CONDITION ASSESSMENT OF EXISTING BRIDGE STRUCTURES

GDOT Project No. RP05 - 01

REPORT OF TASK 1 – APPRAISAL OF STATE-OF-THE-ART OF BRIDGE CONDITION ASSESSMENT

Prepared for

GEORGIA DEPARTMENT OF TRANSPORTATION

By

Naiyu Wang
Bruce R. Ellingwood
Abdul-Hamid Zureick
Curtis O'Malley

GEORGIA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

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Appraisal of State-of-the-art of Bridge Condition Assessment

ABSTRACT

Condition assessment and safety verification of existing bridges and decisions as to whether posting is required currently are addressed through analysis, load testing, or a combination of methods. Bridge rating through structural analysis is by far the most common procedure for rating existing bridges. Load testing may be indicated when analysis produces an unsatisfactory result or when the analysis cannot be completed due to lack of design documentation, information, or the presence of deterioration. The current rating process is described in the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Bridge Evaluation, First Edition*, which allows ratings to be determined through allowable stress methods (AS), load factor methods (LF), or load and resistance factor methods (LRFR), the latter of which is keyed to the new AASHTO *LRFD Bridge Design Specifications*, which now is required for the design of new bridges, effective October, 2007. The State of Georgia currently utilizes the LF method. These three rating methods may lead to different rated capacities and posted limits for the same bridge, a situation that carries serious implications with regard to the safety of the public and the economic well-being of communities that may be affected by bridge postings or closures. To address this issue, the Georgia Institute of Technology has conducted a research program, sponsored by the Georgia Department of Transportation, leading to improvements to the process by which the condition of existing bridge structures in the State of Georgia is assessed and a set of *Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia.* The research program has four tasks. This report summarizes Task 1 – Appraisal of the state-of-the-art for bridge condition assessment.

KEY WORDS:

Bridges; concrete (reinforced); concrete (pre-stressed); condition assessment; loads (forces); reliability; risk; structural engineering.

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Executive Summary

Bridge structures in the United States are at risk from aging, leading to structural deterioration from aggressive environmental attack and other physical mechanisms, service demands from increasing traffic and heavier loads, and deferred maintenance. Condition assessments of an existing bridge may be conducted to develop a bridge load rating, confirm an existing load rating, increase a load rating for future traffic, or determine whether the bridge must be posted in the interest of public safety. Changes in traffic patterns; concern about faulty building materials or construction methods; discovery of a design/construction error after the structure is in service; concern about deterioration discovered during routine inspection; and damage following extreme load events may prompt such evaluations. In the State of Georgia, rating calculations have yet to be performed on 1,587 of the bridges that the GDOT monitors. Moreover, approximately 1,982 of the 8,988 bridges monitored by the Georgia Department of Transportation (GDOT) have been determined to require posting. Posting or other restrictive actions may have a severe economic impact on the State economy, which depends on the trucking industry for distribution of resources and manufactured goods. The economics of upgrading or posting a bridge makes it imperative that condition assessment criteria and methods (either by analysis or by testing) be tied in a rational and quantitative fashion to public safety, function and economics.

Condition assessment and safety verification of existing bridges, and decisions as to whether posting is required are addressed through analysis, load testing, or a combination of methods. Bridge rating through structural analysis is by far the most common (and most economical) procedure for rating existing bridges. Load testing may be indicated when analysis produces an unsatisfactory result or when the analysis cannot be completed due to lack of design documentation, information, or the presence of deterioration. The customary rating process is described in the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Condition Evaluation of Bridges, Second Edition*,[1] which allows ratings to be determined through either allowable stress methods (AS) or load factor methods (LF). The State of Georgia currently utilizes the LF method for most bridges that have been rated. A third rating procedure found in the *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*,[2] is keyed to the new AASHTO Load and Resistance Factor Design (LRFD) method, defined in the *LRFD Bridge Design*

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[1] American Association of State Highway and Transportation Officials (1994). *Manual for Condition Evaluation of Bridges, 2nd Edition (with 1995, 1996, 1998 and 2000 interim revisions)*. AASHTO, Washington D.C.

[2] American Association of State Highway and Transportation Officials (2003). *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges (including 2005 interim revisions)*. AASHTO, Washington D.C.

Specifications, 4th Edition[3]. The LRFR method is being introduced to the bridge maintenance community, and some states are beginning to use it in developing their bridge ratings.[4] These three competing rating methods may lead to different rated capacities and posted limits for the same bridge, a situation that cannot be justified from a professional engineering viewpoint and carries serious implications with regard to the safety of the public and the economic well-being of businesses and individuals who may be affected by bridge postings or closures.

The Georgia Department of Transportation has an urgent need for condition assessment tools that can be used with confidence to determine whether or not to post certain existing bridge structures. To address this need, the Georgia Institute of Technology has conducted a multi-year research program, sponsored by the GDOT, aimed at making improvements to the process by which the condition of existing bridge structures in the State of Georgia is assessed. The end product of this research program is a *Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia*, for practical use by the GDOT in rating bridges. The *Recommended Guidelines* will address condition assessment and evaluation by analysis, load test, or a combination of the two methods, depending on the circumstances and preferences of the GDOT. They will have a sound basis in structural engineering, allowing them to be updated as changing circumstances (traffic demands, additional data, material deterioration, etc) warrant, and will be presented in a relatively simply and familiar form that is suitable for implementation in routine rating assessments.

The research program undertaken for the State of Georgia to develop the *Recommended Guidelines* has four tasks:

Task 1 – Review and critical appraisal of the state-of-the-art of bridge condition assessment
Task 2 – Bridge evaluation by load testing
Task 3 – Advanced analysis techniques
Task 4 – Development of *Recommended Guidelines for Condition Assessment and Evaluation*

This report summarizes the accomplishments in Task 1. A comprehensive review has been undertaken of current procedures for performing condition assessments of existing civil infrastructure, with particular emphasis on bridge structures. The goal of this review was to gain a perspective on technical and other issues associated with condition assessment techniques that have been used successfully in other civil infrastructure applications. As part of this review, ongoing activities in other Departments of Transportation in selected states were scrutinized and current practices

[3] American Association of State Highway and Transportation Officials (2007). *AASHTO LRFD Bridge Design Specifications(including 2008 and 2009 interim revisions), 4th Edition.* AASHTO, Washington D.C.

[4] The *Manual for Condition Evaluation of Bridges* and the *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* have been effectively combined in the new *Manual for Bridge Evaluation, First Edition* (2008).

with regard to bridge inspection, including underwater inspection of load-bearing components, analysis and load testing, and posting of bridges were examined. Reliability-based condition assessment tools and the existence of databases to model uncertainty that would support bridge assessment by the GDOT also were reviewed. Finally, four bridges were selected from the Georgia bridge inventory, with the assistance of bridge engineering staff from the Georgia Department of Transportation, to serve as testbeds for the development of improved rating procedures. Task 1 has established the context for the bridge testing and advanced analysis in Tasks 2 and 3 and for the *Guidelines* developed in Task 4.

CHAPTER 1

Introduction

1.1 BACKGROUND

Bridge structural systems in the United States are at risk from structural aging, leading to structural deterioration from aggressive chemical attack, corrosion, and other physical mechanisms. The problem is amplified by service demands from increasing traffic and heavier loads, coupled with deferred maintenance [Moses, et al, 1994; Fu and Tang, 1995; Saraf and Nowak, 1998]. In the State of Georgia, approximately 1,587[1] bridges require evaluation in the short term by the Georgia Department of Transportation (GDOT) to determine whether they require posting as a result of increases in truck loads. The impact of posting or other restrictive actions on the State economy, which depends on the trucking industry for distribution of resources and manufactured goods, is potentially severe. The economics of upgrading or posting makes it imperative to determine condition assessment criteria and methods (either by analysis or by testing) that are tied in a rational and quantitative fashion to public safety, function and economics.

Bridge evaluation and condition assessment usually is done to confirm an existing load rating or to increase a load rating for future traffic. Changes in use; concern about faulty building materials or construction methods; discovery of a design/construction error after the structure is in service; concern about deterioration discovered during routine inspection; and damage following extreme load events may prompt such evaluations. Condition assessment and safety verification of existing bridges, and decisions as to whether posting is required are addressed through analysis, load testing, or a combination of the methods. Bridge rating through structural analysis is by far the most common (and most economical) procedure for rating existing bridges. Load testing may be indicated when analysis produces an unsatisfactory result or when the analysis cannot be completed due to lack of design documentation, information, or the presence of deterioration [Bakht and Jaeger, 1990]. A properly conducted load test can confirm distribution of forces within the structure [Moses, et al, 1994], validate the assumptions made in a quantitative analysis, and (at extreme load levels) provide proof of load-bearing capacity and a basis for revising the rating of the bridge.

The customary rating process is described in the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Condition Evaluation of Bridges, 2nd Edition* (1994), which allows ratings to be determined through either allowable stress methods (ASR) or load factor methods (LFR). The State of Georgia primarily utilizes the LF method for those bridges in the state that have been rated. A third rating procedure found in the *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* (2003), is keyed to the new AASHTO Load and Resistance Factor Design (LRFD) method, defined in the *LRFD*

[1] As of June 1, 2008

Bridge Design Specification, 4th Edition (2007).[2] The LRFR method is being introduced to the bridge maintenance community, and some states are beginning to use it in developing their bridge ratings. These three competing rating methods may lead to different rated capacities and posted limits for the same bridge, a situation that cannot be justified from a professional engineering viewpoint and carries serious implications with regard to the safety of the public and the economic well-being of businesses and individuals who may be affected by bridge postings or closures.

The Georgia Department of Transportation has an urgent need for condition assessment tools that can be used with confidence to determine whether or not to post certain existing bridge structures. To address this need, the Georgia Institute of Technology has launched a multi-year research program, sponsored by the GDOT, aimed at making improvements to the process by which the condition of existing bridge structures in the State of Georgia is assessed. The end product of this research program is a *Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia*, for practical use by the GDOT in rating bridges. The *Recommended Guidelines* will address condition assessment and evaluation by analysis, load test, or a combination of the two methods, depending on the circumstances and preferences of the GDOT. They will have a sound basis in structural engineering, allowing them to be updated as changing circumstances (traffic demands, additional data, material deterioration, etc) warrant, and will be presented in a relatively simply and familiar form that is suitable for implementation in routine rating assessments.

1.2 RESEARCH OBJECTIVES AND SCOPE

This research will provide the Office of Bridge Maintenance of the State of Georgia Department of Transportation with a set of rational engineering tools for evaluating the need for posting of bridges, for establishing priorities for bridge inspection and rehabilitation, and for determining appropriate strategies for assess fitness-for-purpose of bridges through analysis or load testing. The research program undertaken has four tasks:

Task 1 – Review and critical appraisal of the state-of-the-art of bridge condition assessment
Task 2 – Bridge evaluation by load testing
Task 3 – Advanced analysis techniques
Task 4 – Development of *Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia*

[2] During the period in which the research reported herein was conducted, the two AASHTO rating manuals available to the research team were the *Manual for Condition Evaluation of Bridges, Second Edition* and the *Manual for Condition Evaluation and Load and Resistance Factor Rating, First Edition.* Accordingly, the report of Task 1 is based on the AASHTO documents that were available at the time that the research was performed. The *Manual for Condition Evaluation of Bridges* and the *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* have been effectively combined in the new *Manual for Bridge Evaluation, First Edition* (2008), which became available in 2009. A close scrutiny of the provisions in the new *MBE* has revealed that none of the findings and recommendations in the Report of Task 1are affected by the new document.

This report summarizes the accomplishments in Task 1. A comprehensive review has been undertaken of current procedures for performing condition assessments of existing civil infrastructure, with particular emphasis on bridge structures. The goal of this review was to gain a perspective on technical and other issues associated with condition assessment techniques that have been used successfully in other civil infrastructure applications. As part of this review, ongoing activities in other Departments of Transportation in selected states were scrutinized and current practices with regard to bridge inspection, including underwater inspection of load-bearing components, analysis and load testing, and posting of bridges were examined. Reliability-based condition assessment tools and the existence of databases to model uncertainty that would support bridge assessment by the GDOT also were reviewed, with particular reference to papers and reports that provided technical support for the *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* (2003). This review has established the context for the bridge testing and advanced analysis in Tasks 2 and 3 and for the *Guidelines* developed in Task 4.

CHAPTER 2

Review of Literature on Bridge Evaluation

This chapter reviews current standardized procedures for performing condition assessments of existing bridge structures. The review emphasizes current practices in the United States, but practices in several other industrialized countries are also summarized to provide additional context. This review is aimed at achieving a general perspective on technical issues associated with condition assessment methodologies used for bridges and other civil infrastructure applications. Reliability-based design and condition assessment tools and databases to model uncertainty that would support bridge assessment were reviewed, with particular reference to papers and reports that provide technical support for the AASHTO *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*.

2.1 CURRENT AASHTO GUIDELINES FOR BRIDGE EVALUATION

2.1.1 Bridge Rating by ASR, LFR and LRFR

Until 1970, the sole design philosophy embedded within *AASHTO Standard Specifications for Highway Bridges* was Allowable Stress Design (ASD). The allowable stress is established as a fraction of the load carrying capacity of a structural element (usually the yield or fracture strength in tension or point of instability in compression), and the structural action (stress in tension, bending or compression) from the applied loads may not exceed this allowable limit. Detailed procedures for rating existing bridges based on the ASD method first appeared in 1970 in the *AASHTO Manual for Maintenance Inspection of Bridges*.

Beginning in the early 1970's, as design of reinforced concrete and steel structures was reformulated in terms of "ultimate strength" for concrete and "plastic" design for steel, the load analysis formerly used in ASD was modified as well, with adjustments to the design factors to reflect the relative uncertainty and predictability of different loads, such as vehicle loads, wind and earthquake effects. The new design philosophy was referred to as Load Factor Design (LFD) and was incorporated in the *Manual for Condition Evaluation of Bridges* (*MCE*), which was published by AASHTO in 1994 to replace the earlier *Manual for Maintenance Inspection of Bridges.* Although the 1994 manual contains some guidance for allowable stress rating (ASR), it clearly emphasized the load factor rating (LFR) method. Many State DOTs continue to use the 1994 *Manual*, with 1995, 1996, 1998 and 2000 interim revisions, in their bridge rating work.

In 1994, the AASHTO Bridge Subcommittee voted to adopt the *AASHTO LRFD Bridge Design Specifications* and in 1998 designated LRFD as the primary design method for highway bridges. The *LRFD Specifications* represent the first effort by

AASHTO to integrate modern principles of structural reliability and the probabilistic and statistical models of loads and resistance into the design of highway bridges. LRFD introduced the reliability-based limit states design philosophy to achieve a more uniform and controllable safety levels for each applicable limit state. To extend this philosophy to the evaluation of existing bridges, AASHTO released the 2003 *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*, which presents the first bridge load rating method in the United States to have a structural reliability basis.

At the present time, the ASR, LFR and LRFR methods of bridge rating are all in current use by State DOTs. A summary of these procedures and a critical appraisal of their relative merits are presented in this section.

Allowable Stress Rating (ASR) and Load Factor Rating (LFR)

The rating factors in both ASR and LFR are determined by [*AASHTO MCE*, 1994]:

$$RF = \frac{C - A_1 D}{A_2 L(1+I)} \tag{2.1}$$

in which *RF* is the rating factor for the live load carrying capacity (expressed as a multiple of the design live load effect (from a rating vehicle) that can be carried by the bridge), *C* is the capacity of the structural member, *D* and *L* are, respectively, the dead and live load effect on the member, I is the impact factor to be used with the live load effect, A_1 is the factor on dead load, and A_2 is the factor on live load. The Rating Factor (*RF*) can then be used to compute the rating of the bridge in tons as [*AASHTO MCE*, 1994]:

$$RT = (RF) \times W \tag{2.2}$$

where RT is the bridge member rating in tons, and W is the nominal weight (tons) of the rating truck used in determining the live load effect (*L*).

Both ASR and LFR methods rate bridges at two levels: Inventory and Operating. The Inventory rating level generally corresponds to the customary design level of stresses but reflects the existing bridge and material conditions with regard to structural deterioration. Load ratings based on the Inventory level allow a comparison of the estimated capacity of an existing bridge with the capacity for a new bridge, and therefore result in a live load which can safely carried by the existing bridge structure for an indefinite period of time. Load ratings based on the Operating rating level generally describe the maximum permissible live load to which a structure may be subjected during a limited period of time. Allowing an unlimited number of vehicles to use the bridge at the Operating level may shorten the life of the bridge [*AASHTO MCE*, 1994]. Rating at the Operating level generally is the basis for decisions regarding traffic restriction and load posting.

Although the rating factor format for ASR and LFR is the same, the load factors (A_1, A_2) and the calculation of the capacity (*C)* used in Equation (2.1) are different. In the ASR, $A_1 = A_2 = 1.0$ for both Inventory and Operating level rating; *C* depends on the rating level desired, with the higher value of *C* used for Operating level. In the LFR procedure, $A_1 = 1.3$, while A_2 equals 2.17 for Inventory rating and equals 1.3 for Operating level rating; the nominal capacity *C* is the same regardless of the rating level desired.

Load and Resistance Factor Rating (LRFR) Procedure

The general LRFR rating equation is (*AASHTO LRFR*, 2003):

$$
\begin{aligned}
&RF = \frac{C - \gamma_{CD} DC - \gamma DW \pm \gamma_P P}{\gamma_L LL(1 + IM)} \\
&C = \phi \phi_C \phi_S R_n \\
&\phi_s \phi_c \geq \ 0.85
\end{aligned}
\tag{2.3}
$$

in which *C* is the structural capacity, R_n is the nominal member resistance, *DC* is the dead-load effect of structural components and attachments, *DW* is the dead-load effect of wearing surfaces and utilities, *P* is the permanent loading other than dead loads (post-tensioning for example), *LL* is the live-load effect, *IM* is the dynamic load allowance, γ_{DC} is the load factor applied to the weight of structural components and attachments, γ_{DW} is the load factor for wearing surfaces and utilities, γ_P is the load factor for permanent loads other than dead loads, and γ_L is the live-load factor. The resistance factor (ϕ) accounts for the general uncertainties in the resistance of a bridge member in a satisfactory condition and is the same as that used in LRFD bridge design. The condition factor (ϕ_C) accounts for increasing uncertainties in bridge member resistance once its condition deteriorates, and takes a value of 0.85 for members in poor condition, 0.95 for members in fair condition, and 1.0 for members in good condition. The system factor (ϕ_S) accounts for the level of redundancy in the structure. Bridges that are less redundant or non-redundant are assigned a lower system factor and therefore have reduced calculated capacities.

The LRFR method supports bridge evaluation at three general limit states that were introduced in the *LRFD Bridge Specification*: the strength-limit state (flexural or shear capacity), the service-limit state (deflections and rotations) and the fatigue limit state. The strength limit state is fundamental for public safety and is the main determining factor for bridge posting, closure and repairing. Service and fatigue limit states are applied selectively to bridges. In the LRFR method, bridges are evaluated in a tiered three-step approach for each limit state, as shown in Figure 2.1: design load rating (HL93), legal load rating (AASHTO/state legal trucks), and permit load rating (overweight trucks).

An initial check first is performed using the HL-93 design load (Figure 2.2) using the dimensions and properties corresponding to the present *in situ* condition of a bridge.

The bridge is rated using the same live and dead load factors as those used in the *LRFD Bridge Specifications*, which were calibrated to ensure a safety index of 3.5 (discussed

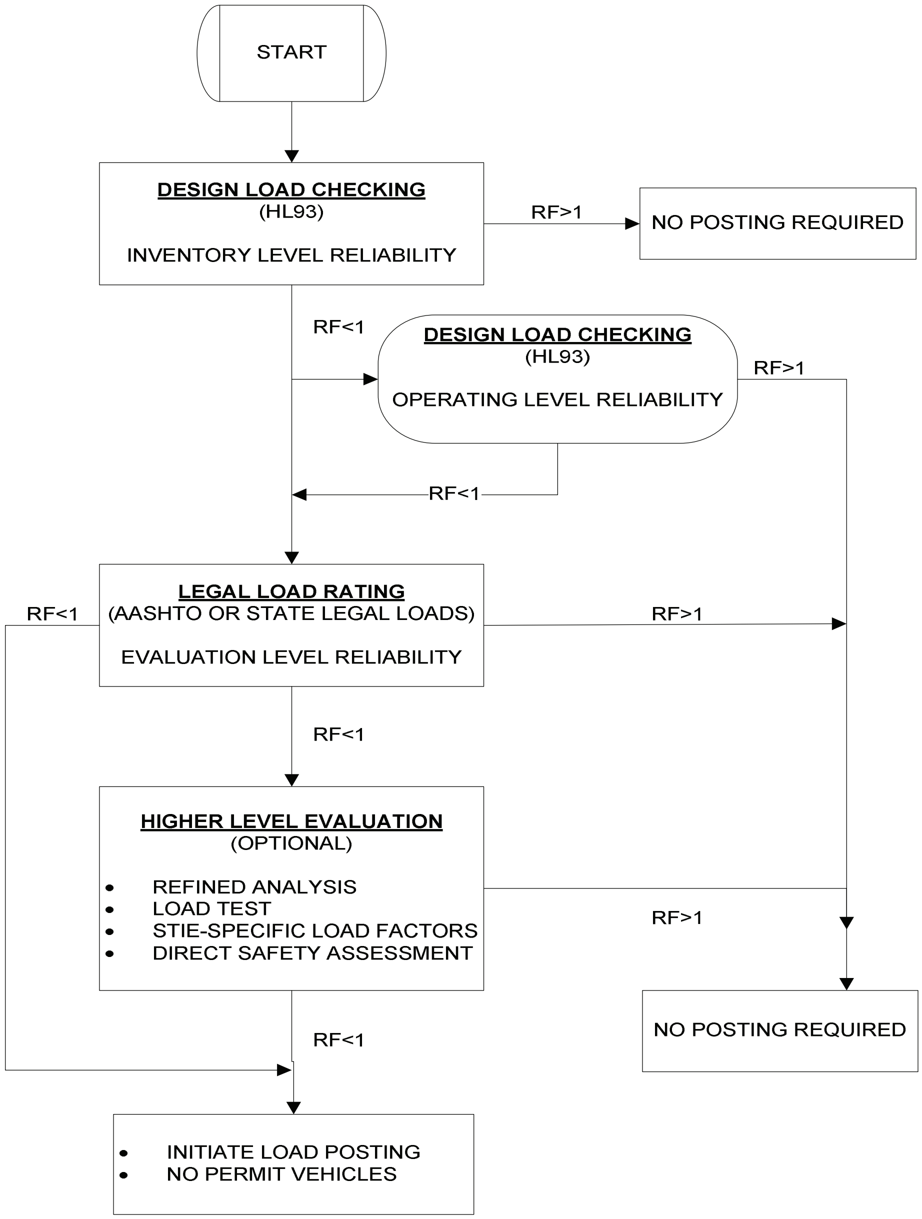

Figure 2.1 Load and Resistance Factor Rating (LRFR) Procedure

subsequently in section 2.1.2 of this chapter). This check aimed at measuring the performance of the existing bridge in comparison to the expected performance of a new bridge, and serves as an initial screening check; a bridge resulting in a RF at this level larger than 1.0 requires no further analysis for any legal loads that result in member forces lower than the HL-93 design load. For example, the HL-93 load is designed to represent the member forces caused by the AASHTO legal loads through a single load case. Therefore any State legal loads that are equal to or less than the AASHTO legal load are covered by a HL-93 design load analysis. On the other hand, if a state has legal loads that surpass the AASHTO legal loads, those states must verify that HL-93 load case incorporates those legal loads.

If the bridge fails to pass the HL-93 design load check, a follow-up evaluation is performed using the AASHTO/State legal trucks (Figure 2.3). The live load factor used at this level is calibrated to a safety index of 2.5 and varies in accordance with local truck traffic conditions at the bridge site (ADTT). The safety criteria, in comparison with the 3.5 in the previous step, are less conservative and reflect the substantial cost impact of strengthening an existing bridge or restricting traffic, as well as the shorter future service period expected compared to the 75 years that is typical for the design of a new bridge [Nowak, 1999; Moses, 2001]. The ratings determined using the legal loads are generally used as the basis for determining whether to post or strengthen the bridge.

Finally, a permit load rating may be performed to check the safety (and serviceability) of the bridge for vehicles above the legally established weight limit. This procedure is only necessary when there an overweight vehicle is to use a bridge, and it is only allowed for bridges that yield RF ≥1 at the previous legal load rating level. The permit live load factors were derived to account for the possibility of the simultaneous presence of one or more non-permit heavy trucks on the bridge when the permit vehicle crosses the span, as well as the site-specific traffic conditions described by the ADTT.

A comparison of the ratings used in the LRFR method (Eq.2.3) with those in the LFR/ASR method (Eq.2.1) shows three key improvements. First, LRFR has attempted to assess the *in situ* bridge resistance systematically and objectively through the use of the system factor (ϕ_S) and the condition factor (ϕ_C). In the LFR/ASR methods, the condition of the bridge, its redundancy, and any deterioration at the time of evaluation must be factored into the estimation of the capacity term (C) in a completely subjective manner. Second, the LRFR method considers dead load from factory-made members, cast-in-place members and wearing surfaces separately, with each assigned an independent dead load factor to account for the different degrees of variability in these components of dead load (discussed subsequently in section 2.1.2 of this chapter). In the LFR/ASR methods, all permanent loads are combined in calculating the dead load effect (*D),* to which an overall dead load factor is applied; adjustments that might be indicated by available *in situ* dead load measurements are difficult to handle in the rating process. Third, the LRFR method has provided a set of live load factors that ranges from 1.4 to 1.8, depending on the bridge's *in situ* traffic condition indicated by ADTT, for rating calculations at the legal load level. This improvement allows site-specific traffic data to be incorporated into the load rating process and present a major advantage of applying probability-based structural reliability theory in existing bridge condition assessment and an important enhancement of the LRFR method over the traditional stress-based rating approaches.

The LRFR method further simplifies the bridge rating process by requiring the use of the HL-93 design load as the starting point in the rating and as a screening check for all other AASHTO/State legal loads. The HL-93 live load envelopes all types of legal loads in the United States and provides a uniform reliability check for various span lengths with just this one load model. In contrast, the HS-20 design load used in the ASR/LFR process does not represent current trucks on the highway system and the ratings determined with this vehicle cannot provide uniform reliability for bridges of varying span lengths. So, to achieve a uniform reliability for highway bridge system, rating calculations have to be applied to all three AASHTO legal loads individually, with each controlling short, medium, or long spans respectively [NCHRP 12-28, 2001; Minervino, et al, 2004]. Finally, permit vehicles that are significantly heavier than the AASHTO/State legal loads may have very different configurations. The LRFR method provides procedures and live load factors specific to permit vehicles ratings for bridges that have been demonstrated to have adequate capacity for AAASHTO/State legal loads. The LFR/ASR methods provided no guidance on permit checking.

Despite these improvements, the LRFR procedure has not been widely adopted for rating or posting bridges in the United States. A survey of State Departments of Transportation on bridge evaluation practices (presented in Chapter 3) has revealed a number of issues and concerns with the LRFR method. Addressing these issues will facilitate the adoption of the LRFR, in a modified form, and provide an improved bridge rating methodology for the State of Georgia. Such improvements are the subject of the current research program, and are presented in Chapter 4, following the survey.

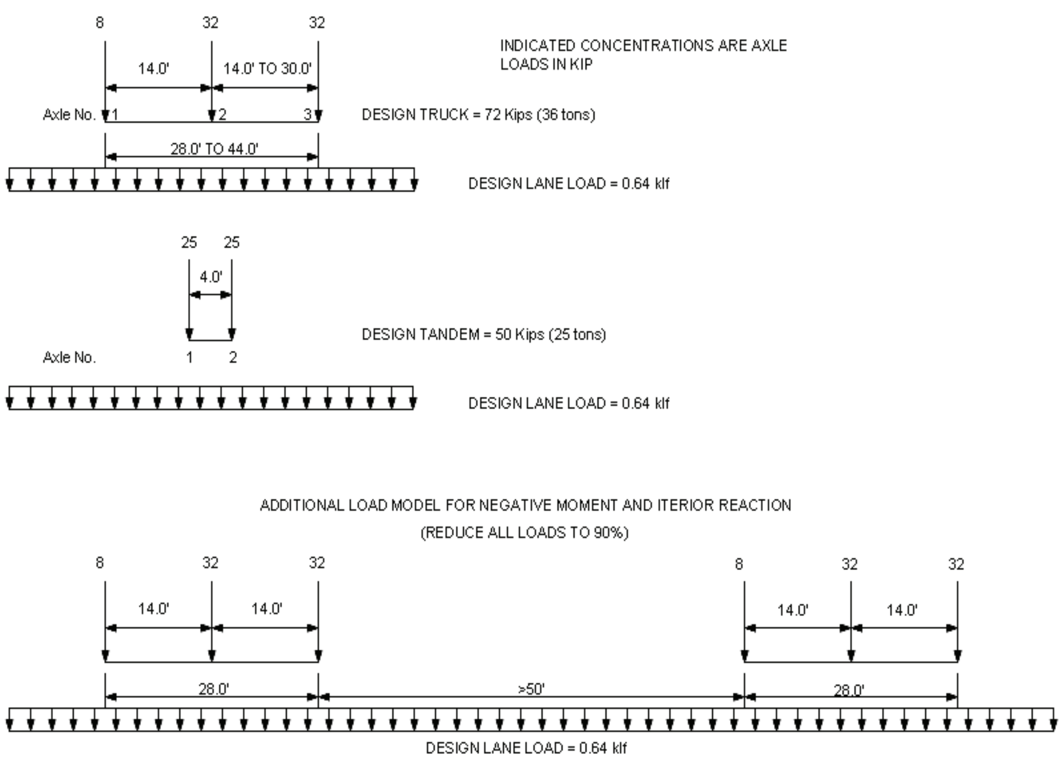

Figure 2.2 LRFD Design Live Loads (HL-93)

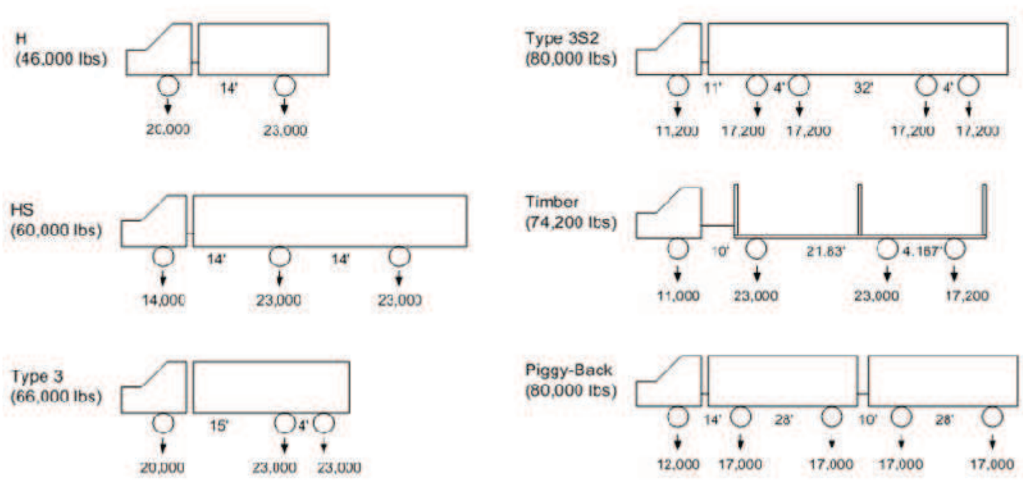

Figure 2.3 State of Georgia Legal Loads

2.1.2 Reliability-based Bridge Rating

As noted previously, the *AASHTO LRFR Guide Manual* is the first bridge load rating method in the United States to be based on modern principles of structural reliability and limit states design. The essential ingredients of a reliability-based design and evaluation include probabilistic models of the structural resistance and loads and a method for analyzing the reliabilities (or, conversely, the limit state probabilities) that are relevant to each bridge limit state. This section provides a brief summary of such methods and tools, as they have been applied to developing the *AASHTO LRFD Specifications* and the *LRFR Guide Manual* and are expected to be relevant to the current research program to develop improved rating methods for the State of Georgia. Details are available in the archival literature [Nowak, 1999; Moses, 2001].

Structural Resistance Models

The capacity of a bridge depends on the strength of its components and connections. The strength, R, is a random variable having uncertainties that fall into three categories [Moses, et al, 1987; Tabsh et al, 1992]: material properties, M, including material strength, modulus of elasticity, cracking stress and chemical composition; fabrication, F, including geometry, dimensions and section modulus; and structural modeling, P, reflecting assumptions and approximate analysis methods. The mean and coefficient of variation for M, F and P are usually determined by material tests, simulations, observations of existing structures and engineering judgment.

In the development of the LRFD bridge design specification [Nowak, 1999], R was determined as the product of the nominal resistance R_n and the three above-mentioned parameters, M, F and P:

$$R = MFPR_n \tag{2.4}$$

As a product of random variables that are assumed to be statistically independent, the resistance is modeled by a lognormal distribution with mean, μ_R, and coefficient of variation (COV), V_R, computed as follows:

$$\mu_R = R_n \mu_M \mu_F \mu_P$$
$$V_R = (V_M^2 + V_F + V_P^2)^{1/2} \quad (2.5)$$

in which μ_M, μ_F and μ_P are the means of *M*, *F* and *P* and V_M, V_F and V_P are the COVs of *M*, *F* and *P*, respectively. The statistical parameters of *R* used in the development of the *LRFD Specifications* for different types of structural components (steel girders, composite and non-composite, reinforced concrete T beams and prestressed concrete AASHTO-Type girders) in different failure modes (bending and shear) are presented in Table 2.1.

Dead Load Model

Dead load is the weight of structural members, nonstructural components and attachments, and traffic wearing surfaces. Because of the different degrees of variability, one must consider the components of bridge dead load from factory-made members (steel and pre-cast concrete), cast-in-place members (T-beams, slabs), and wearing surfaces (asphalt) separately. Generally speaking, dead load can be predicted more accurately than live loads, as long as accurate records have been kept and the as-built condition agrees with the available drawings. In the study by Moses and Verma [1987], the bias (defined as the ratio of the mean to nominal load) and COV of bridge dead loads were taken to be 1.0 and 0.10 respectively. Later in the *AASHTO LRFD* calibration [Nowak 1999], the dead load was divided into four components and each component was modeled with a normal distribution. Finally, Ghosn [2000] used 1.0 and 0.09 for the dead load bias and COV respectively in his study. These components of dead load are listed in Table 2.2 along with their statistical parameters; the "miscellaneous" category is the dead load portion from railings and luminaries.

Live Load Model

Bridge live load is produced by vehicles moving on the bridge. Variability in live load arises from uncertainties in vehicle weight, vehicle position, average daily truck traffic (ADTT), calculations of live load effect (including distribution of live load to supporting girders), and the likelihood of several heavy vehicles being on the bridge at the same time [Moses and Verma, 1987]. Traditionally, the static and the dynamic effects of the live load are considered separately and assumed to be statistically independent [Nowak, 1993; 1999].

Based on weigh-in-motion (WIM) data, Moses and Verma [1987] identified several variables to provide a simplified model for determining the maximum expected single truck load effect:

$$M = aW_{.95} mHIg \quad (2.6)$$

in which M is the predicted maximum dynamic live load effect; a is a constant which relates M to a reference loading model (taken as an AASHTO/legal rating vehicle); $W_{.95}$ is the 95th percentile characteristic value of 75-year maximum truck weight, assumed to be a random variable to reflect the possible errors (epistemic uncertainty) in load estimation and site-to-site differences; the variable m reflects the influence of the dominant vehicle type and configuration at a site; the variable H reflects the overload events due to the multiple vehicle presence, such as side by side or following vehicles, and also reflects the probability that truck weight exceeds the 95th percentile in combination with closely spaced vehicles; variable I is the dynamic impact allowance and variable g is girder distribution factor. Except for the constant a, all of the variables in Eq. (2.6) are random variables with statistics based on studies and data collected on a number of sites.

Table 2.1: Statistical Parameters Defining Component Resistance (Nowak, 1999)

Type of Structure	FM		P		R	
	Bias Factor	COV	Bias Factor	COV	Bias Factor	COV
Non-composite steel member						
Moment (compact)	1.095	0.075	1.02	0.06	1.12	0.100
Moment (noncompact)	1.085	0.075	1.03	0.06	1.12	0.100
Shear	1.12	0.08	1.02	0.07	1.14	0.105
Composite steel member						
Moment	1.07	0.08	1.05	0.06	1.12	0.100
Shear	1.12	0.08	1.02	0.07	1.14	0.105
Reinforced concrete						
Moment	1.12	0.12	1.02	0.06	1.14	0.130
Shear w/steel	1.13	0.12	1.075	0.10	1.20	0.155
Shear no steel	1.165	0.135	1.20	0.10	1.40	0.170
Prestressed concrete						
Moment	1.04	0.045	1.01	0.06	1.05	0.075
Shear w/steel	1.07	0.10	1.075	0.10	1.15	0.140

Table 2.2 Statistical Parameters of Dead Load (Nowak, 1999)

Component	Bias Factor	COV
Factory-made members	1.03	0.08
Cast-in-Place members	1.05	0.10
Asphalt	3.5 inch	0.25
Miscellaneous	1.03-1.05	0.08-0.10

The live load model used to calibrate the *AASHTO LRFD Bridge Design Specifications* is based on the weigh-in-motion data of 10,000 trucks taken at a site in Ontario in 1975, which included axle weights, gross weight and axle spacing for each vehicle [Nowak, 1999]. These 10,000 data points were assumed to define the upper 20% of the truck traffic at the site over a period of about two weeks. By finding the maximum bending moment and shear forces for each Ontario truck on different spans ranging from 10 ft (3 m) to 200 ft (60 m), the cumulative distribution functions (CDFs) of live load effect for various span lengths were obtained. Bridges with both simple spans and two continuous equal spans were considered. These CDFs were then extrapolated to a full lifetime (75 years) consisting of some 75 million truck load events and the 75-year maximum live load was fitted by a normal distribution.

Static and dynamic load effects were studied separately [Tabsh and Nowak, 1991]. On the basis of a finite element study of bridges with various span lengths, it was found that the ratio of the mean value of the 75-year maximum live load (without dynamic impact) to nominal (HL-93) live load is span-length related and its COV is about 12%. The study also concluded that dynamic impact was dependent on three major factors: bridge dynamics, vehicle dynamics and road roughness; the mean value of the dynamic load factor does not exceed 0.15 for a single truck and 0.10 for two trucks side by side, and its COV is about 80%. For the static and dynamic combined load effect, the mean of this 75-yr maximum live load with respect to the design load model (HL93 in Figure 2.1) fell in the range 1.0-1.2, depending on span length, and the COV, was found to be about 0.18.[1]

Structural Reliability Basis for Load and Resistance Factor Design and Evaluation

The starting point for a quantitative evaluation of structural reliability is the description of the limit state of concern (flexural failure, instability, etc) by an expression relating the resistance and load variables described above, derived from principles of structural mechanics. This expression, denoted the limit state function, is given by,

$$G(X) = G(X_1, X_2, \cdots X_m) = 0 \tag{2.7}$$

in which $X = (X_1, X_2, \cdots X_m)$ = vector of random resistance and load variables. The limit state is defined, by convention, as when $G(X) < 0$. Thus, the limit state probability is,

$$P_f = \int f_x(x_1, x_2, \cdots x_m)\, dx_1\ dx_2\ \cdots dx_m \tag{2.8}$$

[1]Imai and Frangopol (2001) found that the maximum bridge live load was best modeled by a Type I distribution of extreme values. Bhattacharya et al. (2006) also found that the Type I distribution fits the experimental measurements of live load effect properly. Finally, Galambos, Ellingwood et al (1982) used the Type I distribution to model the 50-year maximum live load for building structures.

in which $f_x(x)$ = joint probability density function of $\boldsymbol{X}$ and the domain of the multi-fold integration is that region of **x** where $G(\boldsymbol{X}) < 0$. The limit state probability, P_f, is the quantitative metric of structural performance that is consistent with the uncertainties in structural resistance and loads. Modern probability-based limit states design approaches, including *AASHTO LRFD Bridge Design Specifications*, have adopted the *reliability index, β*, as an alternate measure of reliability. For typical structural engineering situations, the reliability index is in the range of 2 to 4.5. The reliability index is related, in a first-order sense, to the limit state probability by $P_f = \Phi(-\beta)$ for well-behaved limit state functions typical of those found in bridge design and condition assessment.

The target reliability index of 3.5 for new bridge structures designed by LRFD, noted above, was determined by calibration to a spectrum of traditional bridge design situations (vintage 1985 and earlier) involving steel, reinforced and prestressed concrete construction. Gravity load situations were emphasized in this calibration exercise. A group of experts from the material specifications participated in assessing the results of this calibration, and selecting target reliabilities. The target index of 2.5 for existing bridges rated using AASHTO/State legal loads was determined by judgment [Moses, 2001; Minervino, et al, 2004]. In the latter case, the implied acceptable annual failure rate of an existing bridge would be at least an order of magnitude higher than a newly constructed bridge, depending on the remaining service life of the existing bridge.

2.1.3 Bridge Rating by Load Testing

Load tests may be performed to provide additional information on future behavior of the bridge, when evaluation by analysis produces an unsatisfactory result or when the analysis cannot be completed due to lack of design documentation, information, or the presence of significant deterioration. The actual performance of bridges during a load test usually is more favorable than what is indicated by the analytical evaluation [Moses, et al, 1994; Nowak, et al, 1988; Barker 2001]. The analytical rating generally is conservative due to several factors which are not considered in routine design and evaluation, including: (a) unintended composite action between non-composite sections; (b) unintended continuity/fixity of simply supported spans; (c) participation of secondary members; (d) participation of nonstructural members; and (e) contribution of the deck to the bridge load-carrying capacity. Load testing is an effective methodology to identify and benefit from the presence of these factors. The *LRFR Guide Manual* (2003), Section 8, provides procedures for the conduct of diagnostic load tests and proof load tests as alternate method for rating bridges.

Diagnostic Load Tests

Diagnostic tests are conducted to determine certain bridge response characteristics or to validate the assumptions made in a quantitative analysis. Such a test improves the bridge engineer's understanding of the behavior of the bridge and reduces uncertainties related to material properties, boundary conditions, cross-section contributions, load distribution, and other factors governing the structural performance and safety. It therefore can be used to revise an existing analytical rating of a bridge. During the test, loads should be placed at various positions on the bridge to determine the response in all critical bridge members. The *AASHTO LRFR Guide Manual*, Section 8, Eq. (8-1), provides a way to modify the analytical load rating following a diagnostic load test:

$$RF_T = RFc \times K \tag{2.9a}$$

where:

RF_T = Load-rating based on the load test result.
RFc = Rating factor based on calculations.
K = Adjustment factor to represent the benefits of the load test

If $K>1$, the response of the bridge is more favorable than predicted by theory. Conversely, if $K<1$, the response of the bridge is more severe and the theoretical bridge rating factor must be reduced. The adjustment factor K is given by Eq. (8-2) in the *LRFR Guide Manual*:

$$K = 1 + K_a \times K_b \tag{2.9b}$$

in which K_a accounts for both the benefit from load test and consideration of the section factor (area, section modulus, etc.) resisting the test load; Eq.(8-3) in the *LRFR Guide Manual* gives the general expression that should be used in determining K_a:

$$K_a = \frac{\varepsilon_c}{\varepsilon_T} - 1 \tag{2.9c}$$

where ε_T and ε_c are the maximum measured strain and the maximum theoretically predicted strain under testing load respectively. K_b in Eq. (2.8b) accounts for the understanding of the load test results when compared with those predicted by theory. In particular, it counts for the contributions that can not be depended on at the rating load level. Table 8-1 in the *LRFR Guide Manual* provides suggested value for K_b ranging from 0 to1 based on the relative magnitude between the unfactored test load and the unfactored gross rating load.

Proof Load Test

Proof load tests are carried out to establish a lower bound on the bridge safe load capacity. From the standpoint of structural reliability, discussed in Section 2.1.2, a successful proof load test truncates a portion of the prior estimated resistance distribution (described by the parameters in Table 2.1), as shown in Figure 2.4. As a result, it often reduces the uncertainty associated with *in situ* bridge resistance. The proof load test therefore can be employed as an effective alternative to rate bridges for which analytical evaluation has produced unsatisfactory rating results or cannot be applied due to lack of information in the bridge files. A target proof load has to be defined prior to the experiment and applied in increments during the test so that the tested bridge can be properly monitored to provide early warnings of possible distress as well as evidence as to whether or not the bridge can carry these imposed loads without damage.

The *LRFR Guide Manual* provides the following equation [Eq. (8-8)] to determine the target proof load L_T:

$$L_T = X_{PA} L_R (1 + IM) \tag{2.10a}$$

where L_R is the unfactored live load due to rating vehicles, IM is the dynamic load allowance, and X_{PA} is the target adjusted live-load factor which is given by Eq. (8-7) in the manual as:

$$X_{PA} = X_P (1 + \frac{\Sigma\%}{100}) \tag{2.10b}$$

In Eq. (2-9b), X_P is the target live load factor needed to bring the bridge to a rating factor of 1 and the $\Sigma\%$ tabulated in Table 8-2 in the *LRFR Guide Manual* represents the appropriate adjustment to X_P accounting for bridge's *in situ* condition, such as deterioration and traffic situation. Once the target load level is established, the proof load test can be performed and subsequently the operating level capacity of the bridge (OP) is calculated from Eq. (8-9) in the *LRFR Guide Manual*:

$$OP = \frac{k_0 L_p}{X_{PA}} \tag{2.10c}$$

in which k_0 equals 1.00 if the proof load test is terminated because the target proof load has been reached or equals 0.88 if distress level has been observed prior to the target load level. In these two cases, L_p equals L_T or the actual maximum applied load respectively. Finally the operating level rating factor can be obtained through Eq. (8-10) in the *LRFR Guide Manual*:

$$RF_o = \frac{OP}{L_R (1 + IM)} \tag{2.10d}$$

The operating capacity, in tons, is the rating factor times the rating vehicle weight in tons.

