = $F_y D t_w$

= (36) (31.39) (0.58)

= 655.4 kips

P_t = $F_y (b_f t_f + A_p)$

= 36 (11.51 × 0.855 + 7.875)

= 637.8 kips

$P_t + P_w < P_c + P_s + P_{rb} + P_{rt}$ ∴ Conditions for Case I are not met

$P_t + P_w + P_c \geq P_s + P_{rb} + P_{rt}$ ∴ The PNA lies in the top flange

$$\bar{Y} = \left(\frac{t_c}{2}\right)\left(\frac{P_w + P_t - P_s}{P_c} + 1\right) = \left(\frac{0.855}{2}\right)\left(\frac{655.4 + 637.8 - 1035.3}{354.3} + 1\right)$$

LRFD Design Table D6.1-1

= 0.739 in. from top of flange

A1A.2.4.1—Classify Section

Following the I-Sections in Flexure Flowchart (section is considered to be constant depth).

LRFD Design Figure C6.4.5-1

A1A.2.4.1a—Check Web Slenderness

Since PNA is in the top flange, the web slenderness requirement is automatically satisfied.

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

A1A.2.4.1b—Check Ductility (LRFD Design 6.10.7.1.2)

D_p = $t_s + \bar{Y}$ = 7.25 + 0.739

= 7.99 in.

$D_t = 33.85 + 7.25 = 41.1$ in.

If $D_p \leq 0.1 D_t$, then $M_n = M_p$

Otherwise, $M_n = M_p\left(1.07 - 0.7\dfrac{D_p}{D_t}\right)$

LRFD Design Eq. 6.10.7.1.2-1

$0.1 D_t = 0.1 \times 41.1 = 4.11$ in.

LRFD Design Eq. 6.10.7.1.2-2

7.99 in. $\not\leq$ 4.11 in. therefore calculate $M_n < M_p$

A1A.2.4.2—Plastic Moment, M_p

Moment arms about the PNA.

Slab:

$$d_s = \frac{t_s}{2} + \overline{Y}$$

$$= \frac{7.25}{2} + 0.739$$

$$= 4.36 \text{ in.}$$

Web:

$$d_w = \frac{D}{2} + t - \overline{Y}$$

$$= \frac{31.39}{2} + 0.855 - 0.739$$

$$= 15.81 \text{ in.}$$

Tension Flange:

$$d_t = t_c - \overline{Y} + D + 0.784$$

$$= 0.855 - 0.739 + 31.39 + 0.784$$

$$= 32.29 \text{ in.}$$

LRFD Design Table D6.1-1

$$M_p = \frac{P_c}{2t_c}\left[\left(\overline{Y}\right)^2 + \left(t_c - \overline{Y}\right)^2\right] + P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_t d_t$$

$$= \left\{ \begin{array}{l} \dfrac{354.3}{2(0.855)}\left[(0.739)^2 + (0.855 - 0.739)^2\right] \\ +(1035.3)(4.36) + 0 + 0 + (655.4)(15.81) + (637.8)(32.29) \end{array} \right\}$$

$$= 35586 \text{ kip-in.} = 2965 \text{kip-ft}$$

A1A.2.4.3—Nominal Flexural Resistance, M_n (LRFD Design 6.10.7.1.2)

LRFD Design Eq. 6.10.7.1.2-1

$$D_p \not\leq 0.1 D_t$$

LRFD Design Eq. 6.10.7.1.2-2

Therefore, $M_n = M_p(1.07 - 0.7\dfrac{D_p}{D_t})$

$$= 2965(1.07 - 0.7 \times 0.194)$$

$$= 2770.0 \text{ kip-ft}$$

A1A.2.4.4—Nominal Shear Resistance, V_n

Classification and Resistance same as for interior.

V_n = 360.3 kips

A1A.2.4.5—Summary for Exterior Stringer

	Dead Load DC_1	Dead Load DC_2	LiveLoad Distribution Factor	Dist. Live Load + Impact	Nominal Capacity
Moment kip-ft	309.5	129.0	*gm* = 0.595	905.4	2770.0
Shear kips	19.0	8.0	*gm* = 0.595	61.2	360.3

A1A.2.5—General Load-Rating Equation

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_{LL})(LL+IM)}$$ Eq. 6A.4.2.1-1

A1A.2.6—Evaluation Factors (for Strength Limit State)

1. Resistance Factor, φ — LRFD Design 6.5.4.2

φ = 1.0 for flexure and shear

Condition Factor, φ_c — 6A.4.2.3

Member is in good condition. NBI Item 59 = 7.

φ_c = 1.0

System Factor, φ_s — 6A.4.2.4

φ_s = 1.0 Multi-girder bridge.

A1A.2.7—Design Load Rating (6A.4.3)

A1A.2.7.1—Strength I Limit State (6A.6.4.1)

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

A1A.2.7.1a—Inventory Level

Table 6A.4.2.2-1

Load	Load Factor γ
DC	1.25
LL	1.75

Flexure:

$$RF = \frac{(1.0)(1.0)(1.0)(2770)-(1.25)(309.5+129)}{(1.75)(905.4)}$$

$$= 1.40$$

Shear:

$$RF = \frac{(1.0)(1.0)(1.0)(360.3)-(1.25)(19+8)}{(1.75)(61.2)}$$

$$= 3.05$$

A1A.2.7.1b—Operating Level

Table 6A.4.2.2-1

Load	Load Factor γ
DC	1.25
LL	1.35

For Strength I Operating Level, only the live load factor changes; therefore the rating factor can be calculated by direct proportions.

Flexure:

$$RF = 1.40\times\frac{1.75}{1.35}$$

$$= 1.81$$

Shear:

$$RF = 3.05 \times \frac{1.75}{1.35}$$

$$= 3.95$$

A1A.2.7.2—Service II Limit State (6A.6.4.1)

For Service Limit States, Capacity C = f_R

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

A1A.2.7.2a—Inventory Level

Allowable Flange Stress for tension flange:

$f_R = 0.95 R_h F_{yf}$ ($f_\ell = 0$) — LRFD Design Eq. 6.10.4.2.2-2

Checking the tension flange as a compression flange typically does not govern for composite sections.

R_h = 1.0 for non-hybrid sections — LRFD Design 6.10.1.10.1

$$f_R = 0.95 \times 1.0 \times 36$$

$$= 34.2 \text{ ksi}$$

$$f_D = f_{DC_1} + f_{DC_2}$$

$$f_D = \frac{(309.5)(12)}{594.7} + \frac{(129)(12)}{730}$$

$$= 6.24 + 2.12 = 8.36 \text{ ksi}$$

$$f_{LL+IM} = \frac{(905.4)(12)}{809} = 13.43 \text{ ksi}$$

$\gamma_{LL} = 1.30$ $\gamma_{DC} = 1.0$ — Table 6A.4.2.2-1

$$RF = \frac{34.2 - (1.0)(8.36)}{1.3(13.43)}$$

$$= 1.48$$

A1A.2.7.2b—Operating Level

$\gamma_{LL} = 1.0$ $\gamma_{DC} = 1.0$ — Table 6A.4.2.2-1

$$RF = \frac{34.2 - (1.0)(8.36)}{1.0(13.43)}$$

$$= 1.92$$

A1A.2.7.3—Fatigue Limit State

The calculations are not shown. See the calculations for interior stringers.

A1A.2.8—Legal Load Rating (6A.6.4.2)

Note: The design load check produced a rating factor greater than 1.0 for the Inventory Design Load Rating. This indicates that the bridge has adequate load capacity to carry all legal loads and need not be subject to load ratings for legal loads. The load rating

computations that follow have been done for illustrative purposes. Shear ratings have not been illustrated.

Live Load: AASHTO Legal Loads—Types 3, 3S2, and 3-3 (Rate for all three) Appendix D6A

g_m = 0.595
IM = 20 percent Table C6A.4.4.3-1

The standard dynamic load allowance of 33 percent is decreased based on a field evaluation certifying that the approach and bridge riding surfaces have only minor surface deviations or depressions.

	Type 3	Type 3S2	Type 3-3	
M_{LL}	660.7	707.2	644.7	kip-ft
gM_{LL+IM}	471.7	504.9	460.3	kip-ft

Live Load: AASHTO Legal Loads—Specialized Hauling Units and Notional Rating Load —SU4, SU5, SU6, SU7, and NRL

Interpolating to determine M_{LL} without impact for 65 ft span Table E6A-2

	SU4	SU5	SU6	SU7	NRL	
M_{LL}	744.7	821.2	913.5	994.1	1037.0	kip-ft
gM_{LL+IM}	531.7	586.3	652.2	709.8	740.4	kip-ft

A1A.2.8.1—Strength I Limit State (6A.6.4.2.1)

Dead load and capacity remain the same

For Types 3, 3S2, and 3-3

Dead Load DC: γ_{DC} = 1.25 Table 6A.4.2.2-1

$ADTT$ = 1,000

Generalized Live-Load Factor for Legal Loads:

γ_{LL} = 1.30 Table 6A.4.4.2.3a-1

Flexure:

$$RF = \frac{(1.0)(1.0)(1.0)(2,770)-(1.25)(309.5+129)}{(1.30)(M_{LL+IM})}$$

	Type 3	Type 3S2	Type 3-3
RF	3.62	3.38	3.72

For Specialized Hauling Units and NRL

Dead Load DC: γ_{DC} = 1.25 Table 6A.4.2.2-1

$ADTT$ = 1,000 Assumed

Generalized Live-Load Factor for Legal Loads γ_{LL} = 1.30 Table 6A.4.4.2.3b-1

$$\text{Flexure: } RF = \frac{(1.0)(1.0)(1.0)(2,770)-(1.25)(309.5+129)}{(1.30)(M_{LL+IM})}$$

	SU4	SU5	SU6	SU7	NRL
RF	3.06	2.70	2.50	2.29	2.21

A1A.2.8.2—Service II Limit State (6A.6.4.2.2)

For Types 3, 3S2, and 3-3, and for Specialized Hauling Units and NRL

γ_{LL} = 1.3 γ_{DC} = 1.0 Table 6A.4.2.2-1

f_R = 34.2 ksi

f_D = 8.36 ksi

$$f_{LL+IM} = \frac{M_{LL+IM} \times 12}{809}$$

$$\text{Service II: } RF = \frac{34.2-(1.0)(8.36)}{(1.3)(f_{LL+IM})}$$

	Type 3	Type 3S2	Type 3-3	
f_{LL+IM}	7.00	7.49	6.82	ksi
RF	2.84	2.65	2.91	

	SU4	SU5	SU6	SU7	NRL	
f_{LL+IM}	7.89	8.70	9.67	10.53	10.98	ksi
RF	2.52	2.29	2.05	1.89	1.81	

No posting is required as for all legal loads, *RF* > 1.0.

A1A.2.8.3—Summary (6A.4.4.4)

Safe Load Capacity (tons), $RT = RF \times W$ Eq. 6A.4.4.4-1

Truck	Type 3	Type 3S2	Type 3-3
Weight (tons)	25	36	40
RF (Service II Controlling)	2.84	2.65	2.91
Safe Load Capacity (tons)	71	95	116

Truck	SU4	SU5	SU6	SU7	NRL
Weight (tons)	27	31	34.8	38.8	40
RF (Service II Controlling)	2.52	2.29	2.05	1.89	1.81
Safe Load Capacity (tons)	68	70	71	73	72

A1A.2.9—Permit Load Rating (6A.6.4.2)

Permit Type: Special (Single-Trip, Escorted)

Permit Weight: 220 kips

Permit Vehicle: Shown in Figure A1A.1.10-1.

ADTT: 1,000

From Live-Load Analysis by Computer Program:

Undistributed Maximum:

M_{LL} = 2,127.9 kip-ft

V_{LL} = 143.5 kips

A1A.2.9.1—Strength II Limit State (6A.6.4.2.1)

Dead load and capacity remain the same as that calculated for the design load rating

γ_{LL} = 1.10 (Single Trip, Escorted) Table 6A.4.5.4.2a-1

γ_{DC} = 1.25

Use the One-Lane Loaded Distribution Factor and divide out the 1.2 multiple presence factor. 6A.4.5.4.2b

$g_{special1}$ = 0.595 (Special method for rigid torsional behavior governs.) LRFD Design 4.6.2.2.2d

$$g_{m1} = g_{v1} = \frac{g_{special1}}{1.2} = 0.496$$

Distributed Live-Load Effects:

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

M_{LL+IM} = (2,127.9) (0.496) (1.2)
= 1,266.5 kip-ft

V_{LL+IM} = (143.5) (0.496) (1.2)
= 85.4 kips

Flexure:
$$RF = \frac{(1.0)(1.0)(1.0)(2,770)-(1.25)(309.5+129)}{(1.10)(1,266.5)}$$
$$= 1.60 > 1.0 \quad \text{OK}$$

Shear:
$$RF = \frac{(1.0)(1.0)(1.0)(360.3)-(1.25)(19+8)}{(1.10)(85.4)}$$
$$= 3.48 > 1.0 \quad \text{OK}$$

A1A.2.9.2—Service II Limit State (Optional)

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

IM = 20 percent (no speed control, minor deviations)

γ_{LL} = 1.0 γ_{DC} = 1.0 Table 6A.4.2.2-1

Dead load and capacity expressed in terms of stresses remain the same as that calculated for the design load rating

f_R = 34.2 ksi

f_D = 8.36 ksi

Live-load effects for the Service II permit rating of an escorted permit are calculated using the same one-lane-loaded procedures as for the Strength II rating. C6A.6.4.2.2

g_{m1} = 0.496

M_{LL+IM} = (2,127.9) (0.496) (1.2) = 1,266.5 kip-ft

= 15,198 kip-in.

$$f_{LL+IM} = \frac{M_{LL+IM}}{S_b} = \frac{15{,}192}{809} = 18.8 \text{ ksi}$$

$$RF = \frac{34.2-(1.0)8.36}{1.0(18.8)} = 1.37 > 1.0 \qquad \text{OK}$$

A1A.2.10—IoH Tier 1 Load Rating (Exterior Beam)

Permit Weight: 82 kips

Gage Width GW = 8 ft

From Live-Load Analysis by Computer Program:

Undistributed Maximum:

M_{LL} = 801.6 kip-ft
V_{LL} = 59.5 kips

A1A.2.10.1—Strength II Limit State

Dead load and capacity remain the same as that calculated for the design load rating

γ_{LL} = 1.30 MBEIoH Table 2A.4.3.2.2-1

γ_{DC} = 1.25

Use the One-Lane Loaded Distribution Factor and divide out the 1.2 multiple presence factor. MBEIoH Table 2A.4.3.2.2-1

$g_{special1,HL93\ truck}$ = 0.595 (Special method for rigid torsional behavior governs.) LRFD Design 4.6.2.2.2d

$$g_{m1,HL93\ truck} = g_{v1,HL93truck} = \frac{g_{special1,HL93truck}}{1.2} = 0.496$$ See A1A.2.3.1c

$$MF_{moment,exterior} = 1 - 0.887 R_1 Ln\left(\frac{GW}{6}\right)\left(\frac{GW}{L}\right)^{0.870} = 1 - 0.887(0.85) Ln\left(\frac{8}{6}\right)\left(\frac{8}{65}\right)^{0.870} = 0.965$$

$$MF_{shear,exterior} = 1 - 0.640 R_1 Ln\left(\frac{GW}{6}\right)\left(\frac{S}{15}\right)^{0.50} = 1 - 0.640(0.85) Ln\left(\frac{8}{6}\right)\left(\frac{7.33}{15}\right)^{0.50} = 0.891$$

$g_{m1} = g_{m1,HL93\ truck}\, MF_{moment,exterior}$ = 0.496 (0.965) = 0.479
$g_{v1} = g_{v1,HL93\ truck}\, MF_{shear,exterior}$ = 0.496 (0.891) = 0.442

Distributed Live-Load Effects:

IM = 20 percent

M_{LL+IM} = (801.6) (0.479) (1.2)
= 460.8 kip-ft

V_{LL+IM} = (59.5) (0.442) (1.2)

= 31.6 kips

Flexure: $RF = \dfrac{(1.0)(1.0)(1.0)(2,770)-(1.25)(309.5+129)}{(1.30)(460.8)}$

= 3.71 > 1.0 OK

Shear: $RF = \dfrac{(1.0)(1.0)(1.0)(360.3)-(1.25)(19+8)}{(1.30)(31.6)}$

= 7.95 > 1.0 OK

A1A.2.10.2—Service II Limit State (Optional)

$RF = \dfrac{f_R - \gamma_{DC} f_D}{\gamma_{LL}\left(f_{LL+IM}\right)}$

IM = 20 percent

γ_{LL} = 1.0 γ_{DC} = 1.0

MBEIoH Table 2A.4.2.2-1

Dead load and capacity expressed in terms of stresses remain the same as that calculated for the design load rating

f_R = 34.2 ksi
f_D = 8.36 ksi

g_{m1} = 0.479
M_{LL+IM} = (801.6) (0.479) (1.2) = 460.8 kip-ft

= 5,529 kip-in.

$f_{LL+IM} = \dfrac{M_{LL+IM}}{S_b} = \dfrac{5,592}{809} = 6.9$ ksi

$RF = \dfrac{34.2-(1.0)8.36}{1.0(6.9)} = 3.74 > 1.0$ OK

A1A.3—Summary of Rating Factors for Load and Resistance Factor Rating Method

Table A1A.3-1—Summary of Rating Factors for Load and Resistance Factor Rating Method—Interior Stringer

Limit State		Design Load Rating		Legal Load Rating								Permit Load Rating	IoH Tier 1 Rating
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL		
Strength I	Flexure	1.29	1.67	3.35	3.12	3.44	2.97	2.70	2.43	2.23	2.13	—	4.87
	Shear	2.29	2.97	—	—	—	—	—	—	—	—	—	6.84
Strength II	Flexure	—	—	—	—	—	—	—	—	—	—	2.01	—
	Shear	—	—	—	—	—	—	—	—	—	—	3.07	—
Service II		1.21	1.57	2.33	2.17	2.38	2.06	1.87	1.68	1.55	1.48	1.53	4.37
Fatigue		0.38	—	—	—	—	—	—	—	—	—	—	—

Table A1A.3-2—Summary of Rating Factors for Load and Resistance Factor Rating Method—Exterior Stringer

Limit State		Design Load Rating		Legal Load Rating								Permit Load Rating	IoH Tier 1 Rating
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL		
Strength I	Flexure	1.40	1.81	3.62	3.39	3.71	3.21	2.92	2.62	2.41	2.31	—	3.71
	Shear	3.05	3.95	—	—	—	—	—	—	—	—	—	7.95
Strength II	Flexure	—	—	—	—	—	—	—	—	—	—	1.60	—
	Shear	—	—	—	—	—	—	—	—	—	—	3.48	—
Service II		1.48	1.92	2.84	2.65	2.91	2.52	2.29	2.05	1.89	1.81	1.37	3.74

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PART B—ALLOWABLE STRESS AND LOAD FACTOR RATING METHODS

A1B.1—EVALUATION OF AN INTERIOR STRINGER

Note: When reference is given as "AASHTO," e.g., "AASHTO 10.38.1," it refers to that Article in the AASHTO *Standard Specifications for Highway Bridges*, 17th Edition.

A1B.1.1—Bridge Data

Refer to Article A1A.1.1, Simple Span Composite Steel Stringer Bridge Data.

A1B.1.2—Section Properties

In unshored construction, the steel stringer must support its own weight plus the weight of the concrete slab. For the composite section, the concrete is transformed into an equivalent area of steel by dividing the area of the slab by the modular ratio. Live load plus impact stresses are carried by the composite section using a modular ratio of n. To account for the effect of creep, superimposed dead load stresses are carried by the composite section using a modular ratio of $3n$ (AASHTO 10.38.1). The as-built section properties are used in this analysis.

A1B.1.2.1—Noncomposite Section Properties

Section properties of rolled shapes are subject to change with changes in rolling practices of the steel industry. Identify steel components from available records, construction date, and field measurements. The section properties for this beam were determined from the *AISC Manual of Steel Construction*, Sixth Edition, printed during the period from July 1963 to March 1967, which is consistent with the "Year Built" date for this bridge.

W 33 × 130 and PL $^5/_8$ in. × $10^1/_2$ in.
$t_f = 0.855$ in.; $b_f = 11.51$ in.; $t_w = 0.58$ in.
$A = 38.26$ in.^{2}

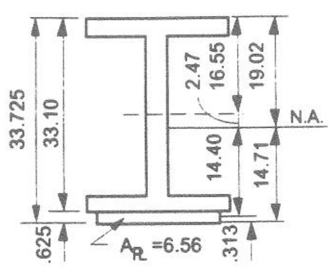


Figure A1B.1.2.1-1 Cross Section—Interior Stringer, Noncomposite

$$\bar{y} = \frac{\overset{W}{(17.175)(38.26)} + \overset{PL}{(0.313)(6.56)}}{38.26 + 6.56}$$

$$\bar{y} = 14.71 \text{ in.}$$

W *W* *PL*

$$I_x = 6,699 + 38.26(2.47)^2 + 6.56(14.40)^2$$
$$= 8,293 \text{ in.}^4$$

$$S_t = \frac{8,293}{19.02} = 436.0 \text{ in.}^3 = S_t^{DL}$$

$$S_b = \frac{8,293}{14.71} = 563.7 \text{ in.}^3 = S_b^{DL}$$

A1B.1.2.2—Composite Section Properties

Effective Flange Width AASHTO 10.38.3.1

$^1/_4$ (65)(12)	=	195 in.	
(7.33)(12)	=	88 in.	
(7.25)(12)	=	87 in.	⇐ Controls

Modular Ratio *n* 6B.5.2.4

for $f_c' = 3,000$ psi $- n = 10$

Composite *n* = *n*: *W* 33 × 130, *PL* $^5/_8$ in. × $10^1/_2$ in. and *Conc.* $7^1/_4$ in. × 87 in.

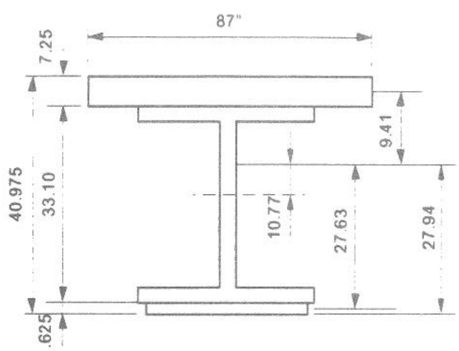


Figure A1B.1.2.2-1—Cross Section—Interior Stringer, Composite *n* = *n*

W *PL* *Conc.*

$$\bar{y} = \frac{(17.175)(38.26) + (0.313)(6.56) + (87 \times 7.25 \div 10)(37.35)}{38.26 + 6.56 + (87 \times 7.25) \div 10}$$

$$\bar{y} = 27.94 \text{ in.}$$

W *W* *PL* *Conc.* *Conc.*

$$I_x = 6,699 + (38.26)(10.77)^2 + (6.56)(27.63)^2 + \frac{(87 \div 10)(7.25)^3}{12} + (87 \times 7.25) \div 10(9.41)^2$$
$$= 22,007 \text{ in.}^4$$

Note: I_x for the bottom cover plate is negligible, however, its Ad^2 term makes a significant contribution.

$$S_t = \frac{22{,}007}{5.79} = 3{,}801 \text{ in.}^3 \quad \text{Section modulus at top of steel}$$

$$S_b = \frac{22{,}007}{27.94} = 787.7 \text{ in.}^3 = S_b^L$$

Use with Live Load.

Composite $n = 3n$: W 33 × 130, PL $^5/_8$ in. × $10^1/_2$ in. and *Conc.* $7^1/_4$ in. × 87 in.

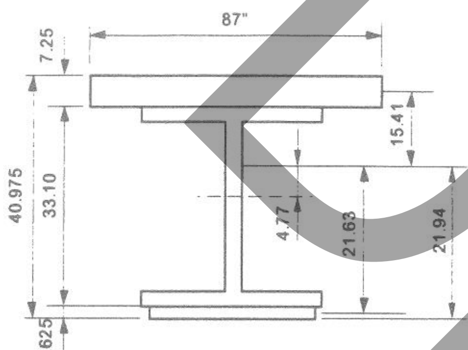


Figure A1B.1.2.2-2—Cross Section—Interior Stringer, Composite *n* = 3*n*

$$\bar{y} = \frac{\overset{W}{(17.175)(38.26)} + \overset{PL}{(0.313)(6.56)} + \overset{Conc.}{(87 \times 7.25 \div 30)(37.35)}}{38.26 + 6.56 + (87 \times 7.25) \div 30}$$

$$\bar{y} = 21.94 \text{ in.}$$

$$I_x = \overset{W}{6{,}699} + \overset{W}{(38.26)(4.77)^2} + \overset{PL}{(6.56)(21.63)^2} + \overset{Conc.}{\frac{(87 \div 30)(7.25)^3}{12} + \left(\frac{87 \times 7.25}{30}\right)(15.41)^2}$$

$$I_x = 15{,}725 \text{ in.}^4$$

$$S_t = \frac{15{,}725}{11.79} = 1{,}333.8 \text{ in.}^3 \quad \text{(Section modulus at top of steel)}$$

$$S_b = \frac{15{,}725}{21.94} = 716.7 \text{ in.}^3 = S_b^{SDL}$$

Use with Superimposed Dead Load (*SDL*).

A1B.1.3—Dead Load Analysis—Interior Stringer

A1B.1.3.1—Dead Loads (Includes an Allowance of Six Percent of Steel Weight for Connections)

Deck $(7.33)\left(\frac{7.25}{12}\right)(150\text{ pcf})$	=	664.3 lb/ft
Stringer $(130)(1.06)$	=	137.8 lb/ft
Cover *PL* $(0.625)(10.5)(490 \div 144)(1.06)(38) \div 65$	=	13.8 lb/ft
Diaphragms $(3)(42.7)(7.33)(1.06) \div 65$	=	15.4 lb/ft
Total per stringer	=	831.3 lb/ft

A1B.1.3.2—Superimposed Dead Loads (AASHTO 3.23.2.3.1.1)

Curb $(1)\left(\frac{10}{12}\right)(150\text{ pcf}) \times \left(\frac{2\text{ curbs}}{4\text{ beams}}\right)$	=	62.5 lb/ft
Parapet $\left[\left(\frac{6 \times 19}{144}\right) + \left(\frac{18 \times 12}{144}\right)\right](150\text{ pcf}) \times \left(\frac{2\text{ parapets}}{4\text{ beams}}\right)$	=	171.9 lb/ft
Railing $(\text{assume 20 plf}) \times \left(\frac{2\text{ railings}}{4\text{ beams}}\right)$	=	10.0 lb/ft
Wearing Surface	=	0.0 lb/ft
Total per stringer	=	244.4 lb/ft

A1B.1.4—Live Load Analysis—Interior Stringer

Live Load: Rate for HS-20

Moments:

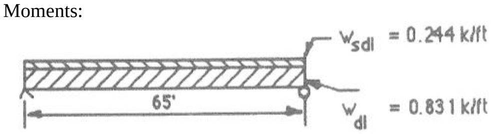


Figure A1B.1.4-1—Load Diagram—Interior Stringer, Dead Load, and Superimposed Dead Load

$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{0.831(65)^2}{8} = 439 \text{ kip-ft}$$

$$M_{SDL} = \frac{w_{SDL}L^2}{8} = \frac{0.244(65)^2}{8} = 129 \text{ kip-ft}$$

M_{LL} Appendix C6B[a]

<u>Span</u>	<u>M_{LL}</u>		
60	403.3		$M_{LL} = \frac{403.3 + 492.8}{2}$
		$\Leftarrow$65 ft	
70	492.8		$M_{LL} = 448$ kip-ft[b]
			(without Impact, without Distribution)

a Note the moments given in the MBE are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MBE value.

b Maximum M_{LL} without impact for 65 ft span, with exact values determined by statics, is 448.02 kip-ft. Nevertheless, judgment should be exercised whether to interpolate tabulated values. The general rule for simple spans carrying moving concentrated loads states that the maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support. It should be understood that locating the precise critical section and load position for rating depends on the combined influence of dead load, live load, and member capacity that make up the general Rating Factor equation.

A1B.1.5—Allowable Stress Rating (6B.3.1, 6B.4.2, and 6B.5.2)

Consider Maximum Moment Section only for this example.

A1B.1.5.1—Impact (Use Standard AASHTO) (6B.6.4, AASHTO 3.8.2.1)

$$I = \frac{50}{L+125} \le 0.3$$
$$I = \frac{50}{65+125} = 0.26$$

A1B.1.5.2—Distribution (Use Standard AASHTO) (6B.6.3, AASHTO 3.23.2.2, and Table 3.23.1)

Thus:

$$DF = \frac{S_s}{5.5} = \frac{7.33\text{ ft}}{5.5} = 1.33$$

$$M_{LL+I} = M_{LL}(1+I) \times DF = 448(1+0.26)(1.33)$$

$$M_{LL+I} = 751\text{ kip-ft}$$

A1B.1.5.3—Inventory Level (Bottom Tension Controls) (6B.5.2.1, Table 6B.5.2.1-1)

For steel with $F_y = 36\text{ ksi} \rightarrow f_I = 0.55 f_y$

Thus:

$$f_I = 0.55(36) = 20\text{ ksi}$$

The Resisting Capacity $(M_{RI}) = f_I S_x^L$

$$M_{RI} = 20\text{ ksi }(787.7\text{ in.})^3 = 15754\text{ kip-in.} = 1,313\text{ kip-ft}$$

Then:

$$RF_I = \frac{M_{RI} - M_{DL}\dfrac{S_b^L}{S_b^{DL}} - M_{SDL}\dfrac{S_b^L}{S_b^{SDL}}}{M_{LL+I}}$$
$$= \frac{1313 - 439\dfrac{787.7}{563.7} - 129\dfrac{787.7}{716.7}}{751} = \frac{557.8}{751}$$

$= \underline{0.74}$ or 0.74 × 36 tons = $\underline{26.7 \text{ tons}}$

Alternatively, in terms of stress:

$$RF_I = \frac{f_s - \dfrac{M_{DL}}{S_b^{DL}} - \dfrac{M_{SDL}}{S_b^{SDL}}}{\dfrac{M_{LL+I}}{S_b^{LL+I}}}$$

$$= \frac{20 \text{ ksi} - \dfrac{439 \text{ ft-kips} \times 12 \text{ in./ft}}{563.7 \text{ in.}^3} - \dfrac{129 \text{ ft-kips} \times 12 \text{ ft-kips}}{716.7 \text{ in.}^3}}{\dfrac{751 \text{ ft-kips} \times 12 \text{ in./ft}}{787.7 \text{ in.}^3}}$$

$$= \frac{20 - 9.345 - 2.160}{11.441}$$

$$= \frac{8.495}{11.441} = 0.74 \text{ as above}$$

A1B.1.5.4—Operating Level (6B.5.2.1, Table 6B.5.2.1-2)

For steel with $F_y = 36 \text{ ksi} \rightarrow f_O = 0.75 f_y$

Thus:

$$f_O = 0.75(36) = 27 \text{ ksi}$$

and

$$M_{RO} = 27(787.7) = 21268 \text{ kip-in.} = 1772 \text{ kip-ft}$$

and:

$$RF_O = \frac{1772 - 439\dfrac{787.7}{563.7} - 129\dfrac{787.7}{716.7}}{751} = \frac{1016.8}{751}$$

$RF_O = \underline{1.35}$ or 1.35 × 36 tons = $\underline{48.7 \text{ tons}}$

A1B.1.5.5—Summary of Ratings for Allowable Stress Rating Method

Table A1B.1.5.5-1—Summary of Ratings for Allowable Stess Rating Method—Interior Stringer

	RF	Tons
Inventory	0.74	26.7
Operating	1.35	48.7

A1B.1.6—Load Factor Rating (6B.3.2, 6B.4.3, and 6B.5.3)

Consider maximum moment section only for this example. See general notes.

A1B.1.6.1—Impact (Use Standard AASHTO) (6B.6.4)

From Allowable Stress Rating $I = 0.26$

A1B.1.6.2—Distribution (Use Standard AASHTO) (6B.6.3)

From Allowable Stress Rating $DF = 1.33$

$$M_{LL+I} = M_{LL}(1+I)DF = 448(1+0.26)(1.33)$$

$$= 751 \text{ kip-ft (as for AS Rating)}$$

A1B.1.6.3—Capacity of Section, M_R (6B.5.3.1)

For braced, compact, composite sections:

$$M_R = M_u$$ AASHTO 10.50.1.1

where M_u is found in accordance with applicable load factor provisions of AASHTO.