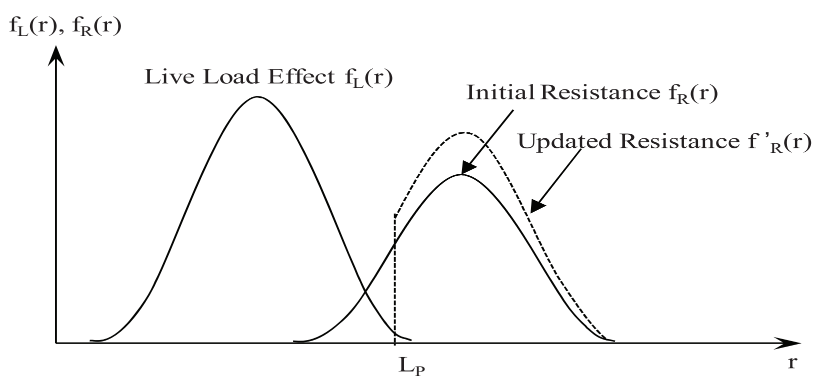

Figure 2.4: Structural Reliability Models for Bridge Proof Load Test

In the *AASHTO MCE* (1994), there is no provision for bridge load tests. In contrast, the *LRFR Guide Manual* provides guidelines on the load testing procedure, target load level and the interpretation of test results. However, as revealed in the survey of state Departments of Transportation, discussed in Chapter 3, there are still some issues that need to be addressed to facilitate broader acceptance of load testing as a tool for condition assessment among bridge engineers. Those issues will be summarized in Section 3.3 and addressed in the current research program, as presented in Chapter 4.

2.2 BRIDGE EVALUATION IN FOREIGN COUNTRIES

A first step in the project was to review and critically appraise current condition assessment procedures for existing bridges through an examination of national and international bridge rating standards and guidelines.

2.2.1 Bridge Rating in Canada

The provisions of Section 11 of the *Ontario Highway Bridge Design Code (OHBDC), 3rd edition* pertain to the evaluation and posting of existing bridges other than soil-steel structures[2] and pedestrian bridges. Provisions are given for the condition inspection, analytical load rating procedure, load testing and calculation of posting limit for bridges. In contrast to the requirements in the United States, evaluation is not mandated for every highway bridges and not required on a periodic basis in Canada.

The *OHBDC* is based on the limit state design philosophy and a target reliability index, β, of 3.5 is used for both design and evaluation. There is no explicit reduction of β in evaluation, while a few adjustments can be applied to reduce those load factors used for design when evaluation is performed. Provisions are provided for ultimate, service and fatigue limit states checking, and only the ultimate limit state is specified to be used for determining the load carrying capacity, stability and load posting of bridges; the exceptions are masonry abutments, masonry piers and masonry retaining walls, for which serviceability is the governing limit state. Fatigue checks are performed only if the bridge owner wants to assess the bridge's remaining life because of the observation of the physical evidence of fatigue-prone details or fatigue related defects. The method of fatigue life assessment is the same as in the *AASHTO LRFR Guide Manual*.

The rating process requires the use of three live load models, designated *OHBEL* levels 1, 2 and 3 respectively, with different gross magnitudes and configurations. These three live load models appear to be similar to the three AASHTO legal loads. The live load factors calibrated for bridge design are adopted in the capacity evaluation for most general cases, with some exceptions: the live load may be reduced by 10% for bridges with inspection intervals less than 5 years; the corresponding lane load equivalent used in evaluation is reduced as a function of *in situ* traffic volume or varies according to the road classes; live load factor may be reduced for multiple lane bridges with a certain level of redundancy, and the dead load factor can be reduced if the nominal dead load is carefully estimated. These reductions are not applied in controlled vehicle rating, which

[2] Defined as a bridge comprised of bolted structural steel plates and engineered soil, designed and constructed so as to utilize structural interaction between the two materials. The OHBDC devotes an entire chapter to this type of bridge, as it does for concrete and steel bridges.

is a procedure that is comparable to the AASHTO permit load checking, and is conducted for specific vehicles for which permission must be granted prior to their using the bridge.

The *OHBDC* presents detailed provisions and curves for establishing posting limits according to the rating calculations performed for the three above-mentioned live load models. The provision regarding posting concrete bridges is similar to that in the AASHTO *LRFR Manual*, that is, a concrete bridge need not to be posted if it has been carrying normal traffic without signs of excessive cracking or deformation.

The OHBDC also states that a load test may be proposed as a part of the evaluation procedure when the analytical approach does not accurately reflect the actual behavior of the bridge. However, no detailed definitions and provisions are provided as to different types of load tests, loading patterns, instrumentation or interpretation of test results.

2.2.2 Bridge Rating in the United Kingdom

Document BD 21/01, *Assessment of Highway Bridges and Structures*, adopts a limit state format with appropriate partial safety factors for condition evaluation of most highway bridges except for cast iron bridges and masonry arch bridges. It is stipulated that bridges built after 1965 should normally be evaluated for serviceability as well as for the ultimate limit states; bridges constructed before 1965 do not need to be assessed for service limit states. Requirements for fatigue endurance however are not included in the standard and the reason stated is that the past stress history of each structure, which could profoundly influence fatigue limit checking, cannot generally be determined to the accuracy level required for assessment.

No reduction in target reliability index from the corresponding values for design is explicitly stated in BD21/01; however, several adjustments are made to the live load model that have the effect of reducing the level of conservatism in the evaluation of existing bridge structures. In the UK, the bridge design live load model consists of a uniform distributed load (UDL) and a knife edge load (KEL) with the intensities of both components decreasing with bridge span lengths. The design load was derived by estimating the worst credible values of relevant loading parameters from available statistics. Adjustments are suggested in the evaluation to scale down this design load model for bridge situations less onerous than the above worst case scenario, while maintaining a consistent reliability level for the whole network; detailed scaling curves for the live load adjustment factors are provides in the document. Furthermore, in the absence of definite information about material characteristics in estimating the resistance of bridge component, the document assigns a set of values to materials which should be used in the initial assessment, mostly according to the construction period of the bridge. Structures which cannot sustain the assessment live loading after the checking calculation, and which are not scheduled for immediate replacement or strengthening should be reevaluated for the other three lower live load models for posting; posting provisions can be found in the document.

Document BA 54/94, *Load testing for bridge assessment*, presents general instructions on load testing practices. The document states that the role of load testing primarily is to seek out the hidden reserves of strength, and the bridges most likely to be involved are those which contain features where such reserves may be found. Load tests are broadly divided into the two categories: Proving load tests and Supplementary load tests, which are analogous to the AASHTO Proof load test and Diagnostic load test, respectively. Because there is a risk of collapse during a proving test, or of damage to essential elements of the structure, such tests therefore are limited in the document only to those bridges which, on the basis of their analytical assessment, would have been closed to traffic or demolished. Bridges that previously have been subjected to proving tests need to be thoroughly inspected and reassessed at more frequent intervals. The document also emphasizes that extreme care has to be taken to extrapolate the results of tests carried out with fairly low levels of loading to those likely to occur at the ultimate limit state.

Instructions provided in BA 54/94 are rather general; detailed guidance on loading patterns and magnitude, testing procedures, and test results interpretations are not provided. Cautionary notes are provided concerning the effectiveness and the accuracy of load testing as a means of load capacity evaluation of existing bridges. Concerns expressed include: whether a static test load can adequately represent the ultimate limit state loading condition; whether a bridge deck should be fully loaded or partially loaded, in view of the fact that the collapse mode of a partially loaded deck may be different from that when the whole deck is loaded as was intended in the design; and whether the benefit of a test is warranted, considering the risk to personnel.

2.2.3 Bridge Rating in Australia

Section 7 of the *Austrian Bridge Design Standard* provides rating guidelines with a commentary. The concept of rating is based on the limit state design philosophy and both serviceability and ultimate limit states are considered. The ultimate action is defined as an action that has a 5% probability of being exceeded during the design life, which represents an average return interval of 2000 years; while the survivability action is defined as one having 5% probability being exceeded per year, corresponding to a return interval of 20 years.

The rating for strength is carried out for all strength limit states, e.g. moment, shear, compression, at all potential critical sections, with the lowest rating factor determined being the rating factor for the bridge. At the service limit state, a structure is checked for vibration and deflection. When a bridge is checked for the fatigue limit state, the cumulative fatigue damage at the critical details of the bridge must be carefully assessed, from which the nominal fatigue life of a bridge can be estimated. For the purpose of rating, the cumulative fatigue damage is defined as the sum of the damage in all previous years; the nominal fatigue life is considered having been reached when the cumulative damage sums to unity[3], in which case, a program of inspection should be initiated to ensure that fatigue cracks are detected and suitably repaired before they endanger the bridge's ability to carry its applied loads.

A bridge may be rated at each limit state, to a general rating capacity or to specific loading cases, using the same partial factor checking format as specified in the *Standard*. For the general rating case, which is comparable to the AASHTO design load

[3] Cumulative damage is assessed using the Palmgren-Miner linear damage accumulation model.

level rating, the live load models and the corresponding load factors are the same as those used in the design of a new bridge. For specific loading cases, the live load can either be a legal load vehicle or an exceptional load, the former case being comparable to the AASHTO legal load level rating, and the latter being comparable to permit load checking. In all cases, the effects of the rating loads for the specific loading cases are determined using the gross weight and the configurations specific to the vehicles under consideration. Since the possibility of overloading at this step is unlikely, a reduced live load factor is permitted. Where the rating for a bridge is less than required for current general access vehicles, consideration shall be given to applying a posted limit on the bridge. Detailed regulation on establishing the limits for specific vehicles however is not presented in the document.

Two types of nondestructive test are defined in the *Australian Standard*: static proof load test and static performance load test, which are comparable to the two types of AASHTO load tests. The difference between the two types of test is in the magnitude of loading, and in the manner and the level of confidence in which the capacity of the bridge to carry the live load is determined from the test results. The *Standard* identifies the load test as an effective method of evaluating the performance and structural capacity of a bridge or bridge type. The document suggests that the proof test loading should be applied incrementally from a base load of 50% of the theoretical rated ultimate capacity in order to protect the bridge and the testing personnel, and the load response should be continuously monitored to ensure that the bridge is behaving in an elastic manner. It also suggests that a numerical model of the structure should be developed prior to the test to assess the ultimate capacity, failure mode and the elastic limit under different loading configurations and to determine the maximum load needed for the test. The *Standard* provides some detailed formulas for updating ratings after a successful load test and also emphasizes that the adoption of the load testing results should only to apply to bridges of similar structural form, taking into consideration material properties and conditions.

2.3 CLOSURE

In general, modern bridge rating procedures worldwide have adopted reliability principles as their basis. They have utilized the limit state philosophy to allow the safety checking to be performed in a deterministic manner without an explicit structural reliability assessment. The reliability indices for design are typically 3.5 or higher over the lifetime of the bridge. However, they permit lower reliability indices in the context of specific evaluations of individual existing bridges, either by explicitly reducing the target safety index in the calibration leading to the load factors for evaluation, which are lower than those used in bridge design, or by directly scaling down the live loads used in the assessment to reflect the lesser requirements for evaluation compared to the design level.

The ultimate limit states are typically required as the governing limit states for safety checking for majority of the bridge types; serviceability and fatigue are not regularly mandated unless signs of distress or fatigue related defects are observed. Rating procedures and the assessment live load models vary the most from country to country, but for the most part, a check on design load is typically performed prior to the capacity estimation respect to actual vehicles; the latter, in general, is the basis for posting.

The view towards load testing is different from country to country, which leads to different treatment of the provisions on this subject in different guidelines. Test protocols and details that are critical for a load test to be successful and informative may not be addressed. The *AASHTO LRFR Guide Manual (2003)* has the most comprehensive provisions on load testing of the condition assessment guidelines reviewed.

CHAPTER 3

Survey of Bridge Rating Practices in the United States

3.1 DEVELOPMENT OF SURVEY OF BRIDGE RATING PRACTICES

As part of NCHRP Project 12-46 that developed the *AASHTO LRFR Guide Manual*(2003), a survey questionnaire had been mailed to State Bridge Engineers in May, 1997, asking for current practices and views on technical issues pertaining to the inspection, evaluation and load rating of bridges. The responses to this questionnaire were valuable in developing the rating criteria in the *LRFR Guide Manual*. However, in the intervening years, the state of bridge evaluation practices in the US has continued to evolve. Accordingly, a follow-up questionnaire was prepared that requested additional information on a subset of topics covered in the older survey, with specific emphasis on bridge capacity evaluation practices that may have changed in the intervening years and would be of particular interest to the current research to develop the *Recommended Guidelines* for Georgia. The questionnaire was sent out to all states in November, 2005, and as of March 15, 2006 forty one responses (Table 3.1) were received and reviewed. A copy of this survey questionnaire can be found in Appendix A to this report.

Table 3.1 Responding States

Alabama	Iowa	Oklahoma
Alaska	Kansas	Oregon
Arizona	Maine	Rode Island
Arkansas	Maryland	South Dakota
California	Minnesota	Tennessee
Colorado	Mississippi	Texas
Connecticut	Missouri	Utah
Delaware	Nevada	Vermont
Florida	New Hampshire	Virginia
Georgia	New York	Washington
Hawaii	New Mexico	West Virginia
Idaho	North Carolina	Wisconsin
Illinois	North Dakota	Wyoming
Indiana	Ohio	

The synthesis of the survey responses in section 3.2 is presented without identifying the state or the respondent. This was done to encourage candor in completing the survey. These responses often were presented in sentence fragments; in that case, an attempt has been made to complete the view expressed in the comment with a minimum of editing. The survey questions fall into several general categories: when to load rate a bridge, when to update existing ratings, how to rate, when to post, and other performance issues (connections, fatigue, and scour). The following synthesis of the survey responses is organized around those categories.

3.2 SYNTHESIS OF SURVEY RESPONSE

When to rate?

In order to comply with FHWA regulations all states either perform a load rating analysis, or make a professional judgment as to the load capacity of their bridges. Most states are working toward 100% load rating, and most of those responding reported to have rated between 80% and 100% of their bridges. This intention is summarized by the response from a Western state: "Our goal is to rate all state owned bridges to determine the maintenance requirements and bridge load carrying capacities and to comply with the *National Bridge Inventory System* (NBIS). Also modeling all bridges will help in overload permit evaluations." The State of Georgia has rated approximately 75% of its bridges,[1] and only five other states have 60% or fewer of their bridges rated. One Western state is in the process of updating all of its ratings to include *both* Inventory and Operating and presently has over 90% rated at Inventory levels and approximately 5% rated at Operating levels. It is the policy of most states to rate all new bridges when they are designed or constructed. Existing unrated bridge structures are being evaluated and rated, as circumstances and resources permit. The rating of existing bridge structures in general begins with those for which design documents are available, and then continues to bridges without them. The rating of bridges without plans is typically performed in one of four ways: using plans from a similar bridge built at about the same time; by load testing the bridge; using results of load tests from a similar bridge structure; or by professional judgment.

When to update ratings?

As to when to update existing load ratings, the following is quoted from the response provided by a Midwestern state and is indicative of other responses:

1. There is a physical change in the condition of a bridge or a structural member, e.g., physical alteration in the structure; new beam or new deck, rusting or spalling or damage occurred to the structural member(s) resulting in section loss; change in the wearing surface; change in the super-imposed dead loads; excessive deflection or settlement observed; occurrence of an accident;
2. There is a request to re-evaluate the rating of a structure for a vehicle different than what was previously used such as for single trip permit load ;
3. There is a change from the method of analysis used for previous rating;

[1] As of December, 2005

4. Special circumstances dictate re-analysis of the structure.
5. There is a change of the rating method (e.g. switch from ASR to LFR), rating software or the truck weight regulations.

A Western state remarked that all their load ratings are being recalculated because: "the previous ratings were done by different individuals and are not consistent." Some other states have specific policies on this issue such as revising load rating when "overlay changes more than 2 inches", "steel section losses are more than 1/16 inches" or "primary member condition rating on the inspection report has changed by more than one point since last routine inspection if the initial rating was 5 or lower." The Georgia DOT revises existing ratings when "deterioration or distresses are observed during inspections, or state legal loads are changed."

<u>What method to use in rating?</u>

Thirty one (31) of the responding states, including Georgia, reported that the LFR method is their primary rating method, but that they occasionally used the ASR method in cases where the LFR method does not appear to be applicable. The remaining ten (10) states reported to use a combination of ASR, LRF, and LRFR depending on what specifications governed the design of the bridge.

Nine (9) of the responding states reported that they were currently using LRFR on either all of their bridges or those bridges designed by *AASHTO LRFD Bridge Design Specifications*. Five (5) states were in the process of evaluating the suitability of LRFR, and were planning a transitioning from LFR to LRFR. Most of the remaining states cited either the lack of resources or readily available software as reasons for not making the transition, but noted that they would change to LRFR if mandated. Several specifically said they were waiting for VIRTIS[2] to offer a version that incorporated the LRFR method before they considered switching from LFR.

Several states which have considered transitioning to LRFR raised some significant questions and issues. One Midwestern state suggested that "the proposed updates to the guidelines do not inspire confidence in the manual." A Western state responded "we are concerned with the high load factors; if we can not lower these factors through WIM (weight in motion) data, we may use older load rating methods on older bridges." Two other Western states simply stated "it was too uncomfortable with the LRFR method to use it" and "not fully confident in this document." The strongest opposition to transitioning to LRFR came from an Eastern state, which observed: "Too much work for no value. Ratings for concrete and timber do not correlate to real world. For timber, LRFR requires a "fudge" factor to get reasonable results for posting. For reinforced concrete bridges, the change from ASR to LFR resulted in a reduction of approximately 20% in posting values and changing from LFR to LRFR will result in another 15% to 20% reduction in the posting limits. On the other hand, with LFR and LRFR, posting values for steel bridges increase." This Eastern state also had serious

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[2] VITRIS is a widely used bridge rating software package, developed by *Cambridge Systematics, Inc.*

questions as to the applicability of LRFR and its ability to perform its main function of providing a uniform reliability for all bridge structure types. A similar concern was expressed by a Midwestern state, which also doubted whether LRFR was suited for all bridge types. These apprehensions about the transition from the older methods to the LRFR method warrant further investigation. As a result, a further investigation will be carried out in Section 3.3 aimed at examining the differences in rating results of these methods through illustrative rating calculations performed for four sample bridges selected from the Georgia bridge inventory.

Most of the states using LFR employ the HS 20-44 vehicle for both Inventory and Operating ratings at the design load level. Some states use the full set of AASHTO Legal vehicles, HS-20, H-20, type 3, type 3-3, and type 3S2, for legal load ratings. In some other states, the AASHTO vehicles are modified and designated as "state legal loads." These modifications typically consist of a scaled-up load and/or a redistribution of the load between the cab and trailer. There are also a few states with unique legal loads, such as logging trucks or other highly used regional vehicles.

When to post?

Answers to the survey question regarding the decision as to when to post a bridge had the widest variation of any of the answers. Twenty (20) of the responding states reported that they post a bridge when its Legal loads exceed the Operating level rating. Georgia and four other states use the Operating rating as the posting limit for bridges on the state system and the Inventory rating for bridges on the local system.

Some other states have more detailed policies regarding posting limit, such as: "use Operating rating for bridges having a condition rating larger than 5, otherwise, use Inventory level rating"; "for fracture critical member use Inventory rating , for others use some value in between the Operating and Inventory levels based on engineer's judgment"; "use Operating rating for concrete members and the average of the two for steel members "; or, "post when the Operating rating is exceeded, or when the Inventory rating is exceeded and posting will have minimal economic effects" One Eastern state specified that all structures need to be considered for posting if the structure's Inventory capacity rating is less than 30 tons for HS20 vehicle, 35 tons for 3S2 vehicle or 18 tons for the H20 vehicle, or when the gross tonnage of a "4 Axle" vehicle exceeds the structure's Operating level capacity." In another Eastern state, a bridge will not be posted if "the bridge can carry H15 at Inventory level and HS20 and all state Legal loads at the Operating level." Several states don't have specific criteria for posting, but will consider it if the structure has a rating factor less than 1.0 at the Inventory level for HS-15 vehicles or if the structure shows signs of major deterioration. There is no consensus among the states as to whether to post a bridge at Operating level or Inventory level ratings. Engineering judgment sometimes is used either to post a bridge whose rating would not normally entail posting, or to not post a bridge that is calculated to require posting.

As to what percentage of state bridge inventory has been posted, twenty (20) of the responding states reported posting fewer than 4% of their bridges, fourteen (14) reported that between 5 to 19% were posted, and the remaining seven (7) have posted over 20% of their bridges. The State of Georgia has reported approximately 22% of its bridges on state and local highway systems need to be posted. This survey question was poorly phrased, however, leading some states to report the total percentage of posted bridges while others reported the percentages of state and local bridges separately. The percentage of posted bridges on local roads is typically anywhere from 10 to 100 times the percentage of posted bridges on state roads.

As to whether serviceability or fatigue limit states are considered when setting up the posting limits, twenty four (24) states, including Georgia, do not consider either; sixteen (16) consider serviceability, and the vast majority of states generally do not consider fatigue. Those that consider serviceability do so only for steel or pre-stressed concrete girder bridges.

<u>When to load test a bridge?</u>

Fourteen (14) out of 41 of the responding states, had performed some form of load testing for the purpose of load rating as a part of bridge evaluation practice. Five (5) other states reported that they had once performed very few load tests for the reason of academic research only. The remaining states, including Georgia, have never used load testing as a tool for bridge condition assessment; one Eastern state remarked that the reason is "testing is too time consuming and expensive."

Most of the load tests have been performed on structures that were in good condition but required posting according to standard rating analysis, on special construction such as FRP bridges, on those bridges without available plans or design documentations, or on those with serious deterioration that prevented an accurate theoretical strength calculation. One Western state noted that they performs test on bridges "deemed to be high risk, or fracture-critical." The benefit of load testing results is best summarized by the response of one Western state:

1) To allow bridges to remain in service without traffic restriction
2) To avoid unnecessary repairs and needless replacement
3) To avoid repairs to bridges scheduled for replacement
4) To get more accurate load distribution factors, and
5) To compare calculated stresses with actual stresses

One other common use of load testing is in evaluation of overload permits. Two of the states that perform load tests do so extensively to prevent having to perform costly repairs, replacement or posting due to "unreliable *AASHTO* rating factors."

Only one Southern state among the fourteen states that performed load tests used the provisions in Chapter 8 of the new *AASHTO LRFR Guide Manual* (2003) to guide their load testing practices, although there is one other state that "follows NCHRP, Nov 1998-No.234, *Manual for Bridge Rating through Load Testing,* which is consistent with Chapter 8 of the *LRFR Guide Manual*." One Western state reported that the reason for not using the *LRFR Guide Manual* is that "we are not yet sufficiently comfortable with it." Two other western states, having performed load tests prior to the issuance of the *Guide Manual*, have also developed their own guidelines and testing procedures, which were reported to be in the process of being compared with the *LRFR Guide Manual*. Some states perform and analyze the load tests themselves, while states that do not have their own guidelines usually leave the testing and interpretation entirely to the

Universities to which they contract the work. One Eastern State "uses the load test to determine live load distribution, which is then applied to LFR formula to update load rating factors." Another Western state has a load testing protocol that involves taking "strain transducer measurements when the structure is under various loads. A model of the bridge is produced based on the strain transducer measurements. This model is then used to predict responses of the bridge to design loads and over-loads."

Other performance issues - connections, fatigue and scour

Thirty seven (37) of the forty one (41) responding states do not assess the capacity of connections on a regular basis. Connections are routinely inspected in most states; however, they are checked for adequate capacity only if engineers suspect that the connections may govern the load rating of a bridge. For the four exceptions, one Eastern state stated that: "Our policy requires load rating of connections for all primary components of a bridge unless the district Bridge Engineer concludes that the connections would not control the rating of the member." A Western state does consider connections, but only those on continuous bridges with a splice at the piers; an Eastern state considers all types of connections, while another Western state examines "all areas of the structure."

Most states normally do not compute remaining fatigue life of a bridge unless fatigue cracking is found during inspections, with the typical reason being lack of sufficient truck volume data. Four states are exceptions. One Eastern state performs a 100% hands-on inspection of fatigue sensitive members; however, can avoid this by calculating the fatigue life of bridges with low traffic counts, and then perform 100% hands-on inspection if the member has a remaining fatigue life of less than 10 years; One Western state "computes remaining fatigue life based on an arms length inspection" and performs such analysis on a 1 to 10 year cycle where the interval is usually 3 years for fatigue prone members as determined by fatigue life; Another Western state performs an in-depth inspection of all fracture critical members regardless of fatigue life, however, when the remaining fatigue life is finite or expired, the frequency of inspections increases. Finally, one Northern state performs fatigue analysis on selected bridges.

All states indicated that they perform some form of scour investigations on a regular basis. Most investigate scour for bridges that cross wade-able waterways during the FHWA-mandated 2 year inspections and all other bridges during a special underwater or scour investigation every 4 to 5 years. Two states report that they perform special scour investigations on any bridges identified as scour-susceptible following floods.

3.3 COMPARISON OF RATING METHODS THROUGH SAMPLE BRIDGES

The survey of current bridge rating practices of State Departments of Transportation, summarized in Section 3.2, revealed considerable differences in current practices and concerns that the ASR, LFR and LRFR methods yielded substantially different ratings. It is apparent that such discrepancies would be a barrier to the implementation of LRFR in routine bridge rating practices.

To determine the extent to which such discrepancies might exist and to quantify the magnitude of the rating differences that might result from the use of ASR, LFR and LRFR methodologies for typical Georgia bridges, a rating analysis with these three methods were performed for each of four sample bridges that had been identified for forthcoming load testing, advanced analysis and demonstration of concept for the *Recommended Guidelines*. The process by which these bridges were selected and detailed description of the engineering characteristics of these sample bridges are provided in Appendix B to this report.[3] Complete rating calculations of these samples bridges are available in Appendices C, D, E and F. Tables 3.2 and 3.3 present a summary of these rating results for flexural and for shear respectively.

In general, rating results by ASR and LFR are consistently close. The difference between Operating and Inventory of these two methods are comparatively much larger than that of LRFR. LRFR legal load ratings for HS20 are falling in between the Inventory and the Operating level ratings computed by either LRF or ASR method, for both moment and shear, for all four bridges. That is to say the LRFR legal level ratings generally are more conservative than the LFR/ASR Operating level ratings and more liberal than the LFR/ASR Inventory level ratings.

Table 3.2 Summary of Sample Bridge Flexural Rating for Interior Girders

Bridge Type	Method	Rating Vehicle			
		HS20		H93	
		Inv	Opr	Inv	Opr
Conc. T (Straight)	ASR	0.70	1.25		
	LFR	0.75	1.25		
	LRFR	0.93		0.65	0.84
Conc. T (Skewed)	ASR	1.36	2.17		
	LFR	1.16	1.93		
	LRFR	1.77		1.27	1.65
Prestressed Girder	ASR				
	LFR	1.54	2.57		
	LRFR	1.95		1.34	1.73
Steel Girder	ASR	0.82	1.33		
	LFR	0.71	1.18		
	LRFR	1.08		0.72	0.93

3.4 CRITICAL APPRAISAL OF BRIDGE RATING PRACTICE IN U.S.

Three different existing rating methods are currently utilized by state Departments of Transportation in their bridge rating work as revealed in the survey responses. The ASR, LFR and LRFR methods are based on different design philosophies and therefore often produce different rating results and lead to different posting limits for

[3] A description of the load testing program and the supporting finite element analyses of the four bridges can be found in the report of Tasks 2 and 3: O'Malley, C., Wang, N., Ellingwood, B. and Zureick, A.-H. (2009). (ftp://ftp.dot.state.ga.us/DOTFTP/Anonymous-Public/Research_Projects/).

the same structure; sometime the difference can be significant. One weak point of the current practice is that no clear policy is provided in the AASHTO rating guidelines as to which method should be used for specific circumstances. The LRFR method is relatively new; while having the most solid and logical theoretical basis of the three, it still need to be tested and validated through research and practice for the bridge engineering community to develop confidence in its use.

Table 3.3 Summary of Sample Bridge Shear Rating for Interior Girders

Bridge Type	Method	Rating Vehicle			
		HS20		H93	
		Inv	Opr	Inv	Opr
Conc. T (Straight)	ASR	0.41	0.75		
	LFR	0.43	0.72		
	LRFR	0.61		0.45	0.58
Conc. T (Skewed)	ASR	0.94	1.44		
	LFR	0.84	1.40		
	LRFR	1.05		0.83	1.08
Prestressed Girder	ASR				
	LFR	1.43	2.39		
	LRFR	1.47		1.05	1.36
Steel Girder	ASR				
	LFR				
	LRFR				

The large number of Inventory, Operating, and Legal loads clutters the analysis and rating process with many redundant calculations, especially in the ASR and LFR related procedures. Consequently, this situation causes differences in interpretations and practices from different DOTs regarding what triggers posting and whether to use an Inventory or Operating rating to post and in which circumstances. These issues should be better stipulated in rating manuals for safe practice and for consistent and unambiguous implementation.

None of the current AASHTO manuals provides clear guidance as to when to revise existing load ratings. Therefore states DOTs normally make their decisions on revising a current rating based on judgment and on what has been observed during the field inspection. Most bi-annual inspections are visual and any insight that might be obtained from such an inspection on existing safety or load-carrying capacity will be qualitative rather than quantitative in nature. Either a way must be found to better quantify what a visual inspection reveals or a more sophisticated inspection strategy, including informative and non-invasive inspection technologies and optimal inspection intervals, should be encouraged, so that the decisions based on inspection data are well-substantiated.

The survey of the state Departments of Transportation also revealed that most states rely solely on analytical methods to evaluate the load-carrying capacity of existing bridges. Load testing as an effective alternative has been largely ignored. Due to the conservative nature of the analytical rating methods, this inevitably leads to some unnecessary bridge repairs, replacements or postings. The fact that most state DOTs do not perform any kind of load testing likely can be traced to a lack of guidance on load test to address practical issues including: under what circumstances a load test will be a good option for bridge rating, and under what circumstances one should choose a diagnostic vs a proof load test, and further, how to design practical load test procedures. General guidance should provide engineers with a good sense before any decision is made as to whether a load test is worthwhile, considering the cost of a test, as well as specific instructions on field data acquisition and interpretation. The current load test guidelines in Section 8 of the *AASHTO LRFR Guide Manual* (2003) do not provide engineers with enough details to bridge the gap between the concept and practice of load testing.

CHAPTER 4

Conclusions and Recommendations

4.1 SUMMARY OF TASK 1 FINDINGS

Given that every existing bridge has a unique as-built condition, operating environment and service load history, a generalized analytical procedure which does not incorporate sufficient *in situ* data may result in inaccurate ratings, as indicated by many load tests. Advancing the current bridge evaluation practice requires better understanding of bridge system behavior, better utilization of available in situ data as well as better modeling of the load process and other physical processes such as fatigue, corrosion and concrete aging. Structural reliability and probabilistic risk analysis methods can provide quantitative tools for the management of uncertainty in condition assessment and are an essential ingredient of risk-informed management decisions regarding bridge infrastructure.

A review of existing assessment approaches and states rating practices have revealed a number of research issues that must be addressed:

- The bridge may contain archaic structural materials. Design documentation may be missing.

- Material strengths *in situ* may be vastly different from the standardized or nominal values assumed in design. On the one hand, concrete strength can increase by as much as 150% beyond the 28-day standard basis due to continued hydration; on the other hand, the strength can deteriorate due to aggressive environmental attack from physical or chemical mechanisms. Failure to consider best estimates of strength and the time-dependent nature of the structural strength and stiffness invariably will lead to an erroneous estimate of in situ strength.

- Analytical approaches to bridge evaluation usually (but not always) yield a conservative measure of actual load-carrying capacities [Bakht and Jaeger, 1990]. This conservatism is the result of assumptions made in the analysis regarding load sharing, composite action, support conditions and nonlinear behavior, in addition to differences in material strengths noted above.

- Discrepancies among the different approved rating methods (ASR, LFR and LRFR) were noted from the survey of state Departments of Transportation. These discrepancies were confirmed by preliminary rating calculations performed on bridges selected from the Georgia bridge inventory, as summarized in Appendices C, D, E and F. The reasons for these differences must be completely understood and addressed in developing the *Recommended Guidelines* for rating bridges in Georgia in Task 4.

- Satisfactory bridge performance over a period of years of service provides additional information not available at the design stage. This information should be taken into account in designing in-service inspection programs and in making decisions regarding upgrading and rehabilitation.

- Current condition assessment relies heavily on visual inspection. More quantitative models of structural deterioration [e.g., Faber, et al, 2000] have been developed but have yet to be incorporated in condition assessment procedures.

- A test load must be a significant fraction of the expected maximum live load for the proof test to be informative and to lower subsequent risk [Moses, et al, 1994; Ellingwood, 1996]. If the test load is increased to an informative level, the probability of damaging the bridge during the load test increases as well. This tradeoff between information gained and likelihood of damage must be part of the decision to load-test a bridge rather than relying on other rating methods.

- Uncertainties in loads and resistances at the design stage are reflected in the safety factors (or load and resistance factors). At the evaluation stage, uncertainties can be either greater (e.g., due to deterioration) or less (measured properties; successful load test). These uncertainties must be identified and analyzed.

The research tasks that follow, Task 2 and Task 3, summarized in section 4.2, are designed to address the research issues identified above.

4.2 OVERVIEW OF TASKS 2, 3 AND 4

Task 2: Evaluation by Load Test

Load tests may be conducted for a number of different purposes [Nowak and Tharmabala, 1988]:

- When calculation has shown that the structure is not capable of meeting the present standards due to changes in loading models or nominal strength;

- When an inspection has revealed significant damage or other changes in the system that may not be captured in an analysis model;

- To establish proof of capacity (rating)

A properly conducted proof load test can be an effective way to update the bridge load capacity assessment in situations where the analytical approach produces low ratings, or analytical analysis is difficult to perform due to deterioration or lack of documentation. A load test indicates a minimum load capacity, and thus might be used if a decision as to whether to post a bridge must be made quickly. It does not reveal the actual bridge capacity; nor does it provide a meaningful measure of the safety of the bridge over a projected service life. Such information must be gained in the context of analysis framework that provides a complete picture of general integrity. A proof load test represents a significant investment in terms of capital, time, personnel and the risks. The tradeoff between the information gain and the probability of damaging the bridge during the test must be part of the decision to load test rather than to employ other rating methods.

An examination of the role and limitations of proof load testing will be performed using FE models that will be developed in Task 3. To provide experimental confirmation of these FE models, the four Georgia bridges, identified at the beginning of the project and documented in Appendix B, will be load tested in Task 2.

Task 3: Evaluation by Analysis

The current design requirements in LRFD provide the starting point for assessment through evaluation. For bridge structures, the AASHTO *LRFD Bridge Design Specifications, 4th Edition* [2007], requirements stipulate that,

$$1.25D + 1.5D_A + 1.75(L + I) < \phi R_n \tag{4.1}$$

in which $D =$ dead load excluding weight of asphalt surface, D_A , and $(L + I)$ represents live load including impact. ϕR_n is the design strength , in which R_n is the nominal resistance and ϕ is the resistance factor, which depend on the particular limit state of interest. This equation is familiar with most designers. Structural reliability methods can be used to update the load and the resistance factors for the purpose of rating for different service scenarios, leaving the bridge engineers with a safety checking procedure with which they are familiar and is consistent with bridge engineer software.

The metric for acceptable performance can be described by the following probability:

$$P_f = P[R < S \mid H] < P_{Target} \tag{4.2}$$

in which, $R =$ structural capacity and $S =$ load. A similar equation is the basis for the *AASHTO LRFD Specifications*; the difference is the present of term H, which represents in a general way what can be gleaned from knowledge regarding prior (successful) performance, what has been learned by the in service inspection, and supporting *in situ* testing, if any. The target probability, P_{Target}, should depend on the economics of rehabilitation/repair, consequences of future outages, and the bridge rating sought. The information gleaned from the review of the Task 1 will be used to determine the values of this metric for preliminary purpose; final values will be stipulated following a test of the methodology on the four existing bridges. To facilitate practical implementation, every effort will be made to make the method procedurally consistent with current GDOT procedures.

Bridge structures are inspected periodically. When a bridge is inspected, the knowledge gained, if properly quantified, can be used to revise estimates of *in situ* strength and load carrying capacity [Eq.(4.1) and (4.2)] or to forecast of remaining service life of a bridge, which provides clear incentives to obtain quantitative *in situ* measurements through modern non-destructive evaluation (NDE) techniques, such as Carpenter hammer sounding, Schmidt rebound hammer and ultrasonic pulse velocity. Knowledge that the structural system has withstood challenges, some of which may have exceed the design basis, during a prior service period provides additional information that should be reflected in its condition assessment [Bartlett and Sexsmith, 1991; Fu and Tang, 1995]. However, NDE and maintenance contribute several sources of uncertainty to condition assessment. First, performance-degrading defects must be detected. Second, the defects must be located and measured accurately in order to determine whether they might impact future performance. These are issues for inspection of civil infrastructure, which involves difficult field conditions. Third, one must determine the impact of maintenance or rehabilitation on strength *in situ*. Research Task 3 will incorporate these sources of uncertainty in the evaluation process encapsulated in Eq. (4.1) and (4.2).

Finite element models of the four sample bridges will be developed in Task 3 as test beds for other research activities and tasks in the project; for comparison with the analytical procedure currently in use by GDOT; as basis for benchmarking the results of the Evaluation by Analysis; and for testing and refining the Load Test protocols developed in Task 2. Finite element modeling, validated through either systematic field inspection supported by NDE technologies or through diagnostic load tests, will be used to conduct “virtual” proof load tests of bridge systems and to support the technical development of the *Guidelines for Bridge Rating* to be developed in Task 4.

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References

Akgul, F. and D.M. Frangopol (2004). “Bridge rating and reliability correlation: comprehensive study for different bridge types.” Journal of Structural. Engineering, ASCE 130(7):1063-1074.

American Association of State Highway and Transportation Officials (2007). *LRFD Bridge Design Specifications*, *4th Edition* (including 2008 and 2009 interim revisions). AASHTO, Washington D.C.

American Association of State Highway and Transportation Officials (1994). *Manual for Condition Evaluation of Bridges*, *2nd Edition* (including 1995, 1996, 1998 and 2000 interim revisions). Washington, DC: AASHTO, 2000.

American Association of State Highway and Transportation Officials (2003). *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges, 1st Edition* (including 2005 interim revisions). AASHTO, Washington D.C.

American Association of State Highway and Transportation Officials (1970). *Manual for Maintenance Inspection of Bridges.* AASHTO, Washington D.C.

Standards Australia International. Australian Standard Bridge Design, Sydney, NSW 2001 Australia

Asantey, S. B. A. and F. M. Bartlett (2005). "Impact of Posted Load Limits on Highway Bridge Reliability." Journal of Bridge Engineering, ASCE 10(3): 321-330.

Bakht, B. and L.G. Jaeger (1990). “Bridge testing – a surprise every time.” Journal of Structural. Engineering, ASCE 116(5):1370-1383.

Barker, M. G. (1995). "Guide Specification Strength Capacity Rating of Existing Girder Bridge." Transportation Research Record 1476: 98-105.

Barker, M. G. (2001). "Quantifying Field-Test Behavior for Rating Steel Girder Bridges." Journal of Bridge Engineering, ASCE 6(4): 254-261.

Bartlett, M. F. and Sexsmith, R.G. (1991). "Bayesian technique for evaluation of material strengths in existing bridges." ACI Materials Journal 88(2):164-169.

BD 21/01 (2001). *Assessment of highway bridges and* structures (includes Correction dated August 2001), DMRB Volume 3 Section 4 Part 3. Highways Agency, London, UK

BA 54/94 (1994). *Load testing for bridge assessment*, DMRB Volume 3 Section 4 Part 8. Highways Agency, London, UK

Bhattacharya, B., D. Li, et al. (2005). "Reliability-Based Load and Resistance Factor Rating Using In-Service Data." Journal of Bridge Engineering, ASCE 10(5): 530-543.

Chajes, M., H. W. Shenton, III, et al. (2000). "Bridge Condition Assessment and Load Rating Using Nondestructive Evaluation Method." Transportation Research Record 1696(2): 83-91.

Chajes, M. J., D. R. Mertz, et al. (1997). "Experimental Load Rating of a Posted Bridge." Journal of Bridge Engineering, ASCE 2(1): 1-10.

Ellingwood, B. (1996)."Reliability-based condition assessment and LRFD for existing structures." Structural Safety 18(2):67-80.

Ellingwood, B. (2000). "LRFD: Implementing Structural Reliability in Professional Practice." Engineering Structures (22): 106-115.

Ellingwood, B., et al. (1980). "Development of a Probability Based Load Criterion for American National Standard A58." NBS Special Publication 577, United States Department of Commerce, National Bureau of Standards, Washington, D.C.

Faber, M.H., D. V. Val and M.G. Stewart (2000). "Proof load testing for bridge assessment and upgrading." Engineering. Structures 22(12):1677-1689.

Frangopol, D. M., J. S. Kong, et al. (2001). "Reliability-Based Life-Cycle Management of Highway Bridges." Journal of Computing in Civil Engineering, ASCE 15(1): 27-34.

Fu, G. and O. Hag-Elsafi (2000). "Vehicular overloads: load model, bridge safety, and permit checking." Journal of Bridge Engineering, ASCE 5(1):49-57.

Fujino, Y. and N. C. Lind (1977). "Proof-Load Factors and Reliability." Journal of the Structural Division, ASCE 103(ST4): 853.

Ghosn, M. (2000). "Development of Truck Weight Regulations Using Bridge Reliability Model." Journal of Bridge Engineering, ASCE 5(4): 293-303.

Ghosn, M. and F. Moses (1998). "Redundancy in Highway Bridge Superstructures." Transportation Research Record, Report 406, Transportation Research Board, National Academy Press, Washington D.C.

Lichtenstein Consulting Engineers Inc. (2001). "NCHRP Web Document 28 (Project C12-46): Contractor's Final Report: Manual for Condition Evaluation and Load Rating of Highway Bridges Using Load and Resistance Factor Philosophy." 3rd Ed.

Minervino, Charles, Bala Sivakumar, Fred Moses, Dennis Mertz, and William Edberg. "New AASHTO Guide Manual for Load and Resistance Factor Rating of Highway Bridges." Journal of Bridge Engineering, ASCE 9(1): 43-54.

Ministry of Transportation (1992). *OHBDC Ontario Highway Bridge Design Code*. Ministry of Transportation, Downsview, Ontario, Canada.

Mori, Y. and Ellingwood, B. (1993). "Reliability-based service life assessment of aging concrete structures." Journal of Structural Engineering, ASCE 119(5):1600-1621.

Moses, F. and Verma, D. (1987). "Load capacity evaluation of existing bridges." NCHRP Report No.301, Transportation Research Board, National Research Council, Washington D.C.

Moses, F., Lebet, J. P. and Bez, R. (1994). "Applications of field testing to bridge evaluation." Journal of Structural Engineering, ASCE 120(6): 1745-1762.

Moses, F., (2001). "Calibration of Load Factors for LRFR Bridge Evaluation." NCHRP Report 405, Transportation Research Board, National Research Council, Washington D.C.

Naus, D.J., Oland, C.B., Ellingwood, B., Hookham, C.J. and Graves, H.L. (1999). "Summary and conclusions of a program to address aging of nuclear power plant concrete structures." Nuclear Engineering and Design 194(1):73-96.

National Cooperative Highway Research Program (NCHRP) (1999a). "Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges." NCHRP 12-16, Pre-final Draft, Transportation Research Board, National Research Council, Washington, D.C.

Nowak, A. S. and Tharmabala, T. (1988). "Bridge reliability evaluation using load test." Journal of Structural Engineering, ASCE 114(10): 2268-2279.

Nowak, A. S. (1993) "Live Load Model for Highway Bridges." Structural Safety, 13(1993): 53-66.

Nowak, A. S. (1999). "Calibration of LRFD Bridge Design Code." NCHRP Report 368, Transportation Research Board, National Research Council, Washington D.C.

Saraf, V. and Nowak, A.S. (1998). "Proof load testing of deteriorated steel girder bridges." Journal of Bridge Engineering, ASCE 3(2):82-90.

van de Lindt, J. W., G. Fu, et al. (2005). "Locality of Truck Loads and Adequacy of Bridge Design Load." Journal of Bridge Engineering, ASCE 10(5): 622-629.

APPENDIX A

Survey Questionnaire on Bridge Evaluation Practice

INSPECTION, EVALUATION AND LOAD RATING OF BRIDGE SURVEY OF CURRENT PRACTICE

The purpose of the survey is to obtain State DOT comments on the current AASHTO Guide Specifications and current practices and views on technical issues pertaining to the inspection, evaluation and load rating of bridges.

Inspection

- Does your agency compute remaining fatigue life and consider it in setting inspection policy and procedures? If yes, please explain.
- Is inspection for scour at the foundation a component of a routine bridge inspection?

Load Rating

- What prompts your decision to rate a bridge?
- When does your agency require that load rating calculations be revised or updated?
- Does your agency use the 2003 AASHTO *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*? If not, please explain why.
- What load rating method or methods are currently used by your agency for steel and concrete bridge?
- What percentage of bridges in your jurisdiction have been load rated?
- Which AASHTO vehicles do you use for Inventory and Operating load rating? (Please insert inventory (Inv) and Operating (Opr) next to the applicable loadings.)

- When load rating bridges, does your agency consider the capacities of connections and joints? Please specify the types of connections or joints routinely checked.

Load Posting and Overload Permit Checking

- Please describe the criteria used by your agency in determining the need for load posting.
- How are inventory and operating ratings used to determine the weight limit for posting?
- Is serviceability or fatigue considered in setting a weight limit for posting? If so, please explain.
- What percentage of bridges in your jurisdiction has been posted?

Nondestructive Load Testing of Bridges

- Describe the criteria used by your agency to select bridges for evaluation by load testing
- Do you use the 2003 AASHTO *Guide Manual* (Chapter 8) for the design, conduct and interpretation of load tests? If not, please explain why.
- How are the results of the load tests used?

APPENDIX B

Documentation of Georgia Bridges Selected for Analysis and Load Testing

In order to accomplish the study objective of providing Georgia Department of Transportation (GDOT) with a set of tools for improving bridge evaluation and rating techniques, a small subset of the state's bridges have been identified through a purposive study of the Georgia state bridge inventory. Appendix B explains the selection process and provides general information regarding these sample bridges.

B.1 SUMMARY OF GEORGIA BRIDGE INVENTORY

In the State of Georgia, there are 8,988 bridges identified in the state's 2005[1] Bridge Inventory Management System (BIMS) Database, of which 82% are multi beam/girder and T-beam bridges. Most of these bridges have been load rated by either allowable stress (AS) or load factor (LF) rating methods and 1,982 of them have been found to require posting (Figure B.1). The posting of a bridge results in economic losses related to the number of vehicles affected and the time required for them to make necessary detours. Ratings of bridges on state or local routes instead of interstates are of particular interest for two reasons: first, these routes make up a much larger percentage of the state's bridges, and second, the repair or replacement of interstate bridges is typically (but not always) planned or conducted once the structure's Inventory load rating falls below 1.0, well before there is any need for posting. The severity of the economic impact of closure is what forces this early action for interstate bridges.

B.2 CRITERIA FOR BRIDGE SELECTION

Discussions with the Georgia Department of Transportation (GDOT) engineers and evaluation of the status of the current bridge infrastructure in the State resulted in a list of several primary and secondary criteria for selecting the bridges to be utilized in Task 2 and Task 3 of this study, which provide the technical bases for the development of the recommended Guidelines in Task 4. These criteria are discussed in the following.

Design Load

The HS-20 load is currently used by the GDOT in bridge rating and was used for design of new bridges prior to October, 2007, the date of adoption of the *AASHTO LRFD Specifications* and its corresponding HL-93 design load. The H-15, HS-15, H-20 together with HS-20 loads presented in Figure B.2 are some of the regular design loads used prior to the adoption of the HL-93. Furthermore, different design loads could be utilized on rural versus urban roadways in the same time period; for example, to cut costs

[1] The bridges selected for detailed analysis and load testing were selected in the first year of the research project.

in rural areas, the HS-15 or H-20 load could be chosen to design a bridge instead of using HS-20. However, as legal loads have increased over time, these older design loads have been gradually phased out of use. Bridges designed for the HS-20 load are comparatively new and unlikely to have experienced significant deterioration or loss of strength. Thus, this project will focus on bridges designed for H-15 loads. Figure B.3 shows that the bridges designed for the H-15 load represent not only the largest number of any single design load category but also the largest number of posted structures where the initial design strength is known.

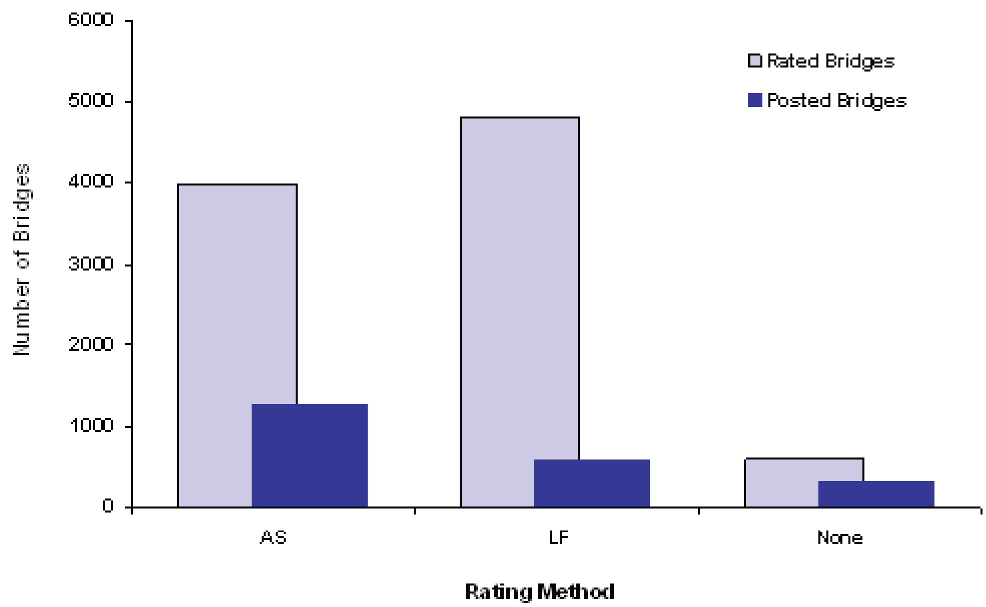


Figure B. 1 Number of Rated and Posted GDOT Bridges, by Rating Method

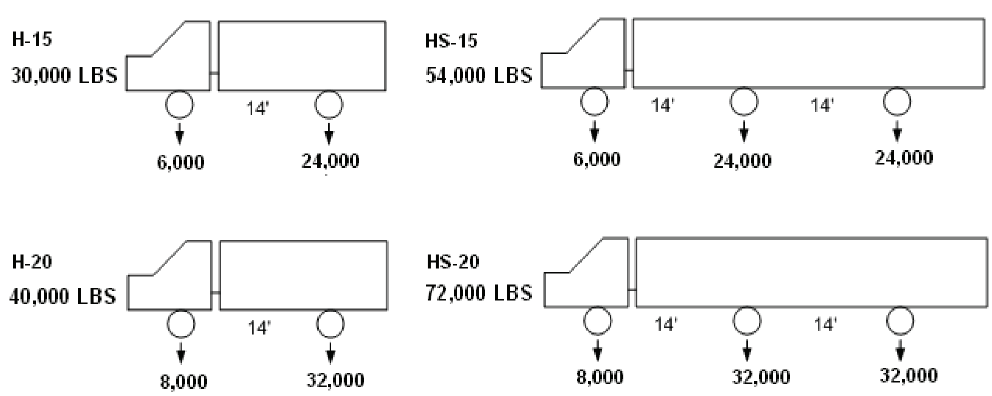


Figure B. 2 Bridge Design Load Prior to HL-93 Live Load Model

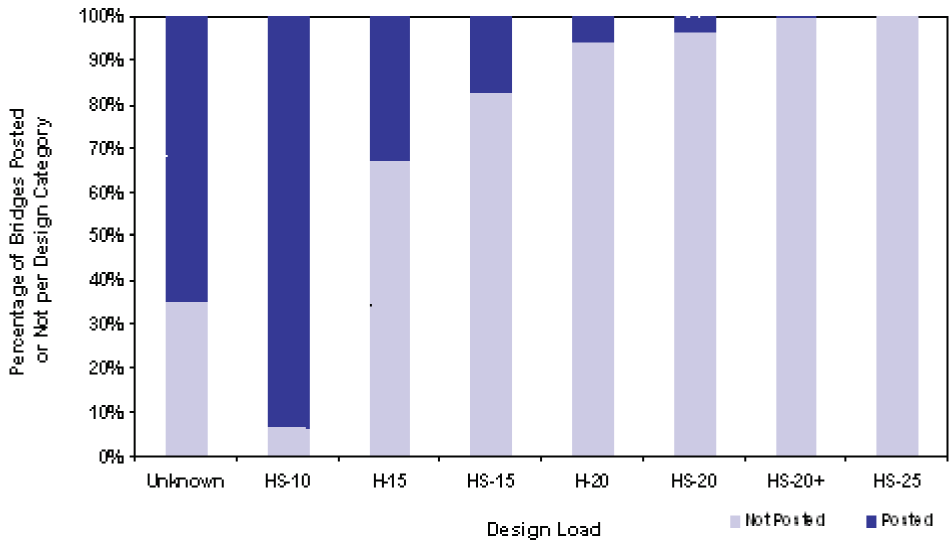


Figure B. 3 Number Posted Bridges per Design Load

Structure Type

Of the 1,982 Georgia bridges that require posting, 77% fall into one of three categories:

- Reinforced concrete T-beam bridges, representing 21%;
- Steel girder bridges, representing 53%; and
- Pre-stressed concrete I-girder bridges, representing 3%.

While the pre-stressed concrete bridges represent a much smaller portion of the posted bridges than reinforced concrete or steel bridges, a higher percentage of relatively new pre-stressed concrete bridges have been found require posting. Of those posted pre-stressed bridges, 57% were constructed after 1980; in contrast, only 2% of the posted reinforced concrete bridges and 10% of the posted steel girder bridges were constructed after 1980. GDOT expressed concern over this bridge category, and it is therefore included in this study. Figure B.4 shows the primary structure type of bridges constructed over each decade from the 1940's to present. Figure B.5 identifies the number of bridges from each category that have been posted as unfit for some or all of the state legal load vehicles.

Wooden bridges have been omitted from this study because they represent about 2% of the bridges in the State, as shown in Figure B.6, and are typically historic structures intended only for light automobile traffic. Suspension, truss, and other long span bridges also represent rather small percentage of the total population and often require independent rating procedures; accordingly, they are not addressed in this study.

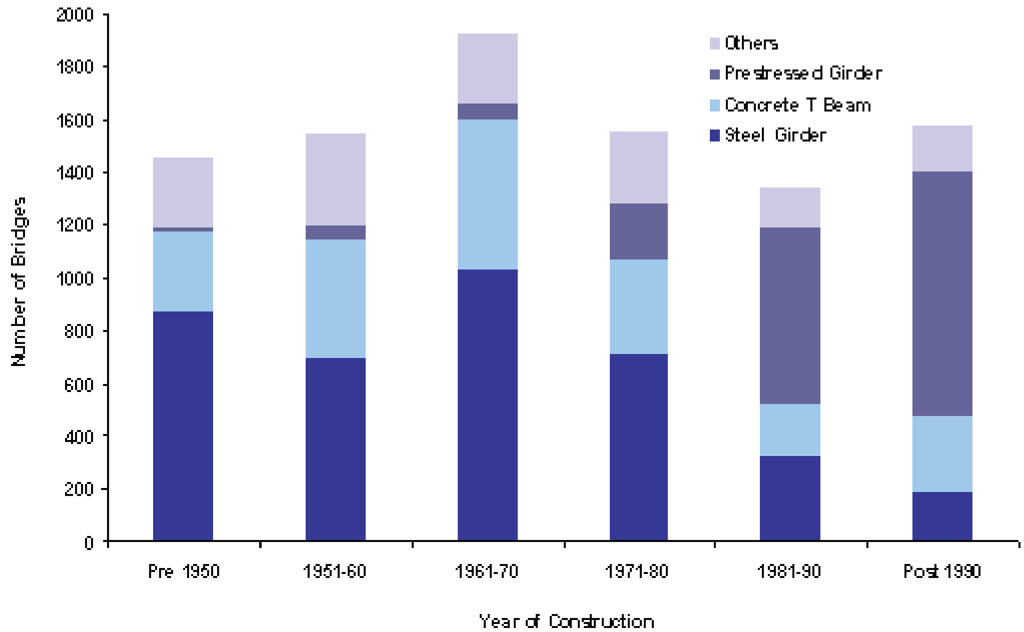


Figure B. 4 Bridge Categories Identified by Decade of Construction

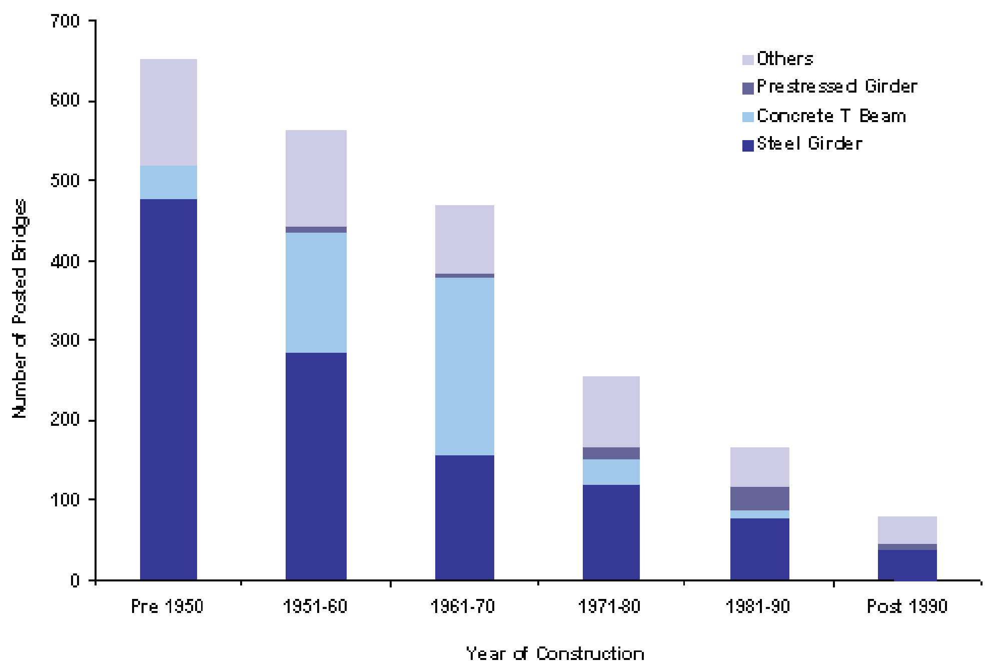


Figure B. 5 Posted Bridges Identified by Decade of Construction

Bridge Span Length

In order to ensure that the selected sample bridges are to some extend statistically representative to the state bridge population, the span length distributions of girder bridges of different material types are briefly studied and presented in Figure B.7. The plots indicate that above 75% of steel spans has a length of 35 ~ 75 ft (11 ~ 23 m) with a mean of 52 ft (16 m). Comparatively, the prestressed concrete span has a wider spread with a larger mean of 73 ft (22 m), while the reinforced concrete span on the other hand has a narrower range with a smaller mean of 34 ft (10 m). It is interesting to notice that roughly 50% of prestressed concrete bridges are either around 45 ft (14 m), or 70 ft (21 m), or 90 ft (27 m), and more than 75% of reinforced concrete spans lands on either 20 ft (6 m), or 35ft (10 m), or 40 ft (12 m). This observation will be appropriately reflected in the quantitative study to characterize the likely performance of a broad selection of state bridges in task 3.

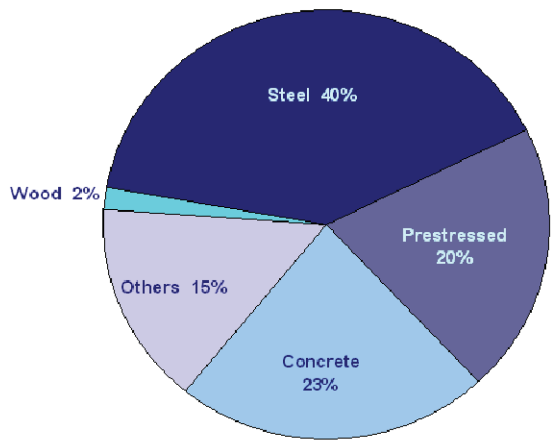
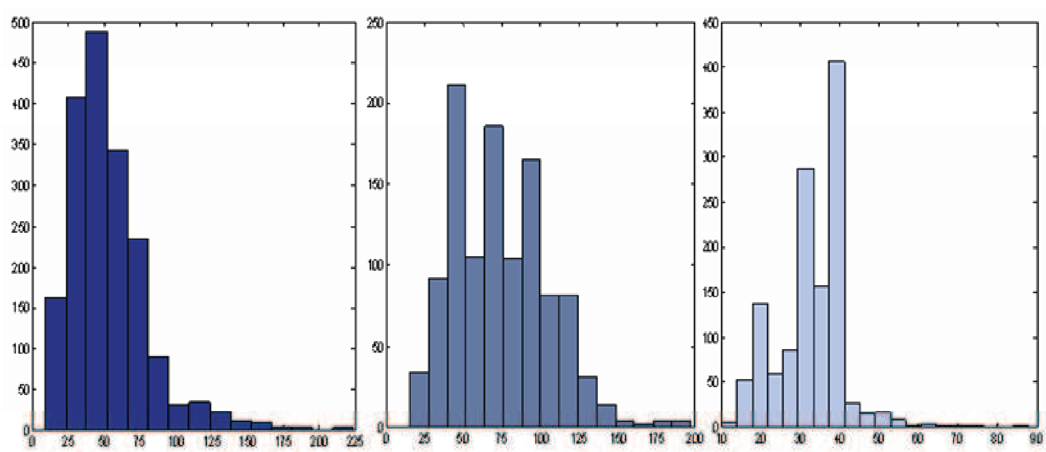


Figure B. 6 Percentages of Bridges of Different Material



a) Steel Span b) Prestressed Concrete Span c) Concrete Span

Figure B. 7 Span Length Distribution of Bridges of Different Type

Bridge Condition

The condition rating of a bridge represents the GDOT bridge inspector's assessment of the overall condition of the structure's primary components. In this study, bridges with moderate condition ratings, typically 5-7, were selected. Highly deteriorated structures such as those with condition assessment levels below 4 were not chosen because, by definition, they suffer from significant deterioration that must be addressed in the immediate future. As such, these highly deteriorated bridges would need to be posted or repaired and are unlikely to benefit from a refined rating procedure. At the opposite end of the scale, bridges in very good condition were not selected either, as they typically are new structures that were designed to modern load levels and are unlikely to require posting. Moreover, any evaluation procedure is developed for moderately deteriorated bridges would also be applicable to those bridges currently in good condition and designed to the same standards.

Accessibility and Ease of Instrumentation

The screening of the GDOT bridge inventory based on structural type, age, material, design load and condition rating led to a bridge population that was still far too large to conduct an in-depth analysis of each of them. Thus, a series of secondary criteria was employed to narrow the selections to a manageable number of bridges. First, all bridges that spanned interstates, railroads or very large rivers were eliminated due to the inaccessibility of their superstructure or substructure for field instrumentation without special equipment. Second, all bridges that had been widened or otherwise modified by adding different types of girders or materials (e.g. FRP) were eliminated from consideration; many of these bridges were T-beam reinforced concrete bridges that had been widened using additional pre-stressed girders. Pre-stressed concrete box girder bridge were also eliminated as these represent a practically small portion of the state bridges, and are difficult to instrument and load-test properly. Finally, candidate bridges for analysis and diagnostic testing were limited to those within approximately 50 miles of Atlanta (and each other) in order to provide greater efficiency in the load testing and inspection process.

B. 3 SELECTED SAMPLE BRIDGES FOR TESTING AND ANALYSIS

Approximately twenty (20) bridges were selected as tentative candidates by screening GDOT database according to above mentioned criteria. Following a site visit to each of these bridges and a review of their fitness and testability with State bridge maintenance engineering staff, four bridges were finally identified for diagnostic load testing and further in-depth analysis. They are:

- Reinforced concrete bridge – straight approach (Bridge ID: 129-0045)
- Reinforced concrete bridge – skewed approach (Bridge ID: 015-0108)
- Pre-stressed concrete girder bridge (Bridge ID: 223-0034), and
- Steel girder bridge (Bridge ID: 085-0018)

Details of the bridge structural systems and of the testing program, including instrumentation, load testing and post-assessment of the test measurements can be found in the Task 2 report.[2] Independent ratings of these four bridges were conducted in Task 1 to verify that the discrepancies between different rating methods revealed in the States survey might exist for bridges in the Georgia inventory, and to determine the sources of the differences observed. The detailed rating calculations for the four bridges can be found in the Appendices C, D, E and F of this report respectively. The following sections provide general descriptions of these four bridges**.**

B.3.1 Reinforced Concrete Bridge – Straight Approach (ID: 129-0045)

Figure B.8 Straight T-Beam Bridge (ID: 129-0045, Gordon County)

This reinforced concrete T-beam bridge (Figure B.8) carries SR 156 over Oothkalooga Creek, was designed using the *AASHTO 1953 specification* for H-15 loading, and was built in 1957. It is located one mile west of Calhoun, GA in Gordon County. SR 156 is a two-lane road. The bridge has eight spans, seven of which are 40 ft (12.19 m) and one (over the channel) 45 ft (13.7 m). The girders are 18 ½ in x 24 ¾ in (46.99 cm x 62.87 cm), except for the long span which is 31 ¾ in, and are spaced 7.2 ft (2.19 m) apart. The bridge has a deck width of 32.3 ft (9.85 m) and a road way width of 25.7 ft (7.83 m). The bridge carries an ADTT of 458. The concrete deck has a condition rating of five, the supporting reinforced concrete T-beam superstructure is rated at seven, and the concrete bent and pier substructure are rated at six. The latest inspection report indicates that all caps have minor hairline cracking, and that several areas of exposed cap reinforcement are present. All beams are reported to show signs of typical flexural cracking. The entire deck has moderate surface deterioration, scaling, and cracking. It has also been repaired in several notably bad sections. The bridge had not been posted, but had been scheduled for replacement in 2008.

B. 3.2 Reinforced Concrete Bridge – Skewed Approach (ID: 015-0108)

This 12-span structure over a long flood plain and a creek carries Old Alabama Rd. over Pumpkinvine Creek 3.7 miles south of Cartersville, GA in Bartow County. The two-lane bridge structure (Figure B.9) has a skew of 30 degrees and an ADTT of 709,

[2] O'Malley, C., N. Wang, B. R. Ellingwood and A.-H. Zureick (2007). Condition assessment of existing bridge structures: Report of Task 2 and Task 3 - Bridge Analysis and Load Testing Program. GDOT Project RP07-01, August, 2009.(ftp://ftp.dot.state.ga.us/DOTFTP/Anonymous-Public/Research_Projects/)

and was designed using the *AASHTO 1977 specifications* for HS-20 loading and dates to 1979. The eleven spans over the flood plain are carried by 40-ft (12.19 m) reinforced concrete T-beams. The 70-ft (21.34 m) span over the channel is supported by AASHTO type II pre-stressed concrete girders. The current bridge ratings for substructure, superstructure, and deck are 6, 6, and 7 respectively, and the bridge is posted for three truck loads: H (21 tons), Tandem (19 tons), and Log (24 tons). There is minor cracking and spalling in a number of the bents and abutments, as well as in the T-beams, but none is in need of immediate repair.

Figure B. 9 Skew T-Beam Bridge (ID: 015-0108, Bartow County)

B.3.3 Pre-stressed Concrete Girder Bridge (ID: 223-0034)

This bridge carries State Route 120 over Little Pumpkinvine Ceek approximately 5 miles south of Dallas in Paulding County GA. It was designed using the AASHTO 1989 for HS-20 loading specifications and was constructed in 1992. The main structural system consists of pre-stressed concrete I-Beams arranged in four simply supported spans. The bridge is 216 ft (65.8 m) long and is comprised of two 40-ft (12.2-m) Type II pre-stressed I-girder spans and two 68-ft (20.7-m) Type III prestressed I-girder spans. The centerline of the bridge is essentially perpendicular to the girder supports. The bridge has a deck width of 43¼ ft (13.2 m) and a roadway width of 40 ft (12.2 m). The 68-ft (20.7-m) spans are comprised of five type III I girders that are composite with the 9⅛ in (232 mm) thick slab (Figure B. 10). The bridge is in good condition, with substructure, superstructure and deck condition numbers of 7, 8 and 7, respectively. It is not posted. The ADT is 6550.

Figure B. 10 Pre-Stress Bridges (ID: 223-0034, Paulding County)

B.3.4 Steel Girder Bridge (ID: 085-0018)

Figure B. 11 Steel Girder Bridge (ID: 085-0018, Dawson County)

This bridge carries SR 136 over the Etowah River 5.7 miles east of Dawsonville, Georgia, in Dawson County. It was designed using the AASHTO 1961 specification, with interim revisions through 1963 for H-15 loading, and was constructed in 1965. The bridge is 196 ft (59.7 m) long and its four 49 ft (12.2 m) spans are supported by four steel girders spaced at 8 ft on centers; the two facia girders are W33x118, while the two interior girders are W33x130, with a full-depth diaphragm located at mid-span (Figure B. 11). The two-lane bridge has a (non-composite) concrete deck, with overall width of 32 ft (9.75 m) and a roadway width of 26 ft (7.92 m). The centerline of the bridge is perpendicular to the girder supports. The bridge was last inspected on June 30, 2005, and at that time the deck and substructure both were assigned a condition assessment rating of 6. The inspection report indicates that there is spalling, aggregate exposure, and transverse cracking in the deck in all spans. The bridge was determined to require posting, and has been posted for a 21-ton H load, 25-ton HS load, 23-ton Tandem load, 32-ton 3-S-2 load, and 27-ton Log load. The piles have minor pitting and the beams have minor deflections. The bridge carries an ADTT of 280.

Rating of a Straight Cast-in-Place Reinforce Concrete T-beam Bridge (GDOT BRIDGE ID # 129-0045)

Example C1:

Rating by the Allowable Stress Method (ASR) Using Load Distribution and Dynamic Allowance Factors Stipulated by AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002).

C1-1 Basic Geometry and Bridge Information

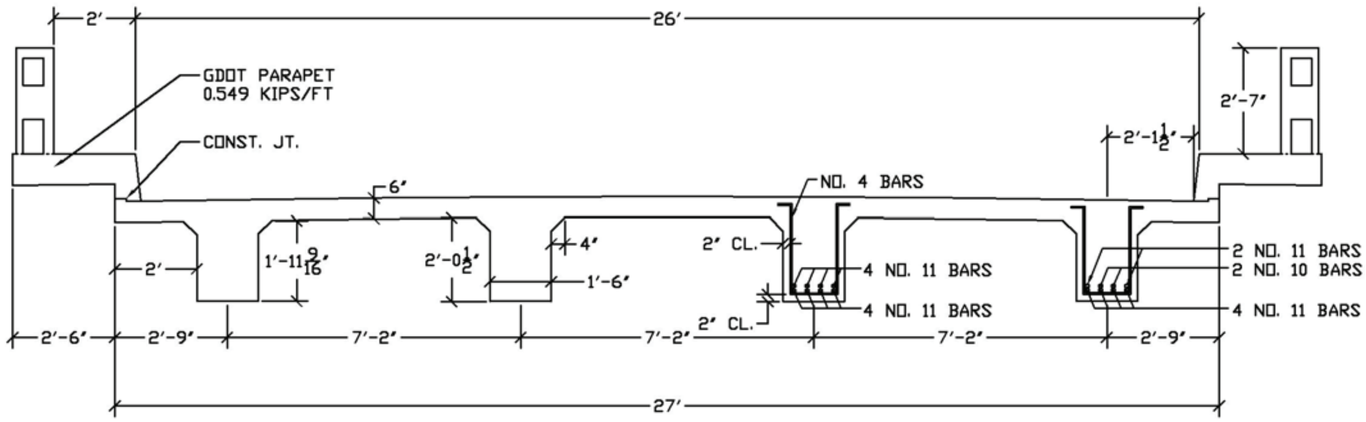


Figure C1-1.1 Bridge Cross Section at Mid-span

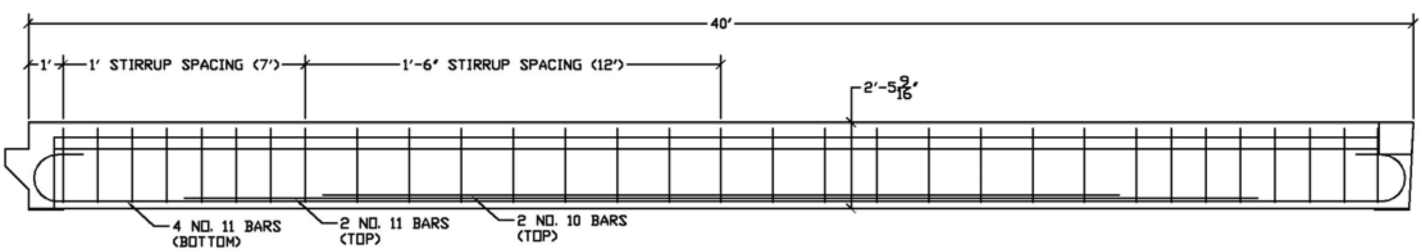


Figure C1-1.2 Exterior Girder Details

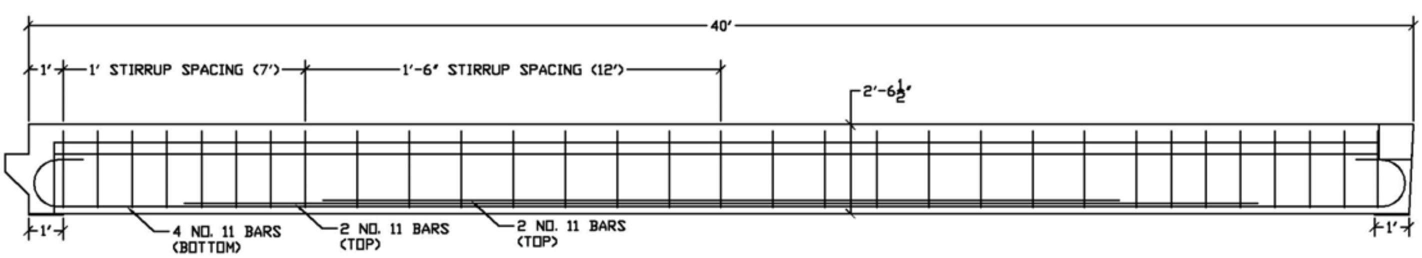


Figure C1-1.3 Interior Girder Details

Concrete strength: $f_c^{'} = 2.5\ \text{ksi}$

Unit weight of concrete: $wc = 0.15\ \text{kips/ft}^3$

Weight per ft of standard GDoT parapet and sidewalk: $wpr = 0.549\ \text{kips/ft}$

C1-2 LOADS

C1-2.1 Permanent Loads

As per Article 3.23.2.3.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) the dead load supported by the outside stringers or beams shall be the portion of the floor slab carried by the stringer or beam. Curbs, railings, and wearing surfaces if placed after the slab has cured, may be distributed equally to all roadway girders.

C1-2.1.1 Interior girder loads

Weight per linear foot of the reinforced concrete slab = $\left(\frac{6}{12}\right)(7.17)(0.150)$ = 0.538 k/ft

Weight per linear foot of cast-in-place beams = $\left[\frac{(24.5)(18)}{144}+\frac{2\frac{(4)(4)}{2}}{144}\right](0.150)$ = 0.476 k/ft

Weight of parapet, rail and sidewalk assembly = $\frac{(2)(0.549)}{4}$ = 0.275 k/ft

Since there is no wearing surface present on the bridge, DW = 0

Total dead load for interior beam = 1.29 k/ft

C1-2.1.2 Exterior girder loads

Weight per linear foot of the reinforced concrete slab = $\left(\frac{6}{12}\right)(6.33)(0.150)$ = 0.475 k/ft

Weight per linear foot of cast-in-place beams = $\left[\frac{(24.5)(18)}{144}+\frac{2\frac{(4)(4)}{2}}{144}\right](0.150)$ = 0.476 k/ft

Weight of parapet, rail and sidewalk assembly = $\frac{(2)(0.549)}{4}$ = 0.275 k/ft

Since there is no wearing surface present on the bridge, DW = 0

Total dead load for exterior beam = 1.23 k/ft

C1-2.2 Vehicular Live Load

The design vehicular live load on the bridge consists of AASHTO HS20 truck with the spacing between the two 32-kip rear-axle loads to be varied from 14 ft to 30 ft to produce extreme force effects. The HS 20 truck is shown below.

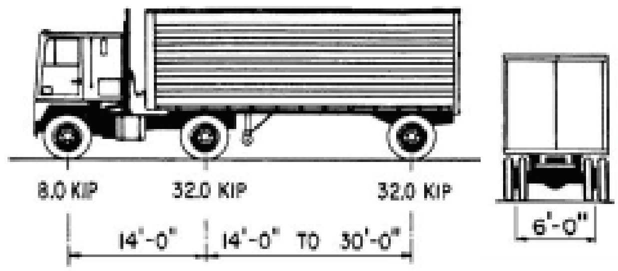


Figure C-2.1 AASHTO HS20 Truck

C1-3 Dynamic Load Allowance

Article 3.8.2.1 of the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002), the dynamic load allowance is taken as:

$$\mathrm{I} = \frac{50}{\mathrm{L}+125} = \frac{50}{39+125} = 0.305 \leq 0.3$$

Where:
I = impact factor (maximum 30 percent)
L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member

C1-4 Live Load Distribution Factors

C1-4.1 Moment distribution factors

C1-4.2.1 Distribution Factor for ***moment*** in interior longitudinal beams

As per Table 3.23.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002), the distribution factors for moment in interior and exterior beams are computed as:

$$DFM = \frac{S}{6} = \frac{7.17}{6} = 1.195$$

C1-4.2.1 Distribution Factor for ***moment*** in exterior longitudinal beams

Using the structure shown in Figure C1.4.1 the moment distribution factor is calculated by computing the reaction at R_B, but not taken less than (Article 3.23.2.3.1 AASHTO 2002):

$$\frac{S}{4.0+0.25S} = \frac{7.17}{4+0.25(7.17)} = 1.24$$

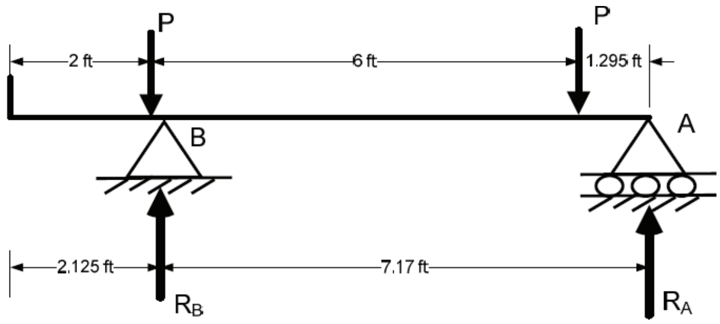


Figure C1-4.1 Exterior Girder Moment Distribution Factor

$$\sum M_A = 0$$
$$R_B(7.17) - P(7.295) - P(1.295) = 0\,.$$
$$R_B = 1.198P$$

Thus, the distribution factor for moment in an exterior beam is:

$$DFM = 1.24$$

C1-4.2 Shear distribution factors

C1-4.2.1 Distribution Factor for ***shear*** in interior longitudinal beams

Article 3.23.1.2 of the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) stipulates that:

> "Lateral distribution of the wheel loads at ends of the beams or stringer shall be that produced by assuming the floors to act as a simple span between stringers or beams. For wheels or axels in other positions on the span, the distribution for shear shall be determined by the method prescribed for moment."

Therefore, by modeling the deck as a series of rigid simply supported beams between the girders, as shown in Figures C1.4.2, the shear distribution factor (DFS) is computing by calculating the reaction R_B.

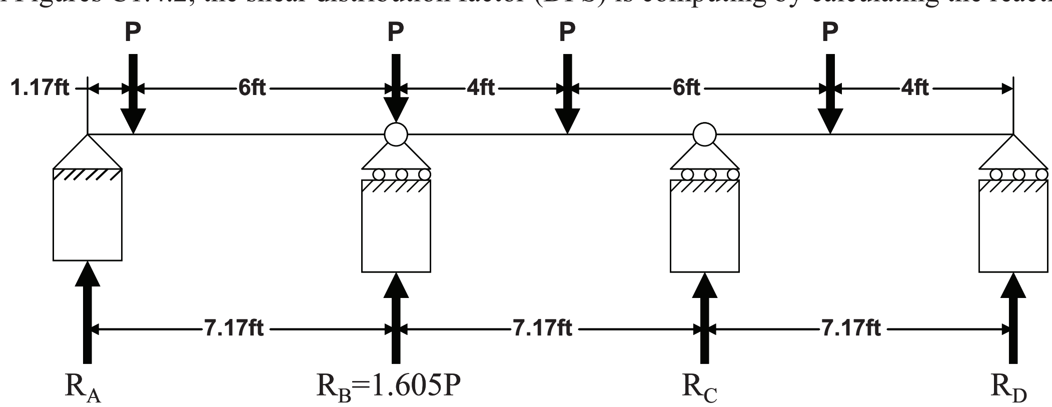


Figure C1-4.2 Interior Girder Shear Distribution Factor

DFS = 1.605 for wheel loads at beam ends

C1-4.2.2 Distribution Factor for ***shear*** in exterior longitudinal beams

Using the structure shown in Figure C1.4.1 the shear distribution factor is calculated by computing the reaction at R_B.

Thus, the distribution factor for shear in an exterior beam is:

$$DFS = 1.198$$

C1-4.2 Summary of distribution factors

Table C1-4.1 Distribution Factors

Action	Interior Beam	Exterior Beam
Bending Moment	1.195	1.24
Shear	1.605	1.198

C1-5 Flexural Analysis

C1-5.1 Maximum live load bending moment

A rudimentary structural analysis of a simply supported beam subjected to a vehicular load having two rear axles and one front axle as shown in Figure C1.5.1 shows that the absolute maximum moment occurs under the middle axle when such an axle is positioned at a distance of 2.33 ft to the left of the beam centerline.

By applying the dynamic allowance factor and the distribution factor for moment of interior beams, we can then compute the maximum live load under the following loads:

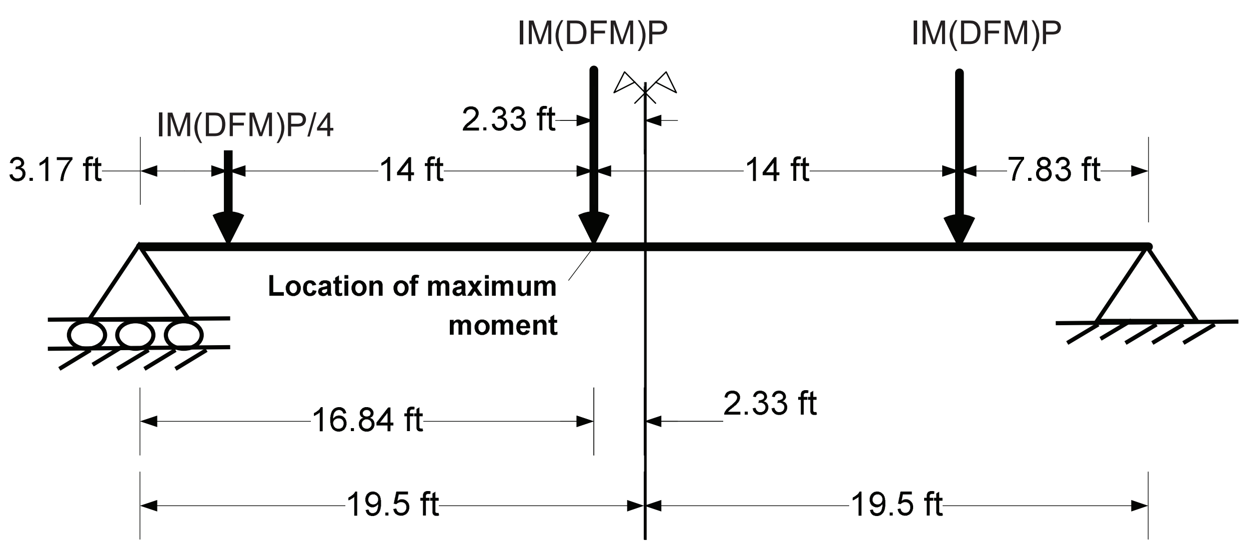


Figure C1-5.1 Max Live Load Moment

C1-5.2.1 Maximum live load moment for an interior beam

With P = 16 kips, $DFM = 1.195$, and IM =1.30, M_{LL+IM} can be computed at the location of the maximum live load moment (at 17.17 ft from the left support) to be:

$$M_{LL+IM} = 335.6\ k - ft$$

Live load moment (at midspan):

$$M_{LL+IM} = 320.5\ k - ft$$

C1-5.2.2 Maximum live load moment for an exterior beam

With P = 16 kips, $DFM = 1.24$, and IM =1.30, M_{LL+IM} can be computed at the location of the maximum live load moment (at 17.17 ft from the left support) to be:

$$M_{LL+IM} = 348.2\ k - ft$$

Live load moment (at midspan):

$$M_{LL+IM} = 332.6\ k - ft$$

C1-5.2 Maximum dead load moment

C1-5.2.1 Maximum dead load moment for an interior beam

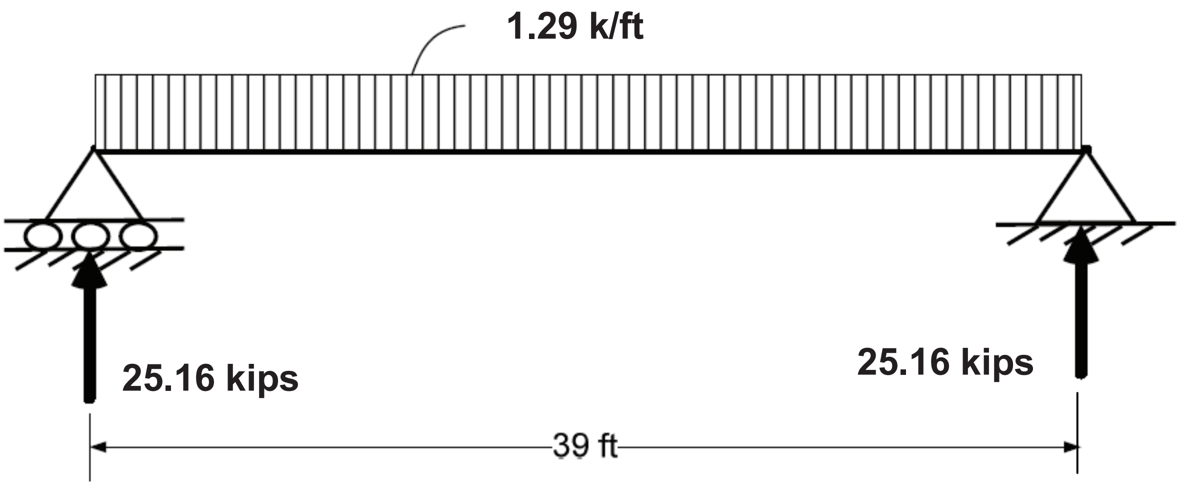


Figure C1-5.3 Interior Girder Dead Load Moment

The dead load moment at the location where the live load produces the maximum effect is

$$M_D = 25.16(17.17) - 1.29\frac{(17.17)^2}{2} = 241.9\ k - ft$$

The dead load moment at midspan is:

$$M_{D(m)} = 1.29\frac{(39)^2}{8} = 245.3k - ft$$

C1-5.2.2 Maximum dead load moment for an exterior beam

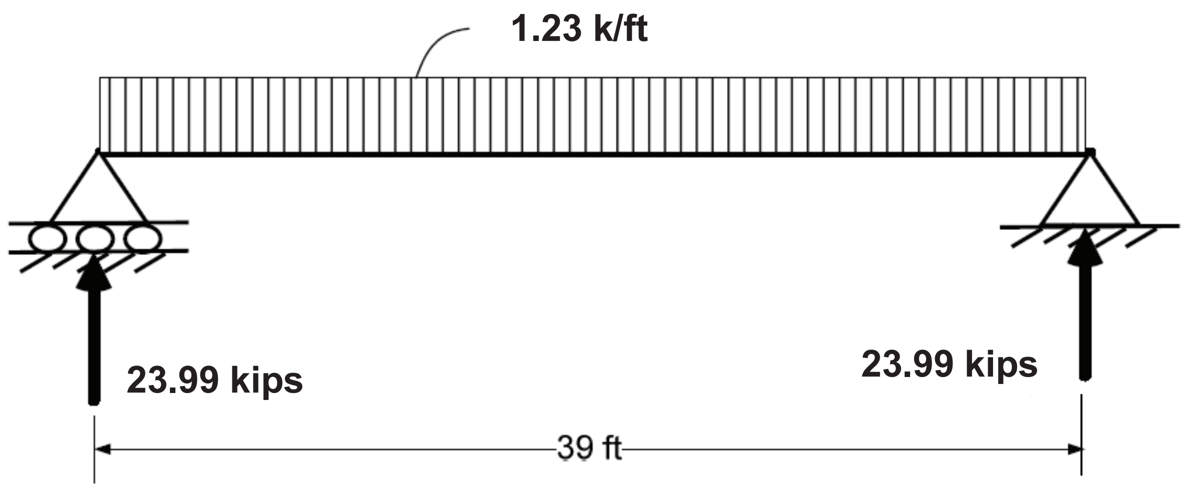


Figure C1-5.4 Exterior Girder Dead Load Moment

The dead load moment at the location where the live load produces the maximum effect is

$$M_D = 23.99(17.17) - 1.23\frac{(17.17)^2}{2} = 230.6\ k - ft$$

The dead load moment at midspan is:

$$M_{D(m)} = 1.23\frac{(39)^2}{8} = 233.9k - ft$$

C1-6 Shear Force Analysis

Based on the shear reinforcement details of the bridge girders, it is necessary to perform shear rating calculations of the bridge at two locations along both the interior and exterior girders. These two critical locations as stipulated in Article 8.15.5.1.4 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) are:

- At d/2 from either support
- At 7.5' from either support at which the stirrup spacing changes from 12" to 18"

C1-6.1 Interior Beam Shear Analysis

C1-6.1.1 Interior beam maximum live load shear force at d/2

The first critical location for the interior girder is located at $\frac{d}{2} = \frac{2.22}{2} =$ 1.11 ft from the support. Article 8.16.6.1.2 of the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) states that the shear at any point between d = 2.22 ft and the support should be designed for the shear computed at d = 2.22 ft plus the full weight of any concentrated load between d = 2.22 ft and the support. Therefore the maximum live load shear force is computed at the location of interest $\frac{d}{2}=\frac{2.22}{2}=$ 1.11 ft from the support as per the shear influence line shown in Figure C1-6.1.