Check assumptions:

2. Section is fully braced along top flange by composite deck (for Live Load and *SDL*).

3. To check if section is compact, need to apply provisions of AASHTO 10.50.1.1.1. These checks follow.

The compressive force in the slab C is equal to the smallest value given by the following equations: AASHTO 10.50.1.1.1(a)

$$C = 0.85 f'_c b t_s + (AF_y)_c$$ AASHTO Eq. 10-123

Neglecting that part of the reinforcement that lies in the compressive zone the equation reduces to:

$$C_{CONC} = 0.85 f'_c b_{eff} t_s = 0.85(3 \text{ ksi})(87 \text{ in.})(7.25 \text{ in.}) = 1608 \text{ kips}^*$$

$$C = (AF_y)_{bf} + (AF_y)_{tf} + (AF_y)_w$$ AASHTO Eq. 10-124

where $(AF_y)_{bf}$ includes cover plate, this equation reduces to:

$$C_{STL} = A_s f_y = (38.26 \text{ in.}^2 + 6.56 \text{ in.}^2)(36 \text{ ksi}) = 1613.5 \text{ kips}$$

$$C_{CONC} < C_{STL} \therefore C_{CONC} = 1{,}608 \text{ controls}$$

Capacity:

$$C' = \frac{\sum(AF_y) - C}{2} = \frac{1{,}613.5 - 1{,}608}{2} = 2.75 \text{ kips}$$ AASHTO Eq. 10-126

$$(AF_y)_{TF} = (11.51 \times 0.855)(36) = 354 \text{ kips} >>> 2.75 \text{ kips} \therefore \text{ NA in top flange}$$ AASHTO 10.50.1.1.1(d)

$$\bar{y} = \frac{C'}{(AF_y)_{TF}} t_{TF} = \frac{2.75}{354}(0.855) = 0.007 \text{ in. neglect. Say NA at top of steel.}$$ AASHTO Eq. 10-127

Since the PNA is at the top of the flange, the depth of the web in compression at the plastic moment, D_{cp}, is equal to zero. Hence, the web slenderness requirement given by Eq. 10-129 in AASHTO Article 10.50.1.1.2 is automatically satisfied.

Check the ductility requirement given by Eq. 10-129a in AASHTO Article 10.50.1.1.2:

$$\left(\frac{D_p}{D'}\right) \le 5$$ AASHTO Eq. 10.129a

$$D' = \beta \frac{(d + t_s + t_h)}{7.5} \qquad \beta = 0.9 \text{ for } F_y = 36{,}000 \text{ psi}$$

$$D' = 0.9 \frac{(33.725 + 7.25 + 0.0)}{7.5} = 4.92$$

$$D_p = 7.25 \text{ in.}$$

$$\left(\frac{D_p}{D'}\right) = \frac{7.25}{4.92} = 1.47 < 5 \text{ OK}$$

Since the top flange is braced by the hardened concrete deck, local and lateral buckling requirements need not be checked. The capacity of composite beams in simple spans satisfying the preceding web slenderness and ductility requirements is given by Eq. 10-129c in AASHTO 10.50.1.1.2 when D_p exceeds D':

$D' < D_p \le 5D'$

4.92 in. < 7.25 in. ≤ 5 x 4.92 in. = 24.6 in.

Therefore:

$$C = M_R = M_U = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right)$$ AASHTO Eq. 10.129c

$$M_y = F_y S = (36)\frac{787.7}{12} = 2{,}363 \text{ kip-ft}$$

Compute the plastic moment capacity, M_p

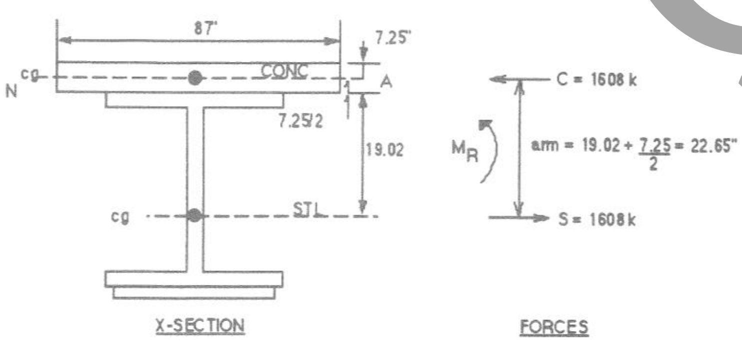


Figure A1B.1.6.3-1—Cross Section—Interior Stringer, for Determining Plastic Moment Capacity, M_p

$$M_p = C \times arm = 1{,}608(22.65) = 36{,}421 \text{ kip-in.} = 3{,}035 \text{ kip-ft}$$

$$M_R = \frac{5(3{,}035) - 0.85(2{,}363)}{4} + \frac{0.85(2{,}363) - 3{,}035}{4}(1.47) = 2{,}914 \text{ kip-ft}$$

A1B.1.6.4—Inventory Level (6B.4.1 and 6B.5.3)

$$RF_I^{LF} = \frac{M_R - A_1 M_D}{A_2 M_{L+I}}$$ Eq. 6B.4.1-1

where: 6B.4.3

$A_1 = 1.3$

$A_2 = 2.17$

Thus:

$$RF_I^{LF} = \frac{(2914) - 1.3(439 + 129)}{2.17(751)}$$

$RF_I^{LF} = \underline{\underline{1.33}}$ or 1.33 × 36 tons = $\underline{\underline{\text{47.9 tons}}}$

A1B.1.6.5—Operating Level (6B.4.3)

Only change is A_2 = 1.3

Thus:

$$RF_O^{LF} = \frac{2.17}{1.3} RF_I^{LF} = \frac{2.17}{1.3}(1.33)$$

$RF_O^{LF} = \underline{\underline{2.22}}$ or 2.22 × 36 tons = $\underline{\underline{\text{79.9 tons}}}$

A1B.1.6.6—Check Serviceability Criteria

For HS loadings overload is defined as $D + 5(L + I)/3$ AASHTO 10.57

A1B.1.6.6a—At Inventory Level (Bottom Steel in Tension Controls)

$f_{DL} + f_{SDL} + 1.67(f_{LL+I}) \leq$ Serv. Strength $= 0.95F_y$ AASHTO 10.57.2

Thus A_1 = 1.0 and A_2 = 1.67 for service rating:

$$RF_I^{LF} = \frac{0.95F_y - (1.0) f_{DL} - (1.0) f_{SDL}}{(1.67) f_{LL+I}}$$

$$= \frac{0.95(36 \text{ ksi}) - \frac{439(12)}{563.7} - \frac{129(12)}{716.7}}{1.67 \frac{751(12)}{787.7}}$$

$= RF_I^{LF} = \underline{\underline{1.19}}$ or 1.19 × 36 tons = $\underline{\underline{\text{42.8 tons}}}$

Check the web compressive stress:

$$C = F_{cr} = \frac{26,200,000\alpha k}{\left(\frac{D}{t_w}\right)^2} \le F_{yw}$$ AASHTO Eq. 10-173

where:

$k = 9\left(D \div D_c\right)^2$

$\alpha = 1.3$

Since D_c is a function of the dead-to-live load stress ratio according to the provisions of AASHTO 10.50(b), an iterative procedure may be necessary to determine the rating factor:

Compute the compressive stresses at the top of the web:

$$f_{DL} = \frac{439(12)(18.165)}{8,293} = 11.5 \text{ ksi}$$

$$f_{SDL} = \frac{129(12)(10.935)}{15,725} = 1.1 \text{ ksi}$$

$$f_{LL+I} = \frac{(751)(12)(4.935)}{22,007} = 2.02 \text{ ksi}$$

$$\sum = 14.62 \text{ ksi}$$

Compute the tensile stresses at the bottom of the web:

$$f_{DL} = \frac{439(12)(13.23)}{8,293} = 8.4 \text{ ksi}$$

$$f_{SDL} = \frac{129(12)(20.46)}{15,725} = 2.0 \text{ ksi}$$

$$f_{L+I} = \frac{(751)(12)(26.46)}{22,007} = 10.84 \text{ ksi}$$

$$\sum = 21.24 \text{ ksi}$$

$$D_c = 31.39\left(\frac{14.62}{14.62 + 21.24}\right) = 12.80 \text{ in.}$$

$$k = 9\left(D \div D_c\right)^2 = 9\left(31.39 \div 12.80\right)^2 = 54.1$$

$$C = F_{cr} = \frac{26,200,000(1.3)(54.1)}{\left(\frac{31.39}{0.58}\right)^2 (1,000)} = 629 \text{ ksi} > F_{yw}$$

$$\therefore F_{cr} = F_{yw} = 36 \text{ ksi}$$

$$RF_I^{LF} = \frac{36 - 11.5 - 1.1}{1.67(2.02)} = \underline{6.9} \text{ or } 6.9 \times 36 \text{ tons} = \underline{248.4 \text{ tons}}$$

Since the computed rating factor would cause the total stresses in the tension flange to far exceed F_y (causing the neutral axis to be higher on the web), further iterations are not necessary in this case. The web compressive stress does not govern the serviceability rating.

A1B.1.6.6b—At Operating Level

$$f_D = RF_O^{LF}\left(f_{L+I}\right) \leq \text{ Service Strength}$$

Thus A_1 = 1.0 and A_2 = 1.0 for service rating:

$$RF_O^{LF} = RF_I^{LF} \times 1.67 = 1.19 \times 1.67$$

$$RF_O^{LF} = \underline{1.98} \text{ or } 1.98 \times 36 \text{ tons } = \underline{71.3 \text{ tons}}$$

A1B.1.6.7—Summary of Ratings for Load Factor Rating Method

Table A1B.1.6.7-1—Summary of Ratings for Load Factor Rating Method—Interior Stringer

	RF	Tons	Controlled
Inventory	1.19	42.8	AASHTO 10.57.2
Operating	1.98	71.3	AASHTO 10.57.2

A1B.1.7—Load Factor Rating—Rate for Single-Unit Formula B Loads

M_{LL+I} from Appendix C6B:

Span	HS-20	NRL	SU4	SU5	SU6	SU7	
60 ft	512.2	595.1	430.2	472.5	525.0	569.9	kip-ft
70 ft	619.2	714.2	510.2	564.4	628.3	685.4	kip-ft

By interpolation:

65 ft	565.7	654.7	470.2	518.5	576.7	627.7	kip-ft

Apply distribution factor DF = 1.33

65 ft	751.0	870.8	625.4	689.6	767.0	834.8	kip-ft

Capacity of Section M_R = 2,914 kip-ft

Dead Load M_{DL} = 439 kip-ft

Superimposed Dead Loads M_{SDL} = 129 kip-ft

$$\text{Inv. } RF = \frac{2,914 - 1.3(439 + 129)}{2.17(M_{L+I})}$$

$$\text{Opr. } RF = \frac{2,914 - 1.3(439 + 129)}{1.3(M_{L+I})}$$

Strength Rating Factors:

	HS-20	NRL	SU4	SU5	SU6	SU7
Inventory	1.33	1.15	1.60	1.45	1.31	1.20
Operating	2.22	1.92	2.67	2.42	2.19	2.00

Check Serviceability Criteria:

$$RF = \frac{0.95F_y - f_{DL} - f_{SDL}}{1.67\, f_{LL+I}}$$

$$RF = \frac{34.2 - 9.35 - 2.16}{1.67\left(M_L + I \times 12 \times 1.0 / 787.7\right)}$$

Serviceability Rating Factors (Controls):

HS-20	NRL	SU4	SU5	SU6	SU7
1.19	1.03	1.43	1.29	1.16	1.07

As the Notional Rating Load NRL RF > 1.0 for strength and serviceability, the bridge has adequate capacity for all legal loads, including the single-unit Formula B trucks.

A1B.1.8—Load Factor Rating for IoH Tier 1 (6B.4.2, 6B.6.2.2, and 6B.6.3)

Consider maximum moment section only for this example.

Gage Width GW = 8 ft

A1B.1.8.1—Impact

From Allowable Stress Rating $I = 0.26$ *(Use Standard AASHTO) (6B.6.4)* > 0.20
Use 0.20 MBEIoH 2B.6.4

A1B.1.8.2—Distribution for One-lane Load

$$DF_{\text{StandardAASHTO}} = \frac{S}{7} = \frac{7.33\text{ ft}}{7} = 1.047$$ *(Use Standard AASHTO) (6B.6.3)*

Modifying factor for non-standrd gage width for interior beam moment:

MF_{moment}=1-0.301 R_1 $Ln(\frac{GW}{6})$ = 1-0.301 (0.85) $Ln\ (\frac{8\,ft}{6})$ =0.926

$DF = DF_{\text{StandardAASHTO}}\ MF_{moment}$=1.047(0.926) = 0.970

$$M_{LL+I} = M_{LL}\left(1+I\right)DF = 400.8\left(1+0.20\right)\left(0.970\right)$$

$$= 466.5\text{ kip-ft}$$

A1B.1.8.3—Capacity of Section

$M_R = 2{,}914$ kip-ft *See A1B.1.6.3*

A1B.1.8.4—Inventory Level Load Rating (6B.4.3)

$$RF_I^{LF} = \frac{M_R - A_1 M_D}{A_2 M_{L+I}}$$ MBEIoH Eq. 2B.4.1-1

Thus:

$$RF_I^{LF} = \frac{(2914) - 1.3(439 + 129)}{2.17(466.5)}$$

$$RF_I^{LF} = \underline{\underline{2.15}}$$

A1B.1.8.5—Operating Level Load Rating (6B.4.3)

Only change is A_2 = 1.3

Thus:

$$RF_O^{LF} = \frac{2.17}{1.3} RF_I^{LF} = \frac{2.17}{1.3}(2.15)$$

$$RF_O^{LF} = \underline{\underline{3.59}}$$

A1B.1.8.6—Check Serviceability Criteria

For HS loadings overload is defined as $D + 5(L + I)/3$ AASHTO 10.57

A1B.1.8.6a—At Inventory Level (Bottom Steel in Tension Controls)

$$f_{DL} + f_{SDL} + 1.67(f_{LL+I}) \le \text{ Serv. Strength } = 0.95F_y$$ AASHTO 10.57.2

Thus A_1 = 1.0 and A_2 = 1.67 for service rating:

$$RF_I^{LF} = \frac{0.95F_y - (1.0) f_{DL} - (1.0) f_{SDL}}{(1.67) f_{LL+I}}$$

$$= \frac{0.95(36 \text{ ksi}) - \frac{439(12)}{563.7} - \frac{129(12)}{716.7}}{1.67 \frac{466.5(12)}{787.7}}$$

$$= RF_I^{LF} = \underline{\underline{1.92}}$$

Check for the web compressive stress is omitted because it will not control. See A1B.1.6.6a.

A1B.1.8.6b—At Operating Level

$$f_D = RF_O^{LF}(f_{LL+I}) \le \text{ Service Strength}$$

Thus A_1 = 1.0 and A_2 = 1.0 for service rating:

$$RF_O^{LF} = RF_I^{LF} \times 1.67 = 1.92 \times 1.67$$

$$RF_O^{LF} = \underline{\underline{3.21}}$$

A1B.1.8.7—Summary of Ratings for Load Factor Rating Method for IoH Tier 1

Table A1B.1.8.7-1—Summary of Ratings for Load Factor Rating Method for IoH Tier 1—Interior Stringer

	RF	Controlled
Inventory	1.92	AASHTO 10.57.2
Operating	3.21	AASHTO 10.57.2

PART C—SUMMARY

A1C.1—Summary of All Ratings for Example A1

Table A1C.1-1—Summary of Rating Factors for All Rating Methods—Interior Stringer

			Design Load Rating (HL-93)		Legal Load Rating								Permit Load Rating	HS-20 Rating		IoH Tier 1
			Inventory	Operating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL		Inventory	Operating	
LRFR Method	Strength I	Flexure	1.29	1.67	2.64	2.46	2.71	2.76	2.50	2.25	2.07	1.98	—			4.87
		Shear	2.29	2.97	—	—	—	—	—	—	—	—	—	—	—	6.84
Limit State	Strength II	Flexure	—	—	—	—	—	—	—	—	—	—	1.92	—	—	—
		Shear	—	—	—	—	—	—	—	—	—	—	2.94	—	—	—
	Service II		1.21	1.57	2.33	2.17	2.38	2.06	1.87	1.68	1.55	1.48	1.53	—	—	4.37
	Fatigue		0.38	—	—	—	—	—	—	—	—	—	—	—	—	—
Allowable Stress Method			—	—	—	—	—	—	—	—	—	—	—	0.74	1.35	—
Load Factor Method	Strength		—	—	—	—	—	1.60	1.45	1.31	1.20	1.15	—	1.33	2.22	2.15(Inventory) 3.59(Operating)
	Serviceability		—	—	—	—	—	1.43	1.29	1.16	1.07	1.03	—	1.19	1.98	1.92(Inventory) 3.21(Operating)

Table A1C.1-2—Summary of Rating Factors for Load and Resistance Factor Rating Method—Exterior Stringer

Limit State		Design Load Rating		Legal Load Rating								*Permit Load Rating*	*IoH Tier 1*
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL		
Strength I	Flexure	1.40	1.81	2.85	2.66	2.93	2.85	2.58	2.32	2.13	2.05	—	3.71
	Shear	3.05	3.95	—	—	—	—	—	—	—	—	—	7.95
Strength II	Flexure	—	—	—	—	—	—	—	—	—	—	1.53	—
	Shear	—	—	—	—	—	—	—	—	—	—	3.33	—
Service II		1.48	1.92	2.84	2.65	2.91	2.52	2.29	2.05	1.89	1.81	1.37	3.74

A1C.2—References

AASHTO. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17. American Association of State Highway and Transportation Officials, Washington, DC, 2002.

AASHTO. *AASHTO LRFD Bridge Design Specifications*, Eighth Edition, LRFD-8. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

NCHRP. *Legal Truck Loads and AASHTO Legal Loads for Posting*. Transportation Research Board, National Research Council, Washington, DC, 2007.

Fu,G., Q.Wang, J.Chi, M.Lwin, and R.Corotis NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202?

A2—REINFORCED CONCRETE T-BEAM BRIDGE: EVALUATION OF AN INTERIOR BEAM

PART A—LOAD AND RESISTANCE FACTOR RATING METHOD

A2A.1—Bridge Data

Span:	26 ft
Year Built	1925
Materials:	
Concrete:	f'_c = 3 ksi
Reinforcing Steel:	Unknown f_y
Condition:	Minor deterioration has been observed, but no section loss. NBI Item 59 = 6
Riding Surface:	Field verified and documented: Smooth approach and deck
ADTT (one direction):	1,850
Skew:	0°

A2A.2—Dead-Load Analysis—Interior Beam

Permanent loads on the deck are distributed uniformly among the beams. LRFD Design 4.6.2.2.1

A2A.2.1—Components and Attachments, *DC*

Structural Concrete:

Consisting of deck + stem + haunches (conservatively, 2½-in. chamfers were not deducted)

$$\left[\frac{6\text{ in.}}{12}\times 6.52\text{ ft}+1.25\text{ ft}\times 2\text{ ft}+2\left(\frac{1}{2}\times\frac{6\text{ in.}}{12}\times\frac{6\text{ in.}}{12}\right)\right]\times(0.150\text{ kcf})$$

$$= 0.902\text{ kip/ft}$$

Railing and curb $0.200\text{ kip/ft}\times\frac{1}{2} = 0.100\text{ kip/ft}$

Total per beam, *DC* $= 1.002\text{ kip/ft}$

$$M_{DC} = \frac{1}{8}\times 1.002\times 26^2 = 84.7\text{ kip-ft}$$

$$V_{DCmax} = 1.002(0.5\times 26) = 13.0\text{ kips}$$

A2A.2.2—Wearing Surface, *DW*

Thickness was field measured: 6A.2.2.3

Asphalt Overlay:

$$\left(\frac{5\text{ in.}}{12}\right)(22\text{ ft})(0.144\text{ kcf})\left(\frac{1}{4}\right) = 0.330\text{ kip/ft}$$

$$M_{DW} = \frac{1}{8}\times 0.330\times 26^2 = 27.9\text{ kip-ft}$$

$$V_{DWmax} = 0.33(0.5\times 26) = 4.3\text{ kips}$$

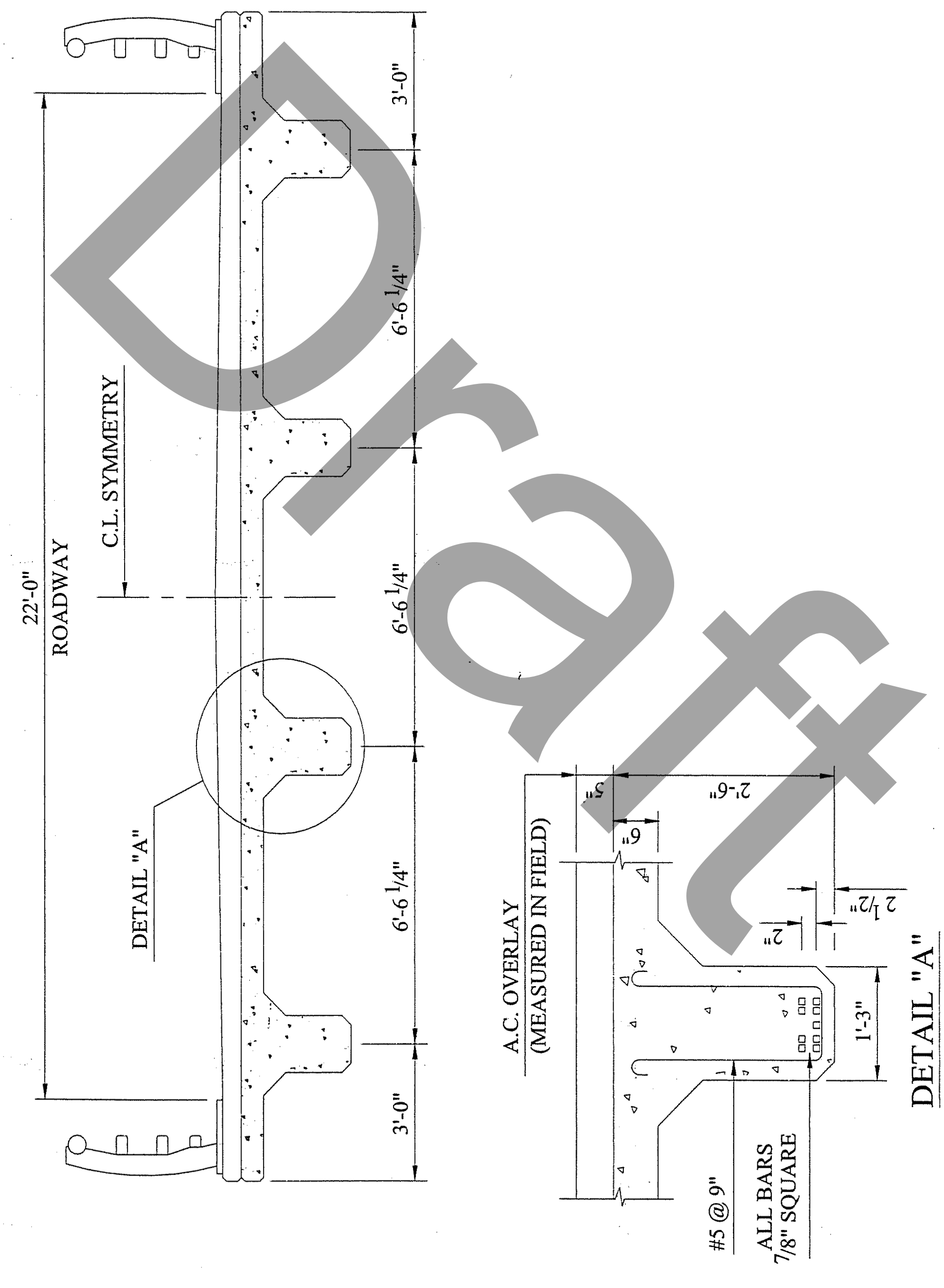


Figure A2A.2.2-1—Reinforced Concrete T-Beam Bridge

A2A.3—Live-Load Analysis—Interior Beam

A2A.3.1—Compute Live-Load Distribution Factor

AASHTO LRFD Type (e) cross section — LRFD Design Table 4.6.2.2.1-1

Longitudinal Stiffness Parameter, K_g

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

LRFD Design Eq. 4.6.2.2.1-1

$$n = 1.0$$

$$I = \frac{1}{12} \times 15 \times 24^3 = 17{,}280 \text{ in.}^4$$

$$A = 15 \times 24 = 360 \text{ in.}^2$$

$$e_g = \frac{1}{2}(24 + 6) = 15 \text{ in.}$$

$$K_g = 1.0\,(17{,}280 + 360 \times 15^2) = 98{,}280 \text{ in.}^4$$

$$\frac{K_g}{12Lt_s^3} = \frac{98{,}280}{12 \times 26 \times 6^3} = 1.46$$

A2A.3.1.1—Distribution Factor for Moment, g_m (LRFD Design Table 4.6.2.2.2b-1)

One Lane Loaded:

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

$$= 0.06 + \left(\frac{6.52}{14}\right)^{0.4}\left(\frac{6.52}{26}\right)^{0.3}(1.46)^{0.1}$$

$$= 0.565$$

Two or More Lanes Loaded:

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

$$= 0.075 + \left(\frac{6.52}{9.5}\right)^{0.6}\left(\frac{6.52}{26}\right)^{0.2}(1.46)^{0.1}$$

$$= 0.703 > 0.565$$

$$\therefore \text{ use } g_m = 0.703$$

A2A.3.1.2—Distribution Factor for Shear, g_v (LRFD Design Table 4.6.2.2.3a-1)

One Lane Loaded:

$$g_{v1} = 0.36 + \frac{S}{25.0}$$

$$= 0.36 + \frac{6.52}{25.0}$$

$$= 0.621$$

Two or More Lanes Loaded:

$$g_{v2} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

$$= 0.2 + \frac{6.52}{12} - \left(\frac{6.52}{35}\right)^{2.0}$$

$$= 0.709 > 0.62$$

$\therefore$ use g_v = 0.709

A2A.3.2—Compute Maximum Live Load Effects

A2A.3.2.1—Maximum Design Live Load (HL-93) Moment at Midspan

Design Lane Load Moment = 54.1 kip-ft

Design Truck Moment = 208.0 kip-ft

Tandem Axles Moment = 275.0 kip-ft Governs

IM = 33 percent (LRFD Design Table 3.6.2.1-1)

M_{LL+IM} = 54.1 + 275.0 × 1.33

= 419.9 kip-ft

A2A.3.2.2—Maximum Design Live Load Shear (HL-93) at Critical Section

See Article A2A.7.

A2A.3.2.3—Distributed Live Load Moments

Design Live Load HL-93:

M_{LL+IM} = 419.9 × 0.703

= 295.2 kip-ft

A2A.4—Compute Nominal Flexural Resistance

A2A.4.1—Compute Effective Flange Width, b_e (LRFD Design 4.6.2.6.1)

Effective Flange Width, b_e, may be taken as the tributary width perpendicular to the axis of the member.

$\therefore$ use b_e = 78.25 in.

A2A.4.2—Compute Distance to Neutral Axis, *c* (LRFD Design 5.6.3.1.1)

Assume rectangular section behavior.

β_1 = 0.85 for f'_c = 3,000 psi (LRFD Design 5.6.2.2)

$$c = \frac{A_s f_y}{0.85 f_c' \beta_1 b}$$ LRFD Design Eq. 5.6.3.1.1-4

$$A_s = 9\left(\frac{7}{8}\right)^2 = 6.89 \text{ in.}^2 \qquad \text{(nine } ^7/_8\text{-in.}^2 \text{ bars)}$$

b = 78.25 in.

f_y = 33 ksi (unknown steel) Table 6A.5.2.2-1

$$c = \frac{6.89 \times 33}{0.85 \times 3.0 \times 0.85 \times 78.25}$$

= 1.34 in. < 6 in.

The neutral axis is within the slab. Therefore, there will be rectangular section behavior.

$a = c\beta_1$

= 1.34 × 0.85

= 1.14 in.

Distance from bottom of section to *CG* of reinforcement, $\bar{y}$

$$\bar{y} = \frac{4 \times 4.5 + 5 \times 2.5}{9}$$

$\bar{y}$ = 3.39 in.

$d_s = h - \bar{y}$

h = 30 in.

d_s = 30 in. – 3.39 in.

= 26.61 in.

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right)$$ LRFD Design 5.6.3.2.3, LRFD Design Eq. 5.6.3.2.2-1

$$= 6.89 \times 33 \left(26.61 - \frac{1.14}{2}\right)\frac{1}{12}$$

= 493.4 kip-ft

A2A.5—Maximum Reinforcement (6A.5.5)

The factored resistance (φ factor) of compression controlled sections shall be reduced in accordance with LRFD Design Article 5.5.4.2. This approach limits the capacity of over-reinforced (compression controlled) sections. C6A.5.5

The net tensile strain, ε_t, is the tensile strain at nominal strength and determined by strain compatibility using similar triangles. LRFD Design C5.6.2.1

Given an allowable concrete strain of 0.003 and depth to neutral axis c = 1.34 in.

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

$$\frac{0.003}{1.34 \text{ in.}} = \frac{\varepsilon_t}{26.61 \text{ in.} - 1.34 \text{ in.}}$$

Solving for ε_t, ε_t = 0.0566.

For ε_t = 0.0566 > 0.005, the section is tension controlled. LRFD Design 5.6.2.1

For conventional construction and tension controlled reinforced concrete sections, resistance factor φ shall be taken as 0.90. LRFD Design 5.5.4.2

A2A.6—Minimum Reinforcement (6A.5.6)

The amount of reinforcement must be sufficient to develop M_r equal to the lesser of: LRFD Design 5.6.3.3

$1.2M_{cr}$ or $1.33M_u$

$$M_r = \varphi_f M_n = 0.90 \times 493.4 \text{ kip-ft}$$

$$= 444.1 \text{ kip-ft}$$

1. $1.33M_u = 1.33\,(1.75 \times 295.2 + 1.25 \times 84.7 + 1.25 \times 27.9)$

   $= 874.3 \text{ kip-ft} > 444.1 \text{ kip-ft}$ No Good

2. $$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$$ LRFD Design Eq. 5.6.3.3-1

M_{dnc} = 0 Total unfactored dead load moment acting on the monolithic or noncomposite section

f_{cpe} = 0 Compressive stress in concrete due to effective prestress forces only at extreme fiber of section where tensile stress is caused by externally applied loads

S_{nc} = $\dfrac{I}{y_t}$ Uncracked section modulus (neglect steel)

γ_1 = flexural cracking variability factor = 1.6

γ_2 = prestress variability factor = 0

γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of nonprestressed reinforcement = 0.67

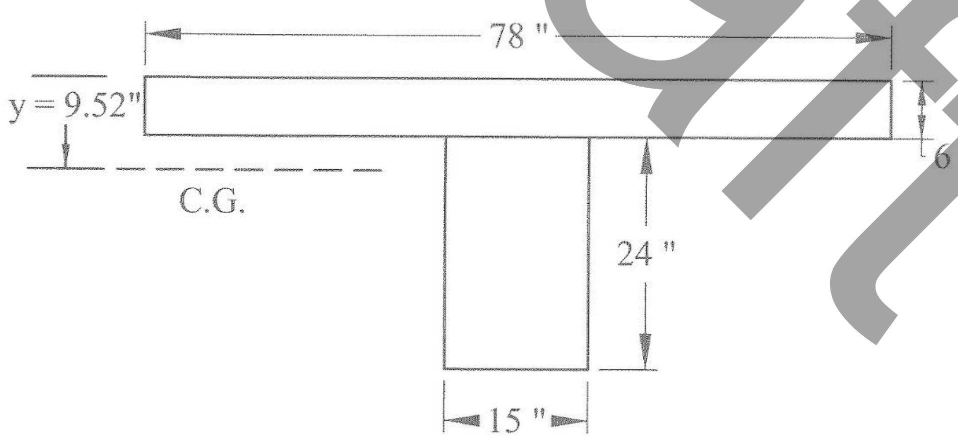


Figure A2A.6-1 Cross Section of Concete T-Beam—Depth to Centroid of Uncracked Section

$$y = \frac{\sum(A_i \times y_i)}{\sum A_i}$$

$$y = \frac{(78\times6\times3)+(24\times15\times18)}{(78\times6)+(24\times15)} = 9.52 \text{ in.}$$

from top of slab to centroid of uncracked section

$I = \sum\left(I_o + A_c d^2\right)$ where $I_0 = bh^3/12$

	y	A_c	$A_c y$	d	Ad^2	I_0
slab	3	468	1,404	6.52	19,895	1,404
stem	18	360	6,480	8.48	25,888	17,280
		828	7,884		45,783	18,684

$I = (18,684 + 45,783) = 64,467$

y_b = 30 in. – 9.52 in. = 20.48 in.

$S_{bc} = \dfrac{64,467}{20.48} = 3,148 \text{ in.}^3$

$f_r = 0.24\sqrt{f_c'} = 0.24\sqrt{3.0} = 0.416$ ksi

$M_{cr} = (0.67)\left[1.6(0.416 + 0)3,148 - 0\right] = 116.99$ kip-ft LRFD Design 5.4.2.6

$M_r = \varphi M_n = 0.9\ (493.4)$
$M_r = 444.1$ kip-ft $> M_{cr} = 116.99$ kip-ft OK

The section meets the requirements for minimum reinforcement.

A2A.7—Compute Nominal Shear Resistance

Stirrups: #5 bars at 9 in.

$$A_v = 2 \times \frac{\pi}{4}\left(\frac{5}{8}\right)^2 = 0.6136 \text{ in.}^2$$

Unknown $f_y \rightarrow 33$ ksi

Critical section for shear: LRFD Design 5.7.3.2
Effective Shear Depth: d_v LRFD Design 5.7.2.8

Distance, meassured perpendicular to the neutral axis, between resultants of the tensile and compressive forces. It need not be taken to be less than the greater of:

$0.9d_e$

$0.72h$

1. $d_v = \dfrac{M_n}{A_s f_y + A_{ps} f_{ps}}$ LRFD Design Eq. C5.7.2.8-1

This quantity depends upon the transfer and development of the reinforcement. Conservatively, we will take d_v as the greater of the remaining criteria to reduce required calculations.