At $\frac{d}{2}=\frac{2.22}{2}=$ 1.11 ft; the length of the portion of the span that is loaded to produce the maximum stress in the member is L = 39-1.11=37.89 ft thus:

$$IM = 1 + I = 1 + \frac{50}{(37.89)+125} = 1.31 \leq 1.30$$

Use IM = 1.30

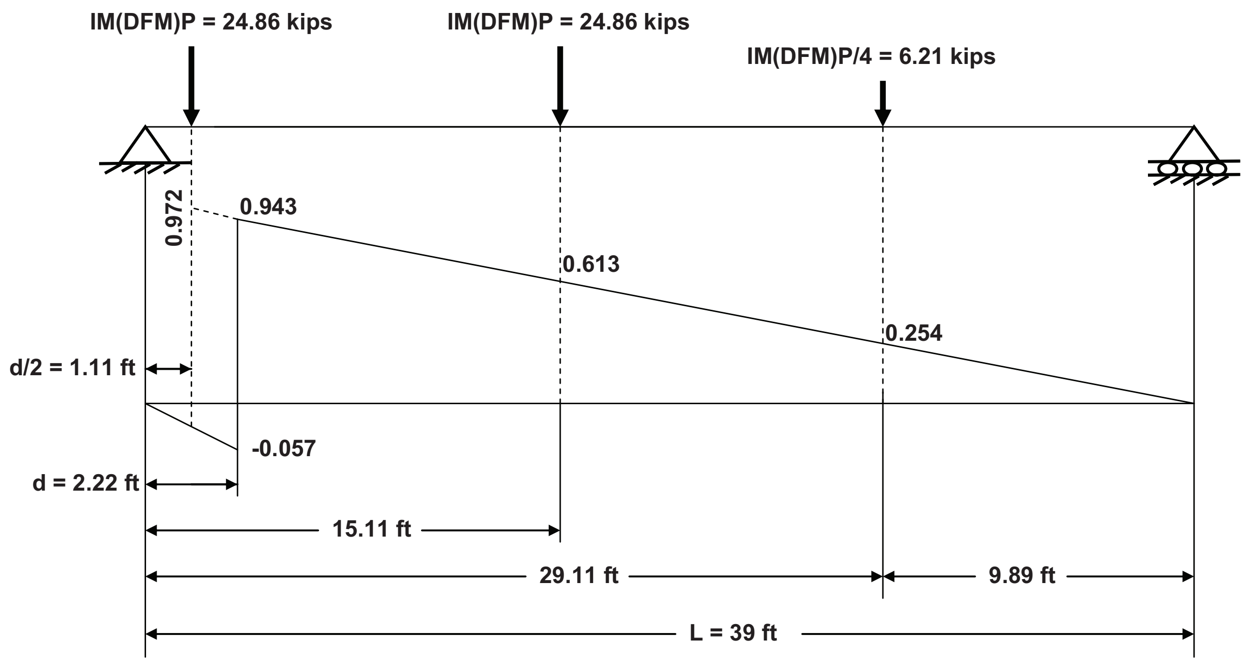


Figure C1-6.1 Shear Influence Line Diagram

$$\mathrm{V}_{\mathrm{HS20}}(1.11') = 24.86 + (24.86)(0.613) + (6.21)(0.254) = 41.7 kips$$

C1-6.1.2 Interior beam dead load shear force at d/2

As stated in section C1-6.1.1 since the dead load is a distributed load the shear at $\frac{d}{2}=\frac{2.22}{2}=$ 1.11 ft is taken as the shear computed at a distance d = 2.22 ft from the support.

$$V_D(1.11') = 1.29(19.5 - 1.11) = 23.72 kips$$
$$V_D(2.22') = 1.29(19.5 - 2.22) = 22.29 kips$$

C1-6.1.3 Interior beam live load shear force at 7.5 ft from the support

An influence line analysis shown in Figure C1-6.2 is used to compute the shear force at 7.5 ft from the support where the stirrup spacing changes from 12" to 18".

At 7.5ft from the support; the length of the portion of the span that is loaded to produce the maximum stress in the member is L = 39-7.5=31.5 ft thus:

$$IM = 1 + I = 1 + \frac{50}{(31.5)+125} = 1.32 \le 1.30$$

Use IM = 1.3

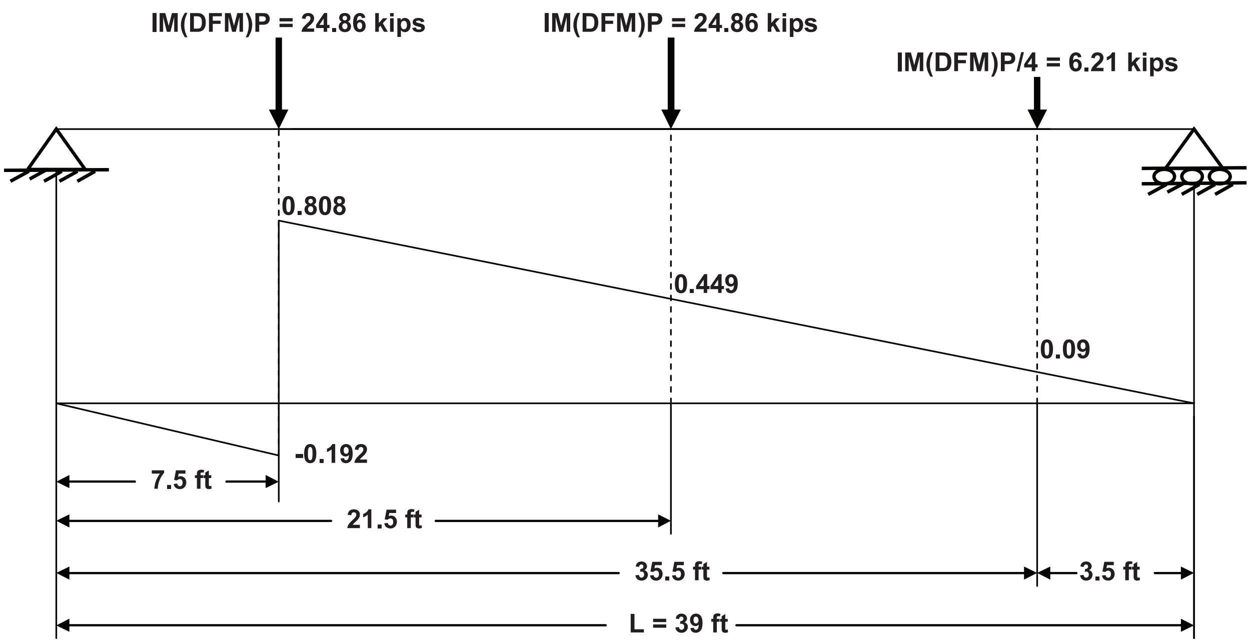


Figure C1-6.2 Shear Influence Line Diagram

$$\mathrm{V}_{\mathrm{HS20}}(7.5') = (24.86)(0.808) + (24.86)(0.449) + (6.21)(0.09) = 31.8kips$$

C1-6.1.4 Interior beam dead load shear force at 7.5 ft from the support

$$V_D(7.5') = 1.29(19.5 - 7.5) = 15.48kips$$

C1-6.2 Exterior Beam Shear Analysis

C1-6.2.1 Exterior beam maximum live load shear force

The exterior beam shear force due to live load at $\frac{d}{2} = \frac{2.14}{2} =$ 1.07 ft from the support is computed in a manner similar to that presented in Section C1-6.1.1.

At $\frac{d}{2} = 1.07$ ft; the length of the portion of the span that is loaded to produce the maximum stress in the member is L = 39-1.07 = 37.93 ft thus:

$$IM = 1 + I = 1 + \frac{50}{(37.93) + 125} = 1.31 \leq 1.30$$

Use IM = 1.3

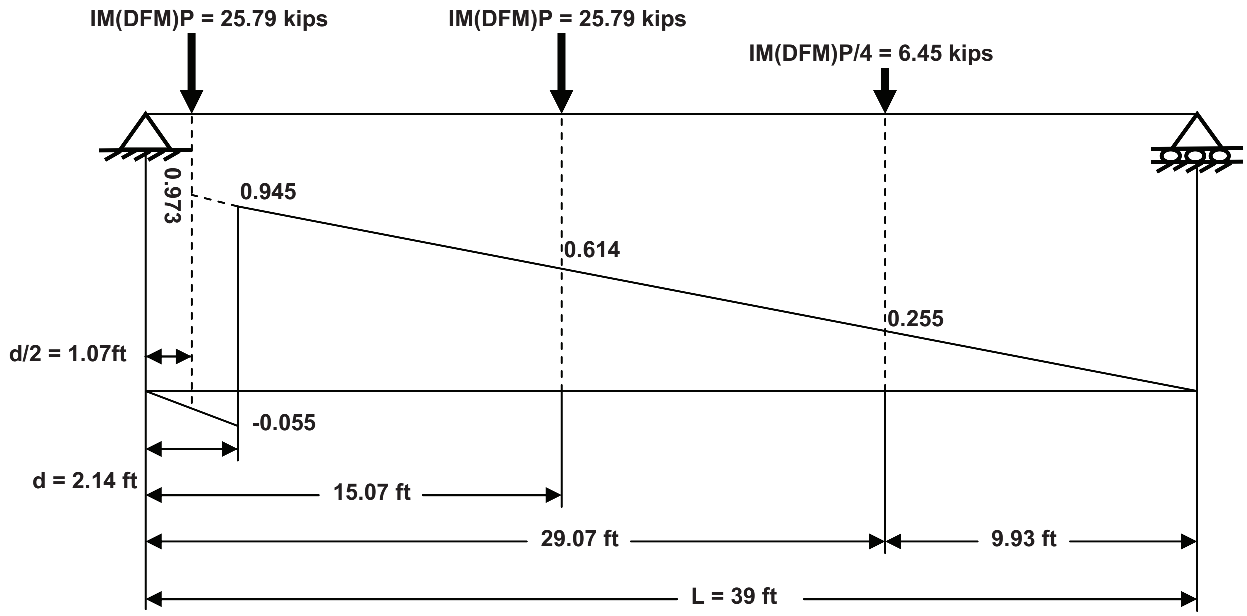


Figure C1-6.3 Shear Influence Line Diagram

$$\mathrm{V}_{\mathrm{HS20}}(1.07') = 25.79 + (25.79)(0.614) + (6.45)(0.255) = 43.3 kips$$

C1-6.2.2 Exterior beam dead load shear force at d/2

The exterior beam shear force due to dead load at $\frac{d}{2} = \frac{2.14}{2} =$ 1.07 ft is computed in a similar manner to that presented in Section C1-6.1.1, and taken as the shear computed at a distance d = 2.17 ft.

$$V_D(1.07') = 1.23(19.5 - 1.07) = 22.67 kips$$

$$V_D(2.14') = 1.23(19.5 - 2.14) = 21.35 kips$$

C1-6.2.3 Exterior beam live load shear force at 7.5 ft from the support

The exterior girder shear due to live load at 7.5 ft from the support is computed in the same manner presented in Section C1-6.1.3.

$$\mathrm{V}_{\mathrm{HS20}}(7.5') = 33.0 kips$$

C1-6.2.4 Exterior beam dead load shear force at 7.5 ft from the support

The exterior girder shear due to dead load at 7.5 ft from the support is computed in the same manner presented in Section C1-6.1.4.

$$V_D\left(7.5'\right) = 1.23\left(19.5 - 7.5\right) = 14.76 kips$$

C1-7 Load Combination

C1-7.1 Interior Girder

Table C1-7.1 shows that the governing moment loading case occurs at the maximum live load moment location (17.17 ft from the support).

Table C1-7.1 Interior Girder Load Combinations

	Moment at 19.5 ft from support	**Moment at 17.17 ft from support**
M_{LL+IM}	320.5 k-ft	335.6 k-ft
M_{DL}	245.3 k-ft	241.9 k-ft
$M_{DL}+ M_{LL+IM}$	565.8 k-ft	577.5 k-ft

C1-7.2 Exterior Girder

Table C1-7.2 shows that the governing moment loading case occurs at the maximum live load moment location (17.17 ft from the support).

Table C1-7.2 Exterior Girder Load Combinations

	Moment at 19.5 ft from support	**Moment at 17.17 ft from support**
M_{LL+IM}	332.6 k-ft	348.2 k-ft
M_{DL}	233.9 k-ft	230.6 k-ft
$M_{DL}+ M_{LL+IM}$	565.5 k-ft	578.8 k-ft

C1-8 Member Capacity

C1-8.1 Interior Girders

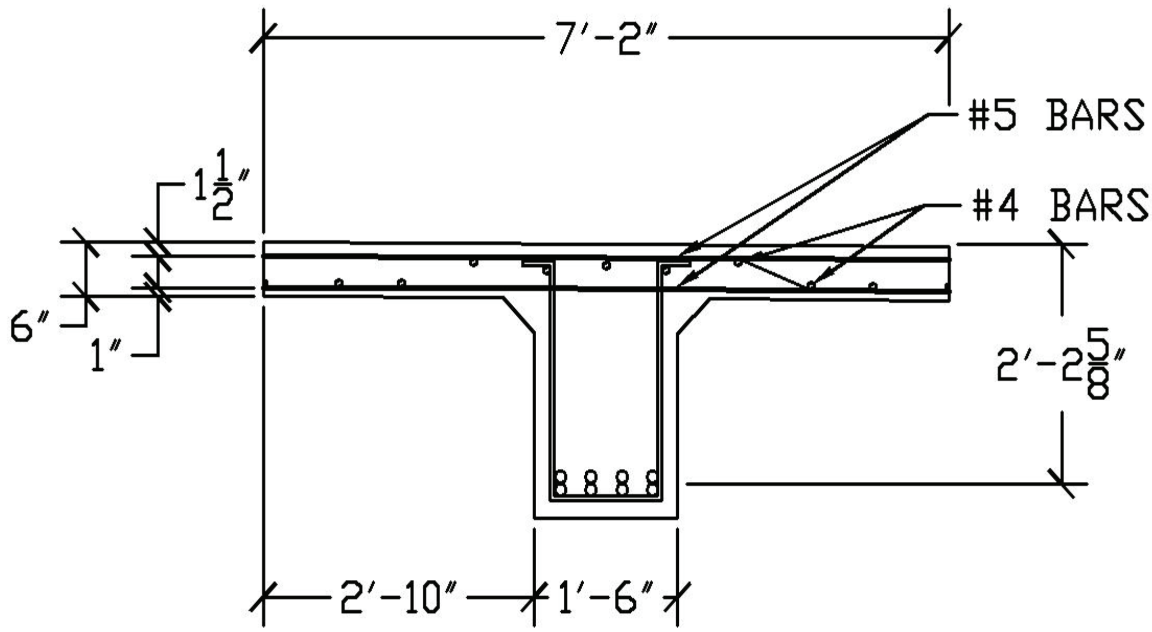


Figure C1-8.1 Member Dimensions

Effective flange width

Article 8.10.1.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002): The effective flange width is limited to the smallest of one-forth the span length, six times the slab thickness or half the distance to the adjacent girders, as per. Thus, $b_e = 86in$

Distance from the extreme compression fiber to centroid of tension reinforcement $d = 26.59in$

Concrete strength $f_c^{'} = 2.5ksi$

Steel reinforcement yield strength $f_y = 40ksi$

Stirrup area $\mathrm{A_v} = 2A_{\#4} = 0.4in^2$

Stirrup spacing $S = 12in$

Shear width $\mathrm{b_v} = 18in$

Angle of inclination of Diagonal Compression Stress $\theta = \pi / 4$

Table C1-8.1 Longitudinal Steel Layer Details

Layer	A_s (in^2)	Depth from top of slab to center of each reinforcement layer
1st layer of Steel includes 5 #4 bars	1.0	2.375
2nd layer of Steel includes 5 #4 bars	1.0	4.125
3rd layer of Steel includes 4 #11 bars	6.24	25.885
4th layer of Steel includes 4 #11 bars	6.24	27.295

Flexural strength of the reinforced concrete member shown in Figure C1-8.1 is performed in Table C1-8.2 in accordance with Article 8.15.3 of AASHTO Standard Specification for Highway Bridge Design, (AASHTO, 2002). In such analysis the contribution of reinforcing steel layer 1 and 2 found in table C1-8.1 is neglected due to the fact that it only provides an increase in member capacity of 0.6 percent.

Table C1-8.2 Interior girder capacity calculation

Flexure	**Inventory Level** f_y = 20 ksi f'$_c$ = 1.0ksi	β=0.85 for $f'_c \le 4$ksi $c = \dfrac{A_s f_y}{.85 f'_c \beta b_e} = \dfrac{12.48(20)}{.85(1).85(86)} = 4.017$ in < t_s = 6 in $a = \beta c = .85(4.017) = 3.415$ in $M_{INV} = A_s f_y \left(d - \dfrac{a}{2}\right)\dfrac{1}{12} = 517.6\ k - ft$	AASHTO MCE 2000 D.6.6.2.3
	Operating Level fy = 28 ksi f'c = 1.5 ksi	$c = \dfrac{A_s f_y}{.85 f'_c \beta b_e} = \dfrac{12.48(28)}{.85(1.5).85(86)} = 3.749$ in < t_s = 6 in $a = c\beta = 3.187$ in $M_{OPR} = A_s f_y \left(d - \dfrac{a}{2}\right)\dfrac{1}{12} = 727.9$ kip-ft	
Shear (d/2) $A_{\#4}$ = .2 in^2 $A_v = 2A_{\#4}$ = .4 in^2 $\theta = \pi/4$, $S = 12$ in b_v = 18 in d = 26.59 in	**Inventory Level** f_y = 20 ksi f'$_c$ = 2.5 ksi	$V_c = 0.95\sqrt{f'_c}\, b_v d = 22.7$ kips $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 17.7$ kips $V_{INV} = V_c + V_s = 40.4$ kips	AASHTO MCE 2000 D.6.6.2.4.
	Operating Level f_y = 28 ksi f'$_c$ = 2.5 ksi	$V_c = 1.3\sqrt{f'_c}\, b_v d = 31.1$ kips $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 24.8$ kips $V_{OPR} = V_c + V_s = 55.9$ kips	

Shear (7.5') $A_{\#4} = .2\ in^2$ $A_v = 2A_{\#4} = .4\ in^2$ $\theta = \pi/4$, $S = 18$ in $b_v = 18$ in d = 26.59 in	**Inventory Level** $f_y = 20$ ksi $f'_c = 2.5$ ksi	$V_c = 0.95\sqrt{f'_c}\,b_v d = 22.7$ kips $V_s = \dfrac{A_v f_y d\ \cot(\theta)}{S} = 11.5$ kips $V_{INV} = V_c + V_s = 34.2$ kips	AASHTO MCE 2000 D.6.6.2.4.
	Operating Level $f_y = 28$ ksi $f'_c = 2.5$ ksi	$V_c = 1.3\sqrt{f'_c}\,b_v d = 31.1$ kips $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 16.5$ kips $V_{OPR} = V_c + V_s = 47.6$ kips	

C1-8.2 Exterior Girders

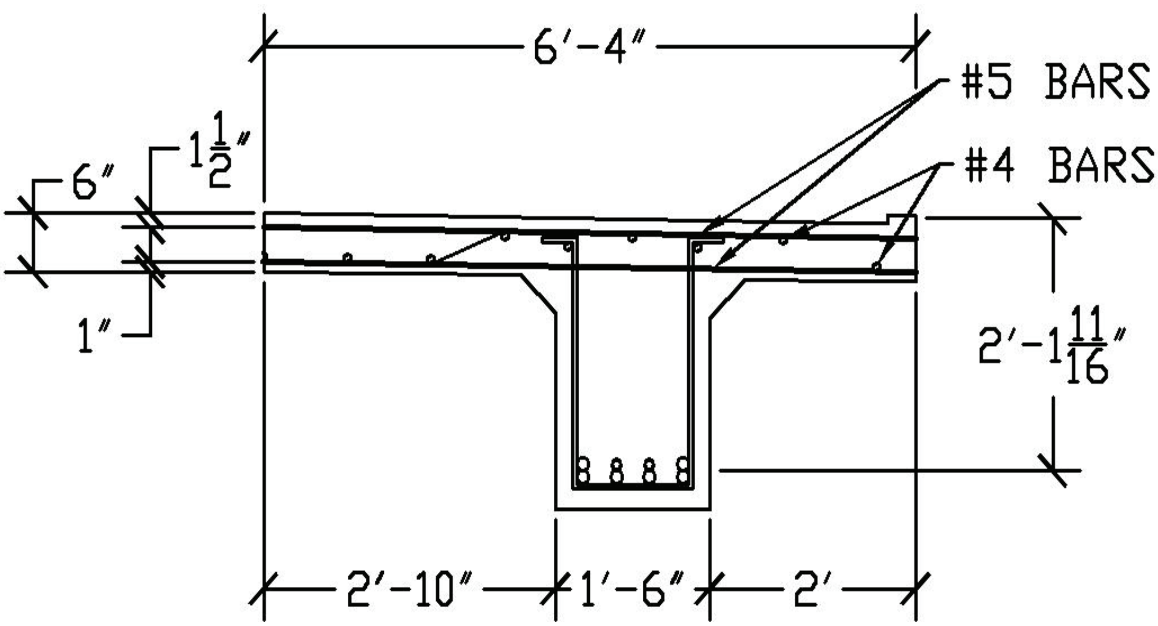


Figure C1-8.2 Member Dimensions

Effective flange width

Article 8.10.1.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002): The effective flange width is limited to the smallest of one-forth the span length, six times the slab thickness or half the distance to the adjacent girders, as per. Thus, $b_e = 76in$

Distance from the extreme compression fiber to centroid of tension reinforcement $d = 25.70in$
Flexural steel area $A_s = 11.9in$
Concrete strength $f_c^{'} = 2.5ksi$
Steel reinforcement yield strength $f_y = 40ksi$
Stirrup area $\mathrm{A_v} = 2A_{\#4} = 0.4in^2$
Stirrup spacing $S = 12in$
Shear width $\mathrm{b_v} = 18in$
Angle of inclination of Diagonal Compression Stress $\theta = \pi / 4$

Table C1-8.3 Exterior girder capacity calculation

Flexure	**Inventory Level** $f_y = 20$ ksi $f'_c = 1.0$ksi	β=0.85 for $f'_c \leq 4$ksi $c = \frac{A_s f_y}{.85 f'_c \beta b_e} = \frac{11.9(20)}{.85(1).85(76)} = 4.334$ in $<$ t_s = 6 in $a = \beta c = .85(4.334) = 3.684$ in $M_{INV} = A_s f_y \left(d - \frac{a}{2}\right)\frac{1}{12} = 473.2\ k - ft$	AASHTO MCE 2000 D.6.6.2.3
	Operating Level fy = 28 ksi f'c = 1.5 ksi	$c = \frac{A_s f_y}{.85 f'_c \beta b_e} = \frac{12.48(28)}{.85(1.5).85(76)} = 4.045$ in $<$ t_s = 6 in $a = c\beta = 3.438$ in $M_{OPR} = A_s f_y \left(d - \frac{a}{2}\right)\frac{1}{12} = 665.9$ kip-ft	

<table>
<tr><td rowspan="2">Shear (d/2)

$A_{\#4} = .2\ in^2$
$A_v = 2A_{\#4} = .4\ in^2$
$\theta = \pi/4$,
$S = 12\ in$
$b_v = 18\ in$
$d = 25.7\ in$</td><td>Inventory Level

$f_y = 20\ ksi$
$f'_c = 2.5\ ksi$</td><td>$V_c = 0.95\sqrt{f'_c}\,b_v d = 21.97\ kips$

$V_s = \frac{A_v f_y d\ \cot(\theta)}{S} = 17.13\ kips$

$V_{INV} = V_c + V_s = 39.1\ kips$</td><td rowspan="2">AASHTO MCE 2000 D.6.6.2.4.</td></tr>
<tr><td>Operating Level

$f_y = 28\ ksi$
$f'_c = 2.5\ ksi$</td><td>$V_c = 1.3\sqrt{f'_c}\,b_v d = 30.07\ kips$

$V_s = \frac{A_v f_y d \cot(\theta)}{S} = 23.97\ kips$

$V_{OPR} = V_c + V_s = 54\ kips$</td></tr>
<tr><td rowspan="2">Shear (7.5')

$A_{\#4} = .2\ in^2$
$A_v = 2A_{\#4} = .4\ in^2$
$\theta = \pi/4$,
$S = 18\ in$
$b_v = 18\ in$
$d = 25.69\ in$</td><td>Inventory Level

$f_y = 20\ ksi$
$f'_c = 2.5\ ksi$</td><td>$V_c = 0.95\sqrt{f'_c}\,b_v d = 21.97\ kips$

$V_s = \frac{A_v f_y d\ \cot(\theta)}{S} = 11.42\ kips$

$V_{INV} = V_c + V_s = 33.4\ kips$</td><td rowspan="2">AASHTO MCE 2000 D.6.6.2.4.</td></tr>
<tr><td>Operating Level

$f_y = 28\ ksi$
$f'_c = 2.5\ ksi$</td><td>$V_c = 1.3\sqrt{f'_c}\,b_v d = 30.07\ kips$

$V_s = \frac{A_v f_y d \cot(\theta)}{S} = 15.99\ kips$

$V_{OPR} = V_c + V_s = 46.1\ kips$</td></tr>
</table>

C1-9 Rating Calculation (ASR)

C1-9.1 ASR for HS20 Vehicle

Table C1-9.1 Allowable Stress Rating (ASR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002))

Flexure **(Interior girder)**	**Inventory Level**	$RF = \frac{M_{INV} - M_D}{M_{LL+IM}} = \frac{517.6 - 241.9}{335.6} = 0.82$	29.5 tons
	Operating Level	$RF = \frac{M_{OPR} - M_D}{M_{LL+IM}} = \frac{727.9 - 241.9}{335.6} = 1.45$	52.2 tons
Flexure **(Exterior girder)**	**Inventory Level**	$RF = \frac{M_{INV} - M_D}{M_{LL+IM}} = \frac{473.2 - 230.6}{348.2} = 0.70$	25.2 tons
	Operating Level	$RF = \frac{M_{OPR} - M_D}{M_{LL+IM}} = \frac{665.9 - 230.6}{348.2} = 1.25$	45.0 tons
Shear at d/2 (Interior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{40.4 - 22.29}{41.7} = 0.44$	15.8 tons
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{55.9 - 22.29}{41.7} = 0.81$	29.2 tons
Shear at d/2 (Exterior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{39.1 - 21.35}{43.3} = 0.41$	14.8 tons
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{54 - 21.35}{43.3} = 0.75$	27.0 tons
Shear at 7.5 ft (Interior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{34.2 - 15.48}{31.8} = 0.59$	21.2 tons
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{47.6 - 15.48}{31.8} = 1.01$	36.4 tons
Shear at 7.5 ft (Exterior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{33.4 - 14.76}{33.0} = 0.56$	20.2 tons
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{46.1 - 14.76}{33.0} = 0.95$	34.2 tons

C1-9.2 ASR for GA HS20-Mod Vehicle

The HS20-Mod axel loads are 14 kips, 23 kips, and 23 kips; each spaced 14 feet apart. The resulting maximum live load shear force is 32.8 kips.

Table C1-9.1 Allowable Stress Rating (ASR) Calculation for GA HS20-Mod (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002))

Shear at d/2 (Exterior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{39.1 - 21.35}{32.8} = 0.54$	19.4 tons
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{54 - 21.35}{32.8} = 1.00$	36.0 tons

Example C2:

Rating by the Load Factor Method (LFR) Using Load Distribution and Dynamic Allowance Factors Stipulated by the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002).

C2-1 Analysis

C2-1.1 Maximum live load Bending Moment

C2-1.1.1 Interior beam

Table C2-1.1 Interior Beam Load Combinations (computed in Section C1-6)

	Moment at 19.5 ft from support	Moment at 17.17 ft from support
M_{LL+IM}	320.5 k-ft	335.6 k-ft
M_{DL}	245.3 k-ft	241.9 k-ft
$M_{DL}+M_{LL+IM}$	565.8 k-ft	577.5 k-ft

C2-1.1.2 Exterior beam

Table C2-1.2 Exterior Beam Load Combinations (computed in Section C1-6)

	Moment at 19.5 ft from support	Moment at 17.17 ft from support
M_{LL+IM}	332.6 k-ft	348.2 k-ft
M_{DL}	233.9 k-ft	230.6 k-ft
$M_{DL}+M_{LL+IM}$	565.5 k-ft	578.8 k-ft

C2-1.2 Shear Analysis

C2-1.2.1 Interior beam

Table C2-1.3 Interior beam shear forces (Computed in Section C1-6)

Location (from supports)	V_{LL+IM}	V_{DL}
d/2 = 1.11ft	41.7 kips	22.29 kips
7.5 ft	31.8 kips	15.48 kips

C2-1.2.2 Exterior beam

Table C2-1.4 Exterior beam shear forces (Computed in Section C1-6)

Location (from supports)	V_{LL+IM}	V_{DL}
d/2 = 1.11ft	43.3 kips	21.35 kips
7.5 ft	33.0 kips	14.76 kips

C2-2 Member Capacity

C2-2.1 Interior Beam

Table C2-2.1 Interior beam capacity calculation

<table>
<tr>
<td>Flexure

f_y = 40 ksi
f'_c = 2.5ksi</td>
<td>$\beta=0.85 \text{ for } f'_c \leq 4\text{ksi}$
$c=\frac{A_s f_y}{.85 f'_c \beta b_e}=\frac{12.48(40)}{.85(2.5).85(86)}=3.214 \text{ in} < t_s = 6 \text{ in}$
$a=\beta c=.85(3.214)=2.732 \text{ in}$
$M_n=A_s f_y\left(d-\frac{a}{2}\right)\frac{1}{12}$
$=12.48(40)\left(26.59-\frac{2.732}{2}\right)\frac{1}{12}=1049\ k-ft$</td>
<td>AASHTO
MCE 2000
D.6.6.2.3</td>
</tr>
<tr>
<td>Shear
(d/2)</td>
<td>$V_c=0.0316\beta\sqrt{f'_c}b_v d$
$=0.0316(2)\sqrt{2.5}(18)26.59=47.8kips$
$V_s=\frac{A_v f_y d \cot(\theta)}{S}$
$=\frac{0.4(40)26.59\cot\left(\frac{\pi}{4}\right)}{12}=35.5kips$
$V_n=V_c+V_s$ = 47.9 + 35.5 = 83.3 kips</td>
<td>AASHTO
MCE 2000
D.6.6.2.4.</td>
</tr>
<tr>
<td>Shear
(7.5 ft)</td>
<td>$V_c=0.0316\beta\sqrt{f'_c}b_v d$
$=0.0316(2)\sqrt{2.5}(18)26.59=47.8kips$
$V_s=\frac{A_v f_y d \cot(\theta)}{S}=\frac{0.4(40)(26.59)\cot\left(\frac{\pi}{4}\right)}{18}=23.6$
$V_n=V_c+V_s$ = 47.9 + 23.7 = 71.4 kips</td>
<td>AASHTO
MCE 2000
D.6.6.2.4.</td>
</tr>
</table>

C2-2.2 Exterior Beam

Table C2-2.2 Exterior beam capacity calculation

Flexure f_y = 40 ksi f'_c = 2.5ksi	$\beta = 0.85$ for $f'_c \leq 4$ksi $c = \frac{A_s f_y}{.85 f'_c \beta b_e} = \frac{11.9(40)}{.85(2.5).85(76)} = 3.467$ in < t_s = 6 in $a = \beta c = .85(3.467) = 2.947$ in $M_n = A_s f_y \left(d - \frac{a}{2}\right)\frac{1}{12}$ $= 11.9(40)\left(25.7 - \frac{2.947}{2}\right)\frac{1}{12} = 961\, k - ft$	AASHTO MCE 2000 D.6.6.2.3
Shear **(d/2)**	$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d$ $= 0.0316(2)\sqrt{2.5}(18)25.7 = 46.2 kips$ $V_s = \frac{A_v f_y d \cot(\theta)}{S} = \frac{0.4(40)25.7\cot\left(\frac{\pi}{4}\right)}{12} = 34.3$ kips $V_n = V_c + V_s$ = 46.2 + 34.4 = 80.5 kips	AASHTO MCE 2000 D.6.6.2.4.
Shear **(7.5 ft)**	$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d$ $= 0.0316(2)\sqrt{2.5}(18)25.7 = 46.2 kips$ $V_s = \frac{A_v f_y d \cot(\theta)}{S} = \frac{0.4(40)(25.7)\cot\left(\frac{\pi}{4}\right)}{18} = 22.8$ kips $V_n = V_c + V_s$ = 46.2 + 22.8 = 69.0 kips	AASHTO MCE 2000 D.6.6.2.4.

C2-3 Rating Calculation (LFR)

Table C2-3.1 Load Factor Rating (LFR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002))

Flexure (Interior girder) $\phi = 0.9$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\varphi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \dfrac{0.9(1049) - 1.3(241.4)}{2.17(335.6)} = 0.87$	31.3 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\varphi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \dfrac{0.9(1049) - 1.3(241.4)}{1.3(335.6)} = 1.44$	51.8 tons
Flexure (Exterior girder) $\phi = 0.9$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \dfrac{0.9(961) - 1.3(230.60)}{2.17(348.2)} = 0.75$	27.0 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \dfrac{0.9(961) - 1.3(230.60)}{1.3(348.2)} = 1.25$	45.0 tons
Shear (Interior Girder) (d/2) $\phi = 0.85$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(83.3) - 1.3(22.29)}{2.17(41.7)} = 0.46$	16.6 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(83.3) - 1.3(22.29)}{1.3(41.7)} = 0.77$	27.7 tons
Shear (Exterior Girder) (d/2) $\phi = 0.85$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(80.5) - 1.3(21.35)}{2.17(43.3)} = 0.43$	<u>15.5 tons</u>
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(80.5) - 1.3(21.35)}{1.3(43.3)} = 0.72$	<u>25.9 tons</u>

Shear (Interior Girder) (7.5 ft) $\phi = 0.85$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(71.4) - 1.3(15.48)}{2.17(31.8)} = 0.59$	21.2 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(71.4) - 1.3(15.48)}{1.3(31.8)} = 0.98$	35.3 tons
Shear (Exterior Girder) (7.5 ft) $\phi = 0.85$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(69.0) - 1.3(14.76)}{2.17(33.0)} = 0.55$	19.8 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(69.0) - 1.3(14.76)}{1.3(33.0)} = 0.92$	33.1 tons

Example C3:

Rating by the Load and Resistance Factor Method (LRFR) Using Load Distribution and Dynamic Allowance Factors Stipulated by the *AASHTO LRFD Specifications* (2007).

C3-1 Dynamic Load Allowance

From Table 3.6.2.1 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007), the dynamic load allowance is taken as 33%. Thus, the dynamic load factor to be applied to the static load is:

$$\left(1+\frac{IM}{100}\right)=1.33$$

C3-2 Live Load Distribution Factors

C3-2.1 Interior Beams

<u>C3-2.1.1 Distribution Factor for ***moment*** in interior longitudinal beams</u>

As per Table 4.6.2.2.2b-1 of AASHTO LRFD Bridge Design Specifications (AASHTO, 2007), the distribution factor for moment in interior beams, g_m , is specified as follows

When one lane is loaded: $$g_{m1}=0.06+\left(\frac{s}{14}\right)^{0.4}+\left(\frac{s}{L}\right)^{0.3}\left(\frac{k_g}{12Lt_s^3}\right)^{0.1}$$

When two or more lane are loaded: $$g_{m2}=0.075+\left(\frac{s}{9.5}\right)^{0.6}+\left(\frac{s}{L}\right)^{0.2}\left(\frac{k_g}{12Lt_s^3}\right)^{0.1}$$

The distribution factor for moment in exterior beams, g_m, is specified as follows

In the case of one lane loaded: g_m is computed by the lever rule
In the case of two or more lane loaded:
The longitudinal stiffness parameter:

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

In which $n=\frac{E_B}{E_D}$ where

E_B = modulus of elasticity of the beam material
E_D = modulus of elasticity of the deck material
e_g = the distance between the centers of gravity of the beams and deck
I = moment of inertia of the beam
A = area of beam

$$n=\frac{E_B}{E_D}=1$$

$$e_g = \frac{24.5}{2} + \frac{6}{2} = 15.25\ in$$

$$I = \frac{(18)(24.5)^3}{12} = 22,059\ in^4$$
$$A = (18)(24.5) = 441\ in^2$$

$$K_g = n\left(I + Ae_g^2\right) = 1\left[22,059 + (441)(15.25)^2\right] = 124,619\ in^4$$

With one lane loaded:
$$g_{m1} = 0.06 + \left(\frac{7.17}{14}\right)^{0.4}\left(\frac{7.17}{39}\right)^{0.3}\left(\frac{124,619}{12(39)(6)^3}\right)^{0.1} = 0.06 + (0.765)(0.6)(1.02) = 0.53$$

With two or more lane loaded:

$$g_{m2} = 0.075 + \left(\frac{7.17}{9.5}\right)^{0.6}\left(\frac{7.17}{39}\right)^{0.2}\left(\frac{124,619}{12(39)(6)^3}\right)^{0.1} = 0.075 + (0.845)(0.713)(1.021) = 0.69$$

$$g_m = \max\left(g_{m1}, g_{m2}\right) = \max\left(0.53, 0.69\right) = 0.69$$

C3-2.1.2 Distribution Factor for ***shear*** in interior longitudinal beams

The distribution factor for shear in interior beams is specified in Table 4.6.2.2.3a-1 of AASHTO LRFD Bridge Design Specifications (AASHT0, 2007) as follows

When one lane is loaded: $g_{v1} = 0.36 + \frac{s}{25} = 0.36 + \frac{7.17}{25} = 0.65$

When two or more lane are loaded: $g_{v2} = 0.2 + \frac{s}{12} - \left(\frac{s}{35}\right)^{2.0} = 0.2 + \frac{7.17}{12} - \left(\frac{7.17}{35}\right)^{2.0} = 0.76$

Thus, $g_v = \max\left(g_{v1}, g_{v2}\right) = \max\left(0.65, 0.76\right) = 0.76$

C3-2.2 Exterior Beams

C3-2.2.1 Distribution Factor for ***moment*** in exterior longitudinal beams

The distribution factor for moment in exterior beams is specified in Table 4.6.2.2.2d-1 of AASHTO LRFD Bridge Design Specifications (2007) as follows:

- When one design lane is loaded, the lever rule is used to determine the distribution factor, g_m

- When two or more lanes are loaded, the distribution factor is computed by multiplying the distribution factor for moment in interior beam by a correction factor, e, defined as

$$e = 0.77 + \frac{d_e}{9.1}$$

Where d_e is the distance from the exterior web of the exterior beam to the interior edge of the curb of traffic barrier.

For computing the distribution factor by the lever rule, a simple structural member such as the one shown below is analyzed

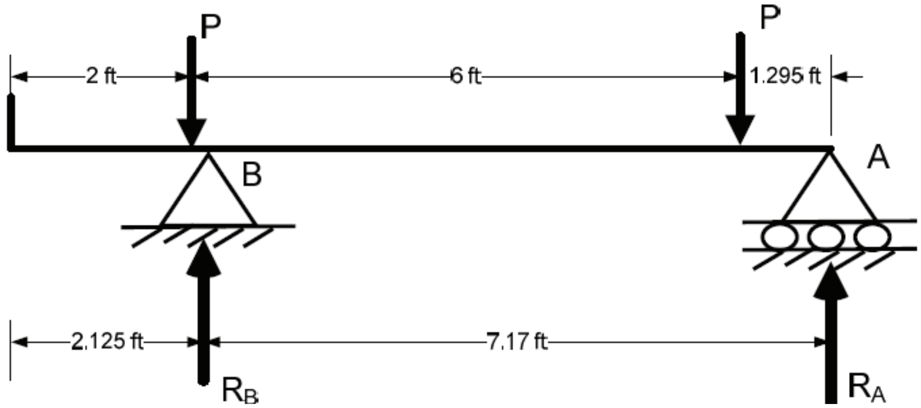


Figure C3-2.1 Exterior Girder Moment Distribution Factor

$$\sum M_A = 0$$
$$R_B(7.17) - P(7.295) - P(1.295) = 0\,.$$
$$R_B = 1.198P$$

Article 3.6.1.1.2 of AASHTO LRFD Bridge Design Specification (AASHTO, 2007) states that a multiple presence factor m = 1.20 must be used when computing girder distribution factors by the lever rule. Thus, when one lane is loaded the distribution factor for the moment in exterior beams is:

$$g_{m1(exterior)} = m\frac{1.198P}{2P} = 1.2(0.6) = 0.72$$

When two or more lanes are loaded:

$$e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{(2.125 - 0.75)}{9.1} = 0.92$$

$$g_{m2(exterior)} = e\, g_{m(interior)} = (0.92)(0.69) = 0.63$$

$$g_{m(exterior)} = \max\left(g_{m1(exterior)}, g_{m2(exterior)}\right) = \max(0.72, 0.63) = 0.72$$

C3-2.3.2.2 Distribution Factor for ***shear*** in exterior longitudinal beams

When one lane is loaded, the distribution factor for shear is computed by the lever rule.
Thus, $g_{v1(exterior)} = 0.72$

When two or more lanes are loaded

$$g_{v2(exterior)} = (e)\, g_{v(interior)} = \left(0.6 + \frac{d_e}{10}\right) g_{v(interior)}$$

$$= \left(0.66 + \frac{2.125 - 0.75}{10}\right)(0.76) = (0.738)(0.76) = 0.61$$

$$g_{v(exterior)} = \max\left(g_{v1(exterior)}, g_{v2(exterior)}\right) = \max(0.72, 0.61) = 0.72$$

C3-2.3 Summary Results of Load Distribution Factors

The following table summarizes the results of calculations concerning the live load distribution factors:

Table C3-2.1 Distribution Factors

Action	Interior Beam	Exterior Beam
Bending Moment	0.69	0.72
Shear	0.76	0.72

C3-3 Analysis

With the LRFR method the HL93 load case in considered at the inventory and operating load level. The HS20 load case is considered for ratings at the legal load level.
The HL93 load consists of two load cases

- **Design truck:** consisting of the HS20 load subjected to distribution and impact factors plus a lane load ($w = 0.64 kips / ft$) that is only subjected to an distribution factors

- **Design tandem:** consisting of 2 25 kip axel spaced 4 ft apart subjected to distribution and impact factors plus a lane load ($w = 0.64 kips / ft$) that is only subjected to an distribution factors

C3-3.1 Maximum Bending Moment

A rudimentary structural analysis of a simply supported beam subjected to a vehicular load having two rear axles and one front axle as shown in Figure C2-4.1 shows that the maximum moment occurs under the middle axle when such an axle is positioned at a distance of 2.33 ft to the left of the beam centerline.

By applying the dynamic allowance factor and the distribution factor for moment of interior beams, we can then compute the maximum live load under the following loads:

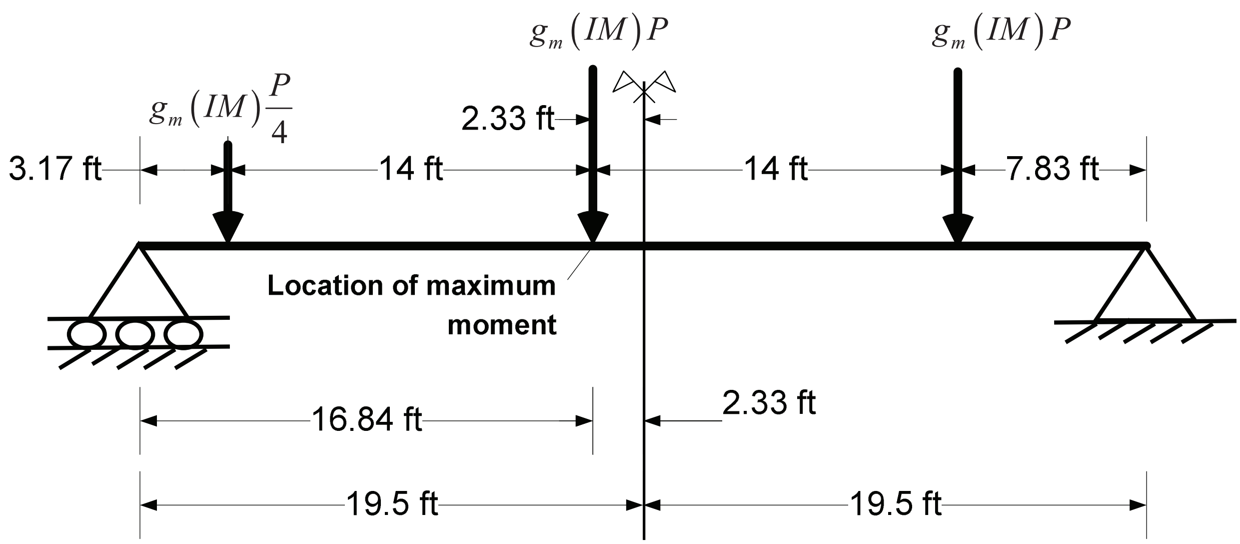


Figure C3-3.1 Max Live Load Moment

C3-3.1.1 Maximum live load moment for an interior beam

With P = 32 kips, $g_m = 0.69$, and IM =1.33, M_{LL+IM} can be computed at the location of the maximum live load moment (at 17.17 ft from the left support) to be:

$$M_{LL+IM} = 396.4\ k - ft$$

Live load moment (at midspan):

$$M_{LL+IM} = 378.71\, k-ft$$

The HL93 load:

$$DesignTruck(17.17) = M_{LL+IM} + g_m\left[12.48(17.17) - w\frac{(17.17)^2}{2}\right]$$

$$= 396.4 + 0.69\left[12.48(17.17) - 0.64\frac{(17.17)^2}{2}\right] = 479.2\, k-ft$$

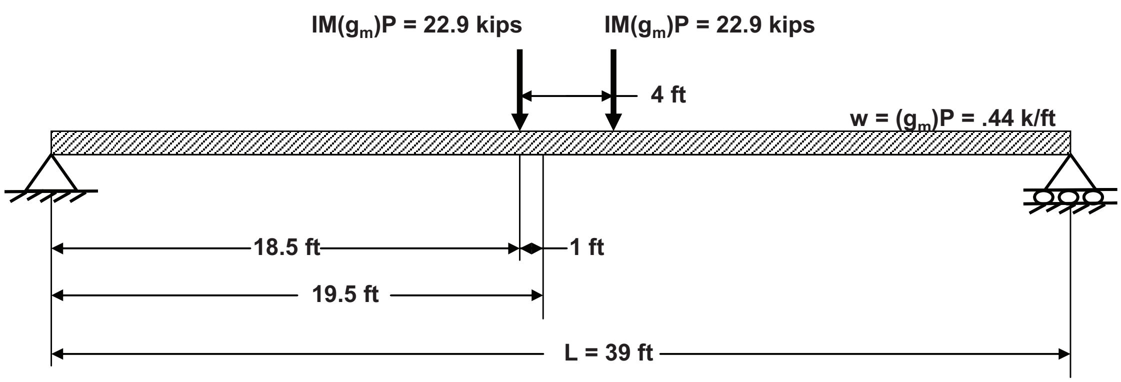


Figure C3-3.2 Maximum Design Tandem Moment

$$DesignTandem(18.5) = 21.7\left(18.5\right) + \left[8.58(18.5) - 0.44\frac{(18.5)^2}{2}\right] = 485\, k-ft$$

$$M_{HL93} = \max\left(DesignTruck, DesignTandem\right) = 485k-ft$$

C3-3.1.2 Maximum live load moment for an exterior beam

With P = 32 kips, $g_{m(exterior)} = 0.72$, and IM =1.33, M_{LL+IM} can be computed at the location of the maximum live load moment (at 17.17 ft from the left support) to be:

$$M_{LL+IM} = 413.7\, k-ft$$

Live load moment (at midspan):

$$M_{LL+IM} = 395.2\, k-ft$$

The HL93 load in this case is:

$$DesignTruck(17.17) = M_{LL+IM} + g_m\left[12.48(17.17) - w\frac{(17.17)^2}{2}\right]$$

$$= 413.7 + 0.72\left[12.48(17.17) - 0.64\frac{(17.17)^2}{2}\right] = 500.1k - ft$$

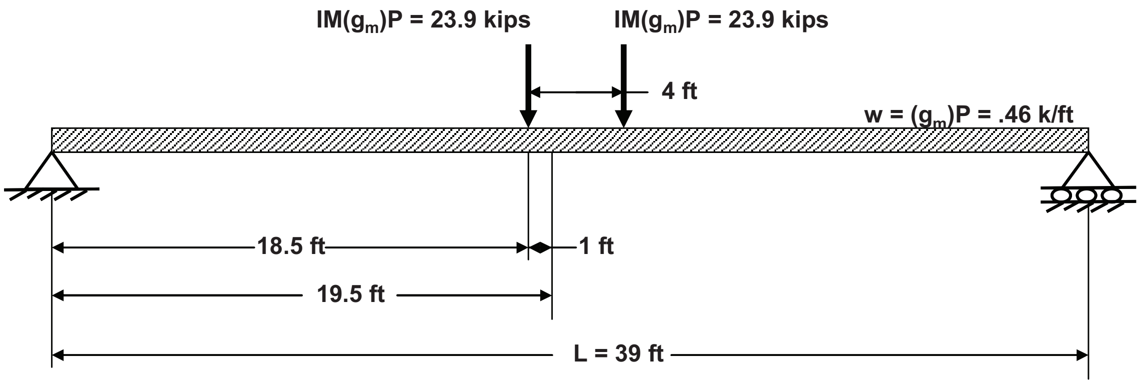


Figure C3-3.3 Maximum Design Tandem Moment

$$DesignTandem(18.5) = 22.7\left(18.5\right) + \left[8.97(18.5) - 0.46\frac{(18.5)^2}{2}\right] = 507\ k - ft$$

$$M_{HL93} = \max\left(DesignTruck, DesignTandem\right) = 507k - ft$$

C3-3.1.3 Maximum dead load moment for an interior beam

The dead load moment at the location where the live load produces the maximum effect is

$$M_D = 25.16(17.17) - 1.29\frac{(17.17)^2}{2} = 241.9\ k - ft$$

The dead load moment at midspan is:

$$M_{D(m)} = 1.29\frac{\left(39\right)^2}{8} = 245.3k - ft$$

C3-3.1.4 Maximum dead load moment for an exterior beam

The dead load moment at the location where the live load produces the maximum effect is

$$M_D = 23.99(17.17) - 1.23\frac{(17.17)^2}{2} = 230.6\ k - ft$$

The dead load moment at midspan is:

$$M_{D(m)} = 1.23\frac{\left(39\right)^2}{8} = 233.9k - ft$$

C3-3.2 Maximum Shear Force

C3-3.2.1 Interior beam maximum live load shear force at $d_v = 2.1ft$

$$\mathrm{V}_{\mathrm{HS20}}\left(\mathrm{d}_v\right) = 51.4kips$$

$$DesignTruck\left(d_v\right) = V_{HS20} + g_v w\left(19.5 - d_v\right) = 51.4 + 0.76\left(0.64\right)\left(19.5 - 2.1\right) = 59.9\ kips$$

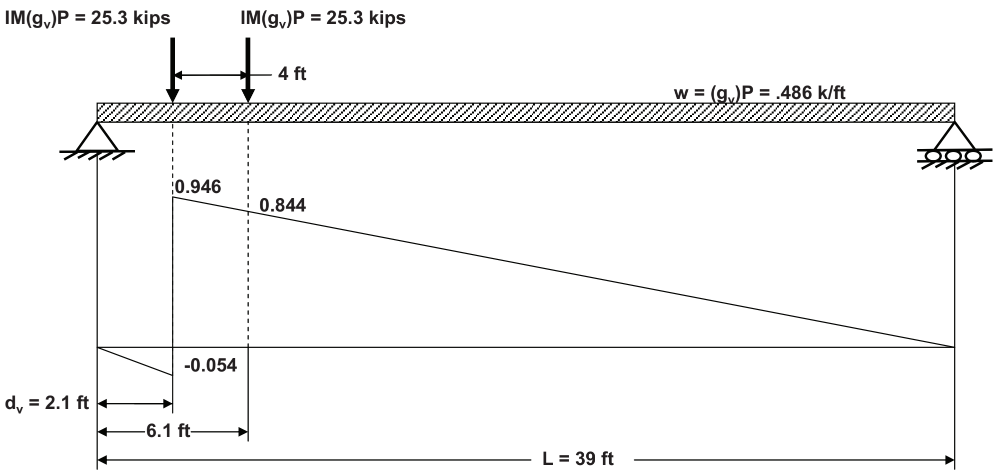


Figure C3-3.4 Interior Beam Shear Design Tandem at d_v

$$DesignTandem\left(d_v\right) = 25.3\left(.946\right) + 25.3\left(.844\right) + .486\left(19.5 - 2.1\right) = 53.7\ kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 59.9kips$$

C3-3.2.2 Interior beam dead load shear force at $d_v = 2.1ft$

$$\mathrm{V}_{\mathrm{DL}}\left(\mathrm{d}_v\right) = 1.29\left(19.5 - 2.1\right) = 22.45kips$$

C3-3.2.3 Interior beam live load shear force at 7.5 ft

$$\mathrm{V}_{\mathrm{HS20}}\left(7.5'\right) = 41.4kips$$

$$V_{HL93}\left(d_v\right) = V_{HS20} + g_{v(ext)} w\left(19.5 - d_v\right) = 41.4 + 0.76\left(0.64\right)\left(19.5 - 2.1\right) = 49.9\ k - ft$$

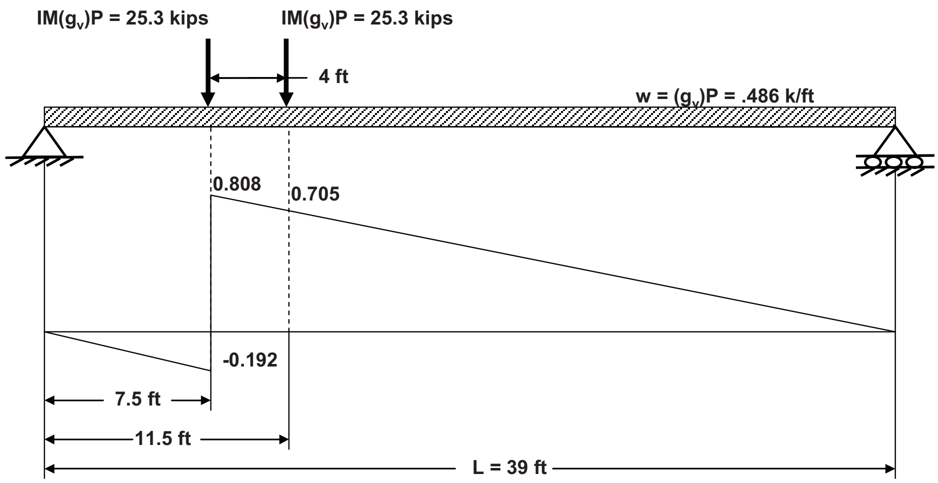


Figure C3-3.5 Interior Beam Shear Design Tandem at 7.5 ft from support

$$DesignTandem\left(d_v\right) = 25.3\left(.808\right) + 25.3\left(.705\right) + .486\left(19.5 - 7.5\right) = 44.1\,kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 49.9kips$$

<u>C3-3.2.4 Interior beam live load shear force at 7.5 ft</u>

$$\mathrm{V_{DL}}\left(7.5'\right) = 1.29\left(19.5 - 7.5\right) = 15.48kips$$

<u>C3-3.2.5 Exterior beam maximum live load shear force at</u> $d_v = 2.0\,ft$

$$\mathrm{V_{HS20}}\left(\mathrm{d}_v\right) = 48.92kips$$

$$V_{HL93}\left(d_v\right) = V_{HS20} + g_{v(ext)}w\left(19.5 - d_v\right) = 48.95 + 0.72\left(0.64\right)\left(19.5 - 2.0\right) = 57.0\,k - ft$$

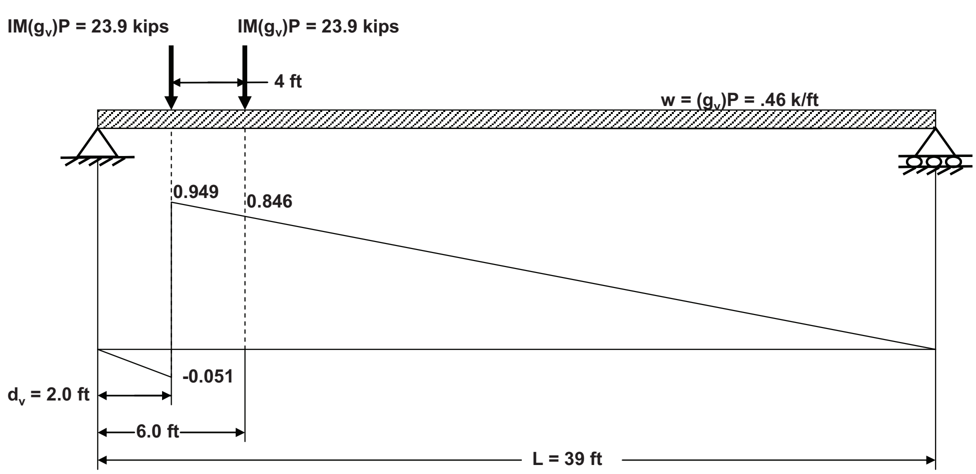


Figure C3-3.6 Exterior Beam Shear Design Tandem at d_v

$$DesignTandem(d_v) = 23.9(.949) + 23.9(.846) + .46(19.5-2) = 51\,kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 57kips$$

<u>C3-3.2.6 Exterior beam maximum live load shear force at</u> $d_v = 2.0\,ft$

$$\mathrm{V_{DL}}(\mathrm{d}_v) = 1.23(19.5-2) = 21.53kips$$

<u>C3-3.2.7 Exterior beam live load shear force at 7.5 ft</u>

$$\mathrm{V_{HS20}}(7.5') = 39.2kips$$

$$V_{HL93}(d_v) = V_{HS20} + g_{v(ext)}w(19.5-d_v) = 39.2 + 0.72(0.64)(19.5-7.5) = 44.7\,k-ft$$

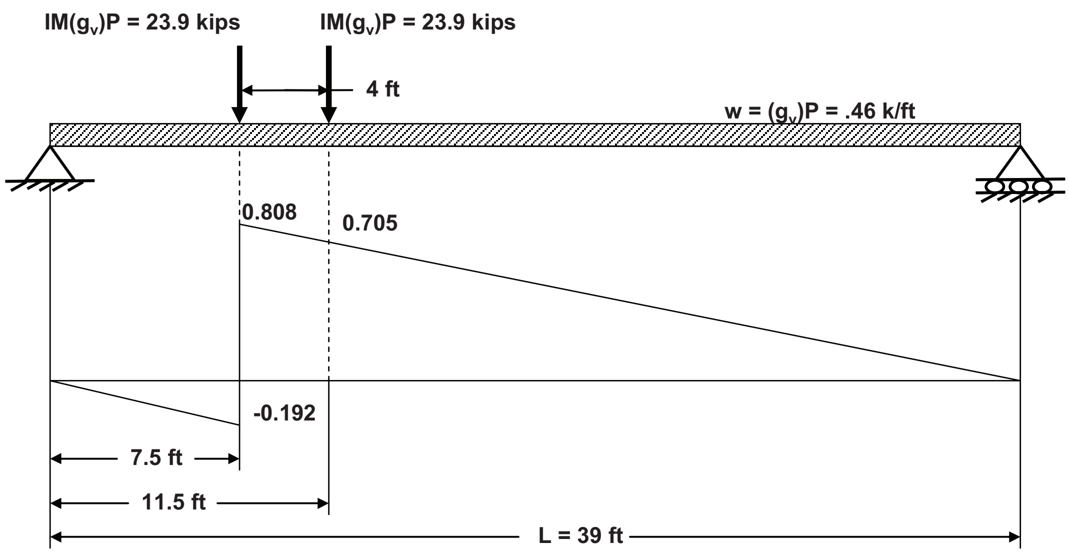


Figure C3-3.7 Exterior Beam Shear Design Tandem at 7.5 ft from support

$$DesignTandem(d_v) = 23.9(.808) + 23.9(.705) + .46(19.5-7.5) = 41.7\,kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 44.7kips$$

<u>C3-3.2.5 Exterior beam maximum live load shear force at</u> $d_v = 2.0\,ft$

$$\mathrm{V_{DL}}(7.5) = 1.23(19.5-7.5) = 14.76kips$$

C3-3.3 Load Combination

C3-3.3.1 Interior beam

Table C3-3.1 Interior Beam Load Combinations

	Moment at 19.5 ft from support	**Moment at 17.17 ft from support**
M_{LL+IM}	378.7 k-ft	396.4 k-ft
M_{DL}	245.3 k-ft	241.9 k-ft
$M_{DL}+M_{LL+IM}$	624.0 k-ft	638.3 k-ft

C3-3.3.2 Exterior Girder

Table C3-3.2 Exterior Beam Load Combinations

	Moment at 19.5 ft from support	**Moment at 17.17 ft from support**
M_{LL+IM}	395.2 k-ft	413.7 k-ft
M_{DL}	233.9 k-ft	230.6 k-ft
$M_{DL}+M_{LL+IM}$	629.1 k-ft	644.3 k-ft

C3-4 Member Capacity

C3-4.1 Interior Beam

Table C3-4.1 Interior beam capacity calculation (Computed in Section C2-2)

Flexure $f_y = 40$ ksi $f'_c = 2.5$ksi	$M_n = A_s f_y \left(d - \frac{a}{2}\right)\frac{1}{12} = 1049\ k-ft$	AASHTO MCE 2000 D.6.6.2.3
Shear at d_v	$V_n = V_c + V_s = 83.3$ kips	AASHTO MCE 2000 D.6.6.2.4.
Shear at 7.5 ft	$V_n = V_c + V_s = 71.4$ kips	

C3-4.2 Exterior Beam

Table C3-4.2 Exterior beam capacity calculation (Computed in Section C2-2)

Flexure $f_y = 40$ ksi $f'_c = 2.5$ksi	$M_n = A_s f_y \left(d - \frac{a}{2}\right)\frac{1}{12} = 961\ k-ft$	AASHTO MCE 2000 D.6.6.2.3
Shear at d_v	$V_n = V_c + V_s = 80.5$ kips	AASHTO MCE 2000 D.6.6.2.4.
Shear at 7.5 ft	$V_n = V_c + V_s = 69.0$ kips	

C3-5 Rating Calculation (LRFR)

The following factors are defined thus:

Live load factor for legal load level $\gamma_{LL} = 1.5$ (LRFR Table 6-5 based on ADTT of 458)

Resistance Factor (for shear and flexure) $\varphi = .9$

Condition Factor (related to NBI Item 59) $\phi_c = 1$

System Factor (related to structural redundancy) $\phi_s = 1$

Table C3-5.1 Load and Resistance Factor Rating (LRFR) Calculation for HL93 load at Inventory and Operating Levels and HS20 load at the Legal Level (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO LRFD Specifications (2007))

Flexure (Interior girder) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(1049) - 1.25(241.9)}{1.75(485)} =$ 0.76	27.4 tons
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(1049) - 1.25(241.9)}{1.35(485)} =$ 0.98	35.3 tons
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.5$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HS20}} = \frac{0.9(1)(1)(1049) - 1.25(241.9)}{1.5(396.4)} =$ 1.08	38.9 tons
Flexure (Exterio r girder) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(961) - 1.25(230.6)}{1.75(507)} = 0.$ 65	23.4 tons
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(961) - 1.25(230.6)}{1.35(507)} = 0.$ 84	30.2 tons
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.5$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HS20}} = \frac{0.9(1)(1)(961) - 1.25(230.6)}{1.5(413.7)} = 0.$ 93	33.5 tons

Shear (Interior Girder at d_v) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(83.3) - 1.25(22.45)}{1.75(59.9)} = 0.4$ 5	<u>16.2 tons</u>
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(83.3) - 1.25(22.45)}{1.35(59.9)} = 0.5$ 8	<u>20.9 tons</u>
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.5$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HS20}} = \frac{0.9(1)(1)(83.3) - 1.25(22.45)}{1.5(51.4)} = 0.6$ 1	<u>22.0 tons</u>
Shear (Exterio r Girder at d_v) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(80.5) - 1.25(21.53)}{1.75(57.0)} = 0.46$	16.6 tons
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(80.5) - 1.25(21.53)}{1.35(57.0)} = 0.59$	21.2 tons
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.5$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HS20}} = \frac{0.9(1)(1)(80.5) - 1.25(21.53)}{1.5(48.92)} = 0.6$ 2	22.3 tons
Shear (Interior Girder at 7.5 ft) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(71.4) - 1.25(15.48)}{1.75(49.9)} = 0.5$ 1	18.4 tons
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(71.4) - 1.25(15.48)}{1.35(49.9)} = 0.6$ 7	24.1 tons
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.5$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HS20}} = \frac{0.9(1)(1)(71.4) - 1.25(15.48)}{1.5(41.4)} = 0.7$ 2	25.9 tons
Shear (Exterio r Girder at 7.5 ft)	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(69.0) - 1.25(14.76)}{1.75(44.73)} = 0.56$	20.2 tons
	Operating Level	$RF = \frac{\phi V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(69.0) - 1.25(14.76)}{1.35(44.73)} = 0.72$	25.9 tons

$\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	$\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$		
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.5$	$RF = \dfrac{\phi\phi_c\phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.9(1)(1)(69.0) - 1.25(14.76)}{1.5(39.2)} = 0.7$ 4	26.6 tons

Rating of a Skew Cast-in-Place Reinforce Concrete T-beam Bridge (GDOT BRIDGE ID #015-0108)

Example D1:

Rating by the Allowable Stress Method (ASR) Using Load Distribution and Dynamic Allowance Factors Stipulated by AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002).

D1-1 Basic Geometry and Bridge Information

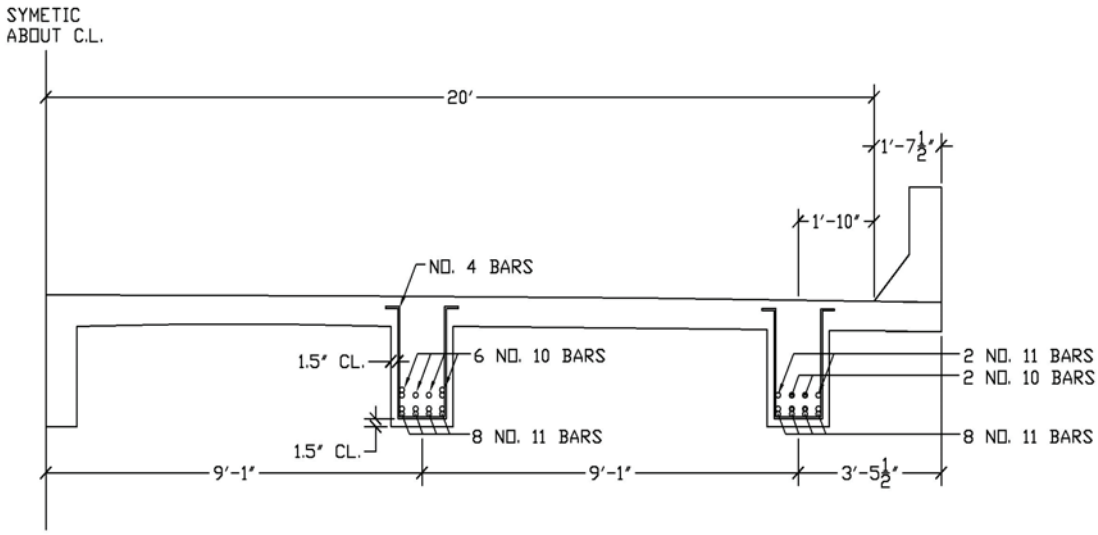


Figure D1-1.1 Bridge Cross Section at Mid-span

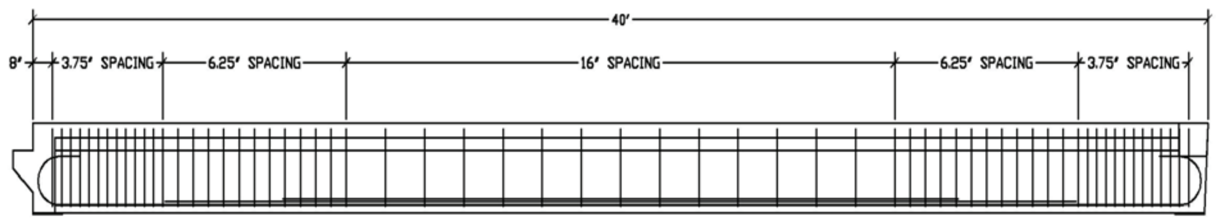


Figure D1-1.2 Girder Details

Concrete strength: $f'_c = 2.5\ \text{ksi}$

Unit weight of concrete: $wc = 0.15\ \text{kips/ft}^3$

Weight per ft of standard GDoT parapet and sidewalk: $wpr = 0.092\ \text{kips/ft}$

D1-2 LOADS

D1-2.1 Permanent Loads

As per Article 3.23.2.3.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) the dead load supported by the outside stringers or beams shall be the portion of the floor slab carried by the stringer or beam. Curbs, railings, and wearing surfaces if placed after the slab has cured, may be distributed equally to all roadway girders.

D1-2.1.1 Interior girder loads

Weight per linear foot of the reinforced concrete slab = $\left(\frac{7.75}{12}\right)(9.0833)(0.150)$ = 0.880 k/ft

Weight per linear foot of cast-in-place beams = $\left[\frac{(25.25)(18)}{144}\right](0.150)$ = 0.473 k/ft

Weight of parapet, rail and sidewalk assembly = $\frac{(2)(0.092)}{4}$ = 0.046 k/ft

Since there is no wearing surface present on the bridge, DW = 0

Total dead load for interior beam = 1.40 k/ft

D1-2.1.2 Exterior girder loads

Weight per linear foot of the reinforced concrete slab = $\left(\frac{7.75}{12}\right)(8)(0.150)$ = 0.775 k/ft

Weight per linear foot of cast-in-place beams = $\left[\frac{(25.25)(18)}{144}\right](0.150)$ = 0.473 k/ft

Weight of parapet, rail and sidewalk assembly = $\frac{(2)(0.092)}{4}$ = 0.046 k/ft

Since there is no wearing surface present on the bridge, DW = 0

Total dead load for exterior beam = 1.29 k/ft

D1-2.2 Vehicular Live Load

The design vehicular live load on the bridge consists of AASHTO HS20 truck with the spacing between the two 32-kip rear-axle loads to be varied from 14 ft to 30 ft to produce extreme force effects. The HS 20 truck is shown below.

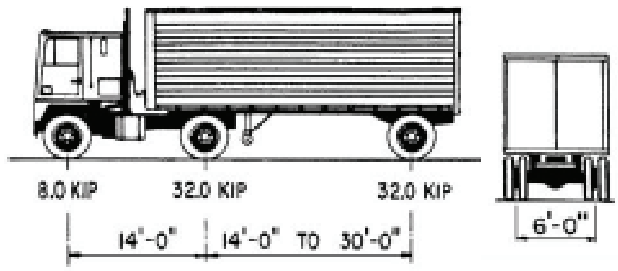


Figure A-2.1 AASHTO HS20 Truck

D1-3 Dynamic Load Allowance

Article 3.8.2.1 of the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002), the dynamic load allowance is taken as:

$$\mathrm{I} = \frac{50}{\mathrm{L}+125} = \frac{50}{39+125} = 0.305 \leq 0.3$$

Where:
I = impact factor (maximum 30 percent)
L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member

D1-4 Live Load Distribution Factors

D1-4.1 Moment distribution factors

<u>D1-4.1.2 Distribution Factor for ***moment*** in interior longitudinal beams</u>

As per Table 3.23.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002), the distribution factors for moment in interior and exterior beams are computed as:

$$DFM = \frac{S}{6} = \frac{9.0833}{6} = 1.514$$

<u>D1-4.1.2 Distribution Factor for ***moment*** in exterior longitudinal beams</u>

Using the structure shown in Figure D1.4.1 the shear distribution factor is calculated by computing the reaction at R_B, but not taken less than (Article 3.23.2.3.1 AASHTO 2002):

$$\frac{S}{4.0+0.25S} = \frac{9.0833}{4+0.25(9.0833)} = 1.45$$

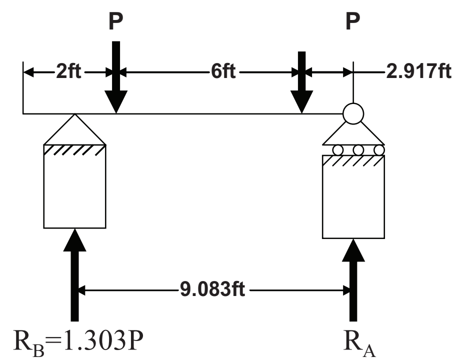


Figure D1-4.1 Interior Girder Moment Distribution Factor

$$\sum M_A = 0$$
$$R_B(9.0833) - P(8.917) - P(2.917) = 0.$$
$$R_B = 1.303P$$

Thus, the distribution factor for moment in an exterior beam is:

$$DFM = 1.45$$

D1-4.2 Shear distribution factors

D1-4.2.1 Distribution Factor for ***shear*** in interior longitudinal beams

Article 3.23.1.2 of the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) stipulates that:

> "Lateral distribution of the wheel loads at ends of the beams or stringer shall be that produced by assuming the floors to act as a simple span between stringers or beams. For wheels or axels in other positions on the span, the distribution for shear shall be determined by the method prescribed for moment."

Therefore, by modeling the deck as a series of rigid simply supported beams between the girders, as shown in Figures D1.4.2, the shear distribution factor (DFS) is computing by calculating the reaction R_B.

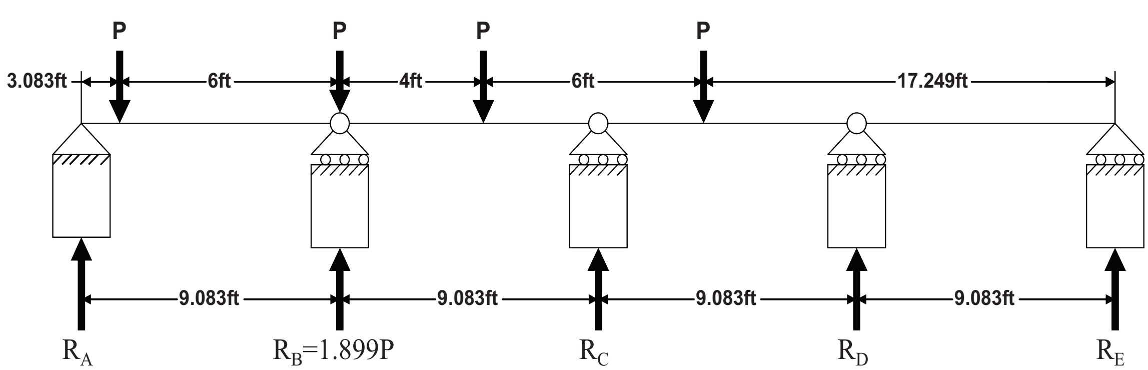


Figure D1-4.2 Interior Girder Shear Distribution Factor

DFS = 1.899 for wheel loads at beam ends

D1-4.2.2 Distribution Factor for ***shear*** in exterior longitudinal beams

Using the structure shown in Figure D1-4.1 the shear distribution factor is calculated by computing the reaction at R_B.

$$DFS = 1.303$$

D1-4.2 Summary of distribution factors

Table D1-4.1 Distribution Factors

Action	Interior Beam	Exterior Beam
Bending Moment	1.514	1.45
Shear	1.889	1.303

D1-5 Flexural Analysis

D1-5.1 Maximum live load bending moment

A rudimentary structural analysis of a simply supported beam subjected to a vehicular load having two rear axles and one front axle as shown in Figure D1.5.1 shows that the absolute maximum moment occurs under the middle axle when such an axle is positioned at a distance of 2.33 ft to the left of the beam centerline.

By applying the dynamic allowance factor and the distribution factor for moment of interior beams, we can then compute the maximum live load under the following loads:

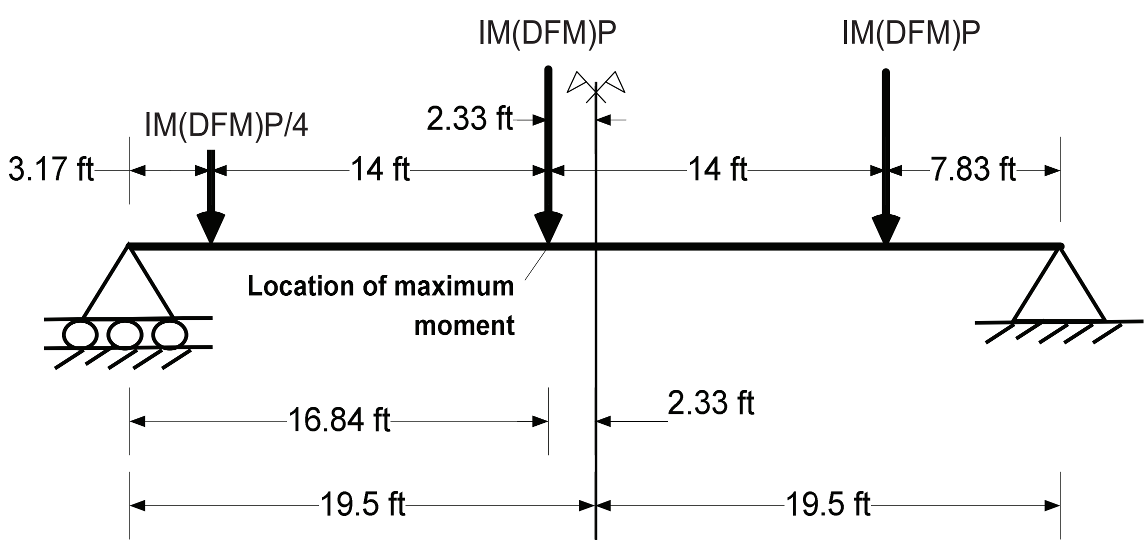


Figure D1-5.1 Max Live Load Moment

D1-5.2.1 Maximum live load moment for an interior beam

With P = 16 kips, $DFM = 1.514$, and IM =1.30, M_{LL+IM} can be computed at the location of the maximum live load moment (at 17.17 ft from the left support) to be:

$$M_{LL+IM} = 425\ k - ft$$

Live load moment (at midspan):

$$M_{LL+IM} = 406\ k - ft$$

D1-5.2.2 Maximum live load moment for an exterior beam

With P = 16 kips, $DFM = 1.45$, and IM =1.30, M_{LL+IM} can be computed at the location of the maximum live load moment (at 17.17 ft from the left support) to be:

$$M_{LL+IM} = 407\ k - ft$$

Live load moment (at midspan):

$$M_{LL+IM} = 389\ k - ft$$

D1-5.2 Maximum dead load moment

D1-5.2.1 Maximum dead load moment for an interior beam

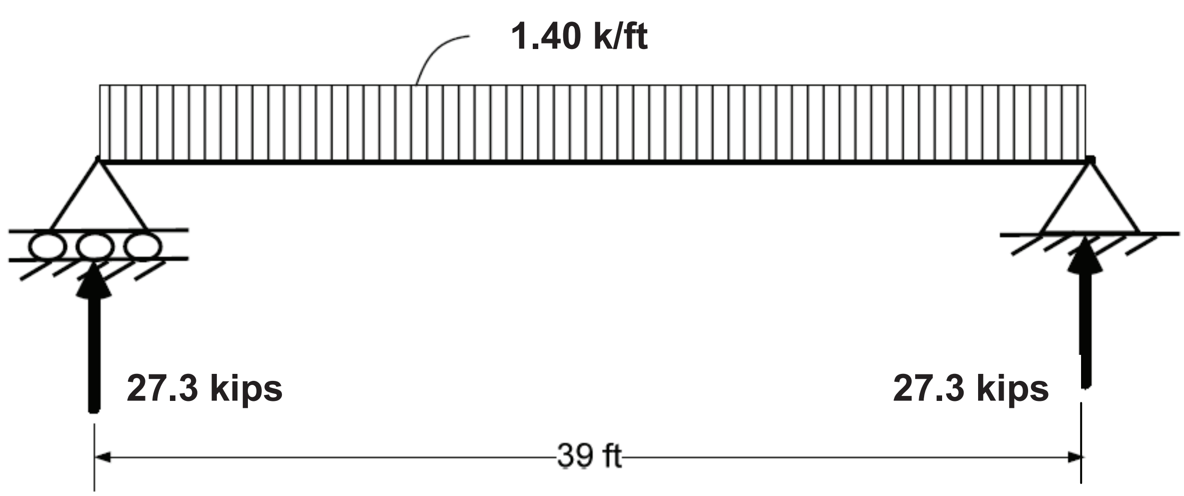


Figure D1-5.3 Interior Girder Dead Load Moment

The dead load moment at the location where the live load produces the maximum effect is

$$M_D = 27.3(17.17) - 1.40\frac{(17.17)^2}{2} = 262\ k - ft$$

The dead load moment at midspan is:

$$M_{D(m)} = 1.29\frac{(39)^2}{8} = 266k - ft$$

D1-5.2.2 Maximum dead load moment for an exterior beam

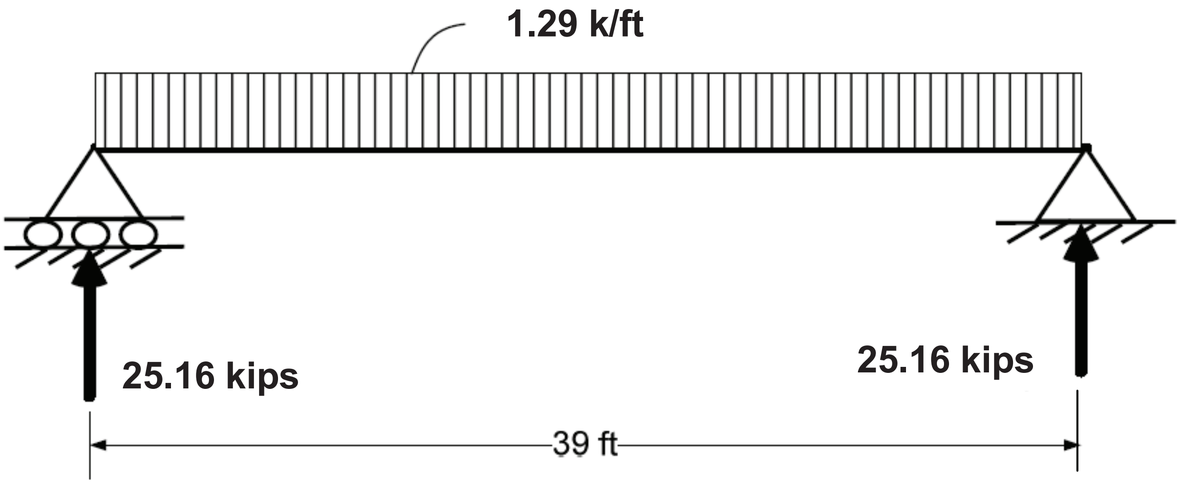


Figure D1-5.3 Exterior Girder Dead Load Moment

The dead load moment at the location where the live load produces the maximum effect is

$$M_D = 25.16(17.17) - 1.29\frac{(17.17)^2}{2} = 242\ k-ft$$

The dead load moment at midspan is:

$$M_{D(m)} = 1.29\frac{(39)^2}{8} = 245k-ft$$

D1-6 Shear Force Analysis

Based on the shear reinforcement details of the bridge girders, it is necessary to perform shear rating calculations of the bridge at two locations along both the interior and exterior girders. These two critical locations as stipulated in Article 8.15.5.1.4 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) are:

- At d/2 from either support
- At 7.5' from either support at which the stirrup spacing changes from 12" to 18"

D1-6.1 Interior Beam Shear Analysis

D1-6.1.1 Interior beam maximum live load shear force at d/2

The first critical location for the interior girder is located at $\frac{d}{2} = \frac{2.1}{2} =$ 1.05 ft from the support. Article 8.16.6.1.2 of the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) states that the shear at any point between d = 2.1 ft and the support should be designed for the shear computed at d = 2.1 ft plus the full weight of any concentrated load between d = 2.1 ft and the support. Therefore the maximum live load shear force is computed at the location of interest $\frac{d}{2}=\frac{2.1}{2}=$ 1.05 ft from the support as per the shear influence line shown in Figure A1-6.1.

At $\frac{d}{2}=\frac{2.1}{2}=$ 1.05 ft; the length of the portion of the span that is loaded to produce the maximum stress in the member is L = 39-1.05=37.95 ft thus:

$$IM = 1 + I = 1 + \frac{50}{(37.95)+125} = 1.31 \le 1.30$$

Use IM = 1.30

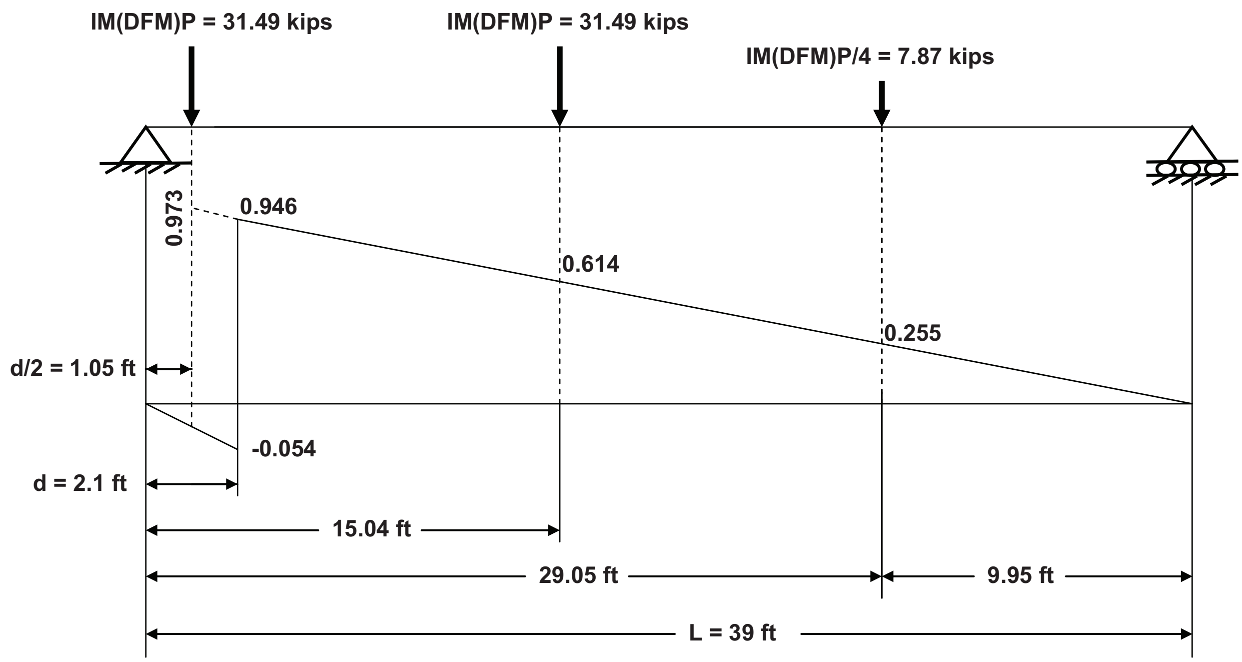


Figure D1-6.1 Shear Influence Line Diagram

$$\mathrm{V}_{\mathrm{HS20}}(1.05') = 31.49 + (31.49)(0.614) + (7.87)(0.255) = 52.8 kips$$

<u>A1-6.1.2 Interior beam dead load shear force at d/2</u>

As stated in section A1-6.1.1 since the dead load is a distributed load the shear at $\frac{d}{2}=\frac{2.1}{2}=$ 1.05 ft is taken as the shear computed at a distance d = 2.1 ft from the support.

$$V_D(1.05') = 1.40(19.5 - 1.05) = 25.8 kips$$
$$V_D(2.1') = 1.40(19.5 - 2.1) = 24.4 kips$$

D1-6.1.3 Interior beam live load shear force at 4.42 ft from the support

An influence line analysis shown in Figure D1-6.2 is used to compute the shear force at 4.42 ft from the support where the stirrup spacing changes from 3.75" to 6.25".

At 4.42ft from the support; the length of the portion of the span that is loaded to produce the maximum stress in the member is L = 39-4.42=34.58 ft thus:

$$IM = 1 + I = 1 + \frac{50}{(34.58)+125} = 1.31 \leq 1.30$$

Use IM = 1.3

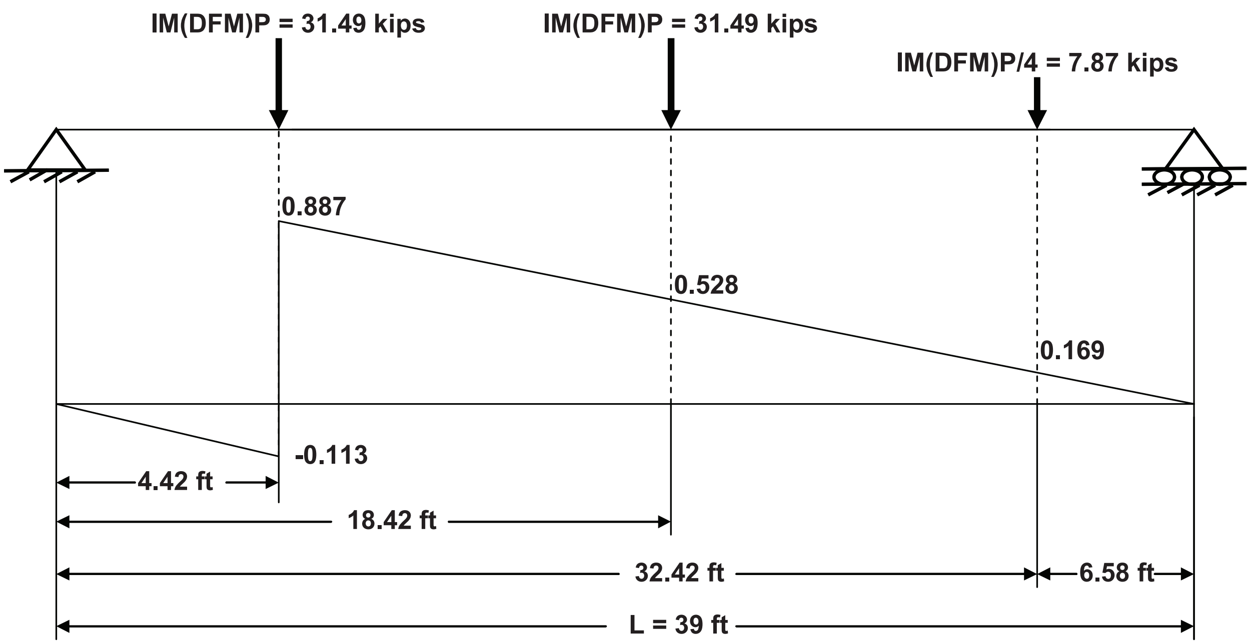


Figure D1-6.2 Shear Influence Line Diagram

$$V_{HS20}(4.42') = (31.49)(0.887) + (31.49)(0.528) + (7.87)(0.169) = 45.9 kips$$

D1-6.1.4 Interior beam dead load shear force at 4.42 ft from the support

$$V_D(4.42') = 1.40(19.5 - 4.42) = 21.1 kips$$

D1-6.1.5 Interior beam live load shear force at 10.67 ft from the support

An influence line analysis shown in Figure D1-6.2 is used to compute the shear force at 10.67 ft from the support where the stirrup spacing changes from 6.25" to 16".