2. 0.9 (26.61) = 23.95 in.
3. 0.72 (30.0) = 21.60 in.

d_v = 23.95 in.

Assume θ = 45°

$0.5d_v \cot \theta \quad = \quad (0.5)(26.04)(\cot 45) = 0.5d_v < d_v \qquad$ Use d_v

Critical section for shear at 23.95 in. from face of support.

Bearing pad width = 4 in.

Calculate shear at $23.95 + \frac{4}{2} = 25.95$ in. from centerline of bearing.

Maximum Shear at Critical Section Near Support (25.95 in.) calculated by statics:

$V_{TANDEM} = 41.9$ kips Governs
$V_{TRUCK} = 41.4$ kips
$V_{LANE} = 7.0$ kips

Total Live-Load Shear = (1.33) (41.9) + 7.0 = 62.7 kips LRFD Design Table 3.6.2.1-1

(including 33 percent increase for dynamic load allowance)

Distributed Shear, V_{LL+IM} = (62.7) (0.709) = 44.5 kips

Dead-Load Shears:

$$V_{DC} = 1.002\left(0.5 \times 26 - \frac{25.95}{12}\right) = 10.8 \text{ kips}$$

$$V_{DW} = 0.33\left(0.5 \times 26 - \frac{25.95}{12}\right) = 3.6 \text{ kips}$$

Resistance:

The lesser of :

$V_n = V_c + V_s + V_p$ LRFD Design Eq. 5.7.3.3-1

$V_n = 0.25f'_c b_v d_v + V_p$ LRFD Design Eq. 5.7.3.3-2

In this case there is no V_p contribution, and:

Effective shear depth, $d_v = 23.95$ in. LRFD Design Eq. 5.7.2.8-1

Minimum web width within the depth d_v, $b_v = 15$ in. LRFD Design Eq. 5.7.2.8-1

$V_c = 0.0316\beta\lambda\sqrt{f'_c}b_v d_v$ LRFD Design Eq. 5.7.3.3-3

$V_s = \frac{A_v f_y d_v \cot\theta}{s}$ (for $\alpha = 90°$) LRFD Design Eq. 5.7.3.3-4

Simplified Approach: LRFD Design 5.7.3.4.1

$\beta = 2.0$
$\theta = 45$

$$V_c = (0.0316)(2)\sqrt{3.0}(15)(23.95) = 39.3 \text{ kips}$$

V_s = $\dfrac{(0.6136)(33)(23.95)\cot 45}{9}$ = 53.9 kips

V_n = 39.3 + 53.9 = 93.2 kips

V_n = 0.25 × 3.0 × 15 × 23.95 = 269.4 kips

93.2 kips < 269.4 kips, therefore V_n = 93.2 kips

A2A.8—Summary for Interior Concrete T-Beam

	Dead Load *DC*	Dead Load *DW*	Live Load Distribution Factor	Dist. Live Load + Impact	Nominal Capacity
Moment, kip-ft	84.7	27.9	g_m = 0.703	295.2	493.4
Shear, kips	10.8	3.6	g_v = 0.709	44.5	93.2

A2A.9—General Load Rating Equation

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)} \text{ s}$$ Eq. 6A.4.2.1-1

For Strength Limit States $C = (\varphi_c)(\varphi_s)(\varphi)R_n$

A2A.10—Evaluation Factors (for Strength Limit States)

1. Resistance Factor, φ LRFD Design 5.5.4.2

φ = ~~1.0~~ 0.90 for flexure and shear of normal weight concrete

2. Condition Factor, φ*c* 6A.4.2.3
No member condition information available. NBI Item 59 = 6.

φ_c = 1.0

3. System Factor, φ_s 6A.4.2.4

φ_s = 1.0 4-girder bridge with S > 4 ft (for flexure and shear)

A2A.11—Design Load Rating (6A.4.3)

A2A.11.1—Strength I Limit State

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

A2A.11.2—Inventory Level (6A.5.4.1)

Table 6A.4.2.2-1

Load	Load Factor
DC	1.25
DW	1.25
LL	1.75

Thickness was field verified

Flexure:

$$RF = \frac{(1.0)(1.0)(0.90)(493.4) - \left[(1.25)(84.7) + (1.25)(27.9)\right]}{(1.75)(295.2)}$$

$$= 0.59$$

Shear:

$$RF = \frac{(1.0)(1.0)(0.90)(93.2) - \left[(1.25)(10.8) + (1.25)(3.6)\right]}{(1.75)(44.5)}$$

$$= 0.85$$

The shear ratings factors for Design Load Rating are calculated for illustration purposes only. In-service concrete bridges that show no visible signs of shear distress need not be checked for shear during design load or legal load ratings. 6A.5.8

A2A.11.3—Operating Level

Table 6A.4.2.2-1

Load	Load Factor γ
DC	1.25
DW	1.25
LL	1.35

For Strength I Operating Level only the live load factor changes; therefore the rating factor can be calculated by direct proportions.

Flexure:

$$RF = 0.59 \times \frac{1.75}{1.35}$$

$$= 0.76$$

Shear:

$$RF = 0.85 \times \frac{1.75}{1.35}$$

$$= 1.10$$

Note: The shear resistance using MCFT varies along the length. The simplified assumptions of $\beta = 2.0$ and $\theta = 45°$ in this example are conservative for high shear–low moment regions. Example A3 demonstrates a case where the shear rating must be performed at multiple locations along the length of the member. Tension in the longitudinal reinforcement caused by moment-shear interaction (LRFD Design Article 5.7.3.5) has not been checked in this example. Example A3 includes demonstrations of this check.

No service limit states apply to reinforced concrete bridge members at the design load check.

A2A.12—Legal Load Rating (6A.5.4.2)

Note: Since the Operating Level Design Load Rating produced *RF* < 1.0 for flexure, load ratings for legal loads should be performed to determine the need for posting.

Live Load: AASHTO Legal Loads—Types 3, 3S2, and 3-3 (Rate for all three) 6A.4.4.2.1

g_m = 0.703

L = 26 ft (L < 40 ft)

IM = 33 percent

Even though the condition of the wearing surface has been field evaluated as smooth, the length of the flexure members prevents the use of a reduced IM. C6A.4.4.3

Table E6A-1

	Type 3	Type 3S2	Type 3-3	
M_{LL+IM} =	250.6	240.7	206.2	kip-ft
gM_{LL+IM} =	176.2	169.2	145.0	kip-ft

Live Load: AASHTO Legal Loads—Specialized Hauling Units and Notional Rating Load—SU4, SU5, SU6, SU7, and NRL 6A.4.4.2.1b

As before:

g_m = 0.703

L = 26 ft (L <40 ft)

IM = 33 percent C6A.4.4.3

Table E6A-2

	SU4	SU5	SU6	SU7	NRL	
M_{LL+IM} =	296.9	323.2	350.1	358.6	360.4	kip-ft
gM_{LL+IM} =	208.7	227.2	246.1	252.1	253.4	kip-ft

A2A.12.1—Strength I Limit State (6A.5.4.2.1)

$ADTT$ = 1850

For AASHTO Legal Loads—Types 3, 3S2, and 3-3

Generalized Live-Load Factor:

Linear interpolation is permitted for other $ADTT$. Therefore: Table 6A.4.4.2.3a-1

$$\gamma_L = 1.30 + \frac{1{,}850 - 1{,}000}{5{,}000 - 1{,}000}(1.45 - 1.30) = 1.33$$

γ_L = 1.33

Flexure:

$$RF = \frac{(1.0)(1.0)(0.90)(493.4) - \left[(1.25)(84.7) + (1.25)(27.9)\right]}{(1.33)(M_{LL+IM})}$$

Truck	Type 3	Type 3S2	Type 3-3
RF =	1.29	1.35	1.57
Vehicle Weight (tons)	25	36	40
Safe Load Capacity (tons)	32	48	62

For Specialized Hauling Units and Notional Rating Load—SU4, SU5, SU6, SU7, and NRL

Generalized Live-Load Factor:

γ_L = 1.33 by interpolation — Table 6A.4.4.2.3b-1

Flexure:

$$RF = \frac{(1.0)(1.0)(0.90)(493.4)-\left[(1.25)(84.7)+(1.25)(27.9)\right]}{(1.33)(M_{LL+IM})}$$

Truck	SU4	SU5	SU6	SU7	NRL
RF	1.09	1.00	0.95	0.90	0.90
Vehicle Weight, tons	27	31	34.8	38.8	40
Safe Load Capacity, tons	29	31	32	34	36

No posting is required for the Types 3, 3S2, and 3-3.

Comparison of the above safe capacities for the SU4, SU5, SU6, and SU7 to the NRL Safe Load Capacity demonstrates that for bridges that do not rate the NRL Load, a posting analysis should be performed to resolve posting requirements for single unit multiaxle trucks. The above results show that the Safe Load Capacity for the SU4 and SU5 vehicle is adequate; however, posting may be required for SU6 and SU7 vehicles. — 6A.8.2 and C6A.8.2

The descision to post a bridge should be made by the Bridge Owner. When for any legal truck the Rating Factor *RF* is between 0.3 and 1.0 then the following formula should be used to establish the safe posting load for that vehicle type. — 6A.8.3

$$\text{Safe Posting Load} = \frac{W}{0.7}\left[(RF)-0.3\right]$$ Eq. 6A.8.3-1

Therefore, for SU6 and SU7, the recommended safe posting loads are:

	SU6	SU7
Safe Posting Load	31	33

No service limit states apply to reinforced concrete bridge members at the legal load check.
This example focused on the interior stringer for illustrative purposes only. Before a final posting descision can be made the exterior beam should be analyzed.

A2A.12.2—Summary

Truck	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL
Weight, tons	25	36	40	27	31	34.8	38.8	40
RF	1.29	1.35	1.57	1.09	1.00	0.98	0.90	0.90
Safe Load Capacity, tons	32	48	62	29	31	32	34	36
Safe Posting Load (tons)	—	—	—	—	—	31	33	

A2A.13—Permit Load Rating (6A.4.5)

Permit Type: Special, Multiple-Trips, no speed control

Permit Weight: 175 kips

Permit Vehicle: Shown in Figure A2A.13-1.

ADTT (one direction): 1,850

IM = 33 percent (L < 40 ft) C6A.4.4.3

Undistributed Maximum:
M_{LL} = 347.3 kip-ft at midspan
V_{LL} = 52.6 kips at 26 in.

A2A.13.1—Strength II Limit State (6A.5.4.2.1)

ADTT (one direction): 1850
Load Factor, γ_L: 1.40 Table 6A.4.5.4.2a-1

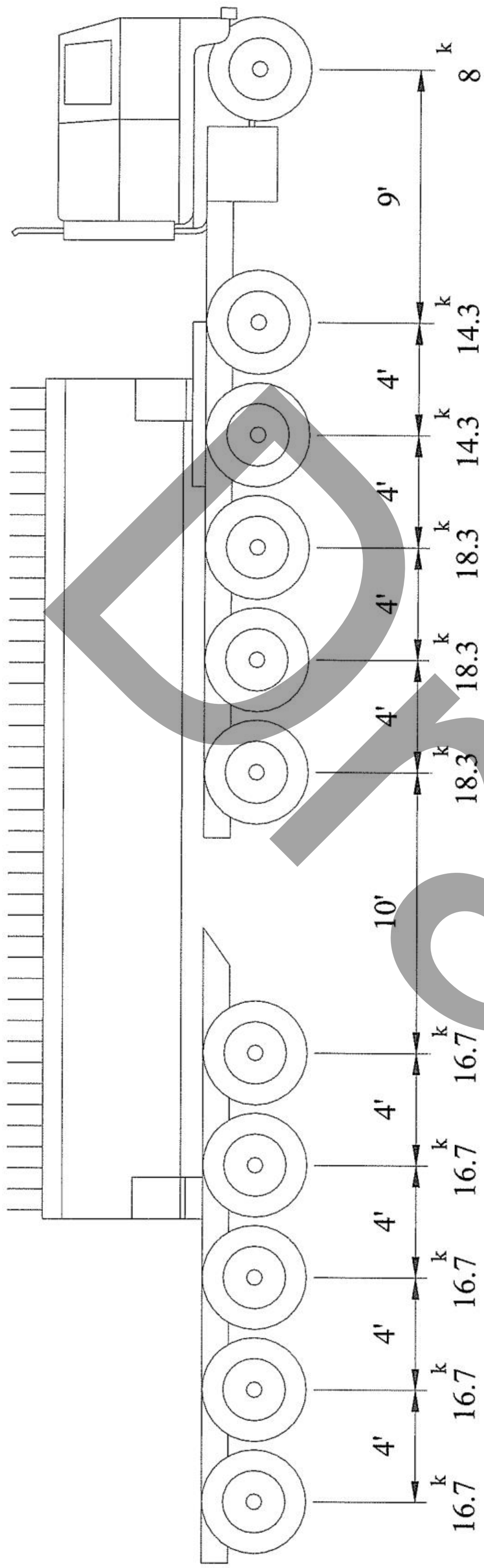


Figure A2A.13-1—Permit Truck Loading Configuration

Use One-Lane Distribution Factor and divide out the 1.2 multiple presence factor. 6A.4.5.4.2b

$$g_{m1} = 0.565 \times \frac{1}{1.2} = 0.471$$

$$g_{v1} = 0.621 \times \frac{1}{1.2} = 0.518$$

Distributed Live-Load Effect:

M_{LL+IM} = (347.3) (0.471) (1.33) = 217.6 kip-ft

V_{LL+IM} = (52.6) (0.518) (1.33) = 36.2 kips

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$ Eq. 6A.4.2.1-1

For Strength Limit States: $C = (\varphi_c)(\varphi_s)(\varphi)R_n$

Flexure:

$$RF_M = \frac{(1.0)(1.0)(0.9)(493.4) - (1.25)(84.7) - (1.25)(27.9)}{(1.40)(217.6)}$$

$= 1.0 \leq 1.0$ No Good

Shear: Shear evaluation may be considered for Permit Load Ratings. 6A.5.8

Since V_n was determined by the simplified approach, it is not dependent upon the vehicle.

$$RF_V = \frac{(1.0)(1.0)(0.9)(93.2) - (1.25)(10.8 + 3.6)}{(1.40)(36.2)}$$

$= 1.30 > 1.0$ OK

A2A.13.2—Service I Limit State (Optional) (6A.5.4.2.2b)

$\gamma_L = \gamma_{DC} = \gamma_{DW} = 1.0$ Table 6A.4.2.2-1

Use the distribution factors that were used for the design and for legal loads. C6A.5.4.2.2b

g_m = 0.703

Distributed Live-Load Effect

M_{LL+IM} = (347.3) (0.703) (1.33) = 324.7 kip-ft

M_{DC} = 84.7 kip-ft

M_{DW} = 27.9 kip-ft

A2A.13.2.1—Simplified Check Using 0.75M_n (C6A.5.4.2.2b)

Unfactored Moments:

$M_{DC} + M_{DW} + M_{LL+IM}$ = 437.3 kip-ft

Nominal flexural resistance:

M_n = 493.4 kip-ft

(Use nominal resistance, not factored.)

$0.75M_n$ = 0.75 × 493.4 = 370.1 kip-ft < 437.3 kip-ft No Good

Moment Ratio = $\dfrac{0.75M_n}{M_{DC}+M_{DW}+M_{LL+IM}} = \dfrac{370.1}{437.3} = 0.86 < 1.0$ No Good

A2A.13.2.2—Refined Check Using 0.9f_y C6A.5.4.2.2b

$M_{DC} + M_{DW}$ = 112.6 kip-ft

The Service I moments act upon the cracked section to produce stress in the reinforcement. An elastic model of the cracked concrete section with transformed steel is used to calculate the stress in the reinforcement due to the Service I loads.

$E_c = 1{,}820\sqrt{f_c'}$ LRFD Design Eq. C5.4.2.4-3

$= 1{,}820\sqrt{3.0}$

= 3,152 ksi

E_s = 29,000 ksi

$n = \dfrac{29{,}000}{3{,}152} = 9.2$ Use $n = 9$

For permanent loads at the Service limit states, use an effective modular ratio of n. LRFD Design 5.6.1

b_e = 78 in.

t_s = 6 in.

t_w = 15 in.

A_s = 6.89 in.^{2}

d_s = 26.61 in.

Assume neutral axis is within the slab.

$$\bar{y} = \frac{(b_e \times \bar{y})\dfrac{\bar{y}}{2} + (n \times A_s)(d_s)}{(b_e \times \bar{y}) + (n \times A_s)}$$

For $n = 9$:

$\bar{y}$ = 5.76 in. (within the slab)

$$I = \frac{1}{12}b_e \times \bar{y}^3 + (b_e \times \bar{y})\left(\frac{\bar{y}}{2}\right)^2 + (n \times A_s)(d_s - \bar{y})^2$$

$$= \frac{1}{12} \times 78 \times 5.76^3 + (78 \times 5.76)\left(\frac{5.76}{2}\right)^2 + (9 \times 6.89)(26.61 - 5.76)^2$$

$$= 31{,}926 \text{ in.}^3$$

Stress in the extreme tension reinforcement:

$$\text{bending stress, } f = n \times \frac{M \times 12 \times \left(h - cov. - \bar{y}\right)}{I}$$

$$f_{LL+IM} = 9 \times \frac{324.7 \times 12 \times (30 - 2.5 - 5.76)}{31,926} = 23.88 \text{ ksi}$$

$$f_D = 9 \times \frac{112.6 \times 12 \times (30 - 2.5 - 5.76)}{31,926} = 8.28 \text{ ksi}$$

f_s = $f_{LL+IM} + f_D$ = 23.88 + 8.28 = 32.2 ksi

f_R = $0.90 f_y$ = 0.90 × 33 ksi = 29.7 ksi 6A.5.4.2.2b

29.7 < 32.2 No Good

Stress Ratio:

$$\frac{f_R - f_{DC} - f_{DW}}{f_{LL+IM}} = \frac{29.7 - 8.28}{23.88} = 0.9 \text{ No Good}$$

Some improvement versus the simplified check, but not enough to allow the permit if this optional check is applied. The truck also has an $RF \leq 1.0$ under flexure.

A2A.14—IoH Tier 1 Load Rating

Vehicle Weight: 82 kips

ADTT (one direction): 1,850
Gage width (GW) = 8 ft

IM = 20 percent MBEIoH 2A.4.3.3

Undistributed Maximum:
M_{LL} = 124.8 kip-ft IoH Tier 1a controls
V_{LL} = 20.4 kips IoH Tier 1a controls

A2A.14.1—Strength I Limit State

ADTT (one direction): 1850
Load Factor, γ_L: 1.33 MBEIoH Table 2A.4.3.2.2-1

Use One-Lane Distribution Factor and divide out the 1.2 multiple presence factor. 6

$$g_{m1,HL93truck} = 0.565 \times \frac{1}{1.2} = 0.471$$

For IoH vehicle with dual-wheel-steering-axle, apply MF: 6A

$$MF_{moment} = 1 - 3.281 R_1 Ln(\frac{GW}{6})(\frac{S}{L})^{1.48} = 1 - 3.281(0.85) Ln(\frac{8}{6})(\frac{6.52}{26})^{1.48} = 0.896$$

$g_{m1} = g_{m1,HL93\ truck}\, MF_{moment}$ =0.471 (0.896) = 0.422

$$g_{v1,HL93truck} = 0.621 \times \frac{1}{1.2} = 0.518$$

For IoH vehicle with singl-wheel-steering-axle, apply MF: 6A

$$MF_{shear} = R_2\, 0.747 (\frac{L}{S})^{0.058} (\frac{14}{S})^{0.392} (\frac{S}{GW})^{0.136} (\frac{t_S}{S})^{0.229}$$

$$= (1.05) 0.747 (\frac{26}{6.52})^{0.058} (\frac{14}{6.52})^{0.392} (\frac{6.52}{8})^{0.136} (\frac{6}{6.52})^{0.229}$$

$$= (1.05) 0.747 (1.084) 1.349 (0.973) 0.981 = 1.095$$

$g_{v1} = g_{v1,HL93\ truck}\, MF_{shear}$ = 0.518 (1.095) = 0.567

Distributed Live-Load Effect:

M_{LL+IM} = (124.8) (0.422) (1.20) = 63.2 kip-ft

V_{LL+IM} = (20.4) (0.567) (1.20) = 13.9 kips

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$ Eq. 6A

For Strength Limit States: $C = (\varphi_c)(\varphi_s)(\varphi)R_n$

Flexure:

$$RF_M = \frac{(1.0)(1.0)(0.9)(493.4) - (1.25)(84.7) - (1.25)(27.9)}{(1.33)(63.2)}$$

= 3.61 > 1.0 OK

Shear:

Since V_n was determined by the simplified approach, it is not dependent upon the vehicle.

$$RF_V = \frac{(1.0)(1.0)(0.9)(93.2) - (1.25)(10.8 + 3.6)}{(1.33)(13.9)}$$

= 3.57 > 1.0 OK

A2A.14.2—Service I Limit State (Optional) (6A.4.2.2)

γ_L = γ_{DC} = γ_{DW} = 1.0 Table 6A

Use the distribution factors that were used for the design and for legal loads. C6A.

$g_{m,HL93,one\text{-}lane}$ = 0.565 Tabl
$g_{m,HL93,one\text{-}lane}$ /1.2 = 0.565/1.2 = 0.471 built in multiple presence factor divided out

For IoH vehicle with dual-wheel-steering-axle, apply MF: 6A.3.2.1.6

$$MF_{moment} = 1 - 3.281 R_1 Ln(\frac{GW}{6})(\frac{S}{L})^{1.48} = 1 - 3.281(0.85) Ln(\frac{8}{6})(\frac{6.52}{26})^{1.48} = 0.896$$

$g_{m1} = g_{m1,HL93\ truck}\, MF_{moment}$ =0.471 (0.896) = 0.422

$g_{m1} = g_{m1,HL93}$
MF_{moment} =0.4
= 0.414

Distributed Live-Load Effect

M_{LL+IM} = (124.8) (0.422) (1.20) = 63.2 kip-ft

M_{DC} = 84.7 kip-ft

M_{DW} = 27.9 kip-ft

A2A.14.2.1—Simplified Check Using 0.75M_n (C6A.5.4.2.2b)

Unfactored Moments:

$M_{DC} + M_{DW} + M_{LL+IM}$ = 175.8 kip-ft

Nominal flexural resistance:

M_n = 493.4 kip-ft

(Use nominal resistance, not factored.)

0.75M_n = 0.75 × 493.4 = 370.1 kip-ft > 175.8 kip-ft OK.

Moment Ratio = $\frac{0.75M_n}{M_{DC} + M_{DW} + M_{LL+IM}} = \frac{370.1}{175.8} = 2.11 > 1.0$ OK.

A2A.14.2.2—Refined Check Using 0.9f_y C

$M_{DC} + M_{DW}$ = 112.6 kip-ft

The Service I moments act upon the cracked section to produce stress in the reinforcement. An elastic model of the cracked concrete section with transformed steel is used to calculate the stress in the reinforcement due to the Service I loads.

$E_c = 1{,}820\sqrt{f'_c}$ L
Ec

$= 1{,}820\sqrt{3.0}$

= 3,152 ksi

E_s = 29,000 ksi

$n = \frac{29{,}000}{3{,}152} = 9.2$ Use n = 9

For permanent loads at the Service limit states, use an effective modular ratio of n. LRFD

b_e = 78 in.

t_s = 6 in.

t_w = 15 in.

A_s = 6.89 in.^{2}

d_s = 26.61 in.

Assume neutral axis is within the slab.

$$\bar{y} = \frac{(b_e \times \bar{y})\frac{\bar{y}}{2} + (n \times A_s)(d_s)}{(b_e \times \bar{y}) + (n \times A_s)}$$

For n = 9:

$\bar{y}$ = 5.76 in. (within the slab)

$$I = \frac{1}{12} b_e \times \bar{y}^3 + (b_e \times \bar{y})\left(\frac{\bar{y}}{2}\right)^2 + (n \times A_s)(d_s - \bar{y})^2$$

$$= \frac{1}{12} \times 78 \times 5.76^3 + (78 \times 5.76)\left(\frac{5.76}{2}\right)^2 + (9 \times 6.89)(26.61 - 5.76)^2$$

$$= 31{,}926 \text{ in.}^3$$

Stress in the extreme tension reinforcement:

$$\text{bending stress, } f = n \times \frac{M \times 12 \times (h - cov. - \bar{y})}{I}$$

$$f_{LL+IM} = 9 \times \frac{63.2 \times 12 \times (30 - 2.5 - 5.76)}{31{,}926} = 4.65 \text{ ksi}$$

$$f_D = 9 \times \frac{112.6 \times 12 \times (30 - 2.5 - 5.76)}{31{,}926} = 8.28 \text{ ksi}$$

f_s = $f_{LL+IM} + f_D$ = 4.65 + 8.28 = 12.93 ksi

f_R = $0.90 f_y$ = 0.90 × 33 ksi = 29.7 ksi 6A.

29.7 > 12.93 OK.

Stress Ratio:

$$\frac{f_R - f_{DC} - f_{DW}}{f_{LL+IM}} = \frac{29.7 - 8.28}{4.65} = 4.61 \text{ OK.}$$

A2A.15—Summary of Rating Factors for Load and Resistance Factor Rating Method

Table A2A.15-1—Summary of Rating Factors for Load and Resistance Factor Rating Method—Interior Beam

Limit State		Design Load Rating		Legal Load Rating								Permit Load Rating	IoH Load Rating
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL		Tier 1
Strength I	Flexure	0.59	0.76	1.29	1.35	1.57	1.69	1.00	0.93	0.90	0.90	—	3.61
	Shear	0.85	1.10	—	—	—	—	—	—	—	—	—	3.57
Strength II	Flexure	—	—	—	—	—	—	—	—	—	—	1.00	-
	Shear	—	—	—	—	—	—	—	—	—	—	1.30	-
Service I		—	—	—	—	—	—	—	—	—	—	Stress Ratio = 0.90	Stress Ratio=4.61

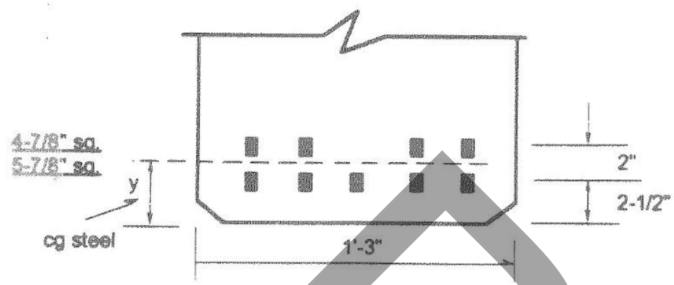
PART B—ALLOWABLE STRESS AND LOAD FACTOR RATING METHODS

A2B.1—Bridge Data

Refer to Article A2A.1 for Bridge Data.

A2B.2—Section Properties

Find cg steel:



$$y = \frac{4(.766)(2 + 2\text{-}1/2) + 5(.766)(2\text{-}1/2)}{4(.766) + 5(.766)}$$

$y = 3.39"$

$d = 30" - 3.39 = 26.61"$

$A_{1BAR} = 7/8" \times 7/8" = 0.766 \text{ in}^2$

$A_s = 9 \times A_{1BAR} = 6.89 \text{ in}^2$

Figure A2B.2-1—Steel Reinforcement Arrangment

Effective Slab Width (for T-Girder): AASHTO 8.10.1.1

$$\frac{1}{4}L = \frac{26 \text{ ft} \times 12 \text{ in./ft}}{4} = 78 \text{ in.}$$

or:

$$CC\ SPCG = 6 \text{ ft} - 6\ \frac{1}{4} \text{ in.} = 78.25 \text{ in.}$$

or:

$$12t_s = 12 \times 6 \text{ in.} = 72 \text{ in.} \Leftarrow \text{Controls}$$

$$\rho_{act} = \frac{As}{b_{eff}d} = \frac{6.89 \text{ in}^2}{78 \text{ in.} \times 26.61 \text{ in.}} = 0.0036$$

(if compression within flange)

A2B.3—Dead-Load Analysis—Interior Beam

Structural Concrete:

$$0.15 \text{ kip/ft}^3\left[\left(\frac{6 \text{ in.}}{12 \text{ in./ft}} \times 6.52 \text{ ft}\right) + \left(1.25 \text{ ft} \times 2.0 \text{ ft}\right) + 2\left(\frac{1}{2}\frac{6}{12}\frac{6}{12}\right)\right] = 0.92 \text{ kip/ft}$$

AC Overlay:

$$0.144 \text{ kip/ft}^3\left(\frac{5 \text{ in.}}{12 \text{ in./ft}} \times 6.52 \text{ ft}\right) = 0.39 \text{ kip/ft}$$

$W_{DL} = 0.902 + 0.39 = 1.292$ kip/ft <u>say 1.3 kip/ft</u>

Midspan Moments:

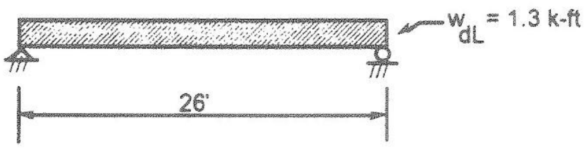


Figure A2B.3-1—Load Diagram for Uniform Dead Load

$$M_{DL} = \frac{W_{DL}L^2}{8} = \frac{1.3 \text{ kip/ft} \times 26^2 \text{ ft}^2}{8} = 109.9 \text{ kip-ft}$$

A2B.4—Live-Load Analysis—Interior Beam

Rate for HS-20 vehicle. Figure 6B.6.2-1

For HS-20—Using Table C6B-1, select from column "Without Impact." Appendix C6B

$M_L = 111.1$ kip-ft (without impact and without distribution)

A2B.5—Allowable Stress Rating (6B.3.1, 6B.4.2, and 6B.5.2)

For this example, we consider only the maximum moment section.

A2B.5.1—Impact (Use standard AASHTO) (6B.6.4, AASHTO 3.8.2.1)

$$I = \frac{50}{L + 125} \leq 0.30$$

$$I = \frac{50}{26 + 125} = 0.33 \text{ use } \underline{\underline{0.30}}$$

A2B.5.2—Distribution (Use standard AASHTO) (6B.6.3, AASHTO 3.23.2.2 and Table 3.23.1)

$$DF = \frac{S_G}{6.0} \text{ Concrete T-Beam}$$

$$DF = \frac{6 \text{ ft} - 6\ 1/4 \text{ in.}}{6.0} = \frac{6.52 \text{ ft}}{6} = 1.087$$

Thus:

$$M_{L+I} = M_L(1 + I)(DF) = 111.1(1 + 0.30)(1.087) = 157 \text{ kip-ft}$$

A2B.5.3—Inventory Level (6B.4.2, 6B.5.2.4)

The inventory unit stresses are determined in accordance with AASHTO Article 8.15, "Service Load Design Method," or taken from 6B.5.2.4[a].

Inventory allowable stresses: AASHTO 8.15.2.1.1

$f_c^I = 1200 \text{ psi} = 1.2 \text{ ksi}$ 6B.5.2.4.1

[a] Note the moments given in the MBE are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MBE values.

For Reinforcing Steel, 6B.5.2.3 controls:

$$f_s^I = 18,000 \text{ psi} = 18 \text{ ksi (unknown steel prior to 1954)}$$ 6B.5.2.3

Capacity (Traditional Approach):

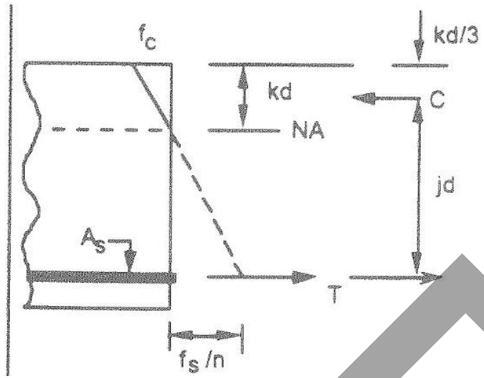


Figure A2B.5.3-1—Stress and Force Diagram, *nts*

The actual steel and concrete stresses are not known and must be found. Since this is a T-beam, assume neutral axis *NA* is within slab. Thus, rectangular beam formulas apply. Check this assumption later.

The following formulas for the Traditional Approach were referenced from *Reinforced Concrete Design Handbook* Working Stresss Method in accordance with ACI 318-63, ACI Publication SP-3.