At 10.67 ft from the support; the length of the portion of the span that is loaded to produce the maximum stress in the member is L = 39-10.67=28.33 ft thus:

$$IM = 1 + I = 1 + \frac{50}{(28.33)+125} = 1.33 \leq 1.30$$

Use IM = 1.3

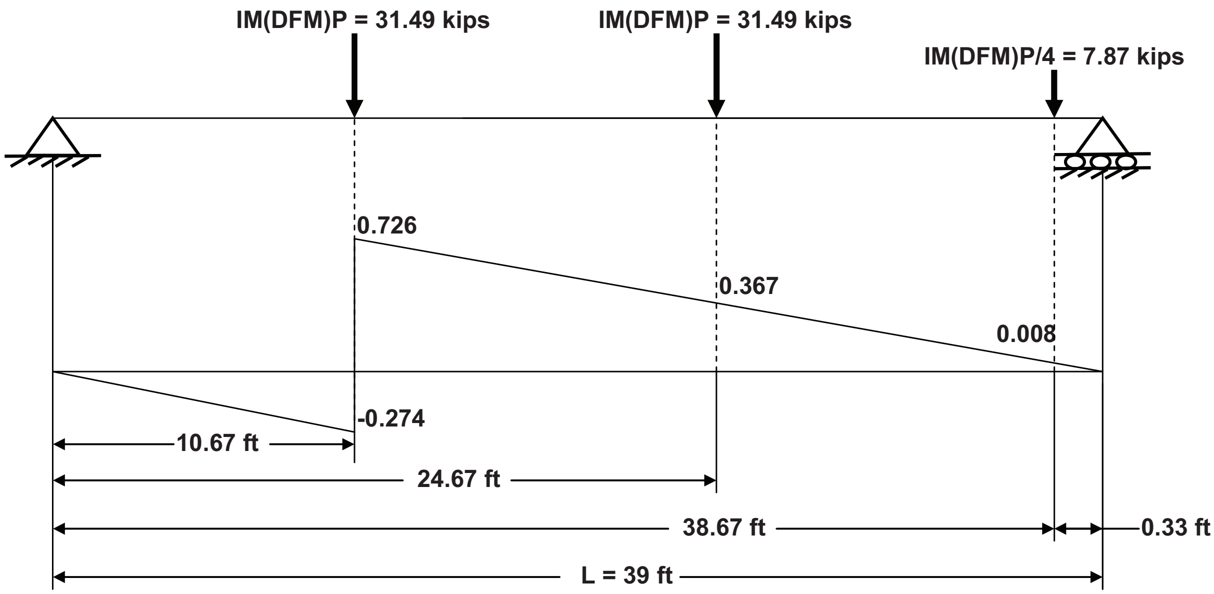


Figure D1-6.2 Shear Influence Line Diagram

$$\mathrm{V}_{\mathrm{HS20}}\left(10.67'\right) = \left(31.49\right)\left(0.726\right) + \left(31.49\right)\left(0.367\right) + \left(7.87\right)\left(0.008\right) = 34.5 kips$$

D1-6.1.6 Interior beam dead load shear force at 10.67 ft from the support

$$V_D\left(10.67'\right) = 1.40\left(19.5 - 10.67\right) = 12.4 kips$$

D1-6.2 Exterior Beam Shear Analysis

D1-6.2.1 Exterior beam maximum live load shear force

$$\frac{d}{2} = \frac{2.1}{2} = 1.05 \text{ ft}$$

L = 39-1.05 = 37.95 ft thus:

$$IM = 1 + I = 1 + \frac{50}{(37.95)+125} = 1.31 \leq 1.30$$

Use IM = 1.3

$$\mathrm{V}_{\mathrm{HS20}}\left(1.05'\right) = 50.6 kips$$

D1-6.2.2 Exterior beam dead load shear force at d/2

As stated in section D1-6.1.1 since the dead load is a distributed load the shear at $\frac{d}{2}=\frac{2.1}{2}=$ 1.05 ft is taken as the shear computed at a distance d = 2.1 ft from the support.

$$V_D(1.05')=1.29(19.5-1.05)=23.8kips$$
$$V_D(2.1')=1.29(19.5-2.1)=22.4kips$$

D1-6.2.3 Exterior beam live load shear force at 4.42 ft from the support

The exterior girder shear due to live load at 4.42 ft from the support is computed in the same manner presented in Section D1-6.1.3.

$$\mathrm{V}_{\mathrm{HS20}}(4.42')=44.0kips$$

D1-6.2.4 Exterior beam dead load shear force at 4.42 ft from the support

The exterior girder shear due to dead load at 4.42 ft from the support is computed in the same manner presented in Section D1-6.1.4.

$$V_D(4.42')=1.29(19.5-4.42)=19.45kips$$

D1-6.2.5 Exterior beam live load shear force at 10.67 ft from the support

The exterior girder shear due to live load at 10.67 ft from the support is computed in the same manner presented in Section D1-6.1.3.

$$\mathrm{V}_{\mathrm{HS20}}(10.67')=33.0kips$$

D1-6.2.6 Exterior beam dead load shear force at 10.67 ft from the support

The exterior girder shear due to dead load at 10.67 ft from the support is computed in the same manner presented in Section D1-6.1.4.

$$V_D(10.67')=1.29(19.5-10.67)=11.4kips$$

D1-7 Load Combination

D1-7.1 Interior Girder

Table D1-7.1 shows that the governing moment loading case occurs at the maximum live load moment location (17.17 ft from the support).

Table D1-7.1 Interior Girder Load Combinations

	Moment at 19.5 ft from support	**Moment at 17.17 ft from support**
M_{LL+IM}	406 k-ft	425 k-ft
M_{DL}	266 k-ft	262 k-ft
$M_{DL}+ M_{LL+IM}$	672 k-ft	678 k-ft

D1-7.2 Exterior Girder

Table D1-7.2 shows that the governing moment loading case occurs at the maximum live load moment location (17.17 ft from the support).

Table D1-7.2 Exterior Girder Load Combinations

	Moment at 19.5 ft from support	**Moment at 17.17 ft from support**
M_{LL+IM}	389 k-ft	407 k-ft
M_{DL}	245 k-ft	242 k-ft
$M_{DL}+ M_{LL+IM}$	634 k-ft	649 k-ft

D1-8 Member Capacity

D1-8.1 Interior Girders

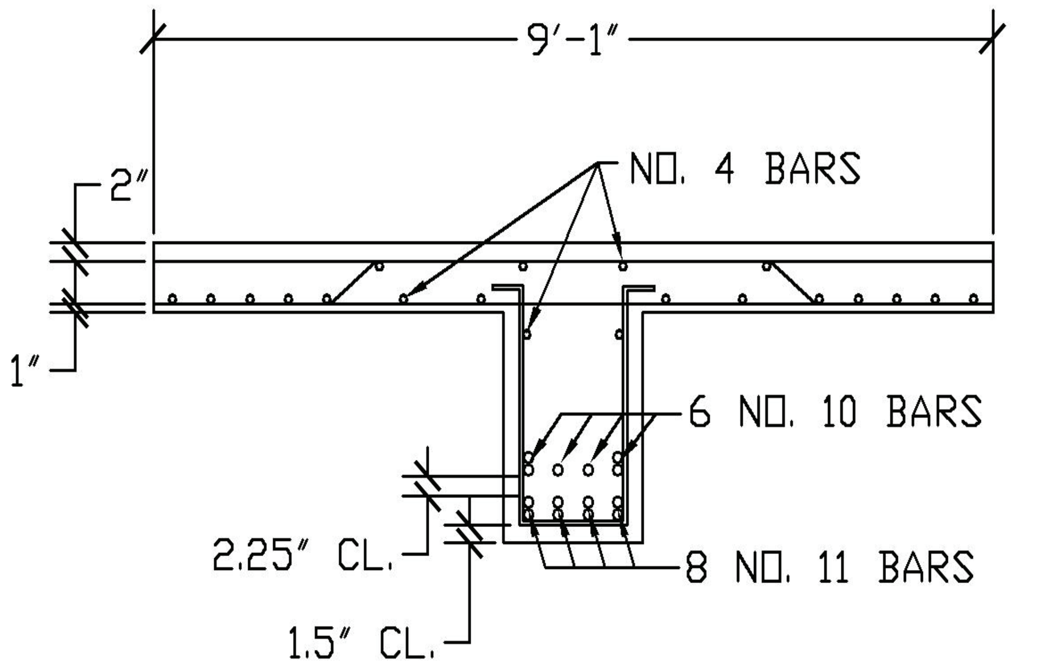


Figure D1-8.1 Member Dimensions

Effective flange width

Article 8.10.1.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002): The effective flange width is limited to the smallest of one-forth the span length, six times the slab thickness or half the distance to the adjacent girders, as per. Thus, $b_e = 109in$

Distance from the extreme compression fiber to:

centroid of the bottom layer of bundled tension reinforcement $d = 29.59in$
centroid of the second layer of bundled tension reinforcement $d = 24.87in$

Concrete strength $f_c^{'} = 3.5ksi$

Steel reinforcement yield strength $f_y = 40ksi$

Stirrup area $\mathrm{A_v} = 2A_{\#4} = 0.4in^2$

Stirrup spacing $S = 3.75in$

Shear width $\mathrm{b_v} = 18in$

Angle of inclination of Diagonal Compression Stress $\theta = \pi / 4$

Table D1-8.2 Interior girder capacity calculation

Flexure	**Inventory Level** $f_y = 20$ ksi $f'_c = 1.2$ksi	$\beta = 0.85$ for $f'_c \le 4$ksi $c = \dfrac{A_s f_y}{.85 f'_c \beta b_e} = \dfrac{12.48(20)+7.62(20)}{.85(1.2).85(109)} = 4.25$ in $< t_s$ $a = \beta c = .85(4.25) = 3.61$ in $M_{INV} = 870\ k - ft$	AASHTO MCE 2000 D.6.6.2.3
	Operating Level fy = 28 ksi f'c = 1.9 ksi	$c = \dfrac{A_s f_y}{.85 f'_c \beta b_e} = \dfrac{12.48(28)+7.62(28)}{.85(1.5).85(109)} = 4.76$ in $< t_s$ $a = c\beta = 4.05$ in $M_{OPR} = 1,209$ kip-ft	
Shear (d/2) $S = 3.75$ in	**Inventory Level** $f_y = 20$ ksi $f'_c = 3.5$ ksi	$V_c = 0.95\sqrt{f'_c}\, b_v d = 30$ kips $V_s = \dfrac{A_v f_y d\ \cot(\theta)}{S} = 63$ kips $V_{INV} = V_c + V_s = 93$ kips	AASHTO MCE 2000 D.6.6.2.4.
	Operating Level $f_y = 28$ ksi $f'_c = 3.5$ ksi	$V_c = 1.3\sqrt{f'_c}\, b_v d = 41$ kips $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 88$ kips $V_{OPR} = V_c + V_s = 129$ kips	
Shear (4.42) $S = 6.25$ in	**Inventory Level** $f_y = 20$ ksi $f'_c = 3.5$ ksi	$V_c = 0.95\sqrt{f'_c}\, b_v d = 30$ kips $V_s = \dfrac{A_v f_y d\ \cot(\theta)}{S} = 38$ kips $V_{INV} = V_c + V_s = 68$ kips	AASHTO MCE 2000 D.6.6.2.4.
	Operating Level $f_y = 28$ ksi $f'_c = 3.5$ ksi	$V_c = 1.3\sqrt{f'_c}\, b_v d = 41$ kips $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 53$ kips $V_{OPR} = V_c + V_s = 94$ kips	

Shear (10.67) $S = 16$ in	**Inventory Level** f_y = 20 ksi f'_c = 3.5 ksi	$V_c = 0.95\sqrt{f'_c}\,b_v d \;= 30$ kips $V_s = \dfrac{A_v f_y d\ \cot(\theta)}{S} = 15$ kips $V_{INV} = V_c + V_s = 45$ kips	AASHTO MCE 2000 D.6.6.2.4.
	Operating Level f_y = 28 ksi f'_c = 3.5 ksi	$V_c = 1.3\sqrt{f'_c}\,b_v d = 41$ kips $V_s = \dfrac{A_v f_y d\cot(\theta)}{S} = 21$ kips $V_{OPR} = V_c + V_s = 62$ kips	

D1-8.2 Exterior Girders

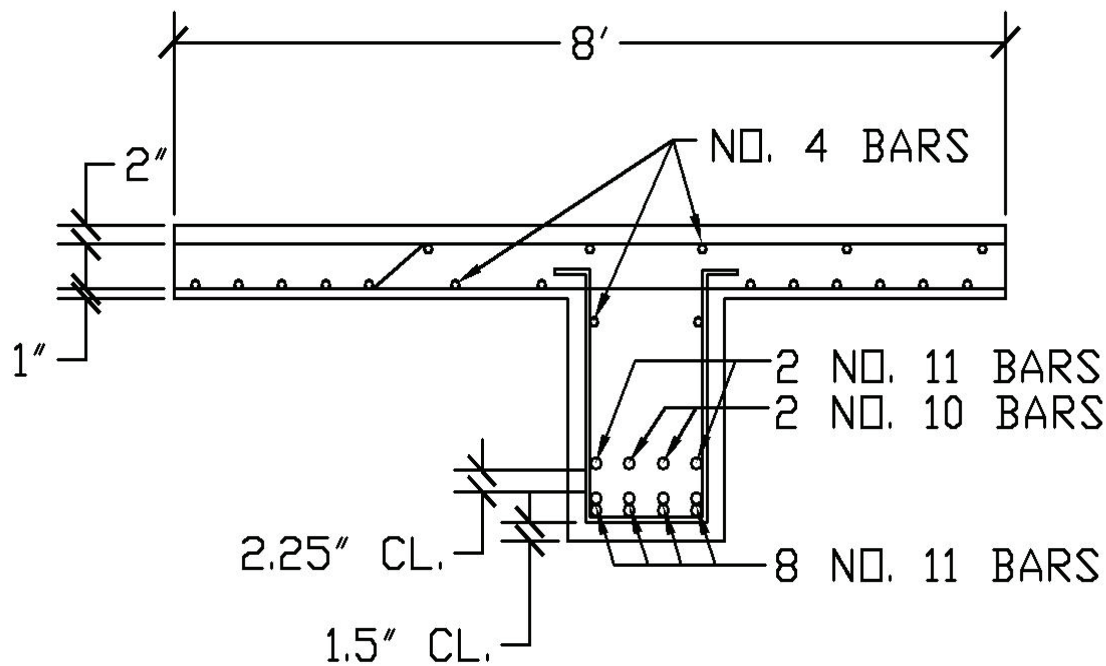


Figure D1-8.2 Member Dimensions

Effective flange width

Article 8.10.1.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002): The effective flange width is limited to the smallest of one-forth the span length, six times the slab thickness or half the distance to the adjacent girders, as per. Thus, $b_e = 96in$

Distance from the extreme compression fiber to:

centroid of the bottom layer of bundled tension reinforcement $d = 29.59in$

centroid of the second layer of bundled tension reinforcement $d = 25.22in$

Flexural steel area $A_s = 12.48in$

Concrete strength $f_c^{'} = 2.5ksi$

Steel reinforcement yield strength $f_y = 40ksi$

Stirrup area $\mathrm{A_v} = 2A_{\#4} = 0.4in^2$

Stirrup spacing $S = 3.75in$

Shear width $\mathrm{b_v} = 18in$

Angle of inclination of Diagonal Compression Stress $\theta = \pi / 4$

Table D1-8.3 Exterior girder capacity calculation

Flexure	**Inventory Level** f_y = 20 ksi f'$_c$ = 1.2ksi	$\beta = 0.85$ for $f'_c \le 4\mathrm{ksi}$ $c = \dfrac{A_s f_y}{.85 f'_c \beta b_e} = \dfrac{12.48(20)+5.66(20)}{.85(1.2).85(96)} = 4.36$ in $< t_s$ $a = \beta c = .85(4.36) = 3.71$ in $M_{INV} = 797$ kip-ft	AASHTO MCE 2000 D.6.6.2.3
	Operating Level fy = 28 ksi f'c = 1.9 ksi	$c = \dfrac{A_s f_y}{.85 f'_c \beta b_e} = \dfrac{12.48(28)+5.66(28)}{.85(1.9).85(96)} = 3.85$ in $< t_s$ $a = c\beta = 3.27$ in $M_{OPR} = 1{,}126$ kip-ft	
Shear (d/2) $S = 3.75$ in	**Inventory Level** f_y = 20 ksi f'$_c$ = 3.5 ksi	$V_c = 0.95\sqrt{f'_c}\, b_v d = 30$ kips $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 63$ kips $V_{INV} = V_c + V_s = 93$ kips	AASHTO MCE 2000 D.6.6.2.4.
	Operating Level f_y = 28 ksi f'$_c$ = 3.5 ksi	$V_c = 1.3\sqrt{f'_c}\, b_v d = 41$ kips $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 88$ kips $V_{OPR} = V_c + V_s = 129$ kips	

<table>
<tr><td rowspan="2">Shear (4.42)

$S = 6.25$ in</td><td>Inventory Level

f_y = 20 ksi
f'$_c$ = 3.5 ksi</td><td>$V_c = 0.95\sqrt{f'_c}\, b_v d = 30$ kips
$V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 38$ kips
$V_{INV} = V_c + V_s = 68$ kips</td><td rowspan="2">AASHTO MCE 2000 D.6.6.2.4.</td></tr>
<tr><td>Operating Level

f_y = 28 ksi
f'$_c$ = 3.5 ksi</td><td>$V_c = 1.3\sqrt{f'_c}\, b_v d = 41$ kips
$V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 53$ kips
$V_{OPR} = V_c + V_s = 94$ kips</td></tr>
<tr><td rowspan="2">Shear (10.67)

$S = 16$ in</td><td>Inventory Level

f_y = 20 ksi
f'$_c$ = 3.5 ksi</td><td>$V_c = 0.95\sqrt{f'_c}\, b_v d = 30$ kips
$V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 15$ kips
$V_{INV} = V_c + V_s = 45$ kips</td><td rowspan="2">AASHTO MCE 2000 D.6.6.2.4.</td></tr>
<tr><td>Operating Level

f_y = 28 ksi
f'$_c$ = 3.5 ksi</td><td>$V_c = 1.3\sqrt{f'_c}\, b_v d = 41$ kips
$V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = 21$ kips
$V_{OPR} = V_c + V_s = 62$ kips</td></tr>
</table>

D1-9 Rating Calculation (ASR)

Table D1-9.1 Allowable Stress Rating (ASR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002))

Flexure (Interior girder)	**Inventory Level**	$RF = \frac{M_{INV} - M_D}{M_{LL+IM}} = \frac{870-262}{425} = 1.43$	51.5 tons
	Operating Level	$RF = \frac{M_{OPR} - M_D}{M_{LL+IM}} = \frac{1209-262}{425} = 2.23$	80.3 tons
Flexure (Exterior girder)	**Inventory Level**	$RF = \frac{M_{INV} - M_D}{M_{LL+IM}} = \frac{797-242}{407} = 1.36$	49.0 tons
	Operating Level	$RF = \frac{M_{OPR} - M_D}{M_{LL+IM}} = \frac{1126-242}{407} = 2.17$	78.1 tons
Shear at d/2 (Interior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{93-24.4}{52.8} = 1.3$	46.8 tons
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{129-24.4}{52.8} = 1.98$	71.3 tons
Shear at d/2 (Exterior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{93-22.4}{50.6} = 1.40$	50.4 tons
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{129-22.4}{50.6} = 2.11$	76.0 tons
Shear at 4.42 ft (Interior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{68-21.1}{45.9} = 1.02$	36.7 tons
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{94-21.1}{45.9} = 1.59$	57.2 tons
Shear at 4.42 ft (Exterior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{68-19.45}{44.0} = 1.10$	39.6 tons
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{94-19.45}{44.0} = 1.69$	60.8 tons
Shear at 10.67 ft (Interior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{45-12.4}{34.5} = 0.94$	<u>33.8 tons</u>
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{62-12.4}{34.5} = 1.44$	<u>51.8 tons</u>
Shear at 10.67 ft (Exterior Girder)	**Inventory Level**	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{45-11.4}{33.0} = 1.02$	36.7 tons
	Operating Level	$RF = \frac{V_{INV} - V_{DL}}{V_{HS20}} = \frac{62-11.4}{33.0} = 1.53$	55.1 tons

Example D2:

Rating by the Load Factor Method (LFR) Using Load Distribution and Dynamic Allowance Factors Stipulated by the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002).

D2-1 Analysis

D2-1.1 Maximum live load Bending Moment

D2-1.1.1 Interior beam

Table D2-1.1 Interior Beam Load Combinations (computed in Section D2-6)

	Moment at 17.17 ft from support
M_{LL+IM}	425 k-ft
M_{DL}	262 k-ft

D2-1.1.2 Exterior beam

Table D2-1.2 Exterior Beam Load Combinations (computed in Section D2-6)

	Moment at 17.17 ft from support
M_{LL+IM}	407 k-ft
M_{DL}	242 k-ft

D2-1.2 Shear Analysis

D2-1.2.1 Interior beam

Table D2-1.3 Interior beam shear forces (Computed in Section D2-6)

Location (from supports)	V_{LL+IM}	V_{DL}
d/2 = 1.05	52.8 kips	24.4 kips
4.42 ft	45.9 kips	21.1 kips
10.67 ft	34.5 kips	12.4 kips

D2-1.2.2 Exterior beam

Table D2-1.4 Exterior beam shear forces (Computed in Section D2-6)

Location (from supports)	V_{LL+IM}	V_{DL}
d/2 = 1.05ft	50.6 kips	22.4 kips
7.5 ft	44.0 kips	19.5 kips
10.67 ft	33.0 kips	11.4 kips

D2-2 Member Capacity

D2-2.1 Interior Beam

Table D2-2.1 Interior beam capacity calculation

Flexure $f_y = 40$ ksi $f'_c = 3.5$ksi	β=0.85 for $f'_c \le 4$ksi $c = \dfrac{A_s f_y}{.85 f'_c \beta b_e} = \dfrac{12.48(40)+7.62(40)}{.85(3.5).85(109)} = 2.917 \text{ in} < t_s$ $a = \beta c = .85(2.917) = 2.479 \text{ in}$ $M_n = 12.48(40)\left(29.59 - \dfrac{2.479}{2}\right) + 7.62(40)\left(24.87 - \dfrac{2.479}{2}\right)$ $= 1,780\ k - ft$	AASHTO MCE 2000 D.6.6.2.3
Shear (d/2)	$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d$ $= 0.0316(2)\sqrt{3.5}(18)29.59 = 63.0 kips$ $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = \dfrac{0.4(40)29.59\cot\left(\frac{\pi}{4}\right)}{3.75} = 126 kips$ V_n=V_c+V_s= 189 kips	AASHTO MCE 2000 D.6.6.2.4.
Shear (4.42 ft)	$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d$ $= 0.0316(2)\sqrt{3.5}(18)29.59 = 63.0 kips$ $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = \dfrac{0.4(40)(29.59)\cot\left(\frac{\pi}{4}\right)}{6.25} = 75.8 \text{ kips}$ V_n=V_c+V_s= 138.8 kips	AASHTO MCE 2000 D.6.6.2.4.
Shear (10.67 ft)	$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d$ $= 0.0316(2)\sqrt{3.5}(18)29.59 = 63.0 kips$ $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = \dfrac{0.4(40)(29.59)\cot\left(\frac{\pi}{4}\right)}{16} = 29.6 \text{ kips}$ V_n=V_c+V_s= 92.6 kips	AASHTO MCE 2000 D.6.6.2.4.

D2-2.2 Exterior Beam

Table D2-2.2 Exterior beam capacity calculation

Flexure $f_y = 40$ ksi $f'_c = 3.5$ksi	$\beta = 0.85$ for $f'_c \le 4$ksi $c = \dfrac{A_s f_y}{.85 f'_c \beta b_e} = \dfrac{12.48(40)+5.66(40)}{.85(3.5).85(96)} = 2.989$ in $< t_s$ $a = \beta c = .85(2.989) = 2.54$ in $M_n = 12.48(40)\left(29.59 - \dfrac{2.54}{2}\right) + 5.66(40)\left(25.22 - \dfrac{2.54}{2}\right)$ $= 1,630\ k-ft$	AASHTO MCE 2000 D.6.6.2.3
Shear (d/2)	$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d$ $= 0.0316(2)\sqrt{3.5}(18)29.59 = 63.0 kips$ $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = \dfrac{0.4(40)29.59\cot\left(\dfrac{\pi}{4}\right)}{3.75} = 126 kips$ $V_n = V_c + V_s =$ 189 kips	AASHTO MCE 2000 D.6.6.2.4.
Shear (4.42 ft)	$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d$ $= 0.0316(2)\sqrt{3.5}(18)29.59 = 63.0 kips$ $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = \dfrac{0.4(40)(29.59)\cot\left(\dfrac{\pi}{4}\right)}{6.25} = 75.8$ kips $V_n = V_c + V_s =$ 138.8 kips	AASHTO MCE 2000 D.6.6.2.4.
Shear (10.67 ft)	$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d$ $= 0.0316(2)\sqrt{3.5}(18)29.59 = 63.0 kips$ $V_s = \dfrac{A_v f_y d \cot(\theta)}{S} = \dfrac{0.4(40)(29.59)\cot\left(\dfrac{\pi}{4}\right)}{16} = 29.6$ kips $V_n = V_c + V_s =$ 92.6 kips	AASHTO MCE 2000 D.6.6.2.4.

D2-3 Rating Calculation (LFR)

Table D2-3.1 Load Factor Rating (LFR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002))

Flexure (Interior girder) $\phi = 0.9$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\varphi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \dfrac{0.9(1,780) - 1.3(262)}{2.17(425)} = 1.3$ 7	49.3 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\varphi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \dfrac{0.9(1,780) - 1.3(262)}{1.3(425)} = 2.2$ 8	82.1 tons
Flexure (Exterior girder) $\phi = 0.9$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \dfrac{0.9(1,630) - 1.3(242)}{2.17(407)} = 1.3$ 0	46.8 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \dfrac{0.9(1,630) - 1.3(242)}{1.3(407)} = 2.1$ 8	78.5 tons
Shear (Interior Girder) (d/2) $\phi = 0.85$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(189) - 1.3(24.4)}{2.17(52.8)} = 1.13$	40.7 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(189) - 1.3(24.4)}{1.3(52.8)} = 1.88$	67.7 tons
Shear (Exterior Girder) (d/2) $\phi = 0.85$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(189) - 1.3(22.4)}{2.17(50.6)} = 1.20$	43.2 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(189) - 1.3(22.4)}{1.3(50.6)} = 2.00$	72.0 tons

Shear (Interior Girder) (4.42 ft) $\phi = 0.85$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(138.8) - 1.3(21.1)}{2.17(45.9)} = 0.91$	32.8 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(138.8) - 1.3(21.1)}{1.3(45.9)} = 1.52$	54.7 tons
Shear (Exterior Girder) (4.42 ft) $\phi = 0.85$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(138.8) - 1.3(19.5)}{2.17(44.0)} = 0.97$	34.9 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(138.8) - 1.3(19.5)}{1.3(44.0)} = 1.62$	58.3 tons
Shear (Interior Girder) (10.67 ft) $\phi = 0.85$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(92.6) - 1.3(12.4)}{2.17(34.5)} = 0.84$	<u>30.2 tons</u>
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(92.6) - 1.3(12.4)}{1.3(34.5)} = 1.40$	<u>50.4 tons</u>
Shear (Exterior Girder) (10.67 ft) $\phi = 0.85$	**Inventory Level** $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(92.6) - 1.3(11.4)}{2.17(33.0)} = 0.89$	32.0 tons
	Operating Level $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$	$RF = \dfrac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{0.85(92.6) - 1.3(11.4)}{1.3(33.0)} = 1.49$	53.6 tons

Example D3:

Rating by the Load and Resistance Factor Method (LRFR) Using Load Distribution and Dynamic Allowance Factors Stipulated by the *AASHTO LRFD Specifications* (2007).

D3-1 Dynamic Load Allowance

From Table 3.6.2.1 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007), the dynamic load allowance is taken as 33%. Thus, the dynamic load factor to be applied to the static load is:

$$\left(1+\frac{IM}{100}\right)=1.33$$

D3-2 Live Load Distribution Factors

D3-2.1 Interior Beams

D3-2.1.1 Distribution Factor for ***moment*** in interior longitudinal beams

As per Table 4.6.2.2.2b-1 of AASHTO LRFD Bridge Design Specifications (AASHTO, 2007), the distribution factor for moment in interior beams, g_m , is specified as follows

When one lane is loaded:

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

When two or more lane are loaded:

$$g_{m2}=0.075+\left(\frac{s}{9.5}\right)^{0.6}+\left(\frac{s}{L}\right)^{0.2}\left(\frac{k_g}{12Lt_s^3}\right)^{0.1}$$

The distribution factor for moment in exterior beams, g_m, is specified as follows

In the case of one lane loaded: g_m is computed by the lever rule

In the case of two or more lane loaded:
The longitudinal stiffness parameter:

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

In which $n=\frac{E_B}{E_D}$ where

E_B = modulus of elasticity of the beam material

E_D = modulus of elasticity of the deck material

e_g = the distance between the centers of gravity of the beams and deck

I = moment of inertia of the beam

A = area of beam

$$n=\frac{E_B}{E_D}=1$$

$$e_g = \frac{25.25}{2} + \frac{7.75}{2} = 16.5\ in$$

$$I = \frac{(18)(25.25)^3}{12} = 24{,}148\ in^4$$
$$A = (18)(25.25) = 455\ in^2$$

$$K_g = n\left(I + Ae_g^2\right) = 1\left[24{,}148 + (455)(16.5)^2\right] = 148{,}022\ in^4$$

With one lane loaded:

$$g_{m1} = 0.06 + \left(\frac{9.083}{14}\right)^{0.4}\left(\frac{9.083}{39}\right)^{0.3}\left(\frac{148{,}022}{12(39)(7.75)^3}\right)^{0.1} = 0.06 + (0.841)(0.646)(0.962) = 0.58$$

With two or more lane loaded:

$$g_{m2} = 0.075 + \left(\frac{9.083}{9.5}\right)^{0.6}\left(\frac{9.083}{39}\right)^{0.2}\left(\frac{148{,}022}{12(39)(7.75)^3}\right)^{0.1} = 0.075 + (0.973)(0.747)(0.962) = 0.77$$

Skew reduction factor Article 4.6.2.2.2e AASHTO 2007

$$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5} = 0.25\left(\frac{148{,}022}{12(39.5)7.75^3}\right)^{0.25}\left(\frac{9.083}{39.5}\right)^{0.5} = 0.108$$
$$1 - c_1\tan(\theta)^{1.5} = 1 - 0.108\tan(30)^{1.5} = 0.953$$

$$g_m = 0.953\max(g_{m1}, g_{m2}) = 0.953\max(0.58, 0.77) = 0.73$$

D3-2.1.2 Distribution Factor for ***shear*** in interior longitudinal beams

The distribution factor for shear in interior beams is specified in Table 4.6.2.2.3a-1 of AASHTO LRFD Bridge Design Specifications (AASHT0, 2007) as follows

When one lane is loaded: $g_{v1} = 0.36 + \frac{s}{25} = 0.36 + \frac{9.083}{25} = 0.72$

When two or more lane are loaded: $g_{v2} = 0.2 + \frac{s}{12} - \left(\frac{s}{35}\right)^{2.0} = 0.2 + \frac{9.083}{12} - \left(\frac{9.083}{35}\right)^{2.0} = 0.89$

Thus, $g_v = \max(g_{v1}, g_{v2}) = \max(0.72, 0.89) = 0.89$

D3-2.2 Exterior Beams

D3-2.2.1 Distribution Factor for ***moment*** in exterior longitudinal beams

The distribution factor for moment in exterior beams is specified in Table 4.6.2.2.2d-1 of AASHTO LRFD Bridge Design Specifications (2007) as follows:

- When one design lane is loaded, the lever rule is used to determine the distribution factor, g_m
- When two or more lanes are loaded, the distribution factor is computed by multiplying the distribution factor for moment in interior beam by a correction factor, e, defined as

$$e = 0.77 + \frac{d_e}{9.1}$$

Where d_e is the distance from the exterior web of the exterior beam to the interior edge of the curb of traffic barrier.

For computing the distribution factor by the lever rule, a simple structural member such as the one shown below is analyzed

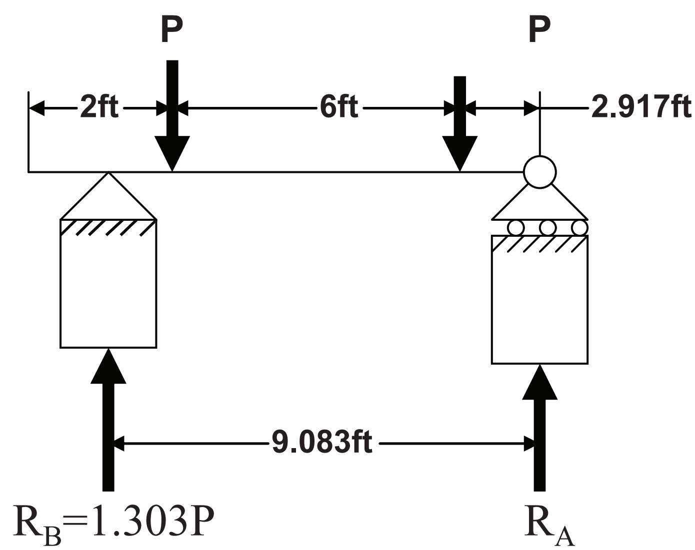


Figure D3-2.1 Exterior Girder Moment Distribution Factor

$$\sum M_A = 0$$
$$R_B(9.0833) - P(8.917) - P(2.917) = 0\,.$$
$$R_B = 1.303P$$

Article 3.6.1.1.2 of AASHTO LRFD Bridge Design Specification (AASHTO, 2007) states that a multiple presence factor m = 1.20 must be used when computing girder distribution factors by the lever rule. Thus, when one lane is loaded the distribution factor for the moment in exterior beams is:

$$g_{m1(exterior)} = m\frac{1.303P}{2P} = 1.2(0.6515) = 0.78$$

When two or more lanes are loaded:

$$e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{(1.834 - 0.75)}{9.1} = 0.889$$

$$g_{m2(exterior)} = e\, g_{m(interior)} = (0.889)(0.77) = 0.68$$

Skew reduction factor Article 4.6.2.2.2e AASHTO 2007

$$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5} = 0.25\left(\frac{148,022}{12(39.5)7.75^3}\right)^{0.25}\left(\frac{9.083}{39.5}\right)^{0.5} = 0.108$$

$$1 - c_1\tan(\theta)^{1.5} = 1 - 0.108\tan(30)^{1.5} = 0.953$$

$$g_{m(exterior)} = 0.953\max\left(g_{m1(exterior)}, g_{m2(exterior)}\right) = \max(0.78, 0.68) = 0.74$$

<u>D3-2.2.2 Distribution Factor for ***shear*** in exterior longitudinal beams</u>

When one lane is loaded, the distribution factor for shear is computed by the lever rule.
Thus, $g_{v1(exterior)} = 0.78$

When two or more lanes are loaded

$$e = 0.6 + \frac{d_e}{10}$$

$$g_{v2(exterior)} = (e)\, g_{v(interior)} =$$

$$\left(0.6 + \frac{1.834 - 0.75}{10}\right)(0.89) = (0.708)(0.89) = 0.63$$

$$g_{v(exterior)} = \max\left(g_{v1(exterior)}, g_{v2(exterior)}\right) = \max(0.78, 0.63) = 0.78$$

D3-2.3 Summary Results of Load Distribution Factors

The following table summarizes the results of calculations concerning the live load distribution factors:

Table D3-2.1 Distribution Factors

Action	Interior Beam	Exterior Beam
Bending Moment	0.73	0.74
Shear	0.89	0.78

D3-3 Analysis

With the LRFR method the HL93 load case in considered at the inventory and operating load level. The HS20 load case is considered for ratings at the legal load level.
The HL93 load consists of two load cases

- **Design truck:** consisting of the HS20 load subjected to distribution and impact factors plus a lane load ($w = 0.64 kips / ft$) that is only subjected to an distribution factors

- **Design tandem:** consisting of 2 25 kip axel spaced 4 ft apart subjected to distribution and impact factors plus a lane load ($w = 0.64 kips / ft$) that is only subjected to an distribution factors

D3-3.1 Maximum Bending Moment

D3-3.1.1 Maximum live load moment for an interior beam

With P = 32 kips, $g_m = 0.73$, and IM =1.33, M_{LL+IM} can be computed at the location of the maximum live load moment (at 17.17 ft from the left support) to be:

$$M_{LL+IM} = 419\ k - ft$$

The HL93 load:

$$DesignTruck(17.17) = M_{LL+IM} + g_m\left[12.48(17.17) - w\frac{(17.17)^2}{2}\right]$$

$$= 419 + 0.73\left[12.48(17.17) - 0.64\frac{(17.17)^2}{2}\right] = 507\ k - ft$$

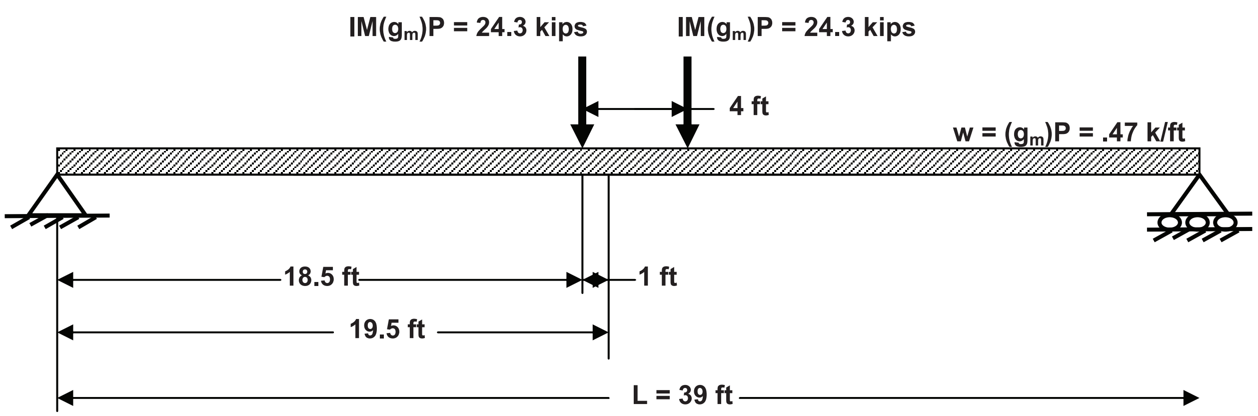


Figure D3-3.1 Maximum Design Tandem Moment

$$DesignTandem(18.5) = 23.03(18.5) + \left[9.17(18.5) - 0.47\frac{(18.5)^2}{2} \right] = 515\ k - ft$$

$$M_{HL93} = \max\left(DesignTruck, DesignTandem\right) = 515k - ft$$

D3-3.1.2 Maximum live load moment for an exterior beam

With P = 32 kips, $g_{m(exterior)} = 0.74$, and IM =1.33, M_{LL+IM} can be computed at the location of the maximum live load moment (at 17.17 ft from the left support) to be:

$$M_{LL+IM} = 425k - ft$$

The HL93 load in this case is:

$$DesignTruck(17.17) = M_{LL+IM} + g_m\left[12.48(17.17) - w\frac{(17.17)^2}{2} \right]$$

$$= 425 + 0.74\left[12.48(17.17) - 0.64\frac{(17.17)^2}{2} \right] = 514k - ft$$

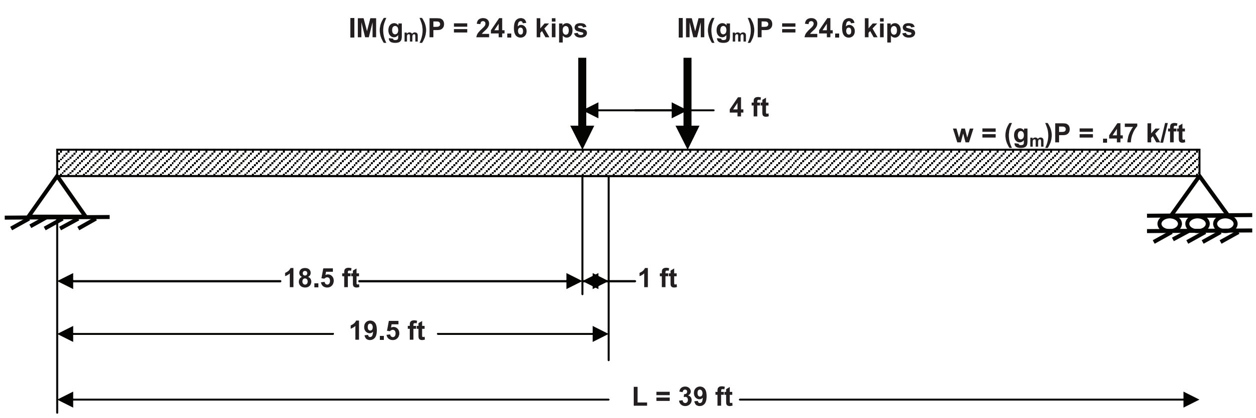


Figure D3-3.2 Maximum Design Tandem Moment

$$DesignTandem(18.5) = 23.3(18.5) + \left[9.17(18.5) - 0.47\frac{(18.5)^2}{2} \right] = 522\ k - ft$$

$$M_{HL93} = \max\left(DesignTruck, DesignTandem\right) = 522k - ft$$

D3-3.2 Maximum Shear Force

Article 5.8.3.2 of AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) states:

> "When the reaction force in the direction of the applied shear introduces compression into the end region of a member, the location of the critical section for shear shall be taken as d_v from the internal face of the support."

$$d_v = d_e - \frac{a}{2} \geq 0.9 d_e \geq 0.72h$$

Where:

d_e = the depth from the extreme compression fiber to the centroid of the tension reinforcement,
a = the depth of the compression block as calculated in Section A.7 of this report,
h = height of the member.

$$d_v = d - \frac{a}{2} = 29.59 - \frac{2.479}{2} = 2.36\,ft \geq 2.2\,ft \geq 2.06\,ft$$

D3-3.2.1 Interior beam maximum live load shear force at $d_v = 2.36\,ft$

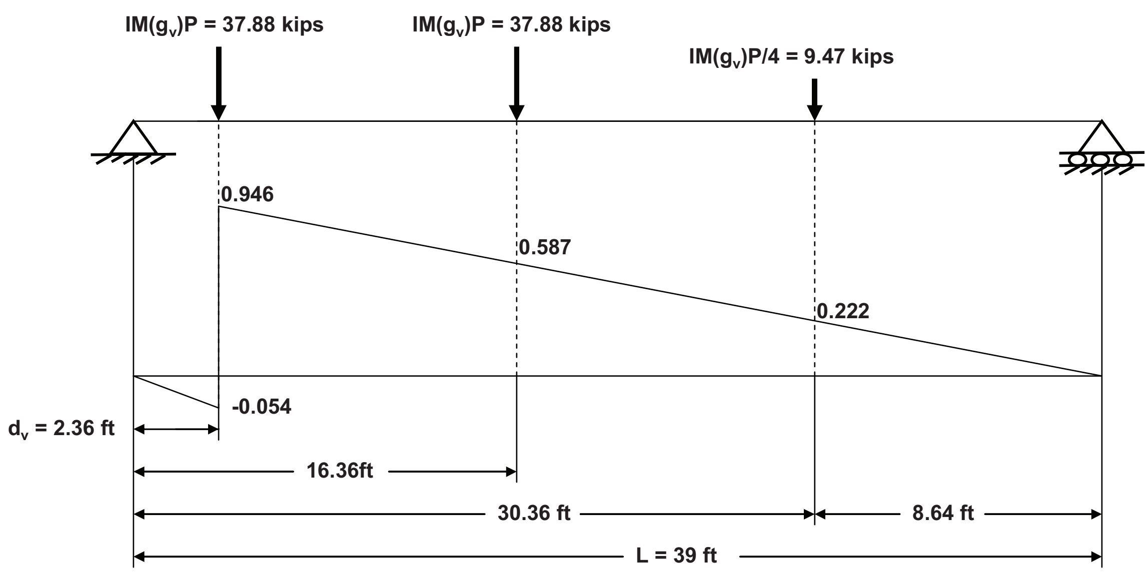


Figure D3-3.3 Shear Influence Line Diagram

$$\mathrm{V_{HS20}}(\mathrm{d}_v) = \left[37.88(0.939) + 37.88(0.581) + 9.47(0.222)\right] = 59.7kips$$

$$DesignTruck(d_v) = V_{HS20} + g_v w(19.5 - d_v) = 59.7 + 0.89(0.64)(19.5 - 2.36) = 69.5\ kips$$

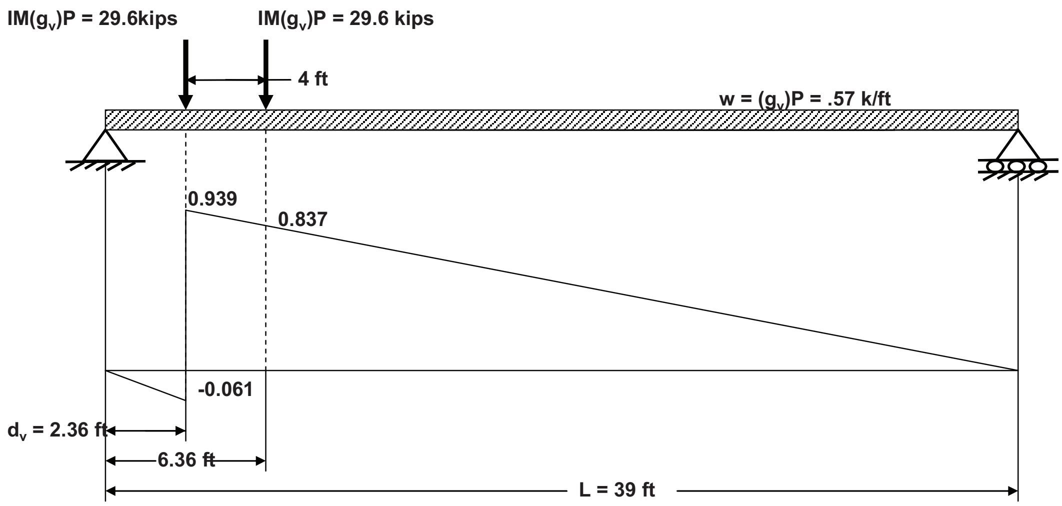


Figure D3-3.4 Interior Beam Shear Design Tandem at d_v

$$DesignTandem\left(d_v\right)=29.6\left(.939\right)+29.6\left(.837\right)+.570\left(19.5-2.36\right)=62.3\ kips$$

$$V_{HL93}=\max(DesignTruck, DesignTandem)=69.5kips$$

D3-3.2.2 Interior beam live load shear force at 4.42 ft

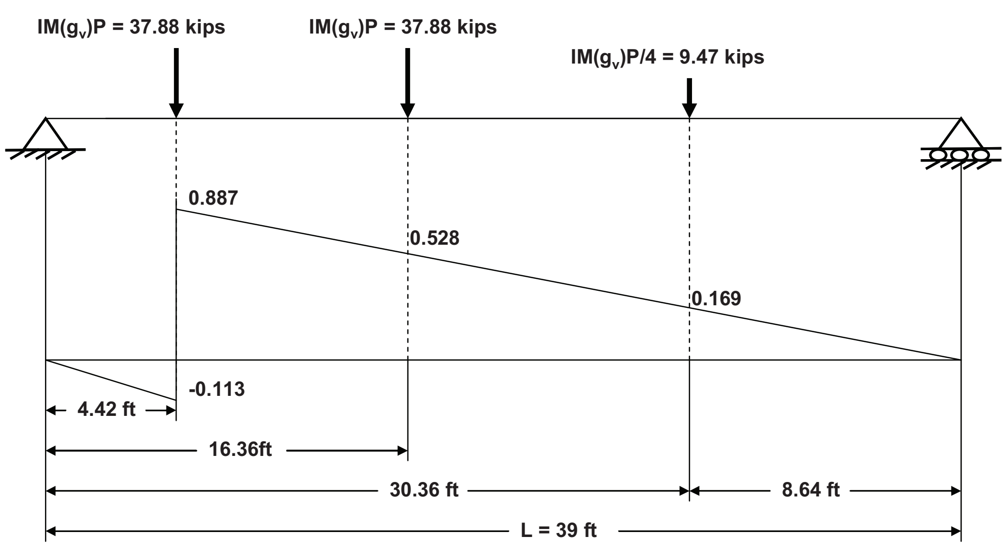


Figure D3-3.5 Shear Influence Line Diagram

$$\mathrm{V}_{\mathrm{HS20}}\left(4.42\right)=\left[37.88\left(0.887\right)+37.88\left(0.528\right)+9.47\left(0.169\right)\right]=55.2kips$$

$$V_{HL93}\left(4.42\right)=55.2+0.89\left(0.64\right)\left(19.5-4.42\right)=63.8\ kips$$

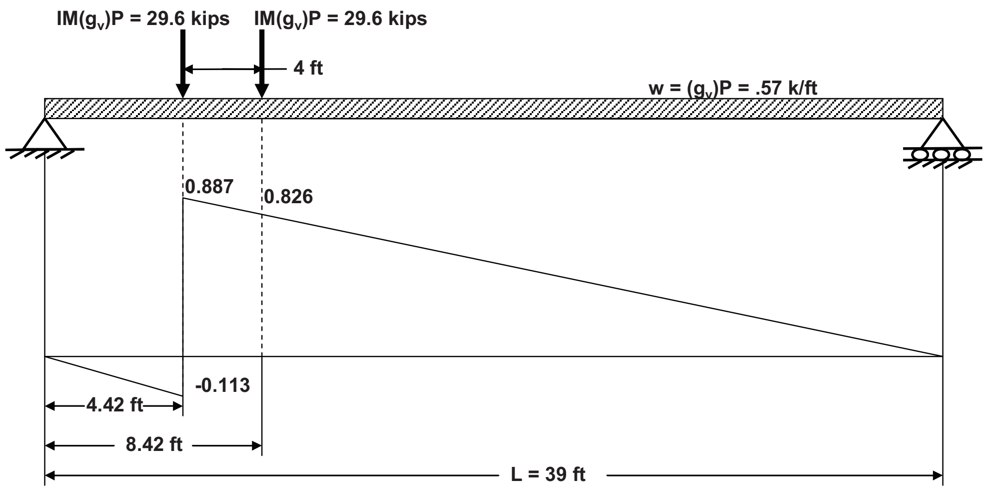


Figure D3-3.6 Interior Beam Shear Design Tandem at 4.42 ft from support

$$DesignTandem(4.42) = 29.6(.887) + 29.6(.826) + .570(19.5 - 4.42) = 59.3\ kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 63.8 kips$$

D3-3.2.3 Interior beam live load shear force at 10.67 ft

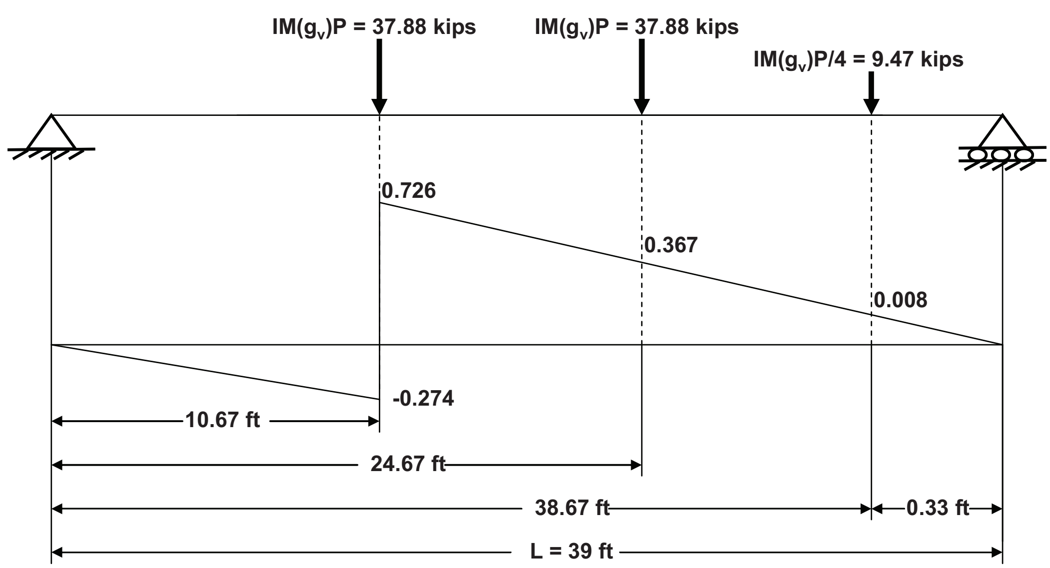


Figure D3-3.7 Shear Influence Line Diagram

$$\mathrm{V}_{\mathrm{HS20}}(10.67) = [37.88(0.726) + 37.88(0.367) + 9.47(0.008)] = 41.5 kips$$

$$V_{HL93}(10.67) = 41.5 + 0.89(0.64)(19.5 - 10.67) = 46.5\ kips$$

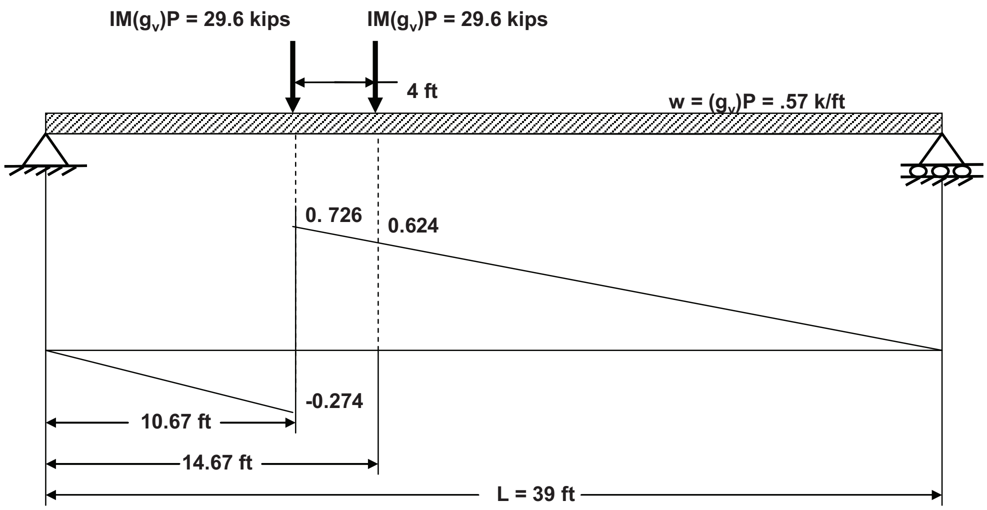


Figure D3-3.8 Interior Beam Shear Design Tandem at 10.67 ft from support

$$DesignTandem(10.67) = 29.6(.726) + 29.6(.624) + .570(19.5 - 10.67) = 45.0\ kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 46.5 kips$$

<u>D3-3.2.4 Exterior beam maximum live load shear force at</u> $d_v = 2.36\,ft$

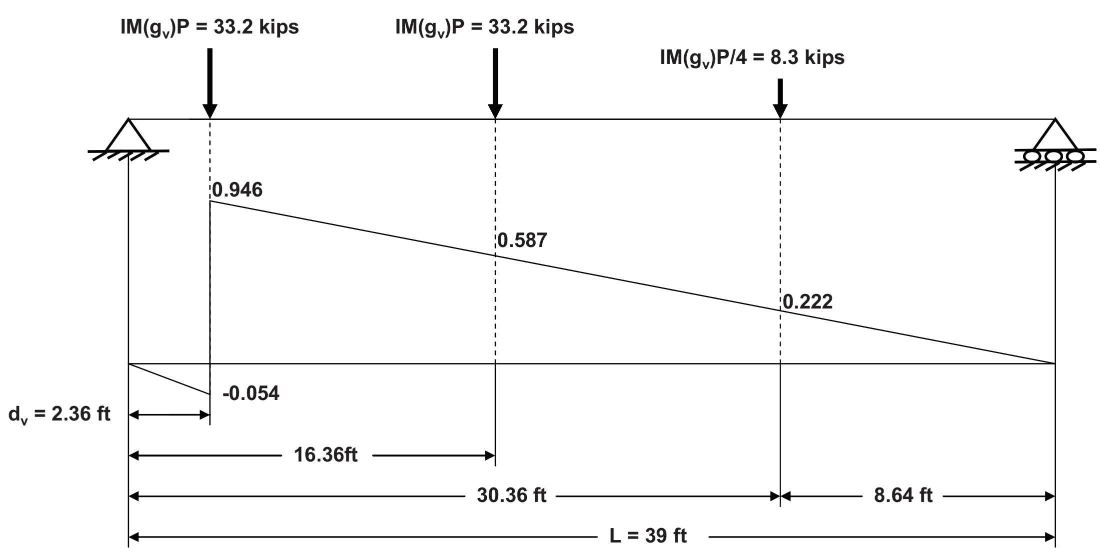


Figure D3-3.9 Shear Influence Line Diagram

$$\mathrm{V}_{\mathrm{HS20}}(\mathrm{d}_v) = [33.2(0.939) + 33.2(0.581) + 8.3(0.222)] = 52.3 kips$$

$$DesignTruck(d_v) = V_{HS20} + g_v w(19.5 - d_v) = 52.3 + 0.78(0.64)(19.5 - 2.36) = 60.9\ kips$$

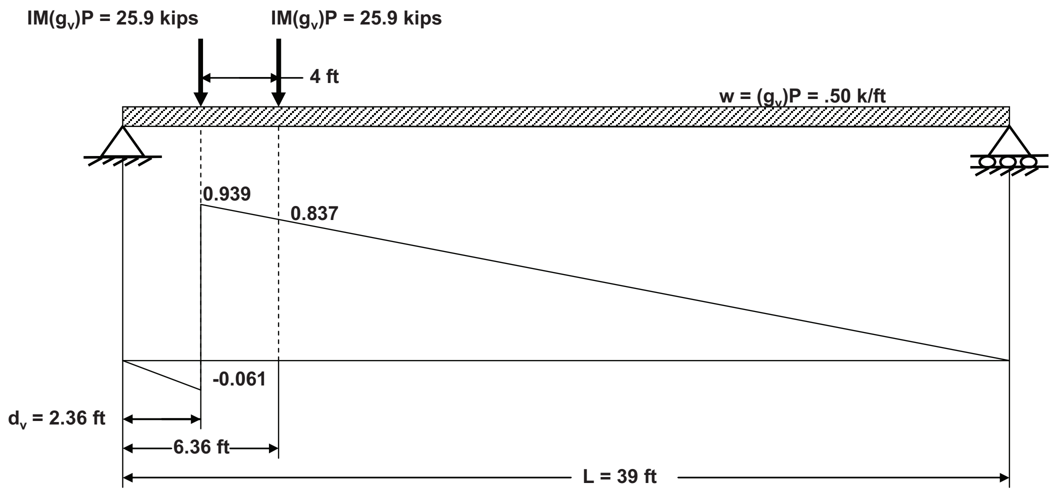


Figure D3-3.10 Exterior Beam Shear Design Tandem at d_v

$$DesignTandem(d_v) = 25.9(.939) + 25.9(.837) + .50(19.5 - 2.36) = 54.6\ kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 60.9 kips$$

<u>D3-3.3.5 Exterior beam live load shear force at 4.42 ft</u>

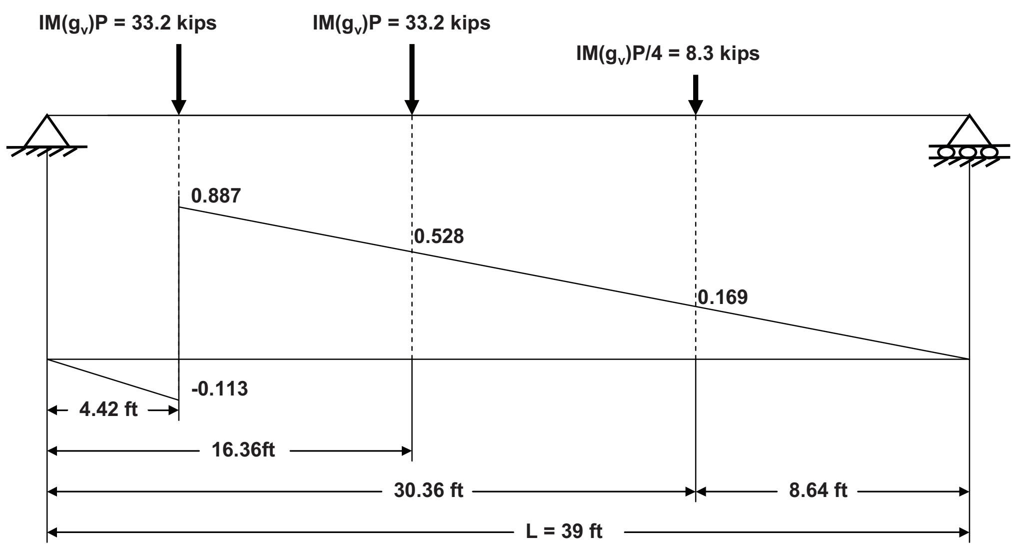


Figure D3-3.11 Shear Influence Line Diagram

$$\mathrm{V}_{\mathrm{HS20}}(4.42) = [33.2(0.887) + 33.2(0.528) + 8.3(0.169)] = 48.4 kips$$

$$V_{HL93}(4.42) = 48.4 + 0.78(0.64)(19.5 - 4.42) = 52.9\ kips$$

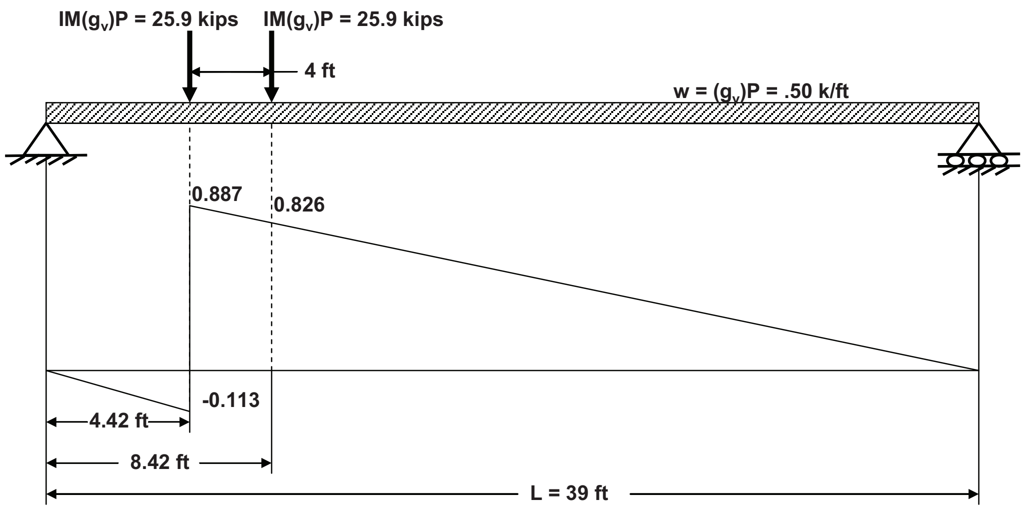


Figure D3-3.12 Exterior Beam Shear Design Tandem at 4.42 ft from support

$$DesignTandem(4.42) = 25.9(.887) + 25.9(.826) + .50(19.5 - 4.42) = 51.4\ kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 52.9kips$$

<u>D3-3.2.6 Exterior beam live load shear force at 10.67 ft</u>

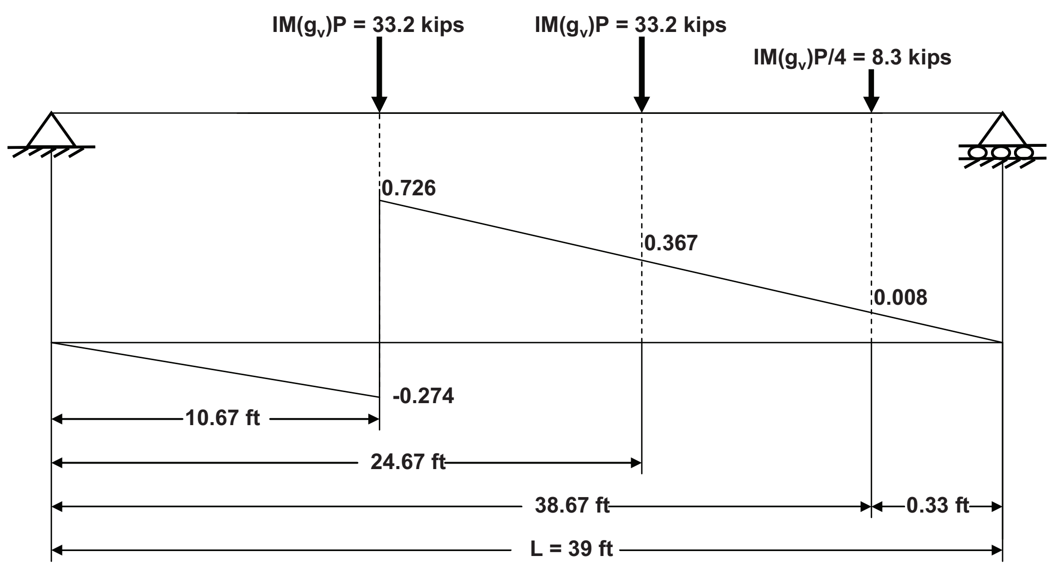


Figure D3-3.13 Shear Influence Line Diagram

$$\mathrm{V_{HS20}}(10.67) = \left[33.2(0.726) + 33.2(0.367) + 8.3(0.008)\right] = 36.4kips$$

$$V_{HL93}(10.67) = 36.4 + 0.78(0.64)(19.5 - 10.67) = 40.8\ kips$$

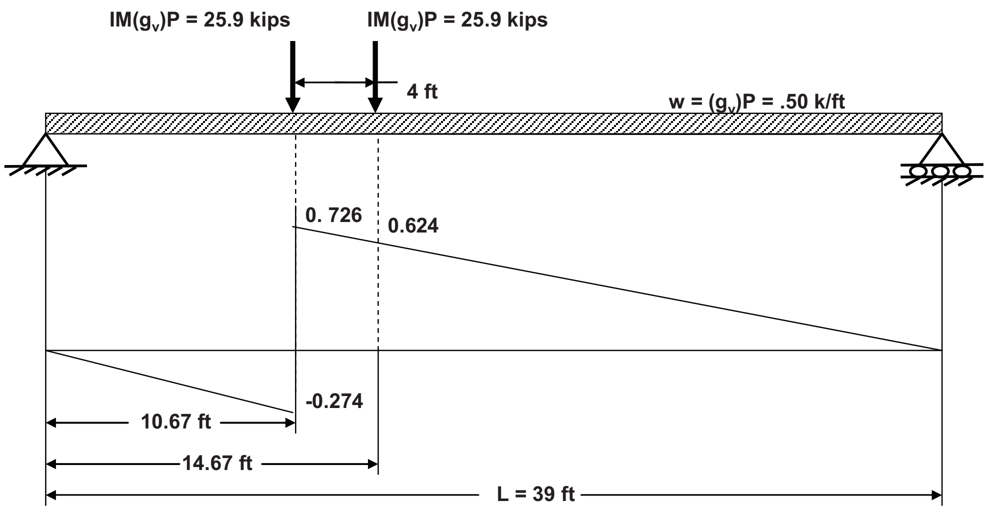


Figure D3-3.14 Exterior Beam Shear Design Tandem at 10.67 ft from support

$$DesignTandem(10.67) = 25.9(.726) + 25.9(.624) + .50(19.5 - 10.67) = 39.4\ kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 40.8 kips$$

D3-4 Member Capacity

D3-4.1 Interior Beam

Table D3-4.1 Interior beam capacity calculation (Computed in Section B2-5)

Flexure f_y = 40 ksi f'_c = 2.5ksi	$M_n = 1,780\ k - ft$	AASHTO MCE 2000 D.6.6.2.3
Shear at d_v	$V_n = V_c + V_s$ = 189 kips	AASHTO MCE 2000 D.6.6.2.4.
Shear at 4.42 ft	$V_n = V_c + V_s$ = 138.8 kips	
Shear at 10.67 ft	$V_n = V_c + V_s$ = 92.6 kips	

D3-4.2 Exterior Beam

Table B4-4.2 Exterior beam capacity calculation (Computed in Section B2-5)

Flexure f_y = 40 ksi f'_c = 2.5ksi	$M_n = 1,630\ k - ft$	AASHTO MCE 2000 D.6.6.2.3
Shear at d_v	$V_n = V_c + V_s$ = 189 kips	AASHTO MCE 2000 D.6.6.2.4.
Shear at 4.42 ft	$V_n = V_c + V_s$ = 138.8 kips	
Shear at 10.67 ft	$V_n = V_c + V_s$ = 92.6 kips	

D3-5 Rating Calculation (LRFR)

The following factors are defined thus:

Live load factor for legal load level $\gamma_{LL} = 1.55$ (LRFR Table 6-5 based on ADTT of 709

Resistance Factor (for shear and flexure) φ = .9

Condition Factor (related to NBI Item 59) $\phi_c = 1$

System Factor (related to structural redundancy) $\phi_s = 1$

Table D3-5.1 Load and Resistance Factor Rating (LRFR) Calculation for HL93 load at Inventory and Operating Levels and HS20 load at the Legal Level (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO LRFD Specifications (2007))

Flexure (Interior girder) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(1,780) - 1.25(262)}{1.75(515)} = 1.41$	50.8 tons
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(1,780) - 1.25(262)}{1.35(515)} = 1.83$	65.9 tons
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.55$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HS20}} = \frac{0.9(1)(1)(1,780) - 1.25(262)}{1.55(419)} = 1.96$	70.6 tons
Flexure (Exterior girder) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(1,630) - 1.25(242)}{1.75(522)} = 1.27$	45.7 tons
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(1,630) - 1.25(242)}{1.35(522)} = 1.65$	59.4 tons

	Legal Level $\gamma_{DC}=1.25$ $\gamma_{LL}=1.55$	$RF=\dfrac{\phi\phi_c\phi_s M_n-\gamma_{DC}M_D}{\gamma_{LL}M_{HS20}}=\dfrac{0.9(1)(1)(1,630)-1.25(242)}{1.55(425)}=1.77$	63.7 tons
Shear (Interior Girder at d/2) $\phi=0.9$ $\phi_c=1$ $\phi_s=1$	**Inventory Level** $\gamma_{DC}=1.25$ $\gamma_{LL}=1.75$	$RF=\dfrac{\phi\phi_c\phi_s V_n-\gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}}=\dfrac{0.9(1)(1)(189)-1.25(24.4)}{1.75(69.5)}=1.15$	41.4 tons
	Operating Level $\gamma_{DC}=1.25$ $\gamma_{LL}=1.35$	$RF=\dfrac{\phi\phi_c\phi_s V_n-\gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}}=\dfrac{0.9(1)(1)(189)-1.25(24.4)}{1.35(69.5)}=1.49$	53.6 tons
	Legal Level $\gamma_{DC}=1.25$ $\gamma_{LL}=1.55$	$RF=\dfrac{\phi\phi_c\phi_s V_n-\gamma_{DC}V_{DL}}{\gamma_{LL}V_{HS20}}=\dfrac{0.9(1)(1)(189)-1.25(24.4)}{1.55(59.7)}=1.51$	54.4 tons
Shear (Exterior Girder at d/2) $\phi=0.9$ $\phi_c=1$ $\phi_s=1$	**Inventory Level** $\gamma_{DC}=1.25$ $\gamma_{LL}=1.75$	$RF=\dfrac{\phi\phi_c\phi_s V_n-\gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}}=\dfrac{0.9(1)(1)(189)-1.25(22.4)}{1.75(60.9)}=1.33$	47.9 tons
	Operating Level $\gamma_{DC}=1.25$ $\gamma_{LL}=1.35$	$RF=\dfrac{\phi\phi_c\phi_s V_n-\gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}}=\dfrac{0.9(1)(1)(189)-1.25(22.4)}{1.35(60.9)}=1.73$	62.3 tons
	Legal Level $\gamma_{DC}=1.25$ $\gamma_{LL}=1.55$	$RF=\dfrac{\phi\phi_c\phi_s V_n-\gamma_{DC}V_{DL}}{\gamma_{LL}V_{HS20}}=\dfrac{0.9(1)(1)(189)-1.25(22.4)}{1.55(52.3)}=1.75$	63.0 tons
Shear (Interior Girder at 4.42 ft)	**Inventory Level** $\gamma_{DC}=1.25$ $\gamma_{LL}=1.75$	$RF=\dfrac{\phi\phi_c\phi_s V_n-\gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}}=\dfrac{0.9(1)(1)(138.8)-1.25(21.1)}{1.75(63.8)}=0.88$	31.7 tons

$\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Operating Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(138.8) - 1.25(21.1)}{1.35(63.8)} = 1.14$	41.0t ons
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.55$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HS20}} = \frac{0.9(1)(1)(138.8) - 1.25(21.1)}{1.55(55.2)} = 1.15$	41.4 tons
Shear (Exterior Girder at 4.42 ft) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(138.8) - 1.25(19.5)}{1.75(52.9)} = 1.09$	39.2 tons
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(138.8) - 1.25(19.5)}{1.35(52.9)} = 1.41$	50.8 tons
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.55$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HS20}} = \frac{0.9(1)(1)(138.8) - 1.25(19.5)}{1.55(48.4)} = 1.34$	48.2 tons
Shear (Interior Girder at 10.67 ft) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(92.6) - 1.25(12.4)}{1.75(46.5)} = 0.83$	29.9 tons
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(92.6) - 1.25(12.4)}{1.35(46.5)} = 1.08$	38.9 tons
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.55$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HS20}} = \frac{0.9(1)(1)(92.6) - 1.25(12.4)}{1.55(41.5)} = 1.05$	37.8 tons

Shear (Exterior Girder at 10.67 ft) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(92.6) - 1.25(11.4)}{1.75(40.8)} = 0.97$	34.9 tons
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(92.6) - 1.25(11.4)}{1.35(40.8)} = 1.25$	45.0 tons
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.55$	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HS20}} = \frac{0.9(1)(1)(92.6) - 1.25(11.4)}{1.55(36.4)} = 1.22$	43.9 tons

Rating of a Prestressed Concrete Bridge (GDOT BRIDGE ID # 223-0034)

Example E1:

Rating by the Load Factor Method (LFR) Using Load Distribution and Dynamic Allowance Factors Stipulated by AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002).

E1-1 Basic Geometry and Bridge Information

Span: L = 66.8125 ft
Steel yield strength: f_y = 60 ksi
Materials:
Concrete strength: f'_c = 3.5 ksi (deck)
f'_c = 6 ksi (Prestressed beam)
Prestressing Steel: ½" dia. Special Lo-Lax Strands
A_{ps} = 0.162 in^2 per Strand
28 prestressing strands
Stirrups: Starting from end 2" space
7 # 5 spaced 6" O.C. over 3'-6"
11" space
3 # 5 spaced 11" O.C. over 2'-9"
14 # 5 spaced 1'-10" O.C. over 25'-8"
Clear space to midspan
Condition: NBI item 59 code = 7
ADTT: 980
Skew: 0 degrees

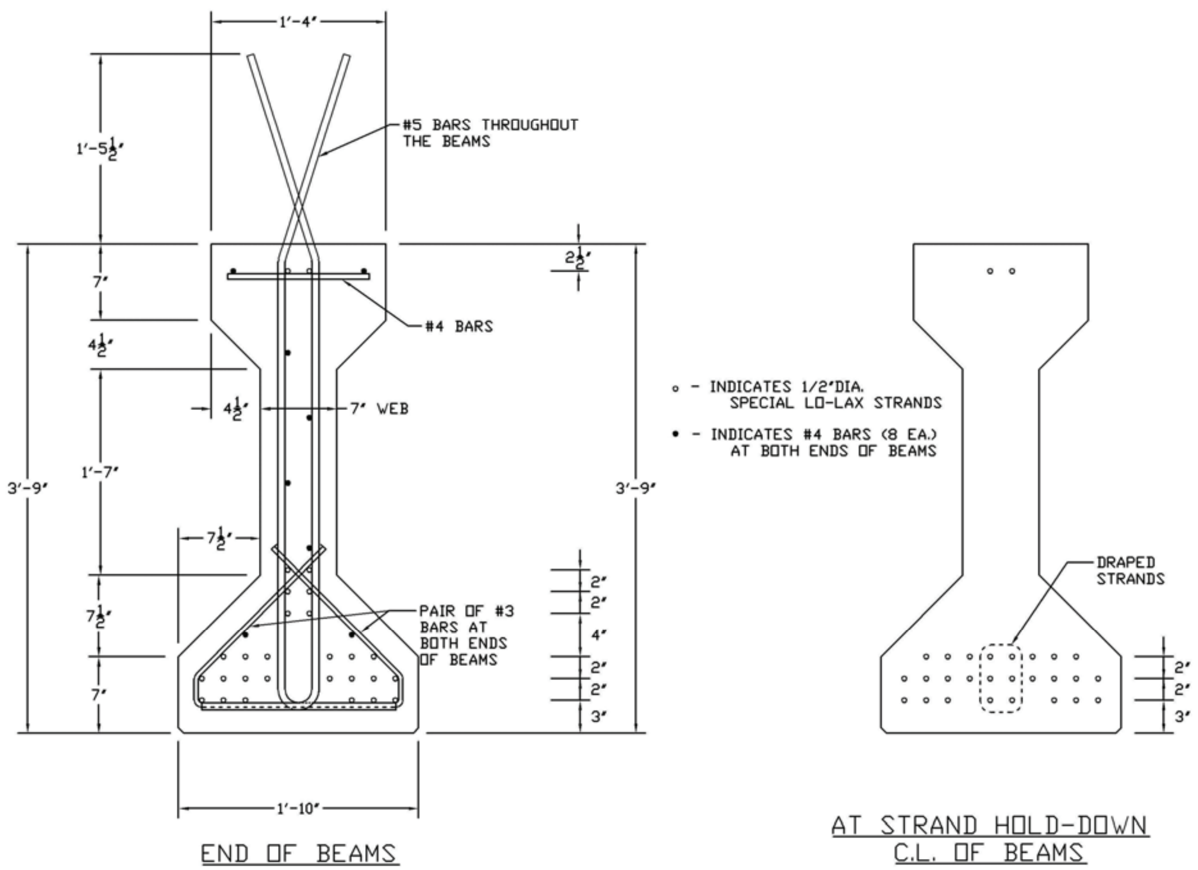


Figure E1-1.1 Beam Reinforcement Details

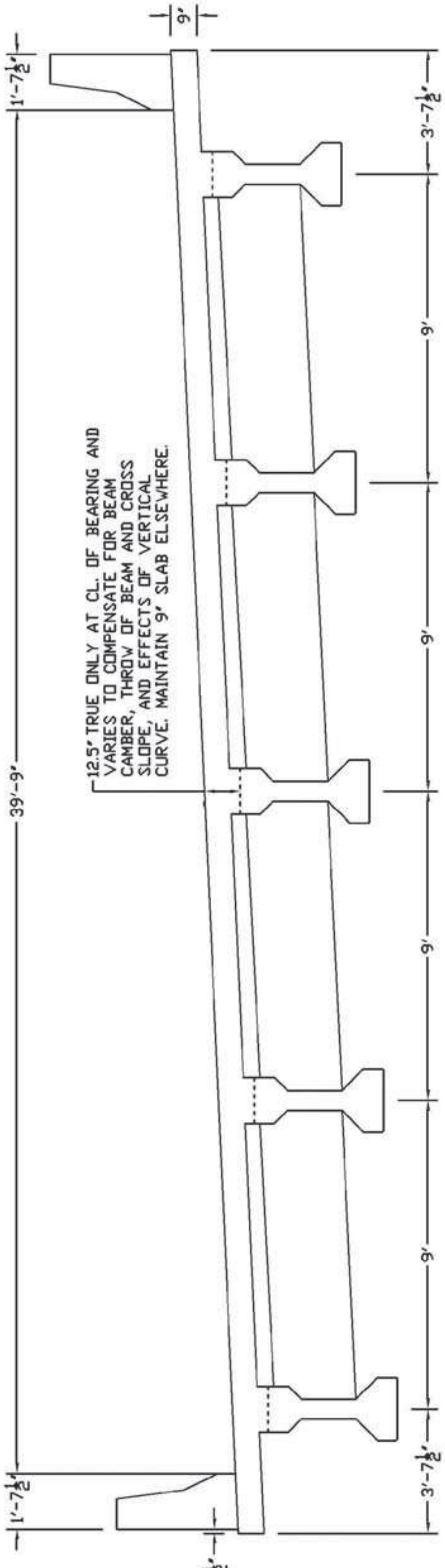


Figure E1-1.2 Bridge Cross Section

E1-2 LOADS

E1-2.1 Permanent Loads

As per Article 3.23.2.3.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) the dead load supported by the outside stringers or beams shall be the portion of the floor slab carried by the stringer or beam. Curbs, railings, and wearing surfaces if placed after the slab has cured, may be distributed equally to all roadway girders.

E1-2.1.1 Dead load interior beam

Weight per linear foot of the reinforced concrete slab	= 1.012 k/ft
Weight per linear foot of beams	= 0.583 k/ft
Weight of parapet, rail and sidewalk assembly	= 0.162 k/ft
Weight per linear foot of coping	= 0.017 k/ft
Since there is no wearing surface present on the bridge, DW	= 0
Total dead load for interior beam	**= 1.774 k/ft**

Weight of Diaphragm $P_d = 3.05 kips$

E1-2.1.1 Dead load interior beam

Weight per linear foot of the reinforced concrete slab	= 0.914 k/ft
Weight per linear foot of beams	= 0.583 k/ft
Weight of parapet, rail and sidewalk assembly	= 0.162 k/ft
Weight per linear foot of coping	= 0.017 k/ft
Since there is no wearing surface present on the bridge, DW	= 0
Total dead load for interior beam	**= 1.676 k/ft**

Weight of Diaphragm $P_d = 1.525 kips$

E1-2.2 Vehicular Live Load

The design vehicular live load on the bridge consists of AASHTO HS20 truck with the spacing between the two 32-kip rear-axle loads to be varied from 14 ft to 30 ft to produce extreme force effects. The HS 20 truck is shown below.

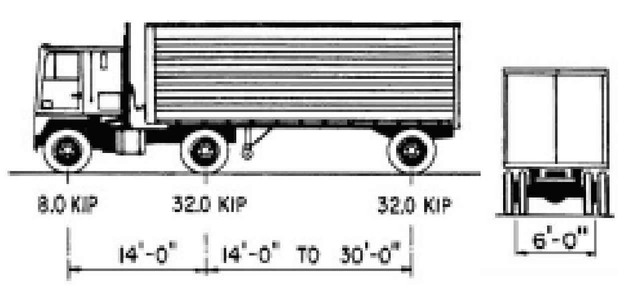


Figure A-2.1 AASHTO HS20 Truck

E1-3 Dynamic Load Allowance

Article 3.8.2.1 of the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002), the dynamic load allowance is taken as:

$$I = \frac{50}{L + 125} = \frac{50}{66.8125 + 125} = 0.261 \le 0.3$$

Where:
I = impact factor (maximum 30 percent)
L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member

E1-4 Live Load Distribution Factors

E1-4.1 Moment distribution factors

E1-4.1.1 Distribution Factor for ***moment*** in interior longitudinal beams

As per Table 3.23.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002), the distribution factors for moment in interior and exterior beams are computed as:

$$\text{DFM} = \frac{s}{5.5} = 1.636$$

E1-4.1.1 Distribution Factor for ***moment*** in interior longitudinal beams

Using the structure shown in Figure E1-4.1 the shear distribution factor is calculated by computing the reaction at R_B, but not taken less than (Article 3.23.2.3.1 AASHTO 2002):

$$\frac{S}{4.0 + 0.25S} = \frac{9}{4 + 0.25(9)} = 1.44$$

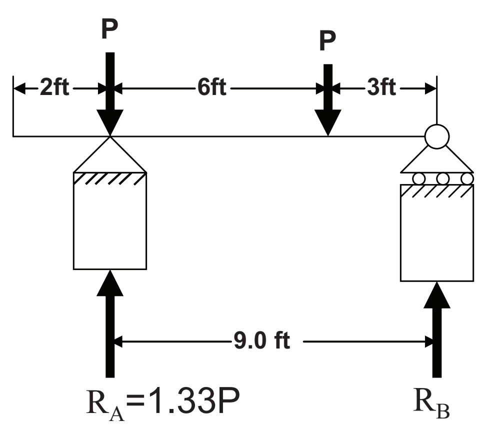


Figure E1-4.1 Exterior Girder Shear Distribution Factor

Thus, the distribution factor for shear in an exterior beam is:

$$DFM = 1.44$$

E1-4.2 Shear distribution factors

E1-4.2.1 Distribution Factor for ***shear*** in interior longitudinal beams

Article 3.23.1.2 of the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) stipulates that:

> "Lateral distribution of the wheel loads at ends of the beams or stringer shall be that produced by assuming the floors to act as a simple span between stringers or beams. For wheels or axels in other positions on the span, the distribution for shear shall be determined by the method prescribed for moment."

Therefore, by modeling the deck as a series of rigid simply supported beams between the girders, as shown in Figures A1.4.2, the shear distribution factor (DFS) is computing by calculating the reaction R_B.

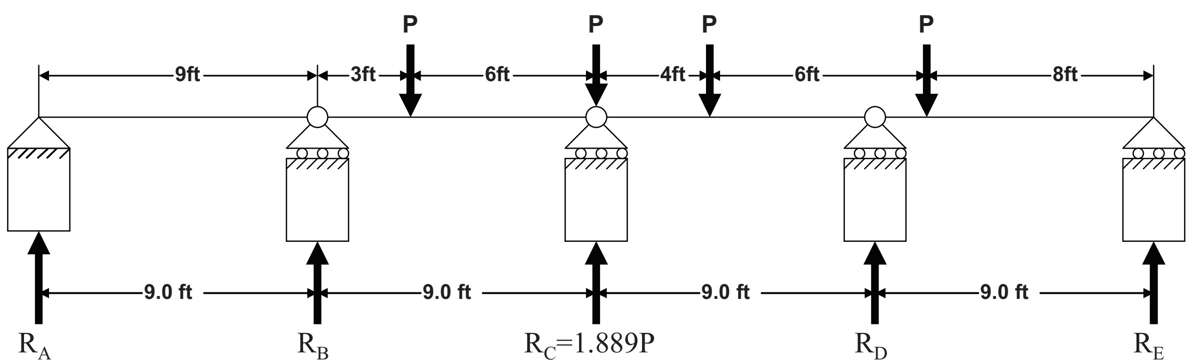


Figure E1-4.2 Interior Girder Shear Distribution Factor

DFS = 1.889 for wheel loads at beam ends

E1-4.2.2 Distribution Factor for ***shear*** in exterior longitudinal beams

Using the structure shown in Figure E1-4.1 the shear distribution factor is calculated by computing the reaction at R_B.

$$DFS = 1.33 \text{ for wheel loads at beam ends}$$

E1-4.2 Summary of distribution factors

Table E1-4.1 Distribution Factors

Action	Interior Beam	Exterior Beam
Bending Moment	1.636	1.44
Shear	1.889	1.33

E1-5 Flexural Analysis

E1-5.1 Live load bending moment

A rudimentary structural analysis of a simply supported beam subjected to a vehicular load having two rear axles and one front axle as shown in Figure A1.5.1 shows that the absolute maximum moment occurs under the middle axle when such an axle is positioned at a distance of 2.33 ft to the left of the beam centerline.

By applying the dynamic allowance factor and the distribution factor for moment of interior beams, we can then compute the maximum live load under the following loads:

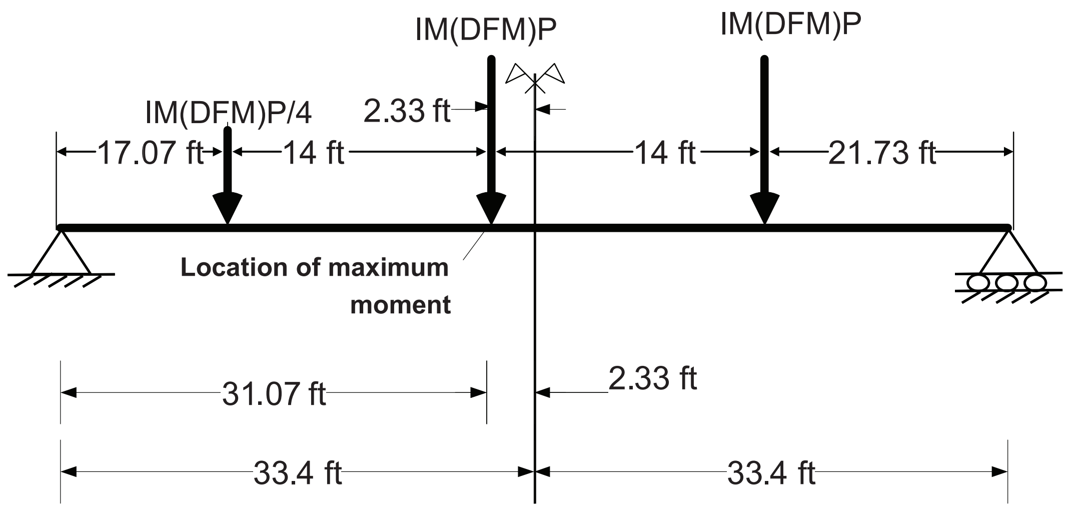


Figure E1-5.1 Max Live Load Moment

E1-5.1.1 Maximum live load moment for an interior beam

With P = 16 kips, $DFM = 1.636$, and IM =1.26, M_{LL+IM} can be computed at the location 31.07 ft from the left support to be:

$$M_{LL+IM} = 957\ k - ft$$

Live load moment (at midspan):

$$M_{LL+IM} = 951\ k - ft$$

E1-5.1.2 Maximum dead load moment for an interior beam

The dead load moment at the location where the live load produces the maximum effect is

$$M_D\left(31.07\right) = \frac{P_d(x)}{2} + w\left[\frac{Lx}{2} - \frac{(x)^2}{2}\right] = \frac{3.05(x)}{2} + 1.774\left[\frac{Lx}{2} - \frac{(x)^2}{2}\right] = 1,032\ k - ft$$

The dead load moment at midspan is:

$$M_D(33.4) = \frac{P_d(x)}{2} + w\left[\frac{Lx}{2} - \frac{(x)^2}{2}\right] = \frac{3.05(x)}{2} + 1.774\left[\frac{Lx}{2} - \frac{(x)^2}{2}\right] = 1{,}041\ k - ft$$

E1-5.1.3 Maximum live load moment for an exterior beam

With P = 16 kips, $DFM = 1.636$, and IM =1.26, M_{LL+IM} can be computed at the location 31.07 ft from the left support to be:

$$M_{LL+IM} = 842\ k - ft$$

Live load moment (at midspan):

$$M_{LL+IM} = 837\ k - ft$$

A1-5.2.1 Maximum dead load moment for an exterior beam

The dead load moment at the location where the live load produces the maximum effect is

$$M_D(31.07) = \frac{P_d(x)}{2} + w\left[\frac{Lx}{2} - \frac{(x)^2}{2}\right] = \frac{3.05(x)}{2} + 1.676\left[\frac{Lx}{2} - \frac{(x)^2}{2}\right] = 978\ k - ft$$

The dead load moment at midspan is:

$$M_D(33.4) = \frac{P_d(x)}{2} + w\left[\frac{Lx}{2} - \frac{(x)^2}{2}\right] = \frac{3.05(x)}{2} + 1.676\left[\frac{Lx}{2} - \frac{(x)^2}{2}\right] = 986\ k - ft$$

E1-6 Shear Force Analysis

To evaluate Shear in the Prestressed Girders the span was broken up into 20 segments and evaluated at each to determine where the critical shear was encountered. The location of this critical shear was found to be 3.341 ft in from the girder support.