Position of Neutral Axis:

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$$ SP-3 Eq. (2)

where:

$$\rho = \frac{A_s}{bd} = \frac{6.89 \text{ in.}^2}{(72 \text{ in.})(26.61 \text{ in.})}$$ SP-3 Table 1

$$\rho = 0.0036$$

$$n = \frac{E_s}{E_c}$$

$$n = 10$$ 6B.5.2.4

$$k = \sqrt{2(0.0036)(10) + [(0.0036)(10)]^2} - (0.0036)(10)$$

$$k = 0.235$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.235}{3} = 0.922$$ SP-3 Table 1

then:

Capacity if concrete allowable stress controls:

$$M_c = \frac{1}{2} f_c jkbd^2$$

$$= \frac{1}{2}(1.2 \text{ ksi})(0.922)(0.235)(72 \text{ in.})(26.61 \text{ in.})^2$$

$$= 6,622.8 \text{ kip-in.} = 552 \text{ kip-ft}$$

Capacity if steel reinforcement allowable stress controls:

$$M_s = A_s f_s jd$$

$$M_s = \left(6.89 \text{ in.}^2\right)(18 \text{ ksi})(0.922)(26.61 \text{ in.})$$

$$M_s = 3,042.8 \text{ kip-in.} = 253 \text{ kip-ft} \Leftarrow \text{Controls since } M_s < M_c$$

Check neutral axis assumption:

k_d = (0.235)(26.61 in.) = 6.25 in. > 6 in. the slab thickness $\therefore NA$ is below bottom of slab and slightly into web. This could be ignored in this case. However, for the sake of completeness, capacity will be figured below based on the *NA* below the slab and ignoring the compression in the stem concrete.

$$kd = \frac{2ndA_s + bt^2}{2nA_s + 2bt}$$

$$kd = \frac{2(10)(26.61 \text{ in.})(6.89 \text{ in.}) + (72 \text{ in.})(6 \text{ in.})^2}{2(10)(6.89 \text{ in.}) + 2(72 \text{ in.})(6 \text{ in.})} = \frac{6258.9}{1001.8}$$

$$kd = 6.25 \text{ in.} \rightarrow k = \frac{kd}{d} = \frac{6.25 \text{ in.}}{26.61 \text{ in.}} = 0.235$$

$$Z = \left(\frac{3kd - 2t}{2kd - t}\right)\frac{t}{3}$$

$$Z = \left(\frac{3(6.25 \text{ in.}) - 2(6 \text{ in.})}{2(6.25 \text{ in.}) - (6 \text{ in.})}\right)\frac{6 \text{ in.}}{3} = \frac{6.75 \text{ in.}}{6.5 \text{ in.}}(2 \text{ in.})$$

$$Z = 2.077 \text{ in.}$$

$$jd = d - Z$$

$$jd = 26.61 \text{ in.} - 2.077 \text{ in.} = 24.53 \text{ in.}$$

$$M_s = A_s f_s jd$$

$$M_s = \left(6.89 \text{ in.}^2\right)(18 \text{ ksi})(24.53 \text{ in.}) = 3042.2 \text{ kip-in.}$$

$$M_s = 253 \text{ kip-ft as before}$$

(Note concrete was not checked since capacity of section is limited by steel allowable stress.)

$$RF_I^A = \frac{M_{RI} - M_D}{M_{L+I}} \qquad \text{Eq. 6B.4.1-1}$$

$$RF_I^A = \frac{253 \text{ kip-ft} - 109.9 \text{ kip-ft}}{157 \text{ kip-ft}} = 0.91$$

A2B.5.4—Operating Level (6B.5.2)

The Operating allowable stresses for concrete with f'_c = 3,000 psi:

$f_c^o = 1900 \text{ psi} = 1.9 \text{ ksi}$ 6B.5.2.4.1

For reinforcing steel:

$f_s^o = 25{,}000 \text{ psi} = 25 \text{ ksi}$ (unknown steel, prior to 1954) 6B.5.2.3

The basic relationships defined previously apply:

Since ρ and *n* do not change, the neutral axis, *k*, *j*, and *Z* terms do not change.

Thus:

$$M_s = A_s f_s jd$$

$$= \left(6.89 \text{ in.}^2\right)(25 \text{ ksi})(24.53 \text{ in.})$$

$$= 4225.3 \text{ kip-in.} = 352 \text{ kip-ft}$$

and checking concrete stress to ensure that concrete does not control:

$$f_c = \frac{f_s}{n}\left(\frac{k}{1-k}\right)$$ SP-3 Table 1

$$f_c = \left(\frac{25 \text{ ksi}}{10}\right)\left(\frac{0.235}{1-0.235}\right) = 0.77 \text{ ksi} << 1.9 \text{ ksi allowable}$$

Therefore, capacity of section is controlled by allowable steel stress.

$$M_{R_O} = 352 \text{ kip-ft}$$

$$RF_O^A = \frac{M_{R_O} - M_{DL}}{M_{L+I}} = \frac{352 \text{ kip-ft} - 109.9 \text{ kip-ft}}{157 \text{ kip-ft}}$$

$$RF_O^A = 1.54$$

A2B.6—Load Capacity Based on Allowable Stress

Inventory: $0.91 \times 36^T = 32.8^T\, HS$

Operating: $1.54 \times 36^T = 55.4^T\, HS$

To transform HS rating to H rating, multiply HS rating factor by ratio of HS moment to H moment:

For 26-ft span:

$$M_L^{\text{HS-20}} = 111.1 \text{ kip-ft}$$

$\rightarrow M_L^{\text{H-15}} = 78 \text{ kip-ft}$ Table C6B-1

Then:

$$M_L^{\text{H-20}} = \frac{20T}{15T} \times 78 \text{ kip-ft} = 104 \text{ kip-ft}$$

and:

$$\text{Ratio} = \frac{M_L^{\text{HS-20}}}{M_L^{\text{H-20}}} = \frac{111.1}{104} = 1.068$$

Thus for H-20 Truck:

Inventory: $0.91 \times 1.068 \times 20^T = 19.4^T\, H$

Operating: $1.54 \times 1.068 \times 20^T = 32.9^T\, H$

A2B.7—Capacity (Alternate Approach)

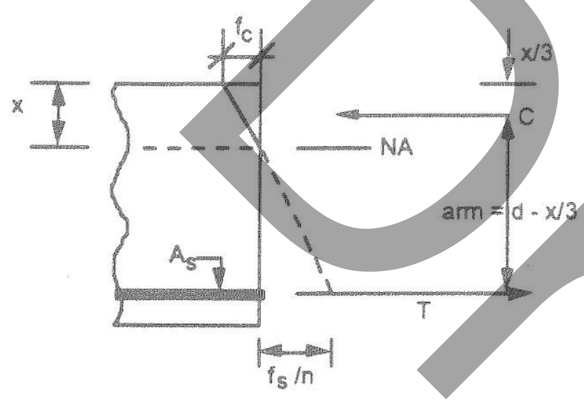


Figure A2B.7-1—Stress and Force Diagram, *nts*

Since the location of the neutral axis *NA* and the corresponding stresses in the steel and concrete are not known, these must be determined consistent with the principles of equilibrium of the cross section.

1. From the stresses on the cross section using similar triangles:

$$\frac{f_c}{x} = \frac{f_s \div n}{d - x} \rightarrow f_c = \frac{f_s}{n}\left(\frac{x}{d - x}\right) \qquad \text{(Eq. A2B.7-1)}$$

2. Assume the steel allowable stress controls the capacity of the section. This will be checked later. Then:

$$T = A_s f_s = \left(6.89 \text{ in.}^2\right)\left(18 \text{ ksi}\right) = 124 \text{ kips}$$

and:

$$C = \frac{1}{2} f_c b x$$

but:

$$C = T$$

thus:

$$\frac{1}{2} f_c b x = A_s f_s$$

$$x = \frac{A_s f_s}{\frac{1}{2} f_c b} \qquad \text{(Eq. A2B.7-2)}$$

Solve Eqs. A2B.7-1 and A2B.7-2 to find location of neutral axis. This may be done by trial and error as follows.

Assume f_s = 18 ksi, i.e., steel allowable stress controls.

Try x = 6.0 in. Then by Eq. A2B.7-1:

$$f_c = \frac{f_s}{n}\left(\frac{x}{d-x}\right) = \frac{18 \text{ ksi}}{10}\left(\frac{6.0 \text{ in.}}{26.61 \text{ in.} - 6.0 \text{ in.}}\right) = 0.524 \text{ ksi} < 1.2 \text{ ksi} \qquad \text{allowable OK}$$

and by Eq. A2B.7-2:

$$x = \frac{A_s f_s}{\frac{1}{2} f_c b} = \frac{\left(6.89 \text{ in.}^2\right)(18 \text{ ksi})}{\frac{1}{2}(0.524 \text{ ksi})(72 \text{ in.})} = 6.57 \text{ in.} > 6.0 \qquad \text{assumed. Try again}$$

Try x = 6.25 in.

$$f_c = \frac{18}{10}\left(\frac{6.25}{26.61 - 6.25}\right) = 0.552 < 1.2 \text{ ksi} \qquad \text{allowable OK}$$

and:

$$x = \frac{(6.89)(18)}{\frac{1}{2}(0.552)(72)} = 6.24 \approx 6.25 \qquad \text{assumed OK}$$

3. Since x = 6.24 > t = 6.0, NA is below bottom of slab and slightly into web. If web concrete in compression is neglected:

$arm \approx d - \frac{x}{3}$ for this example.

$$arm \approx 26.61 - \frac{6.24}{3} = 24.53 \text{ in.}$$

and capacity is:

$$M = A_s f_s \,(arm) = (6.89)(18)(24.53) = 3{,}042.2 \text{ kip-in.} = 253 \text{ kip-ft} \qquad \text{as before}$$

The exact *arm* may be determined from the concrete stress diagram as follows.

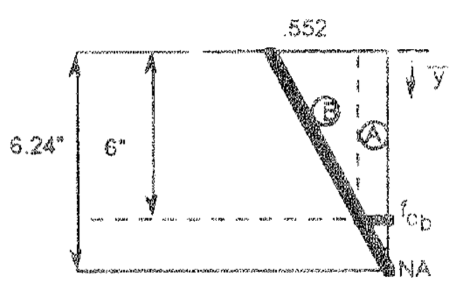


Figure A2B.7-2—Concrete Stress Diagram for Slab Portion of T-Beam, *nts*

at bottom of slab:

$$f_{c_b} = 0.552\left(\frac{0.24}{6.24}\right) = 0.021$$

Next find centroid of stress diagram from top of slab.

$$\bar{y} = \frac{\sum A_y}{\sum A} = \frac{(0.021)(6)\left(\frac{6}{2}\right) + (0.552 - 0.021)(6)\left(\frac{1}{2}\right)\left(\frac{6}{3}\right)}{(0.021)(6) + (0.552 - 0.021)(6)\left(\frac{1}{2}\right)}$$

$$\bar{y} = \frac{3.576}{1.722} = 2.08 \text{ in.}$$

$\therefore arm = 26.61 - 2.08 = 24.53$ in. as found previously

4. The operating capacity may be found as above and will be the same as for the "traditional method." The rating calculations are not shown here since they too will be the same as for the traditional method.

A2B.8—Allowable Stress Rating—Rate for AASHTO Legal Loads

M_{L+I} from Appendix C6B (all values have 30 percent impact):

Span	Type 3	Type 3S2	Type 3-3	
26 ft	122.4	117.7	100.8	kip-ft

Span	NRL	SU4	SU5	SU6	SU7	
26 ft	176.2	145.1	158.0	171.1	175.2	kip-ft

Apply distribution factor DF = 1.087

Span	Type 3	Type 3S2	Type 3-3	
26 ft	133.0	127.9	109.6	kip-ft

Span	NRL	SU4	SU5	SU6	SU7	
26 ft	191.5	157.7	171.7	186.0	190.4	kip-ft

Capacity of section as previously determined in A2B.5.3 and A2B.5.4 respectively:

Inventory Level M_{RI} = 253 kip-ft Operating Level M_{RO} = 352 kip-ft.

Dead Load M_{DL} = 109.9 kip-ft.

For Allowable Stress Method A_1 = 1.0 and A_2 = 1.0 6B.4.2

$$RF_I^A = \frac{M_{RI} - A_1 M_D}{A_2 M_{L+I}} = \frac{253 \text{ kip-ft} - (1.0)109.9 \text{ kip-ft}}{(1.0)M_{L+I}}$$ Eq. 6B.4.1-1

$$RF_O^A = \frac{M_{RO} - A_1 M_{DL}}{A_2 M_{L+I}} = \frac{352 \text{ kip-ft} - (1.0)109.9 \text{ kip-ft}}{(1.0)M_{L+I}}$$

Alowable Stress Method Rating Factors:

	Type 3	Type 3S2	Type 3-3
Inventory	1.08	1.12	1.31
Operating	1.82	1.89	2.21

	NRL	SU4	SU5	SU6	SU7
Inventory	0.75	0.91	0.83	0.77	0.75
Operating	1.26	1.53	1.41	1.30	1.27

Load Capacity in Tons:

Inventory: $RF_I^A \times \text{vehicle weight} = \text{Inv.Cap.}$

Operating: $RF_O^A \times \text{vehicle weight} = \text{Opr.Cap.}$

Load	Type 3	Type 3S2	Type 3-3
Vehicle Weight	25	36	40
Inv. Cap.	27.0	40.3	52.4
Opr. Cap.	45.5	68.0	88.4

Load	NRL	SU4	SU5	SU6	SU7
Vehicle Weight	40	27	31	34.8	38.8
Inv. Cap.	30.0	24.6	25.7	26.8	29.1
Opr. Cap.	50.4	41.3	43.7	45.2	49.3

A2B.9—Summary of Ratings for Allowable Stress Rating Method

Table A2B.9-1—Summary of Ratings for Allowable Stess Rating Method—Interior Beam

Load	HS-20	H-20	Type 3	Type 3S2	Type 3-3
Vehicle Weight (tons)	36	20	25	36	40
Inventory RF	0.91	0.91	1.08	1.12	1.31
Inv. Cap.	32.8	19.4	27.0	40.3	52.4
Operating RF	1.54	1.54	1.82	1.89	2.21
Opr. Cap.	55.4	32.9	45.5	68.0	88.4
Load	NRL	SU4	SU5	SU6	SU7
Vehicle Weight (tons)	40	27	31	34.8	38.8
Inventory RF	0.75	0.91	0.83	0.77	0.75
Inv. Cap.	30.0	24.6	25.7	26.8	29.1
Operating RF	1.26	1.53	1.41	1.30	1.27
Opr. Cap.	50.4	41.3	43.7	45.2	49.3

A2B.10—Load Factor Rating (6B.3.2, 6B.4.3, 6B.5.3)

For this example, we consider only the maximum moment section.

A2B.10.1—Impact (Use standard AASHTO) (6B.6.4, AASHTO 3.8.2.1)

$$I = \frac{50}{L+125} \leq 0.30$$

$$I = \frac{50}{26+125} = 0.33 \text{ use } \underline{\underline{0.30}}$$

A2B.10.2—Distribution (Use standard AASHTO) (6B.6.3, AASHTO 3.23.2.2 and Table 3.23.1)

$$DF = \frac{S_G}{6.0} = \frac{6.52 \text{ ft}}{6} = 1.087$$

Thus:

$$M_{LL+I} = M_L(1+I) \times DF = 111.1(1+0.30)(1.087) = 157 \text{ kip-ft}$$

A2B.10.3—Capacity of Section (6B.5.3.2)

For unknown steel prior to 1954, f_y = 33,000 psi = 33 ksi

M_u is found in accordance with applicable strength requirements of AASHTO Article 8.16.

Consider a rectangular section with compression limited to top slab. Then check 6B.5.3.2 requirement for 75 percent of balanced condition.

$$\rho_{max} = 0.75\rho_{bal} = 0.75\frac{0.85\beta_1 f_c'}{f_y}\frac{87000}{87000+f_y}$$ AASHTO Eq. 8-18

$$\rho_{max} = 0.75\frac{0.85(0.85)(3000)}{33000}\left(\frac{87000}{87000+33000}\right)$$

$$\rho_{act} = 0.0036 << \rho_{max}$$ OK

Then:

$$a = \frac{A_s f_y}{0.85 f_c' b_{eff}}$$ AASHTO Eq. 8-17

$$a = \frac{6.89 \text{ in.}^2 (33 \text{ ksi})}{0.85(3 \text{ ksi})72 \text{ in.}} = 1.24 \text{ in.} < 6 \text{ in.}$$ OK within slab

$$M_R = A_s f_y\left(d - \frac{a}{2}\right)$$ AASHTO Eq. 8-16

$$M_R = \left(6.89 \text{ in.}^2\right)(33 \text{ ksi})\left(26.61 \text{ in.} - \frac{1.24}{2}\right)$$

$$M_R = 5909 \text{ kip-in.} = \underline{\underline{492 \text{ kip-ft}}}$$
$$M_u = \phi M_R$$

where $\phi = 0.90$ AASHTO 8.16.1.2.2

$$M_u = 0.90 \times 492 = 443 \text{ kip-ft.}$$

A2B.10.4—Inventory Level (6B.4.1, 6B.5.3)

M_{DL} is the same as what was estimated for the ASD rating calculation:

$$R_I^{LF} = \frac{M_u - A_1 M_{DL}}{A_2 M_{L+I}}$$ Eq. 6B.4.1-1

where in accordance with 6B.4.3:

$A_1 = 1.3$

$A_2 = 2.17$

Thus:

$$RF_I^{LF} = \frac{443 - 1.3(109.9)}{2.17(157)} = 0.88$$

A2B.10.5—Operating Level (6B.4.1, 6B.5.3)

$$R_O^{LF} = \frac{M_u - A_1 M_{DL}}{A_2 M_{L+I}}$$ Eq. 6B.4.1-1

where in accordance with 6B.4.3:

$A_1 = 1.3$

$A_2 = 1.3$

Thus:

$$RF_O^{LF} = \frac{443 - 1.3(109.9)}{1.3(157)} = 1.47$$

Load capacity based on Load Factor Method, HS-20 truck:

Inventory: $0.88 \times 36^T = 31^T\ HS$

Operating: $1.47 \times 36^T = 52^T\ HS$

Load capacity based on Load Factor Method, H-20 truck, where the ratio of HS moment to H moment has been determined in A2B.6 as 1.068:

Inventory: $0.88 \times 1.068 \times 20^T = 18.8^T\ H$

Operating: $1.47 \times 1.068 \times 20^T = 31.4^T\ H$

A2B.10.6—Summary of Ratings for Load Factor Rating Method

Table A2B.10.6-1—Summary of Ratings for Load Factor Rating Method—Interior Beam

	RF	HS-20 Rating, tons	H-20 Rating, tons
Inventory	0.88	31.7	18.8
Operating	1.47	52.9	31.4

A2B.10.7—Load Factor Rating—Rate for AASHTO Legal Loads

M_{L+I} from Appendix C6B (all values have 30 percent impact)

Span	Type 3	Type 3S2	Type 3-3	
26 ft	122.4	117.7	100.8	kip-ft

Apply distribution factor DF = 1.087

26 ft	133.0	127.9	109.6	kip-ft

Capacity of Section M_U = 443 kip-ft

Dead Load M_{DL} = 109.9 kip-ft

For Inventory level, A_1 = 1.3 and A_2 = 2.17 6B.4.3

Inv. $RF = \dfrac{443-1.3(109.9)}{2.17(M_{L+I})}$

For Operating level, A_1 = 1.3 and A_2 = 2.17 6B.4.3

Opr. $RF = \dfrac{443-1.3(109.9)}{1.3(M_{L+I})}$

Strength Rating Factors:

	Type 3	Type 3S2	Type 3-3
Inventory	1.01	1.05	1.22
Operating	1.74	1.81	2.11

Load Capacity in Tons:

Load	Type 3	Type 3S2	Type 3-3
Vehicle Weight	25	36	40
Inv. Cap.	25.3	37.8	48.8
Opr. Cap.	43.5	65.2	84.4

The bridge has adequate Inventory load capacity for Types 3, 3S2, and 3-3 Legal Loads.

A2B.10.8—Load Factor Rating—Rate for Single-Unit Formula B Loads

M_{L+I} from Appendix C6B (all values have 30 percent impact)

Span	NRL	SU4	SU5	SU6	SU7	
26 ft	176.2	145.1	158.0	171.1	175.2	kip-ft

Apply distribution factor DF = 1.087

Span	NRL	SU4	SU5	SU6	SU7	
26 ft	191.5	157.7	171.7	186.0	190.4	kip-ft

Capacity of Section M_U = 443 kip-ft

Dead Load M_{DL} = 109.9 kip-ft

For Inventory level, $A_1 = 1.3$ and $A_2 = 2.17$ 6B.4.3

$$\text{Inv. } RF = \frac{443 - 1.3(109.9)}{2.17(M_{L+I})}$$

For Operating level, $A_1 = 1.3$ and $A_2 = 2.17$ 6B.4.3

$$\text{Opr. } RF = \frac{443 - 1.3(109.9)}{1.3(M_{L+I})}$$

Strength Rating Factors:

	NRL	SU4	SU5	SU6	SU7
Inventory	0.72	0.88	0.81	0.74	0.73
Operating	1.20	1.47	1.35	1.24	1.22

Load Capacity in Tons:

Load	NRL	SU4	SU5	SU6	SU7
Vehicle Weight	40	27	31	34.8	38.8
Inv. Cap.	28.8	23.8	25.1	25.8	28.3
Opr. Cap.	48.0	39.7	41.9	43.2	47.3

The bridge has inadequate Inventory load capacity for the notional rating load NRL, and the posting loads SU4, SU5, SU6, and SU7.

A2B.11—Load Factor Rating for IoH Tier 1

Impact (Use standard AASHTO capped by 0.20) (6B.6.4, AASHTO 3.8.2.1)

$$I = \frac{50}{26 + 125} = 0.33 > 0.20, \text{ use } \underline{\underline{0.20}}$$

Distribution (Use standardcnn

AASHTO modified) (6B.6.3, AASHTO 3.23.2.2 and Table 3.23.1)

$$DF_{\text{standardAASHTO}} = \frac{S_G}{6.0} = \frac{6.52 \text{ ft}}{6} = 1.087$$

For IoH vehicle with dual-wheel-steering-axle, apply MF: MBEIoH Table 2B.6.3.1-1

$$MF_{moment} = 1 - 3.281 R_1 Ln(\frac{GW}{6})(\frac{S}{L})^{1.48} = 1 - 3.281(0.85) Ln(\frac{8}{6})(\frac{6.52}{26})^{1.48} = 0.896$$

$$DF = DF_{\text{standardAASHTO}} MF_{moment} = 1.087\ (0.896) = 0.974$$

Thus:

$$M_{LL+I} = M_L(1 + I) \times DF = 62.4(1 + 0.20)(0.974) = 72.9 \text{ kip-ft}$$

Capacity of Section

$M_u = \phi M_R$ is the same as in A2B.10.3

where $\phi = 0.90$ AASHTO 8.16.1.2.2

$M_u = 0.90 \times 492 = 443$ kip-ft.

IoH Tier 1 Ratings (6B.4.1, 6B.5.3)

M_{DL} is the same as what was estimated in A2B.10.4, for the inventory level:

$$R_I^{LF} = \frac{M_u - A_1 M_{DL}}{A_2 M_{L+I}}$$ MBEIoH Eq. 2B.4.1-1

where in accordance with 6B.4.3:

$A_1 = 1.3$

$A_2 = 2.17$

Thus:

$$RF_I^{LF} = \frac{443 - 1.3(109.9)}{2.17(72.9)} = 1.90$$

For the operating level:

$$R_O^{LF} = \frac{M_u - A_1 M_{DL}}{A_2 M_{L+I}}$$

where in accordance with 6B.4.3:

$A_1 = 1.3$

$A_2 = 1.3$

Thus:

$$RF_O^{LF} = \frac{443 - 1.3(109.9)}{1.3(72.9)} = 3.17$$

PART C—SUMMARY

A2C.1—Summary of All Ratings for Example A2

Table A2C.1-1—Summary of Rating Factors for All Rating Methods—Interior Beam

Limit State		Design Load Rating				Legal Load Rating								Permit Load Rating	IoH Tier 1 Load Rating
		Inventory	Operating	HS-20 Rating	H-20 Rating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL		
Strength I	Flexure	0.59	0.76			1.29	1.35	1.57	1.09	1.00	0.93	0.90	0.90	—	3.61
	Shear	0.85	1.10			—	—	—	—	—	—	—	—	—	3.57
Strength II	Flexure	—	—			—	—	—	—	—	—	—	—	0.996	—
	Shear	—	—			—	—	—	—	—	—	—	—	1.30	—
Service I		—	—	—	—	—	—	—	—	—	—	—	—	Stress Ratio = 0.90	Stress Ratio=4.61
Allowable Stress Method	Inv.	—	—	0.91	0.97	1.08	1.12	1.31	0.91	0.83	0.77	0.75	0.75	—	—
	Opr.	—	—	1.54	1.64	1.82	1.89	2.21	1.53	1.41	1.30	1.27	1.26		—
Load Factor Method	Inv.	—	—	0.88	0.94	1.01	1.05	1.22	0.88	0.81	0.74	0.73	0.72	—	1.90
	Opr.	—	—	1.47	1.57	1.74	1.81	2.11	1.47	1.35	1.24	1.22	1.20	—	3.17

A2C.2—References

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AASHTO. *AASHTO LRFD Bridge Design Specifications*, Eighth Edition, LRFD-8. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

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A3—SIMPLE SPAN PRESTRESSED CONCRETE: I-GIRDER BRIDGE EVALUATION OF AN INTERIOR GIRDER (LRFR ONLY)

Note: This example illustrates rating an interior prestressed concrete girder at midspan for moment, at the critical section for shear, and at a change in stirrup spacing for shear. The example member contains debonded tendons to illustrate how this affects the rating at the two shear locations.

A3.1—Bridge Data

Span: 80 ft (Total Length = 81 ft)
Year Built: 1985
Materials:
Concrete: $f'_c = 4$ ksi (Deck)
$f'_c = 5$ ksi (P/S Beam)
$f_{ci} = 4$ ksi (P/S Beam at transfer)

Prestressing Steel: $^1/_2$ in. diameter, 270 ksi, Low-Relaxation Strands
$A_{ps} = 0.153$ in.^{2} per strand
32 prestressing strands; ten are debonded over the last 12 ft on each end
Stirrups: #4 at 9 in. over end 20 ft
#3 at 12 in. over center 40 ft
Compression Steel: six #6 Grade 60
Condition: No Deterioration, NBI Item 59 Code = 6
Riding Surface: Minor surface deviations (Field verified and documented)
ADTT (one direction) 5,000
Skew: 0°
Effective Flange Width b_e may be taken as the tributary width perpendicular to the axis of the member — LFRD Design 4.6.2.6.1
Effective Flange Width b_e = 8.5 ft × 12 in. = 102 in.

$$E_c = 33{,}000\, W_c^{1.5}\sqrt{f'_c}$$ LRFD Design Eq. C5.4.2.4-2

For deck, $E_c = 33{,}000\times(0.145)^{1.5}\sqrt{4.0} = 3.64\times10^3$ ksi

For P/S Beam, $E_c = 33{,}000\times(0.145)^{1.5}\sqrt{5.0} = 4.07\times10^3$ ksi

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

Transformed Width, $b_{trans} = 102\text{ in.}\times0.89 = 90.8$ in.

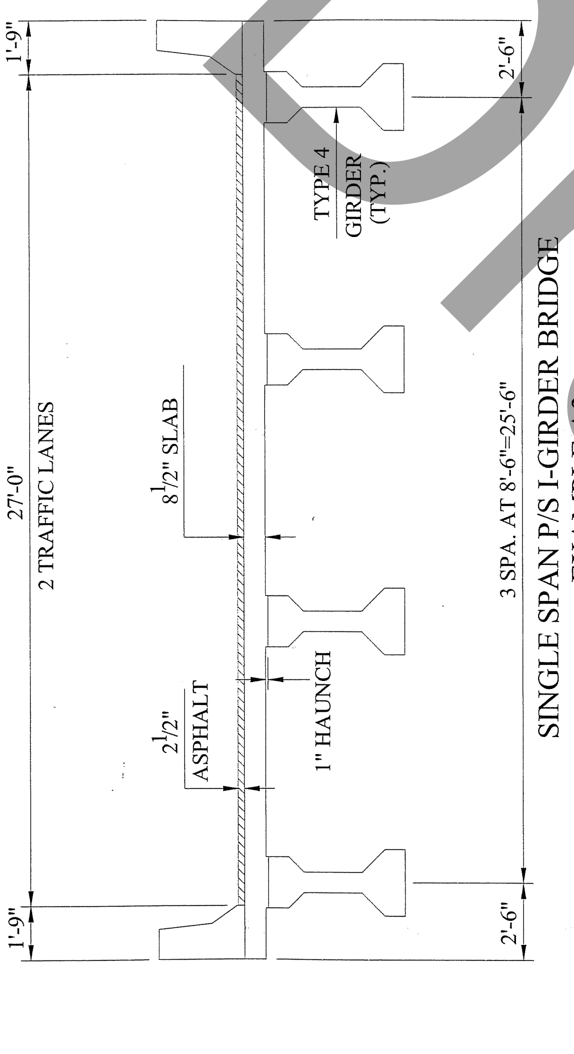

Figure A3.1-1—Cross Section—Single Span Prestressed I-Girder Bridge

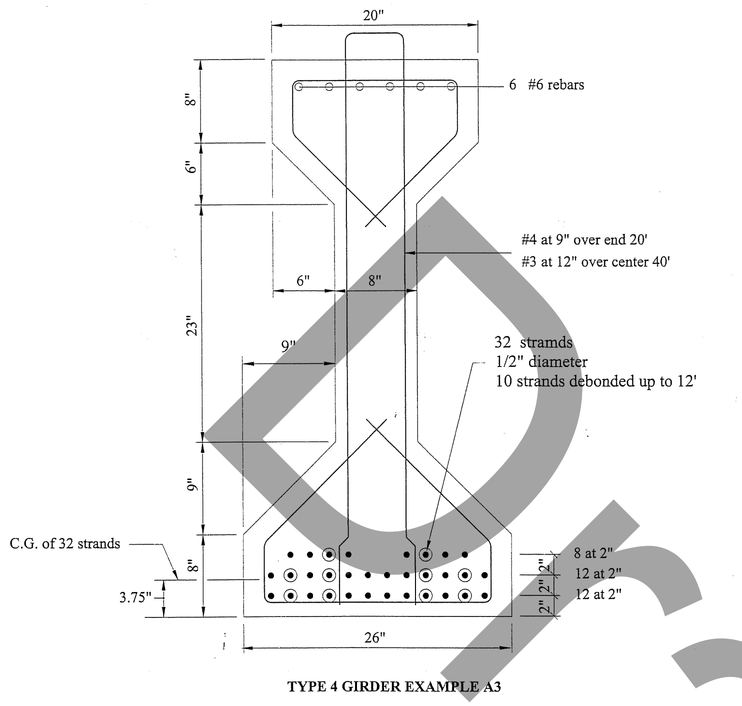

Figure A3.1-2 Cross Section—Interior Girder AASHTO Type 4 I-Girder

A3.2—Summary of Section Properties

Type 4 Girder:

h = 54 in.

A = 789 in.^{2}

I = 260,730 in.^{4}

Y_b = 24.73 in.

S_b = 10,543 in.^{3}

S_t = 8,908 in.^{3}

Composite Section

	Area, in.^{2}	y, in.	Ay	d	Ad^2 in.^{4}	I_0 in.^{4}
P/S beam	789	24.73	19,512	17.07	229,903	260,730
Slab	772	59.25	45,741	17.45	235,076	4,647
Totals	1,561		65,253		464,979	265,377

Area slab = 8.5 in. × 90.8 in. = 772 in.^{2} (uses full slab thickness of deck)

y slab = 54 in. + 1 in. + ½ × 8.5 in. = 59.25 in. (includes 1-in. haunch)

$\bar{y}$ = 65,253 ÷ 1,561 = 41.80 in.

d = $y - \bar{y}$

y_{bot} = $\bar{y}$ = 41.80 in. y_{top} = $h - \bar{y}$ = 54 in. – 41.80 in. = 12.20 in.

$$I_0 \text{ slab} = \frac{bh^3}{12} = \frac{90.8\times(8.5)^3}{12} = 4{,}647 \text{ in.}^4$$

$$I_{comp} = \sum I_0 + \sum Ad^2 = 464{,}979 + 265{,}377 = 730{,}356 \text{ in.}^4$$

$$S_b = \frac{I}{y_{bot}} = 730{,}356 \div 41.80 = 17{,}473 \text{ in.}^3 \quad \text{(Bottom of Beam)}$$

$$S_t = \frac{I}{y_{top}} = 730{,}356 \div 12.20 = 59{,}865 \text{ in.}^3 \quad \text{(Top of Beam)}$$

A3.3—Dead Load Analysis—Interior Girder

A3.3.1—Components and Attachments, *DC*

A3.3.1.1—Noncomposite Dead Loads, DC_1

Girder Self Weight: = 0.822 kip/ft *PCI Design Manual*

Diaphragms: = 0.150 kip/ft

Slab + haunch:

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

Total per Girder DC_1 = 1.90 kip/ft

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

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

A3.3.1.2—Composite Dead Load, DC_2

Concrete Barriers: LRFD Design 4.6.2.2.1

Assuming equal distribution among 4 beams

$$(2\times 0.500 \text{ kip/ft}) \div 4 = 0.25 \text{ kip/ft}$$

$$V_{DC_2} = 0.25\,\text{kip/ft}\times\frac{80\,\text{ft}}{2} = 10 \text{ kips}$$ At support

$$M_{DC_2} = \frac{1}{8}\times 0.25\,\text{kip/ft}\times(80\,\text{ft})^2 = 200 \text{ kip-ft}$$ At midspan

A3.3.2—Wearing Surface, *DW*

Asphalt Overlay: $\frac{2.5\,\text{in.}}{12\,\text{in./ft}} \times 27\,\text{ft} \times 0.144\,\text{kcf} \div 4\,\text{beams} = 0.203\ \text{kip/ft}$

Overlay thickness was not field measured. Use $\gamma_{DW} = 1.5$

$V_{DW} = 0.203\,\text{kip/ft} \times \frac{80\ \text{ft}}{2} = 8.12\ \text{kips}$ At support

$M_{DW} = \frac{1}{8} \times 0.203\ \text{kip/ft} \times 80^2 = 162\ \text{kip-ft}$ At midspan

A3.4—Live Load Analysis—Interior Girder

A3.4.1—Compute Live Load Distribution Factors, *g*

AASHTO LRFD Type (k) cross-section LRFD Design Table 4.6.2.2.1-1

Longitudinal Stiffness Parameter, K_g:

$K_g = n\,(I + Ae_g^{\,2})$ LRFD Design Eq. 4.6.2.2.1-1

$n = \frac{E_B}{E_D} = \frac{4.07 \times 10^3\,\text{ksi}}{3.64 \times 10^3\,\text{ksi}} = 1.12$

$A = 789\ \text{in.}^4$

$I = 260{,}730\ \text{in.}^4$

$L = 80\ \text{ft}$

$t_s = 8.5\ \text{in.}$

$e_g = \text{girder depth} - Y_b + \text{haunch} + t_s/2$

$= (54 - 24.73) + 1 + \frac{8.5}{2}$

$= 34.52\ \text{in.}$

$K_g = 1.12\,(260{,}730 + 789 \times 34.52^2)$

$= 1{,}345{,}038\ \text{in.}$

$\frac{K_g}{12\,Lt_s^{\,3}} = \frac{1{,}345{,}038}{12 \times 80 \times 8.5^3} = 2.28$

A3.4.1.1—Distribution Factor for Moment, g_m (LRFD Design Table 4.6.2.2.2b-1)

One Lane Loaded:

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

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

$$= 0.514$$

Two or More Lanes Loaded:

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

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

$$= 0.724 > 0.514$$

Use $g_m = 0.724$

A3.4.1.2—Distribution Factor for Shear, gv (LRFD Design Table 4.6.2.2.3a-1)

One Lane Loaded:

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

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

$$= 0.70$$

Two or More Lanes Loaded:

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

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

$$= 0.849 > 0.70$$

use $g_v = 0.849$

A3.4.2—Compute Maximum Live Load Effects

A3.4.2.1—Maximum Design Live Load (HL-93)—Moment at Midspan

Note: The general rule for simple spans carrying moving concentrated loads states that the maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support. It should be understood that locating the precise critical section and load position for rating depends on the combined influence of dead load, live load, member capacity, and load factors that make up the general rating factor equation. For simplicity and illustrative purposes only, the moment at midspan is used to closely approximate the maximum moment. See also Example A1, which illustrates that for a beam with a constant section capacity throughout the maximum moment region and a long span, the resulting rating factor obtained by a refined analysis yields only a small difference compared to the rating factor obtained from the maximum moment approximated at midspan.

Calculated by statics with the load centered at midspan:

Design Lane Load Moment = $0.64 \text{ klf} \times \dfrac{(80 \text{ ft})^2}{8}$ = 512 kip-ft

Design Truck Moment = $\dfrac{\left(8^k + 32^k\right) \times 40 \text{ ft} \times 26 \text{ ft}}{80 \text{ ft}} + \dfrac{32^k \times 80 \text{ ft}}{4}$ = 1,160 kip-ft Governs

Tandem Axles Moment = $25^k \times 38$ ft = 950 kip-ft

IM = 33 percent — LRFD Design Table 3.6.2.1-1

M_{LL+IM} = 512 + 1,160 × 1.33

= 2,054.8 kip-ft

Distributed Live Load Moment at Midspan:

$$
\begin{aligned}
M_{LL+IM} &= 2,054.8 \times g_m \\
&= 2,054.8 \times 0.724 \\
&= 1,487.7 \text{ kip-ft}
\end{aligned}
$$

A3.5—Compute Nominal Flexural Resistance at Midspan

f_{ps} = $f_{pu}\left(1 - k\dfrac{c}{d_p}\right)$ — LRFD Design Eq. 5.6.3.1.1-1

k = 0.28 for low-relaxation strands — LRFD Design Table 5.6.3.1.1-1

f_{pu} = 270 ksi

d_p = distance from extreme compression fiber to the centroid of the prestressing tendons

	Strands	y	Strands × y
Layer 1	12	2	24
Layer 2	12	4	48
Layer 3	8	6	48
Total	32		120

$\bar{y} = \dfrac{strands \times y}{strands} = \dfrac{120}{32}$

$\bar{y}$ = 3.75 in. distance from bottom of girder to centroid of prestressing strands

$d_p = (54+1+8.5) - 3.75 = 59.75$ in.

c = distance from the neutral axis to the compressive face

To compute c, assume rectangular section behavior. (Neglect any nonprestressed reinforcement.)

Given A_{ps} = 0.153 in.^{2} for $^1/_2$-in. diameter Low-Relaxation strands:

$$c = \frac{A_{ps} f_{pu}}{\alpha_1 f'_c \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}}$$ LRFD Design Eq. 5.6.3.1.1-4

A_{ps} = 4.896 in^2

b = be = 102 in (Effective Flange Width of Deck)

f'_c = 4.0 ksi (Deck Concrete Strength)

β_1 = 0.85

α_1 = 0.85

$$c = \frac{4.896\,\text{in.}^2 \times 270\,\text{ksi}}{0.85 \times 4.0\,\text{ksi} \times 0.85 \times 102\,\text{in.} + 0.28 \times 4.896\,\text{in.}^2 \times \dfrac{270\,\text{ksi}}{59.75\,\text{in.}}}$$ LRFD Design 5.6.2.2

$a = \beta_1 c = 0.85 \times 4.39 = 3.73$ in. < ts = 8.5 in. LRFD Design 5.6.2.2

Therefore, the rectangular section behavior assumption is valid.

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

Nominal Flexural Resistance (Midspan): LRFD Design Eq. 5.6.3.2.2-1

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) = 4.896 \times 264.4\left(59.75 - \frac{3.73}{2}\right)\frac{1}{12} = 6{,}244.4 \text{ kip-ft}$$

A3.6—Maximum Reinforcement 6A.5.5

The factored resistance (φ factor) of compression controlled sections shall be limited in accordance with LRFD Design Article 5.6.2.1. This approach limits the capacity of over-reinforced (compression controlled) sections. C6A.5.5

The net tensile strain, ε_t, is the tensile strain at nominal strength and determined by strain compatibility using similar triangles. LRFD Design C5.6.2.1

Given an allowable concrete strain of 0.003 and depth to neutral axis c = 4.39 in. and a depth from the extreme concrete compression fiber to the center of gravity of the prestressing strands d_p = 59.75 in. LRFD Design C5.6.2.1

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

$$\frac{0.003}{4.39\text{ in.}} = \frac{\varepsilon_t}{59.75\text{ in.} - 4.39\text{ in.}}$$

$$\varepsilon_t = 0.0378$$

For $\varepsilon_t = 0.0378 > 0.005$, the section is tension controlled and Resistance Factor φ shall be taken as 1.0. LRFD Design 5.6.2.1, 5.5.4.2

A3.7—Minimum Reinforcement 6A.5.6

Amount of reinforcement must be sufficient to develop M_r equal to the lesser of: LRFD Design 5.6.3.3

M_{cr} or 1.33 M_u

$$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$$

Load	Load Factor, γ
DC	1.25
DW	1.50
LL	1.75

LRFD Design Tables 3.4.1-1, 3.4.1-2

M_R = $\varphi M_n = (1.0)\ (6,244.4) = 6,244.4$ kip-ft

1. $1.33M_u$ =
$1.33\ [1.75\ (1,487.7) + 1.25\ (1,520 + 200) + 1.5\ (162)]$ No Good

= 6,645.3 kip-ft > 6,244.4 kip-ft

2. M_{cr} = $\gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$ 6A.5.6, LRFD Design Eq. 5.6.3.3-1

M_{dnc} = $M_{DC1} = 1,520$ kip-ft

S_c = 17,473 in.^{3}

S_{nc} = 10,543 in.^{3}

Modulus of Rupture $f_r = 0.24\lambda\sqrt{f_c'}$ LRFD Design 5.4.2.6

$= 0.24(1)\sqrt{5} = 0.537$ ksi

λ = 1.0

f_{cpe} = compressive stress in concrete due to effective prestress force (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads.

$$f_{cpe} = \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b}$$

where P_{pe} = Effective Prestress Force

A3.7.1—Determine Effective Prestress Force, P_{pe}

$$P_{pe} = A_{ps} f_{pe}$$

Total Prestress Losses $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ immediately before transfer

LRFD Design Eq. 5.9.3.1-1

Effective Prestress $f_{pe} =$ Initial Prestress – Total Prestress Losses

A3.7.1.1—Loss Due to Elastic Shortening and/or External Loads, Δf_{pES}

LRFD Design 5.9.3.2.3a

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp}$$

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e^2}{I} - \frac{M_D e}{I}$$

Initial Prestress immediately prior to transfer = $0.75 f_{pu}$ for low-relaxation prestressing strands

LRFD Design Table 5.9.2.2-1

For estimating P_i immediately after transfer, use $0.90(0.75 f_{pu})$

LRFD Design C5.9.8.2.3a

P_i = $0.90 \times (0.75 \times 270 \text{ ksi})\ 32 \times 0.153$

= 892.3 kips

M_D = Moment due to Self-Weight of Member at Section of Maximum Moment (Midspan)

= $\frac{1}{8} \times 0.822 \times 80^2$

= 657.6 kip-ft

Y_b = 24.73 basic beam section

$\bar{y}$ = 3.75 in. distance from bottom of girder to centroid of prestressing strands

e = 24.73 – 3.75 = 20.98 in.

eccentricity of P/S strands from *CG* of beam

LRFD Design 5.9.3.2.3a

$$f_{cgp} = \frac{892.3}{789} + \frac{892.3 \times 20.98^2}{260,741} - \frac{657.6 \times 12 \times 20.98}{260,741}$$

$$= 1.131 + 1.506 - 0.635$$

$$= 2.002 \text{ ksi}$$

$$E_p = 28,500 \text{ ksi}$$

LRFD Design Eq. C5.4.2.4-2

$$E_{ct} = 33,000(w_c)^{1.5}\sqrt{f'_{ci}}$$
$$= 33,000\ (0.145)^{1.5}\sqrt{4.0}$$
$$= 3,644 \text{ ksi}$$

LRFD Design Eq. C5.9.3.2.3a-1

$$\Delta f_{pES} = \frac{28,500}{3,644} \times 2.002$$

$$= 15.658 \text{ ksi}$$

A3.7.1.2—Approximate Lump Sum Estimate of Time-Dependent Losses, Δf_{pLT}

Time-dependent losses include shrinkage of concrete, creep of concrete, and relaxation of steel. For refined estimates:

LRFD Design Eq. 5.9.3.4.1-1

$$\Delta f_{pLT} = \left(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}\right)_{id} + \left(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS}\right)_{df}$$

for I-Girders, time-dependent losses can be approximated by: LRFD Design C5.9.3.3

LRFD Design Eq. 5.9.3.3-1

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR}$$

where $\gamma_h = 1.7 - 0.01H$ LRFD Design Eq. 5.9.3.3-2

Assuming a relative humidity, H, ranging between 40 to 100 percent.

For this example, assume H = 70 percent or refer to LRFD Design Figure 5.4.2.3.3-1

$$\gamma_h = 1.7 - 0.01(70) = 1.0$$

and: LRFD Design Eq. 5.9.3.3-3

$$\gamma_{st} = \frac{5}{1 + f'_{ci}}$$

$$\gamma_{st} = \frac{5}{1+4} = 1.0$$

and:

Δf_{pR} = an estimate of relaxation loss

LRFD Design 5.9.3.3

$$\Delta f_{pR} = 2.4 \text{ ksi}$$

and:

$$f_{pi} = 0.75 \times 270 \text{ ksi} = 202.5 \text{ ksi}$$

then:

$$\Delta f_{pLT} = 10.0 \times \frac{202.5 \times (32 \times 0.153)}{789} \times 1.0 \times 1.0 + 12.0 \times 1.0 \times 1.0 + 2.4$$

$$\Delta f_{pLT} = 26.97 \text{ ksi}$$

A3.7.1.3—Total Prestress Losses, Δf_{pT}

LRFD Design Eq. 5.9.3.1-1

$$\begin{aligned} \Delta f_{pT} &= \Delta f_{pES} + \Delta f_{pLT} \\ &= 15.658 + 26.97 \\ &= 42.63 \text{ ksi} \end{aligned}$$

$$\begin{aligned} f_{pe} &= \text{Initial Prestress} - \text{Total Prestress Losses} \\ &= 0.75 \times 270 - 42.63 = 159.87 \text{ ksi} \end{aligned}$$

$$\begin{aligned} P_{pe} &= 159.87 \times 32 \times 0.153 \\ &= 782.7 \text{ kips} \end{aligned}$$

Substitute in:

$$\begin{aligned} f_{pb} &= \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b} \\ &= \frac{782.7}{789} + \frac{(782.7)(20.98)}{10,543} \\ &= 2.550 \text{ ksi} \end{aligned}$$

$$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$$ LRFD Design Eq. 5.6.3.3.-1

$\gamma_1 = 1.6$
$\gamma_2 = 1.1$ (for bonded tendons)
$\gamma_3 = 1.0$ (for prestressing steel)

$$M_{cr} = 1.0\left[\left(1.6(0.537) + 1.1(2.550)\right)\frac{17,473}{12} - 1,520\left(\frac{17,473}{10,543} - 1\right)\right]$$

$$M_{cr} = 1.0\left[\left(0.859 + 2.803\right)\frac{17,473}{12} - 1,520\left(0.657\right)\right]$$

Therefore, $M_{cr} =$ 4,333.5 kip-ft and:

$1.33M_u =$ 6,645.3 kip-ft (previously calculated)

$M_{cr} < 1.33\ M_u$ therefore, M_{cr} governs

$M_r =$ 6,244.4 kip-ft (previously calculated)

$M_r =$ 6,244.4 kip-ft > 4,333.5 kip-ft OK 6A.5.6

The minimum reinforcement check is satisfied.

A3.8—Compute Nominal Shear Resistance at First Critical Section

Note: Article 6A.5.8 of this Manual does not require a shear evaluation for the design load and legal loads if the bridge shows no visible sign of shear distress. Shear calculations shown here for HL-93 are for illustrative purposes only.

Shear Location:

Critical section for shear near the supports is the greater of d_v or $0.5d_v \cot\theta$ from the face of support. LRFD Design 5.7.3.2

Effective Shear Depth, d_v: LRFD Design 5.7.2.8

Maximum of:

i) distance between resultants of the tensile and compressive forces

ii) $0.9d_e$

iii) $0.72h$

The first critical section will, by inspection, be within the 12-ft debonded end region. Ten strands have been debonded at the ends.

LRFD Design Eq. 5.6.3.1.1-4

$$c = \frac{A_{ps} f_{pu}}{\alpha_1 f_c' \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

A_{ps} = $(32-10)(0.153) = 3.366 \text{ in}^2$

b = b_e = 102 in. (Effective flange width of deck)

α_1 = 0.85

β_1 = 0.85

f_c' = 4.0 ksi (Deck concrete strength)

k = 0.28 for low-relaxation strands

f_{pu} = 270 ksi

	Strands	y	Strands × y
Layer 1	8	2	16
Layer 2	8	4	32
Layer 3	6	6	36
Total	22		84

$$\bar{y} = \frac{\text{strands} \times y}{\text{strands}} = \frac{84}{22}$$

distance from bottom of beam to 22 strand centroid = 3.82 in.

d_p = $(54 + 1 + 8.5) - 3.82 = 59.68$ in.

$$c = \frac{3.366 \text{ in.}^2 \times 270 \text{ ksi}}{0.85 \times 4.0 \text{ ksi} \times 0.85 \times 102 \text{ in.} + 0.28 \times 3.366 \text{ in.}^2 \times \dfrac{270 \text{ ksi}}{59.68 \text{ in.}}}$$

c = 3.04 in. $a = \beta_1 c = 2.58$ in.

$$d_v = 59.68 - \frac{2.58}{2} = 58.4 \text{ in.}$$

For establishing the critical shear section assume: θ = 30°, a high assumption is conservative.

$$0.5 d_v \cot\theta = (0.5)\ (d_v)\ (\cot 30°)$$
$$= 0.87 d_v < d_v$$

Distance from face of support to centerline of bearing = 6 in. (12-in. bearing pads)

Distance from centerline of bearing to critical shear section:

= 58.4 in. + 6 in. LRFD Design 5.7.3.2

= 64.4 in.

= 5.37 ft

A3.9—Maximum Shear at Critical Section Near Supports

Calculated by statics with the loads applied no closer than 5.37 ft from the supports

$$V_{TANDEM} = 25^k \times \frac{(74.63\text{ ft}+70.63\text{ ft})}{80\text{ ft}} = 45.4\text{ kips}$$

$$V_{TRUCK} = \frac{32^k\,(74.63\text{ ft}+60.63\text{ ft})+8^k\,(46.63\text{ ft})}{80\text{ ft}} = 58.8\text{ kips (Governs)}$$

$$V_{LANE} = \frac{0.64\text{ klf}\,(74.63\text{ ft})^2}{2\times 80\text{ ft}} = 22.3\text{ kips}$$

IM = 33 percent

= 45.4 kips

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

Distributed V_{LL+IM} = 100.5 kips × 0.849 = 85.3 kips

Dead Load Shears:

From A3.3.1, DC_1 = 1.90 kip/ft and DC_2 = 0.25 kip/ft

From A3.3.2, DW = 0.203 kip/ft

V_{DC} = (1.90 klf + 0.25 klf)(0.5 × 80 ft − 5.37 ft) = 74.5 kips

V_{DW} = (0.203 klf) (0.5 × 80 ft − 5.37 ft) = 7.03 kips

A3.10—Compute Nominal Shear Resistance

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

$$V_n = V_s + V_c + V_p$$

LRFD Design Eqs. 5.7.3.3-1, 5.7.3.3-2

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

$V_p = 0.0$ as straight tendons are provided

Critical section for shear near the support is at 64.4 in. from centerline of bearing (within the debonded length). Transverse reinforcement provided at critical section: #4 vertical stirrups at 9-in. spacings.

Minimum Transverse Reinforcement

LRFD Design 5.7.2.5

effective web width, b_v = 8 in.

stirrup spacing, s = 9 in.

Grade 60 rebar, f_y = 60 ksi

$$A_v = 0.0316\sqrt{f'_c}\frac{b_v s}{f_y}$$ LRFD Design Eq. 5.7.2.5-1

$$= 0.0316\sqrt{5}\frac{(8)(9)}{60}$$

$$= 0.0815 \text{ in.}^2$$

Area provided 2 legs × 0.20 in.^{2} = 0.40 in.^{2} > 0.0815 in.^{2} OK

$$V_c = 0.0316\beta\sqrt{f'_c}b_v d_v$$ LRFD Design Eq. 5.7.3.3-3

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$ for α= 90° LRFD Design Eq. 5.7.3.3-4

$$V_n = 0.25 f'_c b_v d_v + V_p = 0.25 \times 5.0 \times 8\text{in.} \times 58.4\text{in.} + 0.0 = 584 \text{ kips}$$ LRFD Design Eq. 5.7.3.3-2

These equations are based on the Modified Compression Field Theory (MCFT) and require the determination of β and θ by detailed analysis. A simplified analysis using θ = 45° and β = 2.0 may be utilized for an initial evaluation before resorting to the MCFT, if necessary, for likely improved shear capacity. C6A.5.8

A3.10.1—Simplified Approach

θ = 45° β = 2.0

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

Effective Web Width: $b_v = 8$ in.

Effective Shear Depth: $d_v = 58.4$ in.

$$V_c = (0.0316)(2.0)\sqrt{5.0}(8)(58.4)$$

$$= 66.0 \text{ kips}$$

Steel:

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

#4 at 9 in.

$$A_v = 2 \times 0.20 = 0.40 \text{ in.}^2$$

$$V_s = \frac{(0.40)(60)(58.4)(\cot 45)}{9}$$

$$= 155.7 \text{ kips}$$

Total Nominal Shear Resistance, V_n:

$$V_c + V_s + V_p = 66.0 + 155.7 + 0.0 = 221.7 \text{ kips}$$

221.7 kips < 584 kips, $\therefore$ V_n = 221.7 kips

Φ = 0.9 LRFD Design 5.5.4.2

φV_n = 0.9 × 221.7= 199.5 kips

Maximum distributed shears at critical section (HL-93 Inventory Loading):

V_{LL+IM} = 85.3 kips

V_{DC} = 74.5 kips

V_{DW} = 7.03 kips

LRFD Design Tables 3.4.1-1, 3.4.1-2

Load	Load Factor γ
DC	1.25
DW	1.50
LL	1.75

Factored Shear:

$$V_u = (1.75)(85.3) + (1.25)(74.5) + (1.5)(7.03)$$
$$= 252.9 \text{ kips} > 199.5 \text{ kips} \quad \text{No Good}$$

Try MCFT approach.

A3.10.2—MCFT Approach

LRFD Design Eq. 5.7.2.8-1

Shear stress on the concrete:

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

$$= \frac{252.9}{(0.9)(8)(58.4)} = 0.601 \text{ ksi}$$

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

OK, Therefore LRFD Design Appendix B5, Table B5.2-1 can be used.

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

Live Load Moments at first critical section determined by statics:

$$M_{TRUCK} = \frac{32^k \times 5.37 \text{ ft}(74.63 \text{ ft} + 60.63 \text{ ft}) + 8^k \times 5.37 \text{ ft}(46.63 \text{ ft})}{80 \text{ ft}} = 315.6 \text{ kip-ft}$$

$$M_{LANE} = 0.64 \text{klf} \times \frac{(74.63 \text{ ft})^2}{2 \times 80 \text{ ft}} \times 5.37 \text{ ft} = 119.6 \text{ kip-ft}$$

$$M_{LL+IM} = 119.6 \text{ kip-ft} + 1.33 \times 315.6 \text{ kip-ft} = 539.6 \text{ kip-ft}$$

Distributed Moment:

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

Dead Load Moments at First Critical Section for Shear:

$M_{DC} = 0.5\ (1.90\text{ klf} + 0.25\text{ klf})\ (5.37\text{ ft})\ (80\text{ ft} - 5.37\text{ ft}) = 430.8$ kip-ft

$M_{DW} = 0.5\ (0.203\text{ klf})\ (5.37\text{ ft})\ (80\text{ ft} - 5.37\text{ ft}) = 40.7$ kip-ft

Load	Load Factor, γ
DC	1.25
DW	1.50
LL	1.75

LRFD Design Tables 3.4.1-1, 3.4.1-2

Factored Moment:

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

$= 1{,}282.9$ kip-ft

Following the approach in the LRFD Shear Design Flowchart in LRFD Design Appendix B5 and LRFD Design Table B5.2-1 in LRFD Design Appendix B5. Transfer Length 60 strand diameters = 30 in. < 64.4 in.

LRFD Design Figure CB5.2-5

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

The Modified Compression Field Theory (MCFT) follows an iterative process in LRFD Design Appendix B5:

$\frac{v}{f_c'} = \frac{0.601}{5.0} = 0.12$ (≤ 0.125, row 3 of LRFD Design Table B5.2-1

LRFD Design Figure CB5.2-5, LRFD Design Table B5.2-1

Assume $\varepsilon_x \leq -0.10 \times 10^{-3}$ ($\varepsilon_s \times 1{,}000 \leq -0.10$)

From LRFD Design Table B5.2-1 (row 3, column 2):

$\theta = 21.9°$ $\beta = 2.99$

Calculate ε_x:

LRFD Design Figure B5.2-3

LRFD Design Appendix B5.2

$$\varepsilon_x = \frac{\dfrac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}}{2\left(E_sA_s + E_pA_{ps}\right)} \le 0.001$$

A_{ps} = $22 \times 0.153 = 3.366$ in.^{2}

f_{po} = $0.7f_{pu} = 0.7 \times 270 = 189$ ksi

$$\varepsilon_x = \frac{\dfrac{|12\times1,282.9|}{58.4} + 0.5|252.9|(\cot 21.9°) - (3.366)(189)}{2(0 + 28,500\times3.366)}$$

$= -0.302 \times 10^{-3}$

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

LRFD Design Figure CB5.2-5

$$\begin{aligned} A_c &= \text{Area below } h/2 \\ &= (8)(26) + 1/2\ (8 + 26)(9) + (10)(8) \\ &= 441 \text{ in.}^2 \end{aligned}$$

LRFD Design Eq. B5.2-5

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

$$\varepsilon_x = \frac{\dfrac{12\times1,282.9}{58.4} + (0.5)(252.9)(\cot 21.9°) - (3.366)(189)}{2\left[(4,030)(441) + 0 + (28,500)(3.366)\right]}$$

$= -0.015\times10^{-3} >$ assumed $\varepsilon_x \le -0.10\times10^{-3}$

Assume $\varepsilon_x \le 0$

From LRFD Design Table B5.2-1 (row 3, column 4):

LRFD Design Eq. 5.7.3.4.2-1, 5.7.3.4.2-2, 5.7.3.4.2-3

$\theta = 23.7°$ $\beta = 2.87$

Calculate ε_x:

$$\varepsilon_x = \frac{\dfrac{12\times1,282.9}{58.4} + (0.5)(252.8)(\cot 23.7°) - (3.366)(189)}{2\left[(4,030)(441) + 0 + (28,500)(3.366)\right]}$$

$= -0.023 \times 10^{-3} <$ assumed $\varepsilon_x \le 0$ OK

Note $-0.023 \times 10^{-3} > -0.05 \times 10^{-3}$ (adjacent column), $\therefore$ no further interactions

Calculate V_n:

LRFD Design Eq. 5.7.3.3-3

$$\begin{aligned} V_c &= 0.0316\,\beta\sqrt{f'_c}b_vd_v \\ &= (0.0316)(2.87)\sqrt{5}(8)(58.4) \\ &= 94.75 \text{ kips} \end{aligned}$$

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

LRFD Design Eq. C5.7.3.3-4

$$= \frac{(0.39)(60)(58.4)(\cot 23.7°)}{9}$$

$$= 345.9 \text{ kips}$$

Total Nominal Shear Resistance:

$$V_n = V_c + V_s$$

LRFD Design Eq. 5.7.3.3-1

$$= 94.75 + 345.9 = 440.7 \text{ kips}$$

(versus 221.7 by simplified method)

$0.25 f_c' b_v d_v + V_p$ = 584 kips (previously calculated)

LRFD Design Eq. 5.7.3.3-2

440.7 kips < 584 kips, $\therefore$ V_n = 440.7 kips

φV_n = 0.9 x 440.7 = 396.6 kips

Alternatively, LRFD Design 5.7.3.4.2 allows for solving for β and θ directly:

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps} f_{po}\right)}{E_s A_s + E_p A_{ps}}$$

LRFD Design Eq. 5.7.3.4.2-4

$$= \frac{\frac{|12 \times 1{,}282.9|}{58.4} + |252.9| - (3.366)(189)}{(0 + 28{,}500 \times 3.366)}$$

$$= -0.0012$$

use 0

LRFD Design 5.7.3.4.2

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)}$$

LRFD Design Eq. 5.7.3.4.2-1

$$= \frac{4.8}{(1 + 750(0))}$$

$$= 4.8$$

$\theta = 29 + 3{,}500\,\varepsilon_s$ LRFD Design Eq. 5.7.3.4.2-3

$= 29 + 3{,}500\,(0)$
$= 29$

Calculate V_n:

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

$$= 0.0316(4.8)\sqrt{5}(8)(58.4)$$

$$= 158.46 \text{ kips}$$

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

$$= \frac{(0.39)(60)(58.4)\cot(29)}{9}$$

$$= 273.9 \text{kips}$$

Total Nominal Shear Resistance:

$V_n = V_c + V_s$

$= 158.5 + 273.9$

$= 432.40$ kips

(versus 440.7 kips by LRFD Design Appendix B5)

A3.10.3—Check Longitudinal Reinforcement (LRFD Design 5.7.3.5)

Tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be proportioned to satisfy LRFD Design Eq. 5.7.3.5-1. “Any lack of full development shall be accounted for.”

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_c} + \left[\left|\frac{V_u}{\varphi_v} - V_p\right| - 0.5V_s\right]\cot\theta$$ LRFD Design Eq. 5.7.3.5-1

Calculate minimum required tensile capacity:

$$V_s = 345.9 \text{ kips} > \frac{V_u}{\varphi} = \frac{252.9}{0.9} = 281.0 \quad \text{use } 281.0 \text{ kips}$$

The right side of LRFD Design Eq. 5.7.3.5-1 yields:

$$= \frac{|(1{,}282.9)(12)|}{(58.4)(1.0)} + \left(\left|\frac{252.9}{0.9}\right| - 0.5(281.0)\right)\cot 23.7°$$

$= 583.7$ kips

Transfer Length: LRFD Design 5.9.4.3.1

ℓ_t = 60 strand diameters

= 60×0.5 in.= 30 in.

Development Length: LRFD Design Eq. 5.9.4.3.2-1

$$\ell_d \geq \kappa\left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b$$

where κ = 1.6 for pretensioned members with a depth greater than 24.0 in.

$$\ell_d \geq 1.6 \times (264.4 - \frac{2}{3} \times 159.87) \times 0.5 = 126.3 \text{ in}$$

The 22 effective strands at the critical shear section are bonded over the full length of the beam. The section at 64.4 in. from centerline of the bearing is between the transfer length (30 in. from end of beam, 26 in. from centerline of bearing) and the development length (126.3 in. from end of beam, 120.3 in. from centerline of bearing). Use a linear growth in strand capacity from f_{pe} at the transfer length to f_{ps} at the development length. LRFD Design 5.9.4.3.2 LRFD Design 5.9.4.3.1

ℓ_{px} = 64.4 in. to critical section LRFD Design Eq. 5.9.4.3.2-3

$$f_{px} = f_{pe} + \frac{\ell_{px} - 60d_b}{\ell_d - 60d_b}(f_{ps} - f_{pe})$$

$$f_{px} = 159.87 + \frac{64.4 - 30}{126.3 - 30}(264.4 - 159.87) = 197.21$$

The left side of LRFD Design Eq. 5.7.3.5-1 yields:

$$= f_{px} \times A_{ps} = 197.21 \text{ ksi} \times 3.366 \text{ in.}^2 = 663.8 \text{ kips}$$

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_c} + \left[\left|\frac{V_u}{\varphi_v} - V_p\right| - 0.5V_s\right]\cot\theta \text{ reduces to}$$

$663.8 \text{ kips} \geq 583.7 \text{ kips}$ OK

A3.11—Compute Nominal Shear Resistance at Stirrup Change/Quarter Point (6A.5.8) C6A.5.8

Multiple locations need to be checked for shear. Typically, locations near the quarter point could be critical because the corresponding moment may be quite low.

(20 ft from centerline of bearing)

Effective Shear Depth, d_v, is based upon 32 strands.

$\rightarrow$ check transfer length LRFD Design 5.9.4.3

60 strand diameters = 30 in.

debonded length = 12 ft

All 32 strands are bonded at: 12 ft + 30 in. = 14.5 ft < 20 ft OK

$$d_v = d_e - \frac{a}{2}$$ LRFD Design 5.7.2.8

$$d_e = h - \bar{y} = 63.5 - 3.75 = 59.75 \text{ in.}$$

$d_v = d_e - \frac{a}{2}$

a = 3.73 in. (from Article A3.5 of this example)

d_v need not be less than the greater of minimum effective shear depth limits $0.9d_e$ or $0.72h$.

d_v = 57.89 in. > $0.9d_e = 53.78$ in.

> $0.72h$ = 45.72 in.

If we base d_v on: LRFD Design Eq. C5.7.2.8-1

$$d_v = \frac{M_n}{A_s f_y + A_{ps} f_{ps}}$$

including the effects of development, then:

$$d_v = \frac{6244.4 \text{ kip-ft} \times 12 \text{ in./ft}}{0 + \left(32 \times 0.153 \text{ in.}^2\right) 264.4 \text{ ksi}} = 57.89 \text{ in.}$$

A3.12—Maximum Shear at Stirrup Change

(20 ft from centerline of bearing)

for HL-93 loading

Calculated by statics with the loads applied no closer than 5.37 ft from the support

$V_{TANDEM} = 25^k \times \frac{(60 \text{ ft} + 56 \text{ ft})}{80 \text{ ft}} = 36.25 \text{ kips} = 36.25 \text{ kips}$

$V_{TRUCK} = \frac{32^k\,(60 \text{ ft} + 46 \text{ ft}) + 8^k\,(32 \text{ ft})}{80 \text{ ft}} = 45.6 \text{ kips}$ Governs

$V_{LANE} = \frac{0.64 \text{ klf}\,(60 \text{ ft})^2}{2 \times 80 \text{ ft}}$

IM = 33 percent

V_{LL+IM} = 14.4 + 1.33 × 45.6 kips = 75.05 kips

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Dead Load Shears:

From A3.3.1, DC_1 = 1.90 kip/ft and DC_2 = 0.25 kip/ft

From A3.3.2, DW = 0.203 kip/ft

V_{DC} = (1.90 klf + 0.25 klf)(0.5 × 80 ft − 20 ft)

V_{DC} = 38 + 5 = 43 kips

V_{DW} = (0.203 klf) (0.5 × 80 ft − 20 ft)

V_{DW} = 4.1 kips

Minimum Transverse Reinforcement:

Effective Web Width:

LRFD Design Eq. 5.7.2.5-1

b_v = 8 in.

Spacing of Transverse Reinforcement:

s = 12 in.

$$A_v = 0.0316\sqrt{f'_c}\frac{b_v s}{f_y}$$

$$A_v = 0.0316\sqrt{5}\frac{(8)(12)}{60} = 0.113 \text{ in.}^2$$

Area provided 2 # 3 = 2 (0.11) = 0.22 in.² > 0.113 in.² OK

A3.12.1—Simplifed Approach

θ = 45°

β = 2.0

Concrete:

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

LRFD Design Eq. 5.7.3.3-3

Effective Shear Depth:

d_v = 57.89 in.

$$V_c = (0.0316)(2.0)\sqrt{5.0}(8)(57.89)$$
$$= 66.0 \text{ kips}$$

Steel:

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

#3 at 12 in.

A_v = 2 (0.110) = 0.22 in.²

$$V_s = \frac{(0.22)(60)(57.89)\cot 45°}{12}$$

V_s = 63.7 kips

Total Nominal Shear Resistance:

$$V_n = V_c + V_s$$
$$= 65.4 + 63.7 = 129.1 \text{ kips}$$

$$0.25 f'_c b_v d_v + V_p = 0.25 \times 5.0 \times 8 \text{ in.} \times 57.89 \text{ in.} + 0.0 = 578.9 \text{ kips}$$

LRFD Design Eq. 5.7.3.3-2

129.1 kips < 578.9 kips $\therefore$ V_n = 129.1 kips

φV_n = 0.9x129.1 = 116.2 kips

Factored Shear V_u:

$V_u = 1.75\ (63.7) + 1.25\ (43) + 1.5\ (4.1) =\ 171.4\,\text{kips}$

116.2 kips < 171.4 kips No Good

Try MCFT Approach.

A3.12.2—MCFT Approach

Shear stress on the concrete:

$$v \quad = \quad \frac{V_u - \varphi V_p}{\varphi b_v d_v} = \frac{172.6}{(0.9)(8)(57.89)} = 0.414 \text{ ksi}$$

LRFD Design Eq. 5.7.2.8-1

$$\frac{v}{f_c'} \quad = \quad \frac{0.414}{5} \quad = \quad 0.0828 < 0.25$$

OK

At Stirrup Change:

$$M_{TRUCK} \quad = \quad \frac{32^k \times 20\,\text{ft.}\left(60\,\text{ft} + 46\,\text{ft}\right) + 8^k \times 20\,\text{ft}\left(32\,\text{ft}\right)}{80\,\text{ft}} = 912.0 \text{ kip-ft}$$

$$M_{LANE} \quad = \quad 0.64\,\text{klf} \times \frac{\left(60\,\text{ft}\right)^2}{2 \times 80\,\text{ft}} \times 20\,\text{ft} = 288 \text{ kip-ft}$$

$$M_{LL+IM} \quad = \quad 288\,\text{kip-ft} + 1.33 \times 912\,\text{kip-ft} = 1{,}501 \text{ kip-ft}$$

$g_m M_{LL+IM}$ = (0.724) (1,501) = 1,087 kip-ft

M_{DC} = 0.5 (1.90 klf + 0.25 klf) (20 ft) (80 ft – 20 ft) = 1,290 kip-ft

M_{DW} = 0.5 (0.203 klf) (20 ft) (80 ft – 20 ft) =121.8 kip-ft

M_u = 1.75 (1087) + 1.25 (1,290) + 1.5 (121.8)

= 3,697.5 kip-ft

Following the approach in the LRFD Shear Design Flowchart in Appendix B5 and LRFD Design Table B5.2-1 in Appendix B5.

LRFD Design Appendix B5

Check upper limit of shear V_n

$$0.25\, f_c' b_v d_v + V_p = 0.25 \times 5.0 \times 8 \text{ in.} \times 57.89 \text{ in.} + 0.0 = 578.9 \text{ kips}$$

LRFD Design Eq. 5.7.3.3-2

$$\frac{v}{f_c'} \quad = \quad 0.0828 \quad \leq 0.100 \quad \text{(2nd row)}$$

LRFD Design Figure CB5.2-5, Table B5.2-1

A_{ps} = $32 \times 0.153 = 4.896$ in.²

f_{po} = $0.7\, f_{pu} = 0.7 \times 270 = 189$ ksi

LRFD Design Eq. B5.2-3

$$\varepsilon_x = \frac{\dfrac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps} f_{po}}{2\left(E_s A_s + E_p A_{ps}\right)} \le 0.001$$

$$\varepsilon_x = \frac{\dfrac{(12)(3,697.5)}{57.89} + 0.5(171.4)\cot\theta - (4.896)(189)}{2(0 + 28,500 \times 4.896)} \le 0.001$$

$$\varepsilon_x = 0.3071 \times 10^{-3} \cot\theta - 0.5694 \times 10^{-3}$$

Assume $\varepsilon_x \le 0.125 \times 10^{-3}$ ($\varepsilon_x \times 1,000 \le 0.125$)

From LRFD Design Table B5.2-1 (row 2, column 5):

$\theta = 24.9°$ $\beta = 2.91$

$$\varepsilon_x = 0.3071 \times 10^{-3} \cot\ 24.9° - 0.5694 \times 10^{-3} = 0.0922 \times 10^{-3}$$

The calculated ε_x is less than the assumed but not less than the adjacent ε_x value 0.0, ∴ the assumption was not too conservative OK

Calculate V_n:

LRFD Design Eq. 5.7.3.3-3

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

$$V_c = (0.0316)(2.91)\sqrt{5}(8)(57.89) = 95.2 \text{ kips}$$

LRFD Design Eq. 5.7.3.3-4

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

$$V_s = \frac{(0.22)(60)(57.89)(\cot 24.9°)}{12} = 137 \text{ kips}$$

LRFD Design Eq. 5.7.3.3-1

$$V_n = V_c + V_s = 95.2 + 137$$

$$V_n = 232.2 \text{ kips}$$

$0.25 f_c' b_v d_v + V_p = 578.9$ kips (previously calculated)

232.2 kips < 578.9 kips therefore V_n = 232.2 kips

$\varphi V_n = 0.9 \times 232.2 = 209.0$ kips

A3.12.3—Check Longitudinal Reinforcement (LRFD Design 5.7.3.5)

LRFD Design Eq. 5.7.3.5-1

$$A_{ps} f_{ps} + A_s f_y \ge \frac{|M_u|}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_c} + \left[\left|\frac{V_u}{\varphi_v} - V_p\right| - 0.5V_s\right]\cot\theta$$

$$V_s = 137 \text{ kips} < \frac{V_u}{\varphi} = \frac{171.4}{0.9} = 190.4 \text{ use } 137 \text{ kips}$$

The right side of LRFD Design Eq. 5.7.3.5-1 yields:

$$= \frac{(3,697.5)(12)}{(57.89)(1.0)} + \left(\frac{171.4}{0.9} - 0.5(137)\right)\cot 24.9° = 1,029 \text{ kips}$$

The 20 fully bonded strands are fully developed at this location (f_{ps} = 264.4 ksi). As a portion of the remaining ten strands are debonded, their development length from the end of the debonded zone is calculated by LRFD Design Eq. 5.9.4.3.2-1 with k = 2.0.

LRFD Design 5.9.4.3

$$\ell_d \geq k(f_{ps} - 2/3 f_{pe})d_b$$

LRFD Design Eq. 5.9.4.3.2-1

$$\ell_d \geq 2 \times (264.4 - \frac{2}{3} \times 159.87)0.5 = 157.8 \text{ in.}$$

Check to see that the debonded strands are fully developed at the stirrup change location.

157.8 in. + 12 ft = 25.2 ft > 20 ft

Therefore, the strands are not fully developed and f_{px} must be determined.

Using a linear increase from f_{pe} at the transfer length to f_{ps} at the development length

From end of debonded zones

$$\ell_{px} = (20 \text{ ft} - 12 \text{ ft}) \times 12 \text{ in./ft} = 96 \text{ in.}$$

$$f_{px} = 159.87 + \frac{96 - 30}{157.8 - 30}(264.4 - 159.87) = 213.9 \text{ ksi}$$

Then, the left side of LRFD Design Eq. 5.7.3.5-1 yields:

$$= \quad 264.4 \times 22 \times 0.153 + 213.9 \times 10 \times 0.153 = 1,217 \text{ kips}$$ OK

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_c} + \left[\left|\frac{V_u}{\varphi_v} - V_p\right| - 0.5V_s\right]\cot\theta$$

LRFD Design Eq. 5.7.3.5-1

reduces to: 1,217 kips ≥ 1,029 kips OK

A3.12.4—Summary

Table A3.12.4-1—Summary of Moments and Shears

Location	Support	Critical Shear	Stirrup Change	Midspan
x/L	0.0	0.067	0.25	0.5
X, ft	0.0	5.37	20	40
V_{DC1}, kips	76	65.8	38	—
V_{DC2}, kips	10	8.7	5	—
V_{DW}, kips	8.12	7.03	4.1	—
$g_m V_{LL+IM}$, kips	—	85.3	63.7	—
V_n, kips, simplified	—	221.7	129.1	—
V_n, kips, MCFT	—	440.7	232.2	—
M_{DC1}, kip-ft	—	380.7	1140	1520
M_{DC2}, kip-ft	—	50.1	150	200
M_{DW}, kip-ft	—	40.7	121.8	162
$g_m M_{LL+IM}$, kip-ft	—	390.5	1087	1487.7
M_n, kip-ft	—	—	—	6244.4

A3.13—General Load Rating Equation (6A.4.2)

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

Eq. 6A.4.2.1-1

A3.13.1 Evaluation Factors (for Strength Limit State)

A3.13.1.1—Resistance Factor, φ (LRFD Design 5.5.4.2)

φ = 1.0 for flexure (previously determined to be a tension-controlled section; see Article A3.6)

φ = 0.9 for shear

A3.13.1.2—Condition Factor, φ_c (6A.4.2.3)

φ_c = 1.0 No member deterioration, NBI Item 59 Code = 6

A3.13.1.3—System Factor, φ_s (6A.4.2.4)

φ_s = 1.0 4-girder bridge with spacing > 4 ft

A3.13.2—Design Load Rating (6A.4.3)

A3.13.2.1—Strength I Limit State (6A.5.4.1)

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

A3.13.2.1a—Inventory Level

Table 6A.4.2.2-1

Load	Load Factor
DC	1.25
DW	1.50 Overlay thickness was not field measured.
LL	1.75

Flexure at Midspan:

$$RF = \frac{(1.0)(1.0)(1.0)(6244.4) - \left[(1.25)(1520 + 200) + (1.5)(162)\right]}{(1.75)(1,487.7)}$$

$$= 1.48$$

The shear rating factors for Design Load Rating are calculated for illustration purposes only. In-service concrete bridges that show no visible signs of shear distress need not be checked for shear during design load or legal load ratings. 6A.5.8

Shear at First Critical Shear Section (64.4 in. from centerline of bearing):

1. Simplified Approach

$$RF = \frac{(1.0)(1.0)(0.9)(221.7) - \left[(1.25)(65.8 + 8.7) + (1.50)(7.03)\right]}{(1.75)(85.3)}$$

$$= 0.64$$

2. MCFT

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

$$= 1.96$$

3. General Procedure

$$RF = \frac{(1.0)(1.0)(0.9)(432.4) - \left[(1.25)(65.8+8.7)+(1.50)(7.03)\right]}{(1.75)(85.3)}$$

$$= 1.91$$

Shear at Stirrup Change (20 ft from centerline of bearing):

1. Simplified Approach

$$RF = \frac{(1.0)(1.0)(0.9)(129.1) - [(1.25)(38+5)+(1.50)(4.1)]}{(1.75)(63.7)}$$

$$= 0.51$$

2. MCFT

$$RF = \frac{(1.0)(1.0)(0.9)(232.2) - [(1.25)(38+5)+(1.50)(4.1)]}{(1.75)(63.7)}$$

$$= 1.34$$

A3.13.2.1b—Operating Level

For Strength I Operating Level only the live load factor changes; therefore the rating factor can be calculated by direct proportions.

Table 6A.4.2.2-1

Load	Load Factor, γ
DC	1.25
DW	1.50
LL	1.35

Flexure at Midspan:

$$RF = 1.48 \times \frac{1.75}{1.35}$$

$$= 1.92$$

Shear: Prestressed concrete shear capacity is load-dependent. Therefore, the change in the rating factor using MCFT will not be linear with the change in the live-load factor. The Operating Design Load Rating for shear is not illustrated here.

This example has illustrated the calculation for the shear rating factor with the longitudinal yield check at the first critical section for shear and at a stirrup change. Due to the variation of resistances for shear along the length of this prestressed concrete I-beam, it is not certain that these two locations govern for the Strength I limit state. A systematic evaluation of the shear and longitudinal yield criteria based on shear-moment interaction should be performed along the length of the beam.

Flexure rating should be checked at maximum moment sections and at sections where there are changes in flexural resistance.

The checks performed for minimum and maximum reinforcement will also vary along the length; the LRFD Design specification requires that these checks are required to be satisfied at each cross section.

A3.13.2.2—Service III Limit State (Inventory Level) (6A.5.4.1)

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

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

f_{pb} = Compressive stress due to effective prestress

= 2.550 ksi (from Article A3.7.1.3 of this example)

Allowable Tensile Stress $= 0.19\sqrt{f_c'}$ LRFD Design 5.9.2.3.2b

$= 0.19\sqrt{5}$

$= 0.425$ ksi

$f_R = 2.550 + 0.425$

$= 2.975$ ksi

Determine Dead Load Stresses at Midspan:

From A3.3.1, M_{DC1} = 1,520 kip-ft and M_{DC2} = 200 kip-ft

From A3.3.2, M_{DW} = 162 kip-ft

From A3.2, $S_{b\,(nc)}$ = 10,543 in.^{3} $S_{b\,(comp)}$ = 17,473 in.

$$f_{DC} = \frac{1{,}520 \times 12}{10{,}543} + \frac{200 \times 12}{17{,}473} \qquad = 1.87 \text{ ksi}$$

$$f_{DW} = \frac{162 \times 12}{17{,}473} \qquad = 0.11 \text{ ksi}$$

Total f_D $= 1.98$ ksi

Live Load Stress at Midspan:

From A3.4.2, M_{LL+IM} = 1,487.7 kip-ft

From A3.2, $S_{b\,(comp)}$ =17,473 in.^{3}

$$f_{LL+IM} = \frac{1{,}487.7 \times 12}{17{,}473} = 1.02 \text{ ksi}$$

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

$$= 1.22$$

A3.13.3—Legal Load Rating (6A.4.4)

Inventory Design Load Rating RF > 1.0, therefore the legal load ratings do not need to be performed and no posting is required. 6A.4.3.1

A3.13.4—Permit Load Rating (6A.4.5)

Permit Type: Special, single-trip, mix with traffic, no escort
Permit Weight: 220 kips

The permit vehicle is shown in Example A1, Figure A1A.1.10-1.

ADTT (one direction): 5,000

From Live-Load Analysis by Computer Program:

Undistributed Maximum M_{LL} = 2,950.5 kip-ft

Undistributed Maximum V_{LL} = 157.9 kips

A3.13.4.1—Strength II Limit State (6A.5.4.2.1)

Load	Load Factor, γ
DC	1.25
DW	1.5
LL	1.2

LRFD Design Tables 3.4.1-1, 3.4.1-2; Table 6A.4.5.4.2a-1

Use One-Lane Distribution Factor and divide out the 1.2 multiple presence factor. 6A.4.5.4.2b

6A.4.5.5

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

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

IM = 20% (Riding surface condition verified by inspection: Minor Deviations)

Maximum Live Load Effect:

$$M_{LL+IM} = (2950.5)(0.428)(1.20)$$

$$= 1515.4 \text{ kip-ft} \quad \text{at midspan}$$

$$V_{LL+IM} = (157.9)(0.583)(1.20)$$

$$= 110.5 \text{ kips}$$

φ factors are the same as those for the design calculations. See Article A3.13.1.

A3.13.4.1a—Flexure

$$RF = \frac{(1.0)(1.00)(1.0)(6244.4) - [(1.25)(1,520 + 200) + (1.5)(162)]}{(1.2)(1515.4)}$$

$$= 2.11 > 1.0$$

OK

Shear evaluation is required for Permit Load Rating. 6A.5.8

A3.13.4.1b—Shear (Using MCFT)

$$RF = \frac{(1.0)(1.0)(0.9)(440.7) - [(1.25)(74.5) + (1.50)(7.03)]}{(1.2)(110.5)}$$

$$= 2.21 > 1.0$$

OK

Shear resistance taken from HL-93. Acceptable and conservative as long as M_u and V_u for HL-93 are both ≥ M_u and V_u for permit. Must be recalculated if permit values are greater.

A3.13.4.2—Service I Limit State (Optional) (6A.5.4.2.2b)

$\gamma_L = \gamma_{DC} = \gamma_{DW} = 1.0$ Table 6A.4.2.2-1

LRFD distribution analysis methods as described in LRFD Design Article 4.6.2 should be used. 6A.4.5.4.2a

$g_m = 0.724$ A3.4.1.1

Distributed Live-Load Effect:

Dead Load Moments at Midspan:

From A3.3.1, M_{DC1} = 1520 kip-ft and M_{DC2} = 200 kip-ft

From A3.3.2, M_{DW} = 162 kip-ft

$M_{LL+IM} = (2,950.5)(0.724)(1.2) = 2,563.4$ kip-ft

$M_{DC} + M_{DW} + M_{LL+IM} = (1,520 + 200) + 162 + 2,563.4 = 4,445.4$ kip-ft

A3.13.4.2a—Simplified Check Using 0.75M_n (C6A.4.2.2)

Nominal flexural resistance: M_n = 6,244.4 kip-ft
(use nominal, not factored resistance)

$0.75M_n = 0.75 \times 6{,}244.4 = 4{,}683.3$ kip-ft > 4,445.4 kip-ft OK

$$\text{Moment Ratio} = \frac{0.75M_n}{M_{DC} + M_{DW} + M_{LL+IM}} = \frac{4,683.3}{4,445.4} = 1.05 > 1.0$$ OK

A3.13.4.2b—Refined Check Using 0.9f_y

Calculate stress in outer reinforcement at Midspan. Stress due to moments in excess of the cracking moment acts upon the cracked section. The moments up to the cracking moment cause stress in the reinforcement equal to the effective prestress.

$f_R = 0.9F_y = 0.9\left(0.9F_{pu}\right) = 0.9\left(0.9 \times 270\right) = 218.7$ ksi 6A.5.4.2.2b, Table 6A.5.4.2.2b-1

M_{cr} = 4,333.5 kip-ft (previously calculated; see Article A3.7.1.3)

Effective prestress: (0.75 × 270 – 42.63) = 159.87 ksi (previously calculated; see Article A3.7.1.3)

$M_{DC} + M_{DW} + M_{LL+IM} - M_{cr} = 4,445.4 - 4,333.5 = 111.9$

Section Properties for the Cracked Composite Section:

b_{trans} = 102 in. × 0.89 = 90.8 in. (see Article A3.2)

h = 54 in. + 1 in. + 8.5 in. = 63.5 in.

A_{ps} = 32 × 0.153 in.^{2} = 4.896 in.^{2}

Modular ratio, n:

$n \quad = \quad \dfrac{E_{ps}}{E_{beam}} = \dfrac{28.5\times10^3}{4.07\times10^3} = 7$

$A_{trans} \quad = \quad 4.896 \text{ in.}^2 \times 7 = 34.3 \text{ in.}^2$

$y \quad = \quad 3.75$ in. (see Article A3.5)

Outer strand $y = 2$ in.

Assume neutral axis is in the slab.

$$c = \frac{\left(\frac{c}{2}\right)(b_{trans}\times c) + (h-y)(A_{trans})}{(b_{trans}\times c) + (A_{trans})}$$

$$c = \frac{\frac{c}{2}(90.8)c + (63.5-3.75)(34.3)}{(90.8)c + 34.3}$$

$$45.4c^2 + 34.3c - 2{,}049.4 = 0$$

Solving for c:

$$c = \frac{-34.3 \pm \sqrt{34.3^2 - 4(45.4)(-2049.4)}}{2(45.4)}$$

$c = 6.35$ in.

$$I_{cr} = \frac{1}{12}(90.8)(6.35)^3 + (90.8)(6.35)\left(\frac{6.35}{2}\right)^2 + (34.3)(63.5-3.75-6.35)^2$$

$$= 105{,}558 \text{ in.}^4$$

Stress beyond the effective prestress (increase in stress after cracking):

$$f_s = n\frac{M_y}{I} = 7\,\frac{(111.9)(12)(63.5-2.0-6.35)}{105{,}558} = 4.9 \text{ ksi}$$

Stress in the reinforcement at Permit crossing Service I:

$f_s = 159.87 + 4.9 = 164.8 \text{ ksi} < f_R = 0.9F_y = 218.7 \text{ ksi}$ OK

Stress Ratio $= \dfrac{0.9 f_y}{f_s} = \dfrac{218.7}{164.8} = 1.33 > 1.0$ OK

All permit checks for an interior girder are satisfied.

A3.13.5—IoH Tier 1 Load Rating (MBEIoH 2A.4.3)

Inventory Design Load Rating *RF* > 1.0, therefore the IoH Tier 1 load rating does not need to be performed.

MBEIoH 2A.4.3.1

A3.14—Summary of Rating Factors

Table A3.14-1—Summary of Rating Factors—Interior Girder

	Design Load Rating (HL-93)		
Limit State	Inventory	Operating	Permit Load Rating
Strength I	—	—	—
Flexure (at midspan)	1.48	1.92	—
Shear (at 64 in.)	1.96	—	—
Shear (at 20 ft)	1.34	—	—
Strength II	—	—	—
Flexure (at midspan)	—	—	2.11
Shear	—	—	2.21
Service III	—	—	—
Flexure (at midspan)	1.22	—	—
Service I	—	—	Stress Ratio = 1.33

A3.15—References

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A4—TIMBER STRINGER BRIDGE: EVALUATION OF AN INTERIOR STRINGER

PART A—LOAD AND RESISTANCE FACTOR RATING METHOD

A4A.1—Bridge Data

Span:	17 ft 10 in.
Year Built:	1930
Year Reconstructed:	1967
Material:	Southern Pine No. 2
Condition:	No deterioration. NBI Item 59 Code = 6
Riding Surface:	Unknown condition
Traffic:	Two Lanes
ADTT (one direction):	150
Skew:	0°

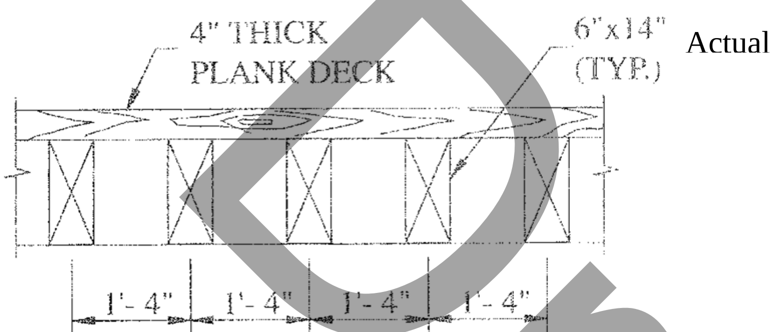

Figure A4A.1-1—Partial Cross Section of Deck

A4A.2—Dead Load Analysis—Interior Stringer in Flexure

A4A.2.1—Components and Attachments, *DC*

Deck: $\frac{16}{12}\times\frac{4}{12}\times 0.050$ = 0.022 kip/ft — LRFD Design Table 3.5.1-1

Stringer: $\frac{6\times 14}{144}\times 0.050$ = 0.029 kip/ft

Total per stringer = 0.051 kip/ft

$$M_{DC} = \frac{1}{8}\times 0.051\times 17.83^2$$
$$= 2.03 \text{ kip-ft}$$

A4A.2.2—Wearing Surface

$DW = 0$

A4A.3—Live Load Analysis—Interior Stringer in Flexure

A4A.3.1—Distribution Factor for Moment and Shear

LRFD Design Type ℓ cross section — LRFD Design Table 4.6.2.2.1-1

One Lane Loaded: — LRFD Design Table 4.6.2.2.2a-1

$$g_1 = \frac{S}{6.7}$$

$$= \frac{\frac{16}{12}}{6.7} = 0.20$$

Two or More Lanes Loaded:

$$g_2 = \frac{S}{7.5}$$

$$= \frac{\frac{16}{12}}{7.5} = 0.18 < 0.20$$

One Lane Loaded Governs

$g = 0.20$

A4A.3.2—Compute Maximum Live Load Effects

A4A.3.2.1—Maximum Design Live Load (HL-93) Moment at Midspan

Design Lane Load Moment	=	25.4 kip-ft		
Design Truck Moment	=	142.6 kip-ft		
Design Tandem Moment	=	175.7 kip-ft	Governs	
IM	=	0 percent		6A.7.5
M_{LL}	=	25.4 + 175.7		
	=	201.1 kip-ft		

A4A.3.2.2—Distributed Live-Load Moments

Design Live Load HL-93:

$g \times M_{LL}$ = 0.20×201.1

= 40.2 kip-ft

A4A.4—Compute Nominal Flexural Resistance

Section Properties for Stringers (based on actual dimensions):

$I_x \;=\; \dfrac{bh^3}{12} = \dfrac{6 \times 14^3}{12} = 1372 \text{ in.}^4$

$S_x \;=\; \dfrac{I_x}{\frac{h}{2}} = \dfrac{1,372}{\frac{14}{2}} = 196 \text{ in.}^3$

$A \;=\; bh = 6 \times 14 = 84 \text{ in.}^2$

A4A.4.1—LRFD Design

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

F_{bo} = 0.85 ksi Reference Design Value LRFD Design Table 8.4.1.1.4-1

C_{KF} = 2.5/ϕ = 2.5 / 0.85 = 2.94 Format Conversion Factor LRFD Design 8.4.4.2

C_M = 1.0 Wet Service Factor LRFD Design 8.4.4.3

(reduction for wet use not required due to species and member size)

C_F = Size Effect Factor for sawn lumber $\left(\dfrac{12}{d}\right)^{\frac{1}{9}} = \left(\dfrac{12}{14}\right)^{\frac{1}{9}} = 0.98 \leq 1.0$ LRFD Design Eq. 8.4.4.4-2

C_{fu} = 1.0 Flat Use Factor LRFD Design 8.4.4.6

C_i = 1.0 Incising Factor (only apply to dimension lumber) LRFD Design 8.4.4.7

C_d = 1.0 Deck Factor LRFD Design 8.4.4.8

C_λ = 0.8 Time Effect Factor for Strength I LRFD Design 8.4.4.9

F_b = 0.85 × 2.94 × 1.0 × 0.98 × 1.0 ×1.0 × 1.0 × 0.8 = 1.96

Adjusted Design Value = F_b = 1.96 ksi

Nominal Resistance $M_n = F_b S C_L$ LRFD Design Eq. 8.6.2-1

C_L = 1.0

M_n = 1.96 ksi × 196 in.^{3} × 1.0 × 1ft/12 in. = 32.01 kip-ft

A4A.5—General Load-Rating Equation (6A.4.2)

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$ Eq. 6A.4.2.1-1

A4A.6—Evaluation Factors (for Strength Limit State)

1. Resistance Factor, φ LRFD Design 8.5.2.2

   φ = 0.85 for Flexure
   φ = 0.75 for Shear

2. Condition Factor, φ_c 6A.4.2.3

   $\varphi_c = 1.0$ Good Condition

3. System Factor φ_s 6A.4.2.4

   $\varphi_s = 1.0$ for flexure and shear in timber bridges

A4A.7—Design Load Rating (6A.4.3)

A4A.7.1—Strength I Limit State (6A.7.4.1)

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

A4A.7.1.1—Inventory Level

Table 6A.4.2.2-1

Load	Load Factor
DC	1.25
LL	1.75

Flexure:

$$RF = \frac{(1.0)(1.0)(0.85)(32.0) - (1.25)(2.03)}{(1.75)(40.2)}$$

$$= 0.35$$

A4A.7.1.2—Operating Level

Table 6A.4.2.2-1

Load	Load Factor
DC	1.25
LL	1.35

Flexure:

$$RF = 0.35 \times \frac{1.75}{1.35}$$

$= 0.45$

A4A.7.1.3—Shear (Horizontal Shear) (LRFD Design 8.7)

Critical Section for Live Load Shear is at a distance d = 14 in. = 1.17 ft from face of support

Place live load to cause maximum shear at lesser of:

1. Three times the depth = 3 × 14 = 42 in. = 3.5 ft Governs
2. $\frac{1}{4}$ of span length = $\frac{1}{4} \times 17.83$ = 4.46 ft

A4A.7.1.4—Compute Maximum Shear at Critical Section (14 in. = 1.17 ft)

A4A.7.1.4a—Dead Load Shear

$$V_{DC} = \frac{1}{2}(0.051)(17.83) - (0.051)(1.17)$$

$$= 0.395 \text{ kips}$$

A4A.7.1.4b—Live Load Shear (HL-93)

Live load placed at 3.5 ft from face of support:

V_{TANDEM} = 34.6 kips Governs

V_{TRUCK} = 26.3 kips

V_{LANE} = 3.7 kips

Undistributed Shear:

V_{LU} = 3.7 + 34.6

= 38.3 kips

Distributed:

V_{LD} = 38.3 × 0.20

= 7.7 kips

For Horizontal Shear:

$$V_{LL} = 0.50\left[\left(0.60V_{LU}\right)+V_{LD}\right]$$ LRFD Design Eq. 4.6.2.2.2a-1

V_{LU} = Maximum vertical shear at 3*d* or *L*/4 due to undistributed wheel loads (kips)

= For undistributed wheel loads, one line of wheels is assumed to be carried by one bending member. LRFD Design 4.6.2.2.2a

$$= \frac{V_{LU}}{2} = \frac{(38.3)}{2} = 19.1 \text{ kips}$$

V_{LD} = Maximum vertical shear at 3*d* or *L*/4 due to wheel loads distributed laterally as specified herein (kips)

= 7.7 kips

V_{LL} = 0.50[(0.60×19.1) + 7.7] = 9.58 kips

A4A.7.1.5—Compute Nominal Shear Resistance

A4A.7.1.5a—LRFD Design

$$V_n = \frac{F_v bd}{1.5}$$ LRFD Design Eq. 8.7-2

F_v = $F_{vo}C_{KF}C_MC_iC_\lambda$ LRFD Design Eq. 8.4.4.1-2

F_{vo} = 0.165 ksi Reference Design Value LRFD Design Table 8.4.1.1.4-1

C_{KF} = 2.5/φ = 2.5 / 0.75 = 3.33 Format Conversion Factor LRFD Design 8.4.4.2

C_M = 1.0 Wet Service Factor LRFD Design 8.4.4.3

(reduction for wet use not required due to species and member size)

C_i = 1.0 Incising Factor LRFD Design 8.4.4.7

C_λ = 0.8 Time Effect Factor for Strength I LRFD Design 8.4.4.9

F_v = $0.165 \times 3.33 \times 1.0 \times 1.0 \times 0.8$

Adjusted Design Value:

F_v = 0.440 ksi

$$V_n = \frac{(0.440)(6)(14)}{1.5} = 24.6 \text{ kips}$$

A4A.7.1.5b—Inventory Level

Load	Load Factor
DC	1.25
LL	1.75

Shear:

$$RF = \frac{(1.0)(1.0)(0.75)(24.6) - (1.25)(0.395)}{(1.75)(9.58)}$$

$$= 1.07$$

A4A.7.1.5c—Operating Level

Shear:

$$RF = 1.07 \times \frac{1.75}{1.35} = 1.39$$

No service limit states apply.

A4A.8—Legal Load Rating (6A.4.4)

Live Load: AASHTO Legal Loads—Types 3, 3S2, and 3-3 (Rate for all three) 6A.4.4.2.1

g = 0.20

IM = 0 percent 6A.7.5

	Type 3	Type 3S2	Type 3-3	
M_{LL}	119.5	108.9	98.4	kip-ft
gM_{LL}	23.9	21.8	19.7	kip-ft

A4A.8.1—Strength I Limit State (6A.7.4.2)

Dead Load *DC*:

Load Factor = 1.25 Table 6A.4.2.2-1

ADTT = 150

Live-Load Factor = 1.30 Table 6A.4.4.2.3a-1

Flexure:

$$RF = \frac{(1.0)(1.0)(0.85)(32.0)-(1.25)(2.03)}{(1.30)(M_{LL})}$$

A4A.8.1.1—Shear Capacity

Live load shear at critical section (14 in.) with live load placed to cause maximum shear effect at 3.5 ft (3*d*).

g = 0.20

IM = 0 percent 6A.7.5

The distributed live load is calculated in the same manner as demonstrated for the design load check.

$$V_{LL} = 0.50\left[(0.60V_{LU})+V_{LD}\right]$$ LRFD Design Eq. 4.6.2.2.2a-1

	Type 3	Type 3S2	Type 3-3	
V_{LU}	11.76	10.72	9.68	kips
V_{LD}	4.70	4.29	3.87	kips
V_{LL}	5.87	5.35	4.83	kips

Shear:

$$RF = \frac{(1.0)(1.0)(0.75)(24.6)-(1.25)(0.395)}{(1.30)(V_{LL})}$$

RF	Type 3	Type 3S2	Type 3-3
Shear	2.35	2.58	2.86
Flexure	0.79	0.87	0.96

A4A.8.2—Summary

Truck	Type 3	Type 3S2	Type 3-3
Weight, tons	25	36	40
RF	0.73	0.80	0.88
Safe Load Capacity, tons	18	28	35

A4A.9—IoH Tier 1 Load Rating

A4A.9.1 – Live Load Moment and Shear

Truck: IoH Tier 1

Gage Width GW =9 ft

Maximum moment for one lane = 75.5(2) =151.0 kip-ft (IoH Tier 1a controls) MBEIoH Table A2B-3

Maximum shear for one lane = 18.7 kips (IoH Tier 1a controls)

IM = 20 percent 6A.4.4.3 and 6A.7.5

M_{LL+IM} = 151.0 (1.2) = 181.2 kip-ft

V_{LL+IM} = 18.7 (1.2) = 22.4 kips

A4A.9.2—Distributed Live-Load Moments

g_{HL93} *= 0.20*

MF_{moment} *for non-standard gage width*

$$= 1 - 0.340 R_1 Ln(\frac{GW}{6}) = 1 - 0.340(0.85) Ln(\frac{9\,ft}{6}) = 0.883$$ MBEIoH Table 2A.3.2.1-1

$g = g_{HL93}\ MF_{moment} = 0.20\ (0.883) = 0.177$

$g \times M_{LL+IM} = 0.177\ (181.2) = 32.1$ kip-ft

A4A.9.3—Rating for Flexure

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

γ_L = 1.30 for ADTT =150 <1,000

$$RF = \frac{(1.0)(1.0)(0.85)(\ 32.0) - (1.25)(2.03)}{(1.30)(32.1)}$$

$$= 0.59$$

A4A.9.4—Rating for Shear

Live load placed at 3.5 ft from face of support (see A4A.7.1.3):

$V_{IoHTier1}$ = 28.0 kips IoH Tier1b governs

Undistributed Shear:

V_{LU} = 28.0 kips

Distributed:

MF_{shear} *for non-standard gage width*

$$= 1 - 0.362 R_1 Ln(\frac{GW}{6})(\frac{t_s}{6})^{0.51}(\frac{S}{9})^{0.17}$$

$$= = 1 - 0.362(0.85) Ln(\frac{9\,ft}{6})(\frac{4in.}{6})^{0.51}(\frac{1.33\,ft}{9})^{0.17}$$

= 0.927

6A.3.2.1.3

g_{HL93} = 0.20

$g = g_{HL93}\ MF_{shear} = 0.20\ (0.927) = 0.185$

V_{LD} = 28.0×0.185

= 5.2 kips

For Horizontal Shear:

V_{LL} = $0.50\left[\left(0.60V_{LU}\right)+V_{LD}\right]$ LRFD Design Eq. 4.6.2.2.2a-1

V_{LU} = Maximum vertical shear at 3*d* or *L*/4 due to undistributed wheel loads (kips)

= For undistributed wheel loads, one line of wheels is assumed to be carried by one bending member. LRFD Design 4.6.2.2.2a

= $\frac{V_{LU}}{2}=\frac{(28.0)}{2}=14.0$ kips

V_{LD} = Maximum vertical shear at 3*d* or *L*/4 due to wheel loads distributed laterally as specified herein (kips)

= 5.2 kips

V_{LL} = 0.50[(0.60×14.0) + 5.2] = 6.8 kips

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

$$RF=\frac{(1.0)(1.0)(0.75)(24.6)-(1.25)(0.395)}{(1.3)(6.8)(1.2)}$$

=1.69

A4A.10—Summary of Rating Factors for Load and Resistance Factor Rating Method

Table A4A.10-1—Summary of Rating Factors for Load and Resistance Factor Method—Interior Stringer

Limit State		Design Load Rating		Legal Load Rating			IoH Tier 1 Load Rating
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	
Strength I	Flexure	0.35	0.45	0.79	0.887	0.96	0.59
	Shear	1.07	1.39	2.35	2.58	2.864	1.69

PART B—ALLOWABLE STRESS RATING METHOD

A4B.1—Bridge Data

Refer to Article A4A.1 for bridge data.

A4B.2—Section Properties

$I_x \quad = \quad \dfrac{bh^3}{12} = \dfrac{6 \times 14^3}{12} = 1{,}372 \text{ in.}^4$

$S_x \quad = \quad \dfrac{I_x}{h \div 2} = \dfrac{1{,}372}{14 \div 2} = 196 \text{ in.}^3$

$A \quad = \quad bh = 6 \times 14 = 84 \text{ in.}^2$

A4B.3—Dead Load Analysis—Interior Stringer

Deck:

$$\frac{(1 \text{ ft} - 4 \text{ in.})4 \text{ in.}}{144 \text{ in.}^2/\text{ft}^2} \times 50 \text{ lb./ft}^3 = 22.2 \text{ lb/ft}$$

Stringer:

$$\frac{6 \text{ in.} \times 14 \text{ in.}}{144} \times 50 = \frac{29.2 \text{ lb/ft}}{51.4 \text{ lb/ft}}$$ say 0.051 kip/ft

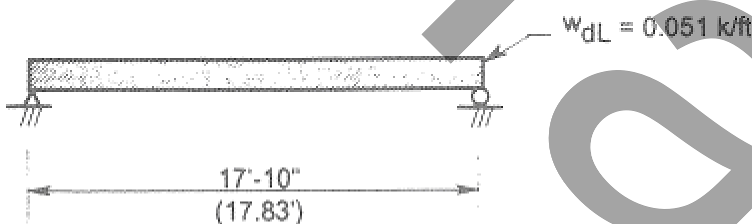

Figure A4B.3-1—Load Diagram for Interior Stringer—Uniform Dead Load

$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{0.051(17.83)^2}{8}$$

$$M_{DL} = 2.03 \text{ kip-ft}$$

A4B.4—Live Load Analysis—Interior Stringer

Live Load: Rate for H-15 truck

Determine the maximum live load moment by statics. For small spans, verify that the maximum moment will occur at midspan with the heaviest wheel positioned at midspan.

$M_L \quad = \quad PL/4$

$M_L \quad = \quad (12 \text{ kips} \times 17.83 \text{ ft})/4 = 53.49 \text{ kip-ft}$

Alternatively, interpolation could be used for estimating. Note that for longer spans and for interpolation between span increments greater than 1 ft, interpolated values yield approximate results. Appendix C6B

Span	M_L	
17 ft	51 kip-ft	
		← For 17.83-ft span, interpolate
18 ft	54 kip-ft	
$M_L = 51 + \frac{17.83-17}{18-17}(54-51) = 53.5$ kip-ft		

A4B.5—Allowable Stress Rating (6B.3.1, 6B.4.2, 6B.5.2)

Consider stringer only; consider maximum moment and shear sections only for this example.

A4B.5.1—Impact (Use standard AASHTO) (6B.6.4)

No impact for timber members: AASHTO 3.8.1.2

$I = 0$

A4B.5.2—Distribution (Use standard AASHTO) (6B.6.3)

For two lanes and plank deck [a]: AASHTO 3.23.2.2, Table 3.23.1

$$DF = \frac{S}{3.75} = \frac{16\text{ in.}/12\text{ in./ft}}{3.75} = 0.36$$

a Note that the moments given in MBE are for one line of wheels. The values given in AASHTO are for the entire rear axle and are therefore twice the MBE values.

Thus:

$$M_{LL} = M_L \times DF = 53.5\,\text{kip-ft} \times 0.36$$

$$M_{LL} = 19.26\,\text{kip-ft}$$

A4B.5.3—Stresses to be Used (Use NDS, *National Design Specification for Wood Construction*, 2005 Edition)

The general equations for adjusted Reference Design Values are:

$F_b' = F_b \times C_D C_M C_t C_L C_F C_{fu} C_i C_r$

$F_V' = F_V \times C_D C_M C_t C_i$

F_b = 850 psi Reference Design Value, NDS Table 4D

F_V = 165 psi Reference Design Value, NDS Table 4D

C_D = 1.15 Load Duration Factor for two months is assumed as cumulative effect of live load. Wood bridges are typically located on low-volume roads; therefore, the accumulated live load duration is lower than 30 days. It is assumed that the live load duration is two months in the reliability analysis.

C_M = 1.0 Wet Service Factor is in NDS Table 4D for Sothern Pine

C_t = 1.0 Temperature Factor

C_L = 1.0 Beam Stability Factor

C_F = 0.98 Size Factor = $(12/d)^{1/9}$ for beam depth exceeding 12 in.

C_{fu} = 1.0 Flat Use Factor; not applicable

C_i = 1.0 Incising Factor

C_r = 1.0 Repetitive Use Factor, not applicable

A4B.5.3.1—Inventory Level Stresses (6B.5.2.7)

$$F_b^I = 850 \times 1.15 \times 0.98 \times 1.0 = 958 \text{ psi} = 0.96 \text{ ksi}$$

$$C_D = 1.15$$

$$C_F = 0.98$$

$$C_i = 1.0$$

and:

$$F_V^I = 165 \times 1.15 \times 1.0 = 190 \text{ psi} = 0.19 \text{ ksi}$$

A4B.5.3.2—Operating Level Stresses (Use standard AASHTO) (6B.5.2.7)

$$F_b^O = F_b^I \times 1.33 = 950 \times 1.33$$

$$F_b^O = 1274 \text{ psi} = 1.27 \text{ ksi}$$

and:

$$F_V^O = 1.33 F_V^I = 1.33 \times 190 \text{ psi} = 253 \text{ psi}$$

A4B.5.4—Inventory Level Rating for Flexure

Capacity:

$$M_{R_I} = F_b^I S_x = 0.96 \text{ ksi} \times 196 \text{in.}^3 = 188 \text{ kip-in.}$$

$$M_{R_I} = 15.68 \text{kip-ft}$$

then:

$$RF_I^M = \frac{M_{R_I} - M_{DL}}{M_{LL}} = \frac{15.68 \text{kip-ft} - 2.03 \text{kip-ft}}{19.26 \text{kip-ft}} \quad \text{Eq. 6B.4.1-1}$$

$RF_I^M = \underline{\underline{0.71}}$ or 0.71 × 15 tons = $\underline{\underline{\text{10.7 tons}}}$ H truck

A4B.5.5—Operating Level Rating for Flexure

Capacity:

$$M_{R_O} = F_b^O S_x = 1.27 \text{ ksi} \times 196 \text{ in.}^3 = 248.9 \text{ kip-in.}$$

$$M_{R_O} = 20.74 \text{kip-ft}$$

then:

$$RF_O^M = \frac{M_{R_O} - M_{DL}}{M_{LL}} = \frac{20.74\,\text{kip-ft} - 2.03\,\text{kip-ft}}{19.26\,\text{kip-ft}} \qquad \text{Eq. 6B.4.1-1}$$

$RF_O^M = \underline{\underline{0.97}}$ or 0.97 × 15 tons = $\underline{\underline{\text{14.6 tons}}}$ H truck

A4B.5.6—Check Horizontal Shear

Computed shear at: AASHTO 13.6.5.2

1. A distance from the support equal to three times the depth of the stringer, or
2. At the quarter point, whichever is less.

Thus by:

1. $3(14\text{ in.}) = 42\text{ in.} \leftarrow \text{ Controls} = 3.5\text{ ft}$
2. $\dfrac{17.83\text{ ft} \times 12\text{ in./ft}}{4} = 53.5\text{ in.}$

For H-15 Truck:

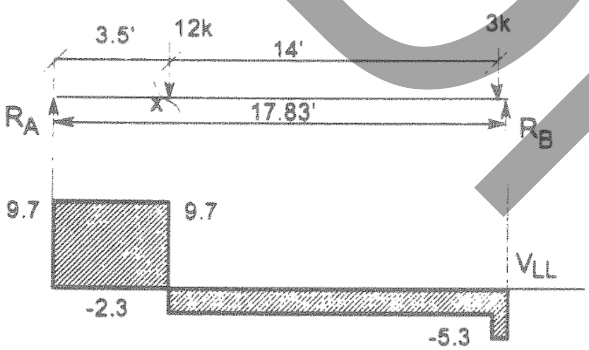

Figure A4B.5.6-1—Shear Diagram for Interior Stringer—H-15 Live Load

$$V_x = \frac{15(x - 2.8)}{L} \qquad \text{Appendix H6B}$$

where L = 17.83 ft

$x = 17.83 - 3.5 = 14.33$ ft

$V_x = \dfrac{15(14.33 - 2.8)}{17.83} = 9.7$ kips per wheel line without distribution

$$V_{L_x} = \frac{1}{2}\left(0.6V_x^{L\ no\ dist.} + DFV_x^{L\ no\ dist.}\right) \qquad \text{AASHTO 13.6.5.2, Eq. 13-10}$$

$$V_{L_x} = \frac{1}{2}\left[0.6(9.7) + 0.36(9.7)\right]$$

$V_{L_x} = 4.7$ kips

For $w_{DL} = 0.051$ kip/ft

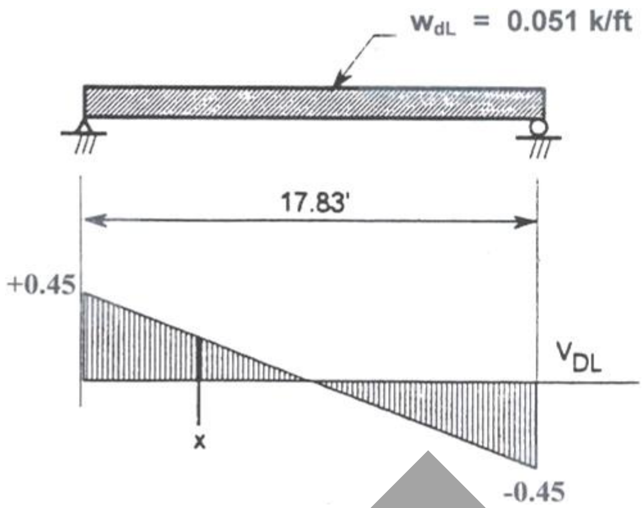

Figure A4B.5.6-2—Load and Shear Diagrams—Uniform Dead Load

$$R_A = R_B = \frac{1}{2} w_{DL} L$$

$$= \frac{1}{2}(0.051) \times 17.83$$

$$= 0.45 \text{ kips}$$

$$V_{D_x} = 0.45 - 0.051 \times 14/12$$

$$V_{D_x} = 0.4 \text{ kips}$$

A4B.5.7—Inventory Level Rating for Shear

Capacity:

$$V_R = \frac{2}{3} b d f_v \qquad \text{AASHTO Eq. 13-9}$$

then:

$$V_{R_I} = \frac{2}{3}(6)(14)(190) \text{ psi} = 10640 \text{ lbs} = 10.64 \text{ kips}$$

$$RF_I^V = \frac{V_{R_I} - V_{D_x}}{V_{L_x}} = \frac{10.64 \text{ kips} - 0.4 \text{ kips}}{4.7 \text{ kips}} \qquad \text{Eq. 6B.4.1-1}$$

RF_I^V = $\underline{\underline{2.18}}$ or 2.18 × 15 tons = $\underline{\underline{32.7 \text{ tons}}}$ H truck

A4B.5.8—Operating Level Rating for Shear

Capacity:

$$V_{R_O} = \frac{2}{3}(6)(14)(253)\text{ psi} = 14168\text{ lbs} = 14.17\text{ kips}$$

$$RF_O^V = \frac{V_{R_O} - V_{D_x}}{V_{L_x}} = \frac{14.17\text{ kips} - 0.4\text{ kips}}{4.7\text{ kips}} \qquad \text{Eq. 6B.4.1-1}$$

$$RF_O^V = \underline{\underline{2.93}}\text{ or } 2.93 \times 15\text{ tons} = \underline{\underline{43.95\text{ tons}}}\text{ H truck}$$

A4B.5.9—Summary of Ratings for Allowable Stress Rating Method

Table A4B.5.9-1—Summary of Ratings for Allowable Stress Rating Method—Interior Stringer

Method/Force	*RF*	H Truck Max. Load, tons
Allowable Stress Moment:		
Inventory	0.71	10.7
Operating	0.97	14.6
Allowable Stress Shear:		
Inventory	2.18	32.7
Operating	2.93	43.9

∴ Rating governed by moment rather than shear.

A4B.6—Load Factor Rating

Not currently available for timber.

PART C—SUMMARY

A4C.1—Summary of All Ratings for Example A4

Table A4C-1—Summary of Rating Factors for All Rating Methods—Interior Stringer

LRFR Method		Design Load Rating		Legal Load Rating			H-15 Rating				IoH Tier 1
							Flexure		Shear		
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	Inv.	Opr.	Inv.	Opr.	
Strength I Limit State	Flexure	0.35	0.45	0.79	0.87	0.96	—	—	—	—	0.59
	Shear	1.07	1.39	2.35	2.58	2.86	—	—	—	—	1.69
Allowable Stress Method		—	—	—	—	—	0.71	0.97	2.18	2.93	—

A4C.2—References

AASHTO. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17. American Association of State Highway and Transportation Officials, Washington, DC, 2002.

AASHTO. *AASHTO LRFD Bridge Design Specifications*, Eighth Edition, LRFD-8. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

NFPA. *National Design Specification for Wood Construction*. National Forest Products Association, Washington, DC, 2005.

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A7—REINFORCED CONCRETE SLAB BRIDGE DESIGN AND LEGAL LOAD CHECK

A7.1—Bridge Data

Span Length:	21.5 ft (simple span)	
Year Built:	1963	
Material:	Concrete	$f_c' = 3$ ksi
	Reinforced Steel	$f_y = 40$ ksi
Condition:	No deterioration. NBI Item 59 Code = 6	
Riding Surface:	Not field verified and documented	
ADTT (one direction):	Unknown	
Skew:	0°	

A7.2—Dead Load Analysis

A7.2.1—Interior Strip—Unit Width

A7.2.1.1—Components, DC

Concrete slab:

$$\left(\frac{14}{12}\right)(1.0)(0.150) = 0.175 \text{ kip/ft}$$

Parapet and curb:

$$\frac{2\left[(1.5)(1.5)+(2.33)(1.0)\right](1.0)(0.150)}{43} = \underline{0.032 \text{ kip/ft}}$$

$$DC = 0.207 \text{ kip/ft}$$

$$M_{DC} = \frac{1}{8}\times 0.207\times 21.5^2$$

$$= 12.0 \text{ kip-ft}$$

A7.2.1.2—Wearing Surface, DW

Asphalt Thickness = $3^1/_2$ in. (field measured)

Asphalt Overlay = $\left(\frac{3.5}{12}\right)(1.0)(0.144) = 0.042$ kip/ft

$$M_{DW} = \frac{1}{8}\times 0.042\times 21.5^2$$

$$= 2.4 \text{ kip-ft}$$

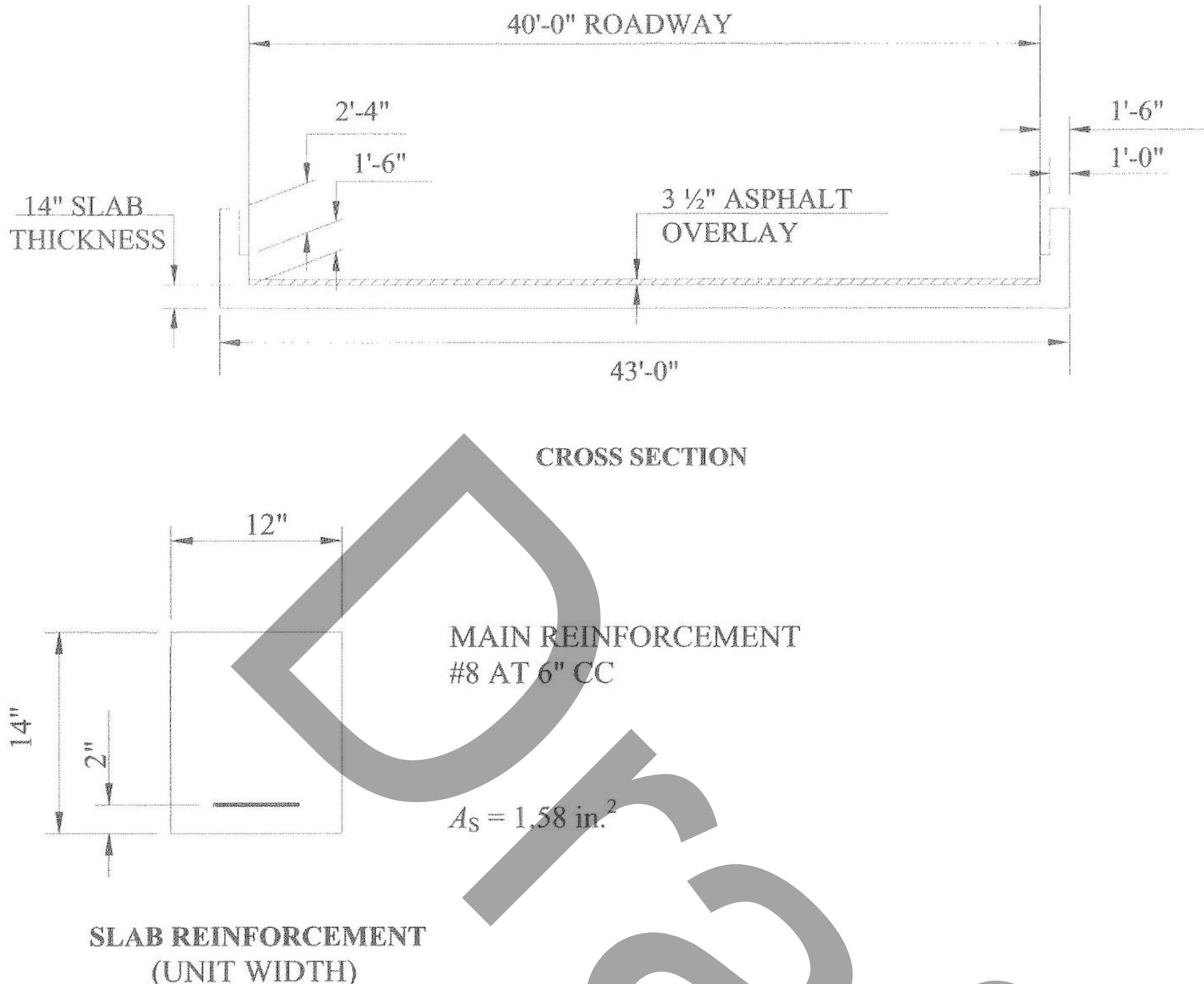

Figure A7.1-1—Reinforced Concrete Slab Bridge

A7.3—Live Load Analysis (Design Load Check)

Equivalent strip width for slab type bridges (Interior Strip) — LRFD Design 4.6.2.3

A7.3.1—One Lane Loaded

$E \quad = \quad 10.0 + 5.0\sqrt{L_1 W_1}$ — LRFD Design Eq. 4.6.2.3-1

$L_1 \quad = \quad 21.5$ ft < 60 ft

$W_1 \quad = \quad$ Lesser of 43.0 ft or 30.0 ft

$\quad = \quad 30.0$ ft

$E \quad = \quad 10.0 + 5.0\sqrt{21.5 \times 30}$

$\quad = \quad 137.0$ in.

$\quad = \quad 11.41$ ft

A7.3.2—More than One Lane Loaded

$E = 84.0 + 1.44\sqrt{L_1 W_1} \leq \dfrac{12.0\,W}{N_L}$

$L_1 = 21.5$ ft < 60.0 ft

$W_1 =$ Lesser of 43.0 ft or 60.0 ft

$= 43.0$ ft

$E = 84 + 1.44\sqrt{21.5 \times 43}$ — LRFD Design Eq. 4.6.2.3-2

$= 127.8$ in. = 10.65 ft < 11.41 ft

$N_L = \dfrac{40.0}{12} = 3$ Design Lanes

$\dfrac{12.0\,W}{N_L} = \dfrac{12 \times 43}{3} = 172$ in. > 127.8 in. OK

Use $E = 10.65$ ft

For longitudinal edge strips, the effective strip width is:

Sum of: — LRFD Design 4.6.2.1.4b

the distance between the edge of the deck and the inside face of the barrier

+ one-quarter the strip width specified in LRFD Design Article 4.6.2.1.3, 4.6.2.3, or 4.6.2.10, as appropriate

+ 12.0 in.

The effective edge strip width shall not exceed either one-half the full strip width or 72.0 in.

$E_2 = 18.0\text{ in.} + 0.25 \times 137.0\text{ in.} + 12.0\text{ in.} = 64.25\text{ in.}$

$E_2 = 0.5 \times 137.0\text{ in.} = 68.5\text{ in.}$

$E_2 = 72$ in.

64.25 in $\leq$ 68.5 in.

$\therefore$ use $E_2 =$ 64.25 in.

LRFD Design Article 4.6.2.1.4b assumes the longitudinal edge strip supports one wheel line and a tributary portion of the design lane load where appropriate.

By comparison of the ratios of the tributary design lane load width to effective slab width, the edge strip is estimated not to govern for this bridge. Note that parapet dead load was assumed to be uniformly distributed across the full bridge width and that parapet width can play an influential role when determining the governing case.

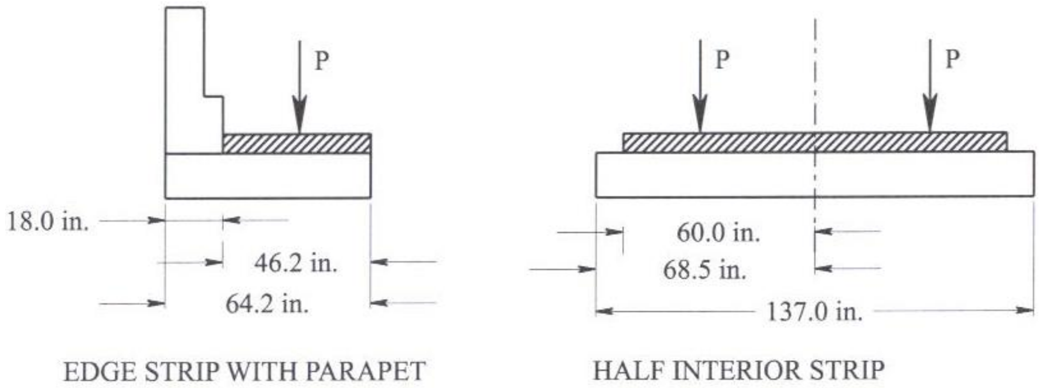

Figure A7.3.2-1—Longitudinal Edge Strip Comparison

Ratio edge strip:

46.25/64.25 = 0.72

Ratio half interior strip:

60.0/68.5 = 0.88 Governs

The rating will consider only the interior strip width.

A7.3.2.1—Midspan Live Load Force Effects (HL-93)

Dynamic Load Allowance	=	33%	
Equivalent Strip Width	=	10.65 ft	
Design-Lane Load Moment	=	37.0 kip-ft	
Design Truck Moment	=	172.0 kip-ft	
Design Tandem Moment	=	219.4 kip-ft	Governs

M_{LL+IM} = $37.0 + 319.4 \times 1.33$

= 328.8 kip-ft

Live Load Moment per unit width of slab:

$$M_{LL+IM} = \frac{328.8}{10.65} = 30.9 \text{ kip-ft/ft}$$

A7.4—Compute Nominal Resistance

Flexural Resistance:

Rectangular Section = $b_w = b = 12$ in. LRFD Design 5.6.3.1

$$c = \frac{A_s f_y}{\alpha_1 f_c' \beta_1 b}$$

LRFD Design Eq. 5.6.3.1.1-4

A_s = 0.79×2 #8 bars at 6 in.

= 1.58 in.^{2}/ft

α_1 = 0.85 LRFD Design 5.6.2.2

β_1 = 0.85 LRFD Design 5.6.2.2

b = 12 in.

$$c = \frac{1.58 \times 40}{0.85 \times 3 \times 0.85 \times 12}$$

= 2.43 in.

a = $c\beta_1$ LRFD Design 5.6.2.2

= 2.43×0.85

= 2.07 in.

d_s = 14 – 2 = 12 in. Distance to *C.G.* of steel

$$M_n = A_s f_y \left(d_s - \frac{a}{2} \right)$$

$$= 1.58 \times 40 \left(12 - \frac{2.07}{2} \right) \times \frac{1}{12}$$

= 57.75 kip-ft/ft

A7.5—Minimum Reinforcement (6A.5.6)

Amount of reinforcement must be sufficient to develop M_r equal to the lesser of: 6A.5.6

M_{cr} or $1.33M_u$

M_r = $\varphi M_n = 0.90 \times 57.75$ kip-ft=51.98 kip-ft

1. $1.33M_u$ = $1.33\, M_u = 1.33 \times (1.75 \times 30.9 + 1.25 \times 12 + 1.25 \times 2.4)$

= 95.9 kip-ft > 51.98 kip-ft No Good

2. M_{cr} = $\gamma_3 \left[(\gamma_1 f_r + \gamma_2 f_{cpc}) S_c - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$ LRFD Design Eq. 5.6.3.3-1

Where a monolithic or non-composite section is designed to resist all the loads, S_{nc} is substituted for S_c. In this case, $f_{cpe} = 0$, therefore:

M_{cr} = $\gamma_3 \left(\gamma_1 f_r \right) S_c$

γ_1 = 1.6

$$\gamma_3 = \frac{F_y}{F_n} = \frac{40 \text{ ksi}}{70 \text{ ksi}} = 0.57$$

$$S_{nc} = \frac{I}{y_t}$$

where:

I = moment of inertia of uncracked section (neglecting reinforcement steel)

y_t = distance from the neutral axis of the uncracked section to the extreme tension fiber

$$= \frac{14}{2} = 7 \text{ in.}$$

$$I = \frac{1}{12} \times 12 \times 14^3 = 2744 \text{ in.}^4$$

$$S_{nc} = \frac{2,744}{7} = 392 \text{ in.}^3$$

$$f_r = 0.24\lambda\sqrt{f'_c}$$ LRFD Design 5.4.2.6

$$\lambda = 1.0$$

$$f_r = 0.24\sqrt{3'} = 0.416 \text{ ksi}$$

$$M_{cr} = 0.57 \times 1.6 \times 0.416 \text{ ksi} \times 392 \text{ in}^3 = 148.7 \text{ kip-in} = 12.4 \text{ kip-ft}$$

$$1.2M_{cr} = 1.2 \times 12.4 = 14.88 \text{ kip-ft} < 51.98 \text{ kip-ft} \quad \text{OK}$$

The section meets the requirements for minimum reinforcement.

A7.6—Maximum Reinforcement (6A.5.5)

Current provisions of the LRFD specification have eliminated the check for maximum reinforcement. Instead, the factored resistance (φ factor) of compression controlled sections shall be reduced in accordance with LRFD Design Article 5.5.4.2. This approach limits the capacity of over-reinforced (compression controlled) sections. C6A.5.5

The net tensile strain, ε_t, is the tensile strain at nominal strength and determined by strain compatibility using similar triangles. LRFD Design C5.6.2.1

Given an allowable concrete strain of 0.003 and depth to neutral axis c = 2.43 in.

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

$$\frac{0.003}{2.43 \text{ in.}} = \frac{\varepsilon_t}{12 \text{ in.} - 2.43 \text{ in.}}$$

$$\varepsilon_t = 0.0118$$

For ε_t = 0.0118 > 0.005, the section is tension controlled and Resistance Factor, φ, shall be taken as 0.90. LRFD Design 5.6.2.1, 5.5.4.2

A7.7—Shear

Concrete slabs and slab bridges designed in conformance with AASHTO specifications may be considered satisfactory for shear. LRFD Design 5.12.2.1

Also shear need not be checked for design load and legal load ratings of concrete members. 6A.5.8

A7.8—General Load-Rating Equation (6A.4.2)

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)}$$ Eq. 6A.4.2.1-1

A7.9—Evaluation Factors (for Strength Limit States)

A7.9.1—Resistance Factor, φ (LRFD Design 5.5.4.2)

φ = 0.90 For flexure

A7.9.2—Condition Factor, φ_c (6A.4.2.3)

φ_c = 1.0 No deterioration

A7.9.3—System Factor, φ_s (6A.4.2.4)

φ_s = 1.0 Slab bridge

A7.10—Design Load Rating (6A.4.3)

A7.10.1—Strength I Limit State (6A.5.4.1)

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

Table 6A.4.2.2-1

Load	Inventory	Operating	
DC, DW	1.25	1.25	Asphalt thickness was field measured
LL + IM	1.75	1.35	

Inventory:

$$RF = \frac{(1.0)(1.0)(0.9)(57.75)-(1.25)(12.0)-(1.25)(2.4)}{(1.75)(30.9)}$$

$$= 0.63$$

Operating:

$$RF = 0.63\times\frac{1.75}{1.35}$$

$$= 0.82$$

A7.10.2—Service Limit State

No service limit states apply to reinforced concrete bridge members. As *RF* < 1.0 for HL-93, evaluate the bridge for legal loads.

A7.11—Legal Load Rating (6A.4.4)

Live Load: AASHTO Legal Loads—Type 3, 3S2, 3-3 (Rate for all 3) 6A.4.4.2.1

E = 10.65 ft 6A.4.4.3

IM = 33 percent Unknown riding surface conditions

	Type 3	Type 3S2	Type 3–3	
M_{LL}	150.4	137.1	123.8	kip-ft
$\frac{M_{LL+IM}}{E}$	18.8	17.1	15.5	kip-ft/ft

A7.11.1—Strength I Limit State 6A.5.4.2.1

Generalized Live-Load Factor: Table 6A.4.4.2.3a-1

γ_L = 1.45

ADTT = Unknown

Flexure:

$$RF = \frac{(1.0)(1.0)(0.90)(57.75) - \left[(1.25)(12.0) + (1.25)(2.4)\right]}{(1.45)(M_{LL+IM})}$$

	Type 3	Type 3S2	Type 3-3
RF	1.25	1.37	1.51

No posting required as *RF* > 1.0 for all AASHTO Legal Loads.

A7.11.2—Service Limit State

No service limit states apply to reinforced concrete bridge members at the legal load rating.

A7.11.3—Shear LRFD Design 5.12.2.1

Concrete slabs and slab bridges designed in conformance with AASHTO Specifications may be considered satisfactory for shear.

Shear need not be checked for legal loads. 6A.5.8

A7.11.4—Summary

Truck	Type 3	Type 3S2	Type 3-3
Weight, tons	25	36	40
RF	1.25	1.37	1.51
Safe Load Capacity, tons	31	49	60

A7.12—IoH Tier 1 Load Rating

IoH Tier 1 load's gage width GW = 8ft

Dynamic Load Allowance = 20%

Maximum moment due to one lane of load = 97.6 kip-ft

Equivalent Strip Width according to LRFD Design E = 11.41 ft, see A7.3.1.

Modifying Factor for LRFD Design E:

$$MF = \frac{1}{1-0.155R_1 Ln(\frac{GW}{6})} = \frac{1}{1-0.155(0.85)Ln(\frac{8\,ft}{6})} = 1.039$$

E = MF × LRFD Design E=1.039 (11.41) = 11.86 ft

M_{LL+IM} = 97.6 × 1.20 = 117.1 kip-ft Tier 1a controls

Live Load Moment per unit width of slab:

$$M_{LL+IM} = \frac{117.1}{11.86} = 9.87 \text{ kip-ft/ft}$$

For unknown ADTT, live load factor γ_{IoH} =1.45 with one lane loading MBEIoH Table 2A.4.3.2.2-1

$$RF = \frac{(1.0)(1.0)(0.9)(57.75)-(1.25)(12.0)-(1.25)(2.4)}{(1.45)(9.87)}$$

$$= 2.37$$

A7.13—Summary of Rating Factors

Table A7.12-1 Summary of Rating Factors—Concrete Slab Interior Strip

Limit State		Design Load Rating		Legal Load Rating			IoH Tier 1
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	
Strength I	Flexure	0.63	0.82	1.25	1.37	1.51	2.37

A7.14—References

AASHTO. *AASHTO LRFD Bridge Design Specifications*, Eighth Edition, LRFD-8. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

Fu,G., Q.Wang, J.Chi, M.Lwin, and R.Corotis NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202?

A9—P/S CONCRETE ADJACENT BOX-BEAM BRIDGE: DESIGN LOAD AND PERMIT LOAD RATING OF AN INTERIOR BEAM

Note: This example demonstrates the rating calculations for moment at the centerline of a prestressed concrete adjacent box beam bridge.

A9.1—Bridge Data

Span Length:	70 ft (simple span)
Year Built:	1988
Concrete:	f'_c = 5 ksi (P/S beam)
	f'_{ci} = 4 ksi (P/S beam at transfer)
Prestressing Steel:	$^1/_2$ in. diameter, 270 ksi stress-relieved strand
Reinforcing Steel:	Grade 60
Condition:	No deterioration. NBI Item 59 Code = 7
Riding Surface:	Field verified and documented: Smooth approach and deck
ADTT (one direction):	4600
Skew:	0°

A9.1.1—Section Properties

48 in. × 33 in. Box Beams

A = 753 in.^{2}

I_x = 110,499 in.^{4}

S_{bot} = 6,767 in.^{3}

S_{top} = 6,629 in.^{3}

A9.2—Dead Load Analysis—Interior Beam

The beams are sufficiently transversley post-tensioned to act as a unit. Conditions given in LRFD Design Article 4.6.2.2.1 are also satisfied. Therefore, permanent loads due to barrier, wearing surface, and utilities may be uniformly distributed among the beams.

A9.2.1—Components and Attachments, *DC*

Beam Self Weight (including diaphragms) = 0.815 kip/ft

Sidewalks:

$$2\left(\frac{10.25}{12}\times 7\times 0.150\right)\frac{1}{12}=0.150 \text{ kip/ft}$$

Parapets:

$$2\left(1.0\times 2.25\times 0.150\right)\frac{1}{12}=0.056 \text{ kip/ft}$$

Railing:

$$2\times 0.02 \text{ kip/ft}\times\frac{1}{12}=0.003 \text{ kip/ft}$$

Total DC = 1.024 kip/ft

M_{DC} = $M_{DC}=\frac{1}{8}\times 1.024\times 70^2$

= 627.2 kip-ft

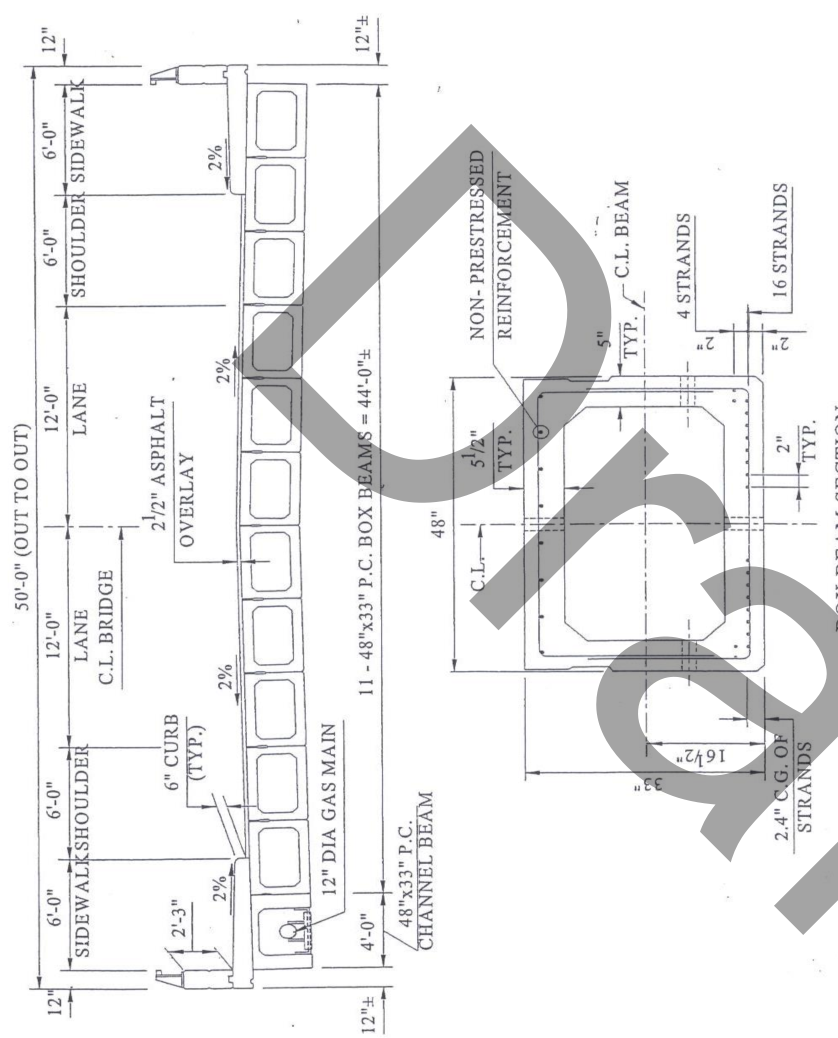

Figure A9.2.1-1—Bridge Cross Section and Interior Box Beam Cross Section

Asphalt thickness = $2^{1}/_{2}$ in. (not field measured)

A9.2.2—Wearing Surface and Utilities, *DW*

Asphalt Overlay:

$$\frac{2.5}{12} \times 36.0 \times 0.144 \times \frac{1}{12} = 0.09 \text{ kip/ft}$$

12-in. Gas Main:

$$0.05 \text{ kip/ft} \times \frac{1}{12} = 0.005 \text{ kip/ft}$$

Total DW = 0.095 kip/ft

$$M_{DW} = \frac{1}{8} \times 0.095 \times 70^2$$

$$= 58.2 \text{ kip-ft}$$

A9.3—Live Load Analysis—Interior Girder

Type (g) cross section. LRFD Design Table 4.6.2.2.1-1

The beams are transversely post-tensioned to act as a unit.

A9.3.1—Compute Live Load Distribution Factors for an Interior Beam (LRFD Design Table 4.6.2.2.2b-1)

N_b = 12

$$k = 2.5\left(N_b\right)^{-0.2} \geq 1.5$$

$$= (2.5)(12)^{-0.2} = 1.52 \qquad \text{Say } 1.5$$

I = 110,499 in.^{4}

b = 48 in.

For closed thin-walled shapes: LRFD Design Eq. C.4.6.2.2.1-3

$$J = \frac{4A_o^2}{\Sigma \frac{s}{t}}$$

A_o = Area enclosed by the centerlines of elements

$$= (48 - 5)(33 - 5\ ^{1}/_{2}) = 1{,}182.5 \text{ in.}^2$$

s = Length of a side element

$$J = \frac{4 \times 1{,}182.5^2}{\frac{2(48-5)}{5.5} + \frac{2(33-5.5)}{5}}$$

$$= 209{,}985 \text{ in.}^4$$

A9.3.1.1—Distribution Factor for Moment

One Lane Loaded:

$$g_{m1} = k\left(\frac{b}{33.3L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25}$$

$$= 1.50\left(\frac{48}{33.3\times 70}\right)^{0.50}\left(\frac{110,499}{209,985}\right)^{0.25}$$

$$= 0.183$$

Two or More Lanes Loaded:

$$g_{m2} = k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}$$

$$= 1.50\left(\frac{48}{305}\right)^{0.6}\left(\frac{48}{12\times 70}\right)^{0.2}\left(\frac{110,499}{209,985}\right)^{0.06}$$

$$= 0.268 > 0.183$$

$$g_m = g_{m2} = 0.268$$

A9.3.2—Maximum Live Load (HL-93) Moment at Midspan

Design Lane Load:

$$0.64\,\text{klf} \times \frac{(70\ \text{ft})^2}{8} = 392.0\ \text{kip-ft}$$

Design Truck (with the middle axle positioned at midspan):

$$\frac{32^K \times 70\,\text{ft}}{4} + \frac{\left(8^K + 32^K\right)\times 21\,\text{ft}\times 35\,\text{ft}}{70} = 980.0\ \text{kip-ft} \quad \text{Governs}$$

Design Tandem (with tandem centered on midspan):

$$25^K \times 33\,\text{ft} = 825.0\ \text{kip-ft}$$

IM = 33 percent

M_{LL+IM} = 392.0 + 980.0 × 1.33

= 1,695.4 kip-ft

g × M_{LL+IM} = (0.268)(1,695.4)

= 454.4 kip-ft

A9.4—Compute Nominal Flexural Resistance

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

LRFD Design
Eq. 5.6.3.1.1-1

$k = 2(1.04 - \frac{f_{py}}{f_{pu}}) = 2(1.04 - \frac{229.5}{270}) = 0.38$ LRFD Design Eq. 5.6.3.1.1-1 Table 6A.5.4.2.2b-1

f_{pu} = 270 ksi

d_p = distance from extreme compression fiber to the *C.G.* of prestressing tendons

= 33 in. – 2.4 in.

= 30.6 in.

For rectangular section:

$$c = \frac{A_{ps} f_{pu}}{\alpha_1 f_c' \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$$ LRFD Design Eq. 5.6.3.1.1-4

Commented [REH1]: This doesn't print correctly.

Neglects nonprestressed reinforcement.

A_{ps} = 20 × 0.153

= 3.06 in.^{2}

b = 48 in.

f'_c = 5 ksi

α_1 = 0.85 LRFD Design 5.6.2.2

β_1 = 0.85–0.05(f_c'–4) = 0.85–0.05(5–4) = 0.80 LRFD Design 5.6.2.2

$$c = \frac{3.06 \times 270}{0.85 \times 5 \times 0.80 \times 48 \times 0.38 \times 3.06 \times \frac{270}{30.6}}$$ LRFD Design 5.6.3.1.1

= 4.76 in.

$a = \beta_1 c$ LRFD Design 5.6.2.2

= 0.80 × 4.76

= 3.81 in. < 5.5 in.

Therefore, the rectangular section behavior assumption is valid.

$$f_{ps} = 270\left(1 - 0.38 \times \frac{4.76}{30.6}\right)$$

= 254.0 ksi

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$ LRFD Design Eq. 5.6.3.2.2-1

$$= 3.06 \times 254.0\left(30.6 - \frac{3.81}{2}\right)\frac{1}{12}$$

= 1,858.6 kip-ft

A9.5—Maximum Reinforcement (C6A.5.5)

The factored resistance (φ factor) of compression controlled sections shall be limited in accordance with LRFD Design Article 5.6.2.1. This approach limits the capacity of over-reinforced (compression controlled) sections. C6A.5.5

The net tensile strain, ε_t, is the tensile strain at nominal strength and determined by strain compatibility using similar triangles. LRFD Design C5.6.2.1

Given an allowable concrete strain of 0.003 and depth to neutral axis c = 4.76 in. and a depth from the extreme concrete compression fiber to the center of gravity of the prestressing strands, d_p = 30.6 in.

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

$$\frac{0.003}{4.76\text{ in.}} = \frac{\varepsilon_t}{30.6\text{ in.} - 4.76\text{ in.}}$$

$$\varepsilon_t = 0.0163$$

For ε_t = 0.0163 > 0.005, the section is tension controlled and Resistance Factor, φ, shall be taken as 1.0. LRFD Design 5.6.2.1, 5.5.4.2

A9.6—Minimum Reinforcement 6A.5.6

Amount of reinforcement must be sufficient to develop M_r equal to the lesser of: LRFD Design 5.6.3.3

$1.33M_u$ or M_{cr}

$$M_r = \varphi M_n = (1.0)(1,858.6) = 1,858.6$$

$$M_u = 1.75(454.4) + 1.25(627.2) + 1.5(58.2) = 1,666.5$$

$1.33M_u$ = 2,216.4 > M_r check $M_r \geq M_{cr}$

$$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$$

LRFD Design Eq. 5.6.3.3-1

Where a monolithic or noncomposite section is designed to resist all the loads, S_{nc} is substituted for S_c. Therefore:

$$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_{nc}\right]$$

γ_3 = 1.0 for prestressing steel LRFD Design 5.6.3.3

γ_1 = 1.6

γ_2 = 1.1 for bonded tendons

S_{nc} = S_b = 6,767 in.^{3}

f_{cpe} = compressive stress in concrete due to effective prestress force (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads

$$f_{cpe} = \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b}$$

where:

P_{pe} = effective prestress force

Modulus of Rupture: LRFD Design 5.4.2.6

$$f_r = 0.24\lambda\sqrt{f_c'}$$

λ = 1.0

$$f_r = 0.24(1.0)\sqrt{5}$$ LRFD Design 5.4.2.8

$$= 0.537 \text{ ksi}$$

A9.6.1—Determine Effective Prestress Force, P_{pe}

P_{pe} = $A_{ps}f_{pe}$

Total Prestress Losses: LRFD Design Eq. 5.9.3.1-1

Δf_{pT} = $\Delta f_{pES} + \Delta f_{pLT}$ immediately before transfer

Effective Prestress:

f_{pe} = Initial Prestress – Total Prestress Losses

A9.6.1.1—Loss Due to Elastic Shortening, Δf_{pES} (LRFD Design 5.9.3.2.3a)

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp}$$ LRFD Design Eq. 5.9.3.2.3a-1

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e^2}{I} - \frac{M_D e}{I}$$

Initial Prestress immediately prior to transfer = $0.7f_{pu}$ if not available in plans. 6A.5.2.3
(This value assumes stress-relieved strand.)

For estimating P_i immediately after transfer, use $0.90(0.7f_{pu})$. LRFD Design C5.9.3.2.3a

P_i = $0.90 \times (0.7 \times 270)\ 20 \times 0.153$

= 520.5 kips

A = 753 in.^{2}

I = 110,499 in.^{4}

e = 16.5 in. – 2.4 in.

= 14.1 in.

M_D = Moment due to self-weight of the member

$$= \frac{1}{8} \times 0.815 \times 70^2 = 499.2 \text{ kip-ft}$$

f_{cgp} = $\frac{520.5}{753}+\frac{520.5\times14.1^2}{110,499}-\frac{499.2\times14.1\times12}{110,499}$

= 0.691 + 0.936 – 0.764

= 0.863 ksi

E_{ct} = $33,000\,K_1\,(w_c)^{1.5}\sqrt{f'_{ct}}$ LRFD Design Eq. C5.4.2.4-2

= $33,000(1.0)(0.145)^{1.5}\sqrt{4.0}$

= 3,644 ksi

E_p = 28,500 ksi LRFD Design 5.4.4.2

Δf_{pES} = $\frac{28,500}{3,644}\times0.863$ LRFD Design Eq. 5.9.3.2.3a-1

= 6.750 ksi

A9.6.1.2—Refined Estimates of Time-Dependent Losses (LRFD Design 5.9.3.4)

Δf_{pLT} = $(\Delta f_{pSR}+\Delta f_{pCR}+\Delta f_{pR1})id+(\Delta f_{pSD}+\Delta f_{pCD}+\Delta f_{pR2}-\Delta f_{pSS})df$ LRFD Design Eq. 5.9.3.4-1

$(\Delta f_{pSD}+\Delta f_{pCD}+\Delta f_{pR2}-\Delta f_{pSS})=0$ (No composite deck)

Shrinkage of Girder Concrete

Δf_{pSR} = $E_{bid}E_pK_{id}$ LRFD Design Eq. 5.9.3.4.2a-1

E_{bid} = $K_sK_{hs}K_fK_{td}0.48\times10^{-3}$ LRFD Design Eq. 5.4.2.3.3-1

K_s = $1.45-0.13\left(\frac{V}{S}\right)\geq1.0$ LRFD Design Eq. 5.4.2.3.2-2

$\frac{V}{S}$ = $\frac{753}{(48+48+33+33+81)}$ = 3.10 in.

K_s = 1.45–0.13(3.10) = 1.05

K_{hs} = 2.00–0.014H LRFD Design Eq. 5.4.2.3.3-2

H = average annual humidity; assume 70 percent

K_{hs} = 2.00–0.014(70) = 1.02

K_f = $\frac{5}{1+f'_{ci}}$ LRFD Design Eq. 5.4.2.3.2-4

K_f = $\frac{5}{1+4}=1.0$

K_{td} = $\frac{t}{12\left(\frac{100-4(f'_{ci})}{f'_{ci}+20}\right)+t}$ LRFD Design Eq. 5.4.2.3.2-5

For time development factor at time of transfer of prestressing force, assume 0.75 days.

$$K_{td1} = \frac{0.75}{12\left(\frac{100-4(4)}{4+20}\right)+0.75} = 0.018$$

For time development factor at final design time, assume 30 years.

t_f = 30 × 365 = 10,950 days

$$K_{td2} = \frac{10,950}{12\left(\frac{100-4(4)}{4+20}\right)+10,950} = 0.996$$

E_{bid} = 1.05(1.02)(1.0)(0.996–0.018)(0.48×10^{-3}) = 5.02×10^{-4}

$$K_{id} = \frac{1}{1+\frac{E_p}{E_{ci}}\frac{A_{ps}}{A_g}\left(1+\frac{A_g e_{pg}^2}{I_g}\right)\left[1+0.7\psi_b\left(t_f,t_i\right)\right]}$$

LRFD Design Eq. 5.9.3.4.2-2

$$\psi_b\left(t_f,t_i\right) = 1.9K_s K_{hc} K_f K_{td} t_i^{-0.118}$$

LRFD Design Eq. 5.4.2.3.2-1

K_{hc} = 1.56 – 0.008H

LRFD Design Eq. 5.4.2.3.2-3

= 1.56 – 0.008(70) = 1.0

$$\psi_b\left(t_f,t_i\right) = 1.9\left(1.05\right)\left(1.0\right)\left(1.0\right)\left(0.996-0.018\right)\left(0.75\right)^{-0.118}$$
$$= 2.02$$

$$K_{id} = \frac{1}{1+\left(\frac{28,500}{3,644}\right)\left(\frac{3.06}{753}\right)\left(1+\frac{753\left(14.1\right)^2}{110,499}\right)\left[1+0.7\left(2.02\right)\right]}$$
$$= 0.847$$

f_{pSR} = (5.02 × 10^{-4})(28,500)(0.847) = 12.12 ksi

Creep of Girder Concrete

$$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp}\psi_b\left(t_d,t_i\right)K_{id}$$

LRFD Design Eq. 5.9.3.4.2b-1

$$= \frac{28,500}{3,644}\left(0.863\right)\left(2.02\right)\left(0.847\right)$$
$$= 11.55 \text{ ksi}$$

Relaxation of Prestressing Strands

$$\Delta f_{pR1} = \frac{f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}}-0.55\right)$$

LRFD Design Eq. 5.9.3.4.2c-1

f_{pt} = max(0.70(f_{pn}) – Δf_{pES}, 0.55f_{py})

0.70(f_{pn}) – Δf_{pES} = 0.70(270) – 6.750 = 182.25 ← Governs

0.55f_{py} = 0.55(0.85)(270) = 126.23

K_L = 7.0 (Stress Relieved Strands)

$$\Delta f_{pR1} = \frac{182.25}{7}\left(\frac{182.25}{0.85(270)} - 0.55\right) = 6.36 \text{ ksi}$$

Total Time Dependent Losses

Δf_{pLT} = $\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}$

= 12.12 + 11.55 + 6.36 = 30.03 ksi

A9.6.1.3—Total Prestress Losses, Δf_{pT}

Δf_{pT} = $\Delta f_{pES} + \Delta f_{pLT}$ LRFD Design Eq. 5.9.3.1-1

= 6.75 + 30.03

= 36.78 ksi

f_{pe} = Initial Prestress – Total Prestress Losses

= (0.7 × 270) –36.78

= 152.22 ksi

P_{pe} = 152.22 × 20 × 0.153

= 465.79 kips

$$f_{cpe} = \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b}$$

$$= \frac{465.79}{753} + \frac{465.79(16.5-2.4)}{6,767}$$

= 1.589 ksi

$$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_{nc}\right]$$

$$= (1.0)\left[\left(1.6(0.537)+1.1(1.589)\right)6,767\times\frac{1}{12}\right]$$

= 1,470.2 kip-ft

M_r = φM_n

= 1.0 × 1,858.6 = 1,858.6 kip-ft

M_r = 1,858.6 > M_{cr} = 1,470.2 OK

Minimum reinforcement check is satisfied. 6A.5.6

A9.7—General Load-Rating Equation (6A.4.2)

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)}$$ Eq. 6A.4.2.1-1

A9.7.1—Evaluation Factors for Strength Limit States

A9.7.1.1—Resistance Factor, φ LRFD Design 5.5.4.2

φ = 1.0 for flexure

A9.7.1.2—Condition Factor, φ_c 6A.4.2.3

φ_c = 1.0 no deterioriation

A9.7.1.3—System Factor, φ_s 6A.4.2.4

φ_s = 1.0

A9.7.2—Design Load Rating (6A.4.3)

A9.7.2.1—Strength I Limit State (6A.5.4.1)

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

Table 6A.4.2.2-1

Load	Inventory	Operating	
DC	1.25	1.25	
DW	1.5	1.50	Asphalt thickness was not field measured
LL + IM	1.75	1.35	

A9.7.2.1a—Flexure at Midspan

Inventory:

$$RF = \frac{(1.0)(1.0)(1.0)(1{,}858.6)-(1.25)(627.2)-(1.50)(58.2)}{(1.75)(454.4)}$$

$$= 1.24$$

Operating:

$$RF = 1.24\times\frac{1.75}{1.35}$$

$$= 1.61$$

Shear need not be checked for the design load as the bridge does not exhibit signs of shear distress.

A9.7.2.2—Service III Limit State for Inventory Level (6A.5.4.1)

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

Flexural Resistance:

f_R = f_{pb} + Allowable tensile stress

f_{pb} = compressive stress due to effective prestress

= 1.589 (See previous calculation, A9.6.1.3)

Allowable Tensile Stress $= 0.19\sqrt{f'_c}$ — LRFD Design Table 5.9.2.3.2b-1

$= 0.19\sqrt{5}$

= 0.425 ksi

f_R = 1.589 + 0.425

= 2.014 ksi

Dead Load Stress:

$$f_{DC} = \frac{627.2 \times 12}{6,767} = 1.112 \text{ ksi}$$

$$f_{DW} = \frac{58.2 \times 12}{6,767} = 0.103 \text{ ksi}$$

Total f_D = 1.215 ksi

Live Load Stress:

$$f_{LL+IM} = \frac{454.4 \times 12}{6,767} = 0.806 \text{ ksi}$$

γ_L = 0.80

γ_D = 1.0 — Table 6A.4.2.2-1

$$RF = \frac{2.014 - (1.0)(1.215)}{(0.80)(0.806)}$$

= 1.24 > 1.0 OK

A9.7.3—Legal Load Rating (6A.4.4)

Inventory design load rating *RF* > 1.0, therefore the legal load ratings do not need to be performed and no posting is required. — 6A.4.3.1

A9.7.4—Permit Load Rating (6A.4.5)

Permit Type: Routine

Permit Weight: 220 kips

Permit Vehicle: Shown in Example A1, Figure A1A.1.10-1

ADTT (one direction): 4,600

From Live Load Analysis by Computer Program:

Undistributed maximum:

M_{LL} = 2,592 kip-ft

A9.7.4.1—Strength II Limit State 6A.4.5.4.2a

$$\frac{GVW}{AL} = \frac{220}{51} = 4.3 > 3.0$$ Table 6A.4.5.4.2a-1

$$\gamma_L = \frac{1.30-1.20}{5,000-1,000} = \frac{\gamma_L - 1.20}{4,600-1,000}$$

$$= 1.29$$

For a routine permit, use a multi-lane loaded distribution factor. 6A.4.5.4.2a

g_m = 0.268 (two lanes loaded distribution factor)

IM = 10 percent Field inspection verified: Smooth Riding Surface Table C6A.4.4.3-1

Distributed Live Load Effects:

$$M_{LL+LL} = (2,592)(0.268)(1.10) = 764.1 \text{ kip-ft}$$

Flexure:

$$RF = \frac{(1.0)(1.0)(1.0)(1,858.6) - 1.25(627.2) - 1.5(58.2)}{(1.29)(764.1)}$$

RF = 1.00 = 1.0 OK

Note: Permit trucks should be checked for shear incrementally along the length of the member. Not illustrated here; see Example A3. 6A.5.8

A9.7.4.2—Service I Limit State 6A.6.4.2.2

$\gamma_L = \gamma_{DC} = \gamma_{DW} = 1.0$ Table 6A.4.2.2-1

LRFD distribution analysis methods as described in LRFD 4.6.2 should be used. C6A.6.4.2.2

g_m = 0.268

M_{LL+LL} = (2,592)(0.268)(1.10) = 764.1 kip-ft

M_{DC} = 627.2 kip-ft

M_{DW} = 58.2 kip-ft

M_{cr} = 1,470.2 kip-ft (previously calculated)

f_{pe} = 159.3 ksi (previously calculated)

$M_{DC} + M_{DW} + M_{LL+IM} - M_{cr}$ = 627.2 + 58.2 + 764.1 – 1470.2 = –20.70 kip-ft

A9.7.4.2a—Simplified check using $0.75M_n$ C6A.5.4.2.2b

$M_{DC} + M_{DW} + M_{LL+IM} = 1449.5$ *kip-ft*

$0.75M_n$ = $0.75 \times 1{,}858.6$ kip-ft

= 1,394.0 kip-ft < 1,449.5 kip-ft NO GOOD

Moment Ratio:

$$\frac{0.75\,M_n}{M_{DC} + M_{DW} + M_{LL+IM}} = \frac{1{,}394.0}{1{,}449.5} = 0.96 < 1.0$$ NO GOOD

A9.7.4.2b—Refined check using $0.90f_y$

Calculate stress in outer reinforcement at midspan. Stress due to moments in excess of the cracking moment acts upon the cracked section. The moments up to the cracking moment cause stress in the reinforcement equal to the effective prestress.

f_R = $0.9f_y = 0.9(0.85f_{pu}) = 0.9(0.85 \times 270) = 206.6$ ksi Table 6A.5.4.2.2b-1

Section Properties for the Cracked Section:

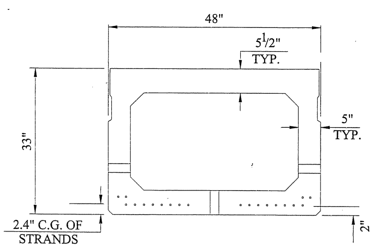

Figure A9.7.4.2b-1—Box Beam Cross Section

Assume neutral axis is in the top flange.

A_{ps} = 3.06 in.^{2} LRFD Design 5.6.1

f'_c = 5 ksi

Effective modular ratio of $2n$ is applicable

n = $\dfrac{E_p}{E_c} = \dfrac{28{,}500}{4{,}000}$

n = 7; therefore, $2n = 14$

A_{trans} = $A_{ps}2n = 3.06 \times 14 = 42.8\text{in.}^2$

$$c = \frac{\frac{c}{2}(b)(c) + (d_e)(A_{trans})}{(b)(c) + A_{trans}}$$

$$c = \frac{\frac{c}{2}(48)(c) + (33 - 2.4)(42.8)}{(48)(c) + 42.8}$$

$$24c^2 + 42.8c - 1,309.7 = 0$$

Solving for c:

c = 6.55 in. > 5.50 in. assumed; therefore, find neutral axis depth by trial and adjustment

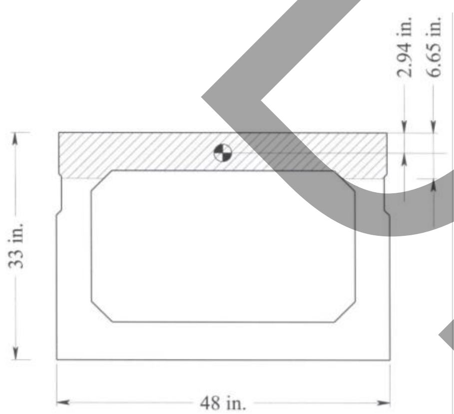

Figure A9.7.4.2b-2—Box Beam Cross Section for Determining *c*

Table A9.7.4.2b-1—Trial and Adjustment Values for *c*

Trial c	Centroid	Area Concrete	Calculated c	Difference Trial – Calculated
5.5	2.75	264.0000	6.6383	–1.138
5.8294	2.8041	264.7872	6.6749	–0.846
6.3	2.8827	271.3392	6.6621	–0.362
6.6	2.9318	275.1264	6.6596	–0.060
6.65	2.9400	275.7456	6.6594	–0.009
6.7	2.9482	276.3504	6.6595	+0.041

By trial and adjustment, c approximately equals 6.65:

$$c = \frac{2.94(275.7456) + \left[(16 \times .153\text{ in.}^2 \times 31\text{ in.}) + (4 \times .153\text{ in.}^2 \times 29\text{ in.})\right] \times 14}{(275.7456) + (20 \times .153\text{ in.}^2 \times 14)} = 6.65$$

$$I_{cr} = \begin{bmatrix} \frac{1}{12}(47.25)(5.5)^3+(47.25)(5.5)\left(6.65-\frac{5.5}{2}\right)^2+ \\ 2\times\left[\frac{1}{12}(6.9)(1.15)^3+(6.9)(1.15)(0.58)^2\right]+ \\ (42.8)(33-2.4-6.65)^2 \end{bmatrix} = 29165 \text{ in.}^4$$

Stress beyond the effective prestress (increase in stress after cracking):

$$f = n\frac{M_y}{I} = 7\frac{(-20.7)(12)(33-2-6.65)}{29{,}165} = -1.45 \text{ ksi}$$

Stress in the reinforcement at permit crossing Service I:

$$f_s = 152.2-1.45 = 150.75 \text{ ksi} < f_R = 0.9F_y = 206.6 \text{ ksi} \quad \text{OK}$$

Stress Ratio:

$$\frac{0.9f_y}{f_s} = \frac{206.6}{150.75} = 1.37 > 1.0$$ Allowable steel stress; actual on cracked section

OK

For this bridge, the simplified check indicates that the Service I condition is violated for the permit truck; the more detailed check indicates that the condition is acceptable.

A9.8—IoH Tier 1 Load Rating (6A.4.6)

Inventory design load rating *RF* > 1.0, therefore the ioH Tier 1 load rating does not need to be performed. MBEIoH 2A.4.3.1

A9.9—Summary of Rating Factors

Table A9.9-1—Summary of Rating Factors—Interior Box Beam

Limit State		Design Load Rating (HL-93) Inventory	Design Load Rating (HL-93) Operating	Permit Load Rating
Strength I	Flexure	1.24	1.61	—
Strength II	Flexure	—	—	1.00
Service III		1.24	—	—
Service I	Approximate	—	—	Stress Ratio = 0.96
	Refined	—	—	Stress Ratio = 1.37

A9.10—Reference

AASHTO. *AASHTO LRFD Bridge Design Specifications*, Eighth Edition, LRFD-8. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

Fu,G., Q.Wang, J.Chi, M.Lwin, and R.Corotis NCHRP Report ??? *Proposed New AASHTO Load Rating Provisions for Implements of Husbandry*, Transportation Research Board, 202?