E1-6.1 Interior Beam Shear Analysis

E1-6.1.1 Interior beam maximum live load shear force at 3.341 ft

The critical location for the interior girder is located at 3.341 ft from the support.

At 3.341 ft; the length of the portion of the span that is loaded to produce the maximum stress in the member is L = 66.8125 - 3.341 = 63.47 ft thus:

$$IM = 1 + I = 1 + \frac{50}{(66.8125 - 3.341) + 125} = 1.265 \le 1.30$$

Use IM = 1.265

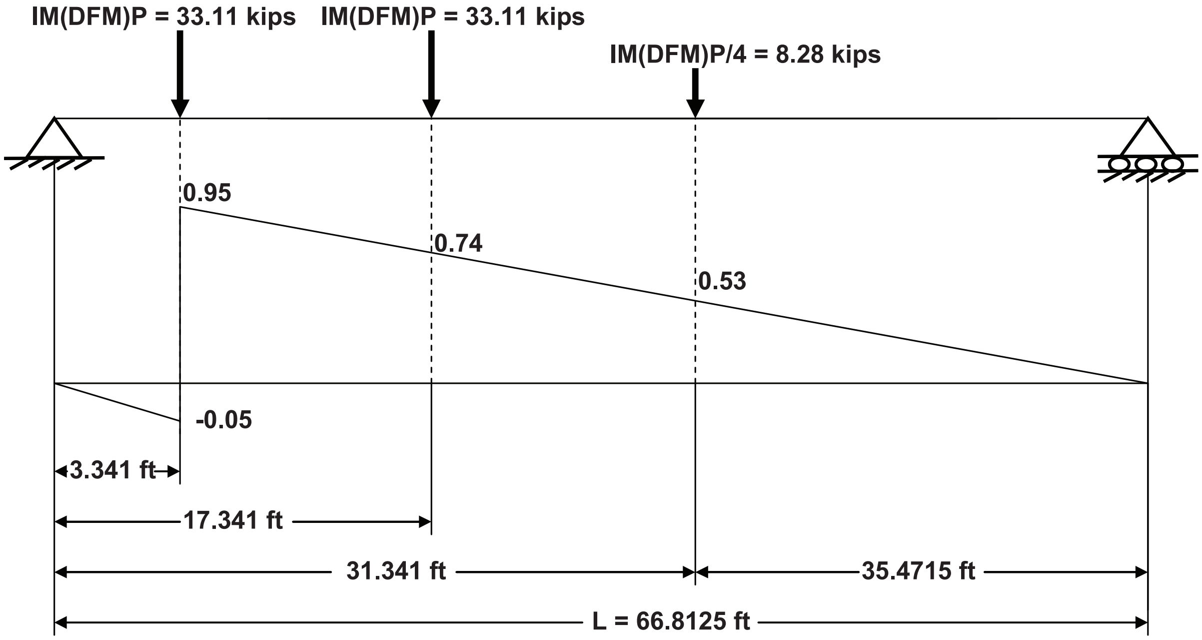


Figure E1-6.1 Shear Influence Line Diagram

$$\mathrm{V}_{\mathrm{HS20}}(3.341) = 33.11(0.95) + (33.11)(0.74) + (8.28)(0.53) = 60.3 kips$$

<u>E1-6.1.2 Interior beam dead load shear force at 3.341 ft</u>

$$\mathrm{V}_{\mathrm{dc}}(3.341) = 1.774\left(\frac{\mathrm{L}}{2} - 3.341\right) + \frac{3.05}{2} = 54.9\mathrm{kips}$$

E1-6.2 Exterior Beam Shear Analysis

<u>E1-6.2.1 Exterior beam maximum live load shear force at 3.341 ft</u>

The exterior beam live load shear is computed by the same means as that shown in Section E1-6.1, but with the exterior distribution factor.

$$\mathrm{V}_{\mathrm{HS20}}(3.341) = 53.1 kips$$

<u>E1-6.2.2 Exterior beam dead load shear force at 3.341 ft</u>

$$\mathrm{V}_{\mathrm{dc}}(3.341) = 1.676\left(\frac{\mathrm{L}}{2} - 3.341\right) + \frac{1.525}{2} = 51.2\mathrm{kips}$$

E1-8 Interior Beam Analysis

$A_{ps} = 4.676\ in^2$ (area of prestressing)
$f'_s = 270$ ksi (ultimate strength of prestressing)
$b_{deck} = 108$ in (effective width of deck Article 9.9.1 AASHTO, 2002)
$h = 45$ in (height of prestressed beam)
$h_{slab} = 9$ in (height of slab)
$y_{nc} = 20.27$ in (distance from centroid of beam to bottom of beam)
$y_c = 36.94$ in (distance from centroid of composite beam and slab to bottom of beam)
$y_s = 7.679$ in (distance from centroid of prestressing to bottom of beam)
$\beta_1 = .85$ in (concrete strength factor)
$A = 559.5\ in^2$ (area of beam)
$I_{NC} = 125,400 in^4$ (moment of inertia of beam)
$I_C = 402,900 in^4$ (moment of inertia of composite beam and slab)
$f_{ci} = 5,000\ psi$ (compressive strength of concrete at time of initial prestressing)
$\gamma = 0.28$ (Lo-Lax Prestressing Factor

e = Eccentricity of Prestressing strands
$e(31.07) = 12.426$ in $\qquad e_{max} = e(33.406) = 12.595$ in
d = distance from extreme compression fiber to centroid of prestressing

$d = h + h_{slab} - (y_{nc} - e) =$ 46.15 in at 31.07 ft from the support
$d = h + h_{slab} - (y_{nc} - e) =$ 46.3 in at 33.406 ft from the support
$p = A_{ps} / (b_{deck} d)$ Article 9.1.1 (AASHTO, 2002)

E1-8.1 Moment Capacity

$$\varphi = 1$$

$$f_{su} = f'_s \left(1 - \frac{\gamma}{\beta_1} p \frac{f'_s}{f_{deck}} \right) \text{ Article 9.17.4 (AASHTO, 2002)}$$

$$\varphi Mn(x) = A_{ps} f_{ps} d \left(1 - 0.6 \frac{p(f_{su})}{f'_c} \right) \text{ Article 9.17.2 (AASHTO, 2002)}$$

At 31.07 ft from support

$f_{ps}(31.07) = 263.56 ksi$

$\varphi Mn(31.07) =$ 4,542 kip-ft

At 33.406 ft from support

$f_{ps}(33.406) = 263.58 ksi$

$\varphi Mn(33.406) =$ 4,558 kip-ft

E1-8.2 Prestress Losses (Article 9.16 AASHTO, 2002)

Total non-composite dead load

$$\mathrm{M}_{\mathrm{dl}_{\mathrm{nc}}}\left(33.406\right)=1.774\frac{(66.8125)^2}{8}+\frac{3.05}{2}\frac{66.8125}{2}= 11{,}404 \text{ kip-in}$$

Total composite dead load

$$\mathrm{M}_{\mathrm{dl}_{\mathrm{c}}}\left(33.406\right)=0.162\frac{(66.8125)^2}{8}=1{,}085\ k-in$$

Dead load moment due to self weight of the girder:

$$M_g\left(33.406\right)=0.583\frac{(66.8125)^2}{8}=3{,}904\ k-in$$

Shrinkage Losses:
RH is equal to 70 as per Article 9.16.2.1.1 of AASHTO Standard Specifications for Highway Bridge Design (AASHTO, 2002))

$$SH=17{,}000-150\left(RH\right)=17{,}000-150\left(70\right)=6.5ksi$$

Elastic Shortening:
E_s = 28 x 10^6 psi as per Article 9.16.2.1.2 of AASHTO Standard Specifications for Highway Bridge Design (AASHTO, 2002)). The prestressing force after initial losses P_{si} is required in order to compute the elastic shortening (ES), and relaxation of the prestressing steel CR_s; however, the prestressing force after initial losses is affected by ES and CR_s therefore a first approximation of P_{si} is taken as:

$$P_{si}=0.9\left(0.75f_s^{'}A_{ps}\right)=852.2kips$$

After computing ES and CR_s the initial prestressing force converges to:

$$P_{si}=A_{ps}\left(0.75f_s^{'}-ES-0.3CR_s\right)=874.3kips$$

" f_{cir} the average concrete stress at the c.g. of the prestressing steel at time of release" Article 9 AASHTO Standard Specification for Highway Bridge Design (AASHTO, 2002) is needed for the computation of ES.

$$\begin{aligned} f_{cir} &= -\frac{P_{si}}{A}-\frac{P_{si}ee}{I}-\frac{M_g e}{I} \\ &= -\frac{874.3}{559.5}-\frac{874.3\left(-12.595\right)^2}{125{,}400}-\frac{3{,}904\left(-12.595\right)}{125{,}400}=2.277ksi \end{aligned}$$

$$ES=\frac{E_s}{E_{ci}}f_{cir}=\frac{28\times10^6}{33\left(150^{3/2}\right)\sqrt{5{,}000}}\left(2.277\right)=14.9ksi$$

Creep of Concrete:
As per Article 9.16.2.1.3 of AASHTO Standard Specifications for Highway Bridge Design (AASHTO, 2002)):

f_{cds} is the "average concrete compressive stress at the c.g. of prestressing steel under full dead load."
Article 9 AASHTO Standard Specification for Highway Bridge Design (AASHTO, 2002):

$$f_{cds} = \frac{\left(M_{DL_{nc}} - M_g\right)(e)}{I_{NC}} + \frac{M_{DL_c}\left(y_c - y_s\right)}{I_C} = \frac{7500(12.595)}{125,400} + \frac{1085(36.9-7.679)}{402,900} = 0.832ksi$$

$$CR_c = 12f_{cir} - 7f_{cds} = 12(2.227) - 7(0.832) = 21.5ksi$$

Relaxation of Prestressing Steel:
As per Article 9.16.2.1.4 of AASHTO Standard Specifications for Highway Bridge Design (AASHTO, 2002)):

$$\begin{aligned} CR_s &= 5,000 - 0.1ES - 0.05\left(SH + CR_c\right) \\ &= 5,000 - 0.1(14,900) - 0.05(6,500 + 21,500) = 2.1ksi \end{aligned}$$

Total Prestressing Losses:

$$\Delta f_s = SH + ES + CR_c + CR_s = 6.5 + 14.9 + 21.5 + 2.1 = 45ksi$$

Stress after Initial Losses: (check for convergence)

$$P_{si} = \left(0.75f_s^{'} - ES - 0.3CR_s\right)A_{ps} = \left(0.75(270) - 14.9 - 0.3(2.1)\right)4.676 = 874.3kips$$

Stress after Final Losses:

$$P_s = \left(0.75f_s^{'} - \Delta f_s\right)A_{ps} = \left(0.75(270) - 45\right)4.676 = 736.5kips$$

E1-8.3 Serviceability Limit State: (allowable stress)

Allowable stress in top flange of prestressed beam: (Article 9.15.2.2 AASHTO, 2002)

$$S_t = .4f_c^{'} = 2,400 \text{ psi}$$

Allowable stress in bottom flange of prestressed beam: (Article 9.15.2.2 AASHTO, 2002)

$$S_b = -6\sqrt{f_c^{'}} = -464.8 \text{ psi}$$

$$S_{tNC} = \frac{I_{NC}}{h - y_{nc}} = \frac{125,400}{45 - 20.27} = 5,071 in^3$$

$$S_{tC} = \frac{I_C}{h - y_c} = \frac{402,900}{45 - 36.94} = 49,990 in^3$$

$$S_{bNC} = \frac{I_{NC}}{y_{nc}} = \frac{125,400}{20.27} = 6,186 in^3$$

$$S_{bC} = \frac{I_C}{y_c} = \frac{402,900}{36.94} = 10,907 in^3$$

E1-8.3.1 Stress at top of beam from prestressing and dead load

$$S_{tdl} = \frac{P_s}{A_b} - \frac{P_s e_{\max}}{S_{tNC}} + \frac{M_{dl_{nc}}}{S_{tNC}} + \frac{M_{dl_c}}{S_{tC}}$$
$$= \frac{736.5}{559.5} - \frac{736.5(12.595)}{5,071} + \frac{11,404}{5,071} + \frac{1,085}{49,990} = 1.758 ksi$$

E1-8.3.2 Stress at top of beam from live load (including impact and distribution factors)

$$S_{tll} = \frac{M_{ll_c}}{S_{tC}} = \frac{11,410}{49,990} = 0.228 ksi$$

E1-8.3.3 Stress at bottom of beam from prestressing and dead load

$$S_{bdl} = \frac{P_s}{A_b} + \frac{P_s e_{\max}}{S_{bNC}} - \frac{M_{dl_{nc}}}{S_{bNC}} - \frac{M_{dl_c}}{S_{bC}}$$
$$= \frac{736.5}{559.5} + \frac{736.5(12.595)}{6,186} - \frac{11,404}{6,186} - \frac{1,085}{10,907} = 0.873 ksi$$

E1-8.3.4 Stress at bottom of beam from live load (including impact and distribution factors)

$$S_{bll} = \frac{M_{ll_c}}{S_{bC}} = \frac{11,410}{10,907} = 1.046 ksi$$

E1-8.4 Shear Capacity (Evaluated 3.341 ft from either support)

$N_d = 6$ (number of draped strands)

$F_{ps} = 26.3$ kips per strand

$F_{se} = F_{ps} N_d = 158$ kips

$e_{max} = 12.595$ in

$e(3.341) = 11.07$ in

$P_s = 736.5$ kips (total pre-stress force)

$A_v = 2(0.31) = 0.62$ in^2(area of stirrups)

$d_v = 0.8d = 44.8$ in (shear depth)

$S = 11$ in (stirrup spacing)

$b_v = 7$ in (width of web)

$A_b = 559.5$ in (beam area)

$h_e = 8in$ (Draped height at end of beam)

$$w_{dl} = w_{coping} + w_{slab} + w_{girder}$$

$$M_{DL_{nc}}(3.341) = \left(w_{dl} \frac{L}{2} + \frac{P_d}{2} \right) 3.341 - w_{dl} \frac{3.341^2}{2} = 176 \text{ kip-ft}$$

$$V_s = \frac{A_v f_y d_v}{S} = 151.5 \text{ kips}$$

$$f_{pc} = \frac{P_s}{A_b} - \frac{P_s e(y_c - y_{nc})}{I_{NC}} + \frac{M_{dl}(y_c - y_{nc})}{I_{NC}} = 513 \text{ psi}$$

$$V_c = \left(3.5\sqrt{f'_{cGirder}} + 0.3 f_{pc}\right) b_v d_v = 133.3 \text{kips}$$

The draped length of the strands L_d for this beam is equal to half the span length plus the distance from the center of bearing to the end of the beam (5in).

$$L_d = \frac{L}{2} + 5in = 33.82\,ft$$

$$V_P = \sin\left(\tan^{-1}\left(\frac{h_e}{L_d} \right) \right) F_{se} = 3.11 \text{ kips}$$

$$V_n = V_c + V_s + V_p = 287.9 \text{ kips}$$

$$\phi V_N = 259 kips$$

E1-9 Exterior Beam Analysis

$A_{ps} = 4.676 \text{ in}^2$ (area of prestressing)

$f_s^{'} =$ 270 ksi (ultimate strength of prestressing)

$b_{deck} =$ 97.5 in (effective width of deck Article 9.9.1 AASHTO, 2002)

$h =$ 45 in (height of prestressed beam)

$h_{slab} =$ 9 in (height of slab)

$y_{nc} =$ 20.27 in (distance from centroid of beam to bottom of beam)

$y_c =$ 36.20 in (distance from centroid of composite beam and slab to bottom of beam)

$y_s =$ 7.679 in (distance from centroid of prestressing to bottom of beam)

$\beta_1 =$ .85 in (concrete strength factor)

A = 559.5 in^2 (area of beam)

$I_{NC} = 125,400 in^4$ (moment of inertia of beam)

$I_C = 390,400 in^4$ (moment of inertia of composite beam and slab)

$f_{ci} = 5,000 psi$ (compressive strength of concrete at time of initial prestressing)

$\gamma = 0.28$ (Lo-Lax Prestressing Factor)

e = Eccentricity of Prestressing strands

$e(31.07) = 12.426$ in

$e_{max} = e(33.406) = 12.595$ in

d = distance from extreme compression fiber to centroid of prestressing

$d = h + h_{slab} - (y_{nc} - e) =$ 46.15 in at 31.07 ft from the support

$d = h + h_{slab} - (y_{nc} - e) =$ 46.3 in at 33.406 ft from the support

E1-9.1 Moment Capacity

$\varphi = 1$

At 31.07 ft from support	At 33.406 ft from support
$f_{su}(31.07) = 262.87 ksi$	$f_{su}(33.406) = 262.89 ksi$
φMn(31.07)= 4,510 kip-ft	φMn(33.406) = 4,526 kip-ft

E1-9.2 Prestress Losses (Article 9.16 AASHTO, 2002)

Total non-composite dead load:

$$M_{dl_{nc}}(33.406) = 1.514\frac{(66.8125)^2}{8} + \frac{1.525}{2}\frac{66.8125}{2} = 10,749 \text{ kip-in}$$

Total composite dead load: (same as interior)

$$\mathrm{M}_{\mathrm{dl_c}}(33.406) = 1,085\ k - in$$

Dead load moment due to self weight of the girder: (same as interior)

$$M_g(33.406) = 3,904\ k - in$$

Shrinkage Losses: (same as interior)

$$SH = 6.5ksi$$

Elastic Shortening:
$E_s = 28 \times 10^6$ psi as per Article 9.16.2.1.2 of AASHTO Standard Specifications for Highway Bridge Design (AASHTO, 2002)). The prestressing force after initial losses P_{si} is required in order to compute the elastic shortening (ES), and relaxation of the prestressing steel CR_s; however, the prestressing force after initial losses is affected by ES and CR_s therefore a first approximation of P_{si} is taken as:

$$P_{si} = 0.9\left(0.75 f_s' A_{ps}\right) = 852.2kips$$

After computing ES and CR_s the initial prestressing force converges to:

$$P_{si} = A_{ps}\left(0.75 f_s' - ES - 0.3CR_s\right) = 874.3kips$$

"f_{cir} the average concrete stress at the c.g. of the prestressing steel at time of release" Article 9 AASHTO Standard Specification for Highway Bridge Design (AASHTO, 2002) is needed for the computation of ES.

$$\begin{aligned} f_{cir} &= -\frac{P_{si}}{A} - \frac{P_{si}ee}{I} - \frac{M_g e}{I} \\ &= -\frac{874.3}{559.5} - \frac{874.3(-12.595)^2}{125,400} - \frac{3,904(-12.595)}{125,400} = 2.277ksi \end{aligned}$$

$$ES = \frac{E_s}{E_{ci}} f_{cir} = \frac{28\times10^6}{33\left(150^{3/2}\right)\sqrt{5,000}}(2.277) = 14.9ksi$$

Creep of Concrete:
As per Article 9.16.2.1.3 of AASHTO Standard Specifications for Highway Bridge Design (AASHTO, 2002)):

f_{cds} is the "average concrete compressive stress at the c.g. of prestressing steel under full dead load." Article 9 AASHTO Standard Specification for Highway Bridge Design (AASHTO, 2002):

$$f_{cds} = \frac{\left(M_{DL_{nc}} - M_g\right)(e)}{I_{NC}} + \frac{M_{DL_c}\left(y_c - y_s\right)}{I_C} = \frac{6,845(12.595)}{125,400} + \frac{1,085(36.202 - 7.679)}{390,400} = 0.767ksi$$

$$CR_c = 12 f_{cir} - 7 f_{cds} = 12(2.227) - 7(0.767) = 21.4ksi$$

Relaxation of Prestressing Steel:
As per Article 9.16.2.1.4 of AASHTO Standard Specifications for Highway Bridge Design (AASHTO, 2002)):

$$CR_s = 5,000 - 0.1ES - 0.05(SH + CR_c)$$
$$= 5,000 - 0.1(14,900) - 0.05(6,500 + 21,400) = 2.1ksi$$

Total Prestressing Losses:

$$\Delta f_s = SH + ES + CR_c + CR_s = 6.5 + 14.9 + 21.5 + 2.1 = 45ksi$$

Stress after Initial Losses: (check for convergence)

$$P_{si} = \left(0.75 f_s^{'} - ES - 0.3CR_s\right) A_{ps} = \left(0.75(270) - 14.9 - 0.3(2.1)\right) 4.676 = 874.3kips$$

Stress after Final Losses:

$$P_s = \left(0.75 f_s^{'} - \Delta f_s\right) A_{ps} = \left(0.75(270) - 45\right) 4.676 = 736.5kips$$

E1-9.3 Serviceability Limit State: (allowable stress)

$$S_{tNC} = \frac{I_{NC}}{h - y_{nc}} = \frac{125,400}{45 - 20.27} = 5,071in^3$$
$$S_{tC} = \frac{I_C}{h - y_c} = \frac{390,400}{45 - 36.202} = 44,374in^3$$
$$S_{bNC} = \frac{I_{NC}}{y_{nc}} = \frac{125,400}{20.27} = 6,186in^3$$
$$S_{bC} = \frac{I_C}{y_c} = \frac{390,400}{36.202} = 10,784in^3$$

E1-9.3.1 Stress at top of beam from prestressing and dead load

$$S_{tdl} = \frac{P_s}{A_b} - \frac{P_s e_{\max}}{S_{tNC}} + \frac{M_{dl_{nc}}}{S_{tNC}} + \frac{M_{dl_c}}{S_{tC}}$$
$$= \frac{736.5}{559.5} - \frac{736.5(12.595)}{5,071} + \frac{10,749}{5,071} + \frac{1,085}{44,374} = 1.631ksi$$

E1-9.3.2 Stress at top of beam from live load (including impact and distribution factors)

$$S_{tll} = \frac{M_{ll_c}}{S_{tC}} = \frac{10,104}{44,374} = 0.228ksi$$

E1-9.3.3 Stress at bottom of beam from prestressing and dead load

$$S_{bdl} = \frac{P_s}{A_b} + \frac{P_s e_{\max}}{S_{bNC}} - \frac{M_{dl_{nc}}}{S_{bNC}} - \frac{M_{dl_c}}{S_{bC}}$$
$$= \frac{736.5}{559.5} + \frac{736.5(12.595)}{6,186} - \frac{10,749}{6,186} - \frac{1,085}{10,784} = 0.978ksi$$

E1-9.3.4 Stress at bottom of beam from live load (including impact and distribution factors)

$$S_{bll} = \frac{M_{ll_c}}{S_{bC}} = \frac{10,104}{10,784} = 0.937ksi$$

E1-9.4 Shear Capacity (Evaluated 3.341 ft from either support)

$N_d = 6$ (number of draped strands)

$F_{ps} = 26.3$ kips per strand

$F_{se} = F_{ps} N_d = 158$ kips

$e_{max} = 12.595$ in

$e(3.341) = 11.07$ in

$P_s = 736.5$ kips (total pre-stress force)

$A_v = 2(0.31) = 0.62$ in^2(area of stirrups)

$d_v = 0.8d = 44.8$ in (shear depth)

$S = 11$ in (stirrup spacing)

$b_v = 7$ in (web width)

$A_b = 559.5$ in (area of beam)

$h_e = 8in$ (Draped height at end of beam)

$$w_{dl} = w_{coping} + w_{slab} + w_{girder}$$

$$M_{DL_{nc}}(3.341) = \left(w_{dl}\frac{L}{2} + \frac{P_d}{2}\right)3.341 - w_{dl}\frac{3.341^2}{2} = 163 \text{ kip-ft}$$

$$V_s = \frac{A_v f_y d_v}{S} = 151.5 \text{ kips}$$

$$\mathrm{f_{pc}} = \frac{\mathrm{P_s}}{A_b} - \frac{P_s e\left(y_c - y_{nc}\right)}{I_{NC}} + \frac{M_{dl}\left(y_c - y_{nc}\right)}{I_{NC}} = 301\ \text{psi}$$

$$\mathrm{V_c} = \left(3.5\sqrt{\mathrm{f'_{cGirder}}} + 0.3\mathrm{f_{pc}}\right)\mathrm{b_v d_v} = 113.3\text{kips}$$

The draped length of the strands L_d for this beam is equal to half the span length plus the distance from the center of bearing to the end of the beam (5in).

$$L_d = \frac{L}{2} + 5in = 33.82\,ft$$

$$\mathrm{V_P} = \sin\left(\tan^{-1}\left(\frac{h_e}{L_d}\right)\right)\mathrm{F}_{se} = 3.11\ \text{kips}$$

$$V_n = V_c + V_s + V_p = 267.9\ \text{kips}$$

$$\phi V_N = 241 kips$$

E1-9 Rating Calculation (LFR)

Inventory Level

$\gamma_{DC} = 1.3$

$\gamma_{LL} = 2.17$

Operating Level

$\gamma_{DC} = 1.3$

$\gamma_{LL} = 1.3$

Table E1-9.1 Load Factor Rating (LFR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002))

Flexure (Interior beam) (at 31.07) $\phi = 1$	**Inventory Level**	$RF = \frac{\phi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \frac{4,542 - 1.3(1032)}{2.17(957)} = 1.54$	55.4 tons
	Operating Level	$RF = \frac{\phi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \frac{4,542 - 1.3(1032)}{1.3(957)} = 2.57$	92.5 tons
Flexure (Exterior beam) (at 31.07) $\phi = 1$	**Inventory Level**	$RF = \frac{\phi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \frac{4,510 - 1.3(978)}{2.17(842)} = 1.77$	63.7 tons
	Operating Level	$RF = \frac{\phi M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{LL+IM}} = \frac{4,510 - 1.3(978)}{1.3(842)} = 2.96$	106.6 tons
Shear (Interior beam) (3.341) $\phi = 0.9$	**Inventory Level**	$RF = \frac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \frac{259 - 1.3(54.9)}{2.17(60.3)} = 1.43$	51.5 tons
	Operating Level	$RF = \frac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \frac{259 - 1.3(54.9)}{1.3(60.3)} = 2.39$	<u>86.0 tons</u>
Shear (Exterior beam) (3.341) $\phi = 0.9$	**Inventory Level**	$RF = \frac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \frac{241 - 1.3(51.2)}{2.17(53.1)} = 1.51$	54.4 tons
	Operating Level	$RF = \frac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \frac{241 - 1.3(51.2)}{1.3(53.1)} = 2.52$	90.7 tons
Interior Beam Serviceability (at midspan)	**Top of Beam Inventory Level**	$RF_t = \frac{S_t - S_{tdl}}{S_{tll}} = \frac{2.4 - 1.758}{0.228} = 2.82$	101.5 tons
	Bottom of Beam Inventory Level	$RF_b = \frac{S_b - S_{bdl}}{-S_{bll}} = \frac{-.4648 - 0.873}{-1.046} = 1.28$	<u>46.1 tons</u>
Exterior Beam Serviceability (at midspan)	**Top of Beam Inventory Level**	$RF_t = \frac{S_t - S_{tdl}}{S_{tll}} = \frac{2.4 - 1.631}{0.228} = 3.37$	121.3 tons
	Bottom of Beam Inventory Level	$RF_b = \frac{S_b - S_{bdl}}{-S_{bll}} = \frac{-.4648 - 0.978}{-0.937} = 1.54$	55.4 tons

Example E2:

Rating by the Load and Resistance Factor Rating (LRFR) Using Load Distribution and Dynamic Allowance Factors Stipulated by AASHTO LRFD Bridge Design Specification (AASHTO, 2007).

E2-1 Basic Geometry and Bridge Information

Span: L = 66.8125 ft
Steel yield strength: f_y = 60 ksi
Materials:
Concrete strength: f'_c = 3.5 ksi (deck)
f'_c = 6 ksi (Prestressed beam)
Prestressing Steel: ½" dia. Special Lo-Lax Strands
A_{ps} = 0.162 in^2 per Strand
28 prestressing strands
Stirrups: Starting from end 2" space
7 # 5 spaced 6" O.C. over 3'-6"
11" space
3 # 5 spaced 11" O.C. over 2'-9"
14 # 5 spaced 1'-10" O.C. over 25'-8"
Clear space to midspan
Condition: NBI item 59 code = 7
ADTT: 980
Skew: 0 degrees

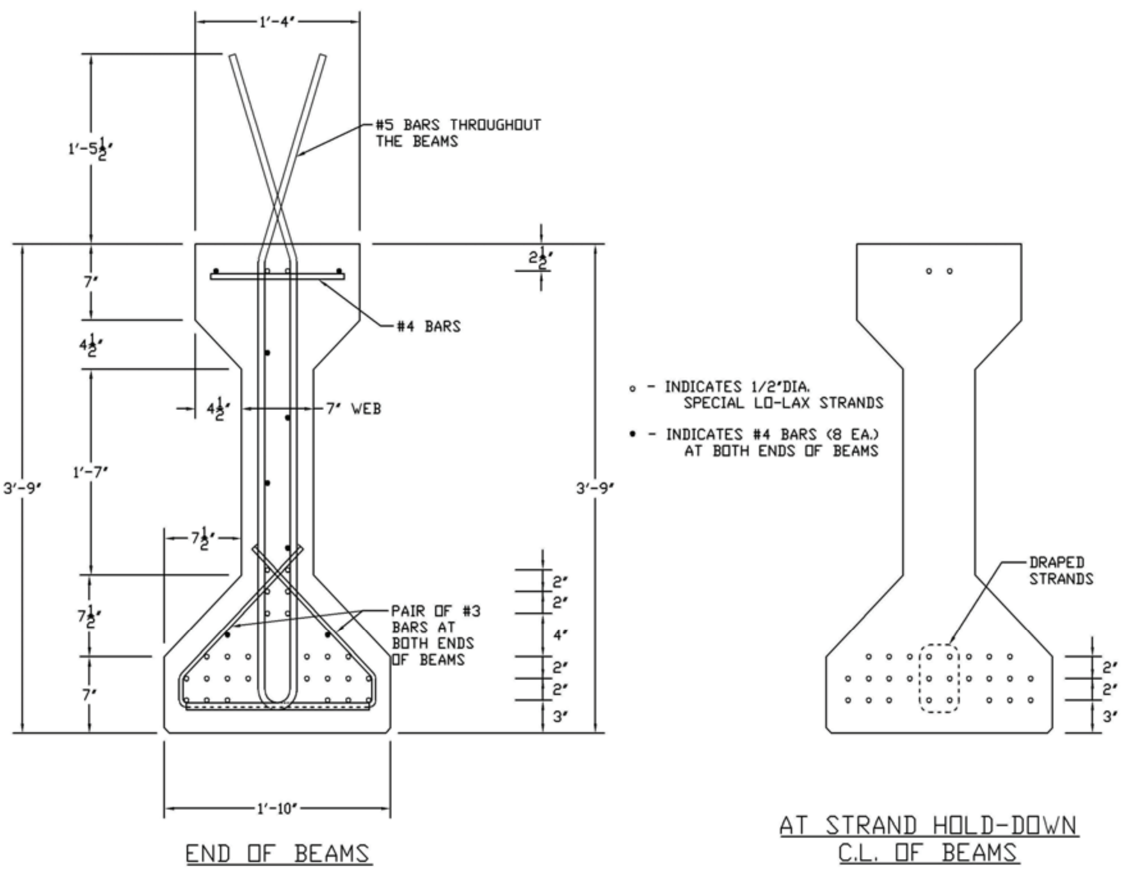


Figure E2-1.1 Beam Reinforcement Details

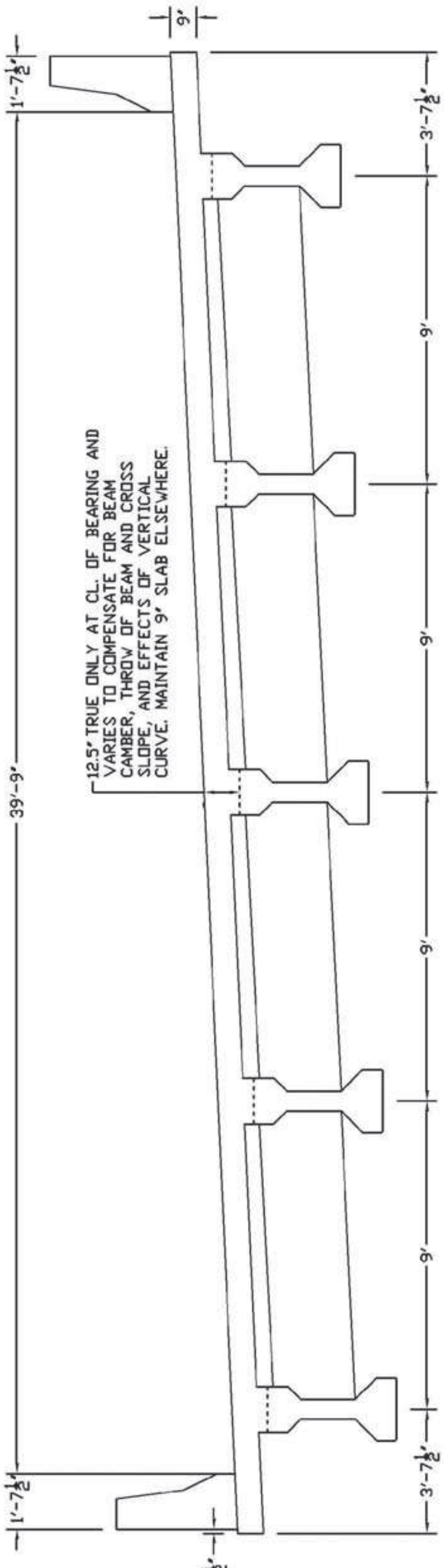


Figure E2-1.2 Bridge Cross Section

E2-2 LOADS

E2-2.1 Dead load interior beam

Weight per linear foot of the reinforced concrete slab	= 1.012 k/ft
Weight per linear foot of beams	= 0.583 k/ft
Weight of parapet, rail and sidewalk assembly	= 0.162 k/ft
Weight per linear foot of coping	= 0.017 k/ft
Since there is no wearing surface present on the bridge, DW	= 0
Total dead load for interior beam	**= 1.774 k/ft**

Weight of Diaphragm $P_d = 3.05 kips$

E2-2.1 Dead load interior beam

Weight per linear foot of the reinforced concrete slab	= 0.914 k/ft
Weight per linear foot of beams	= 0.583 k/ft
Weight of parapet, rail and sidewalk assembly	= 0.162 k/ft
Weight per linear foot of coping	= 0.017 k/ft
Since there is no wearing surface present on the bridge, DW	= 0
Total dead load for interior beam	**= 1.676 k/ft**

Weight of Diaphragm $P_d = 1.525 kips$

E2-3 Dynamic Load Allowance

From Table 3.6.2.1 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007), the dynamic load allowance is taken as 33%. Thus, the dynamic load factor to be applied to the static load is:

$$\left(1+\frac{IM}{100}\right)=1.33$$

E2-4 Live Load Distribution Factors

E2-4.1 Moment distribution factors

E2-4.1.1 Distribution Factor for ***moment*** in interior longitudinal beams

As per Table 4.6.2.2.2b-1 of AASHTO LRFD Bridge Design Specifications (AASHTO, 2007), the distribution factor for moment in interior beams, g_m , is specified as follows

When one lane is loaded: $$g_{m1} = 0.06 + \left(\frac{s}{14}\right)^{0.4} + \left(\frac{s}{L}\right)^{0.3}\left(\frac{k_g}{12Lt_s^3}\right)^{0.1}$$

When two or more lane are loaded: $$g_{m2} = 0.075 + \left(\frac{s}{9.5}\right)^{0.6} + \left(\frac{s}{L}\right)^{0.2}\left(\frac{k_g}{12Lt_s^3}\right)^{0.1}$$

The distribution factor for moment in exterior beams, g_m, is specified as follows

In the case of one lane loaded g_m is computed by the lever rule
In the case of two or more lane loaded:
The longitudinal stiffness parameter:

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

In which $n = \frac{E_B}{E_D}$ where

E_B = modulus of elasticity of the beam material
E_D = modulus of elasticity of the deck material
e_g = the distance between the centers of gravity of the beams and deck
I = moment of inertia of the beam
A = area of beam

$$n = \frac{E_B}{E_D} = 1$$

Single lane loaded $$g_{m1} = .6 + \left(\frac{s}{14}\right)^{.4}\left(\frac{s}{L}\right)^{.3}\left(\frac{K_g}{12Lt_s{}^3}\right)^{.1} = .611$$

Two lanes loaded $$g_{m2} = .075 + \left(\frac{s}{9.5}\right)^{.6}\left(\frac{s}{L}\right)^{23}\left(\frac{K_g}{12Lt_s{}^3}\right)^{.1} = .853$$

$$g_m = \max(g_{m1}, g_{m2}) = .853$$

E2-4.1.1 Distribution Factor for ***moment*** in exterior longitudinal beams

The distribution factor for moment in exterior beams is specified in Table 4.6.2.2.2d-1 of AASHTO LRFD Bridge Design Specifications (2007) as follows:

- When one design lane is loaded, the lever rule is used to determine the distribution factor, g_m
- When two or more lanes are loaded, the distribution factor is computed by multiplying the distribution factor for moment in interior beam by a correction factor, e, defined as

$$e = 0.77 + \frac{d_e}{9.1}$$

Where d_e is the distance from the exterior web of the exterior beam to the interior edge of the curb of traffic barrier.

For computing the distribution factor by the lever rule, a simple structural member such as the one shown below is analyzed

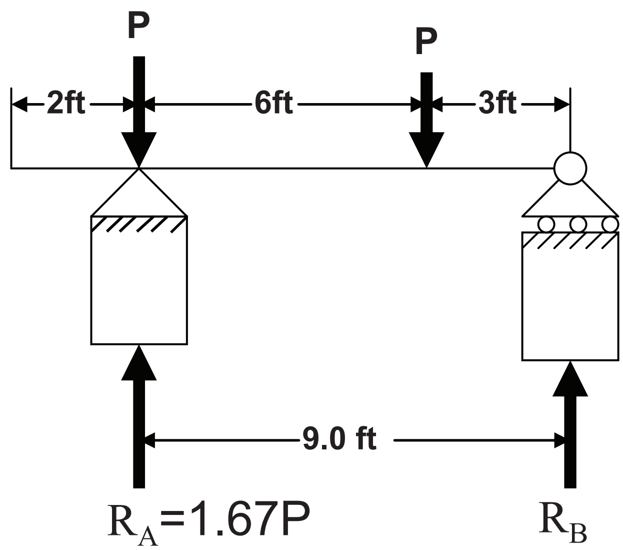


Figure E2-4.1 Exterior Girder Distribution Factor

Article 3.6.1.1.2 of AASHTO LRFD Bridge Design Specification (AASHTO, 2007) states that a multiple presence factor m = 1.20 must be used when computing girder distribution factors by the lever rule. Thus, when one lane is loaded the distribution factor for the moment in exterior beams is:

$$g_{m1(exterior)} = m\frac{1.67P}{2P} = 1.2(0.835) = 1.00$$

When two or more lanes are loaded:

$$e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{(2.0 - 0.29)}{9.1} = 0.96$$

$$g_{m2(exterior)} = e\, g_{m(interior)} = (0.96)(0.853) = 0.819$$

$$g_{m(exterior)} = \max\left(g_{m1(exterior)}, g_{m2(exterior)}\right) = \max(1.00, 0.819) = 1.00$$

E2-4.2 Shear distribution factors

E2-4.2.1 Distribution Factor for ***shear*** in interior longitudinal beams

The distribution factor for shear in interior beams is specified in Table 4.6.2.2.3a-1 of AASHTO LRFD Bridge Design Specifications (AASHT0, 2007) as follows

Single lane loaded $$g_{v1} = .36 + \left(\frac{s}{25}\right) = .72$$

Two lanes loaded $$g_{v2} = .2 + \left(\frac{s}{12}\right) - \left(\frac{s}{35}\right)^2 = .884$$

$$g_v = \max(g_{v1}, g_{v2}) = .884$$

E2-4.2.2 Distribution Factor for ***shear*** in exterior longitudinal beams

When one lane is loaded, the distribution factor for shear is computed by the lever rule.
Thus, $g_{v1(exterior)} = 1.00$

When two or more lanes are loaded

$$g_{v2(exterior)} = (e)\, g_{v(interior)} = \left(0.6 + \frac{d_e}{10}\right) g_{v(interior)}$$

$$= \left(0.66 + \frac{2.125 - 0.75}{10}\right)(0.76) = (0.738)(0.76) = 0.61$$

$$g_{v(exterior)} = \max\left(g_{v1(exterior)}, g_{v2(exterior)}\right) = \max(1.00, 0.61) = 1.00$$

E2-4.3 Summary of distribution factors

Table E2-4.1 Distribution Factors

Action	Interior Beam	Exterior Beam
Bending Moment	0.853	1.00
Shear	0.884	1.00

E2-5 Flexural Analysis

E2-5.1 Live load bending moment

E2-5.1.1 Interior beam HS20 load:

From Section E2-5 when P = 32 kips, $g_m = 0.853$, and IM =1.33, the HS20 live load M_{LL+IM} can be computed at the location 31.07 ft from the left support to be:

$$M_{LL+IM} = 1,053\ k - ft$$

E2-5.1.2 Interior beam HL93 load: (Max of Design Truck and Tandem Truck Load cases)

Design Truck: HS20 load, w = 0.64 kips/ft

$$DesignTruck(31.07) = M_{LL+IM} + g_m \left[21.38(31.07) - w\frac{(31.07)^2}{2} \right]$$

$$= 1,053 + 0.853\left[21.38(31.07) - 0.64\frac{(31.07)^2}{2} \right] = 1,356k - ft$$

Tandem Truck: P = 25 kips, w = 0.64 kips/ft

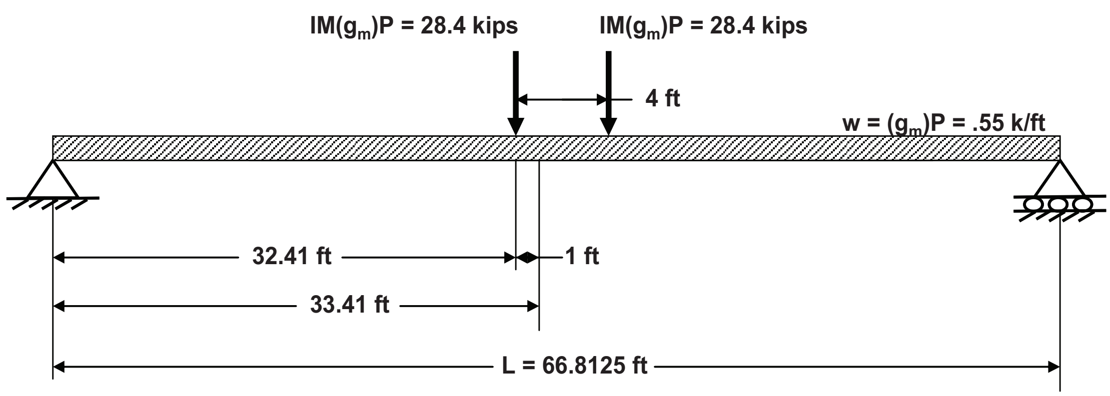


Figure E2-5.1 Maximum Design Tandem Moment

$$DesignTandem(32.41) = 27.5\left(32.41\right) + \left[18.24(32.41) - 0.55\frac{(32.41)^2}{2} \right] = 1,194k - ft$$

$$M_{LL+IM(HL93)} = \max\left(DesignTruck, DesignTandem\right) = 1,356k - ft$$

E2-5.1.3 Maximum dead load moment for an interior beam (Section E1-5)

$$M_D\left(31.07\right) = 1,032\ k - ft$$

$$M_D\left(33.4\right)=1,041\ k-ft$$

E2-5.1.4 Exterior beam HS20 load:

From Section E2-5 when P = 32 kips, $g_m = 1.0$, and IM =1.33, the HS20 live load M_{LL+IM} can be computed at the location 31.07 ft from the left support to be:

$$M_{LL+IM} = 1,234\ k-ft$$

E2-5.1.2 Exterior beam HL93 load: (Max of Design Truck and Tandem Truck Load cases)

Design Truck: HS20 load, w = 0.64 kips/ft

$$DesignTruck(31.07) = M_{LL+IM} + g_m\left[21.38(31.07) - w\frac{(31.07)^2}{2}\right]$$
$$= 1,234 + 1.0\left[21.38(31.07) - 0.64\frac{(31.07)^2}{2}\right] = 1,589\ k-ft$$

Tandem Truck: P = 25 kips, w = 0.64 kips/ft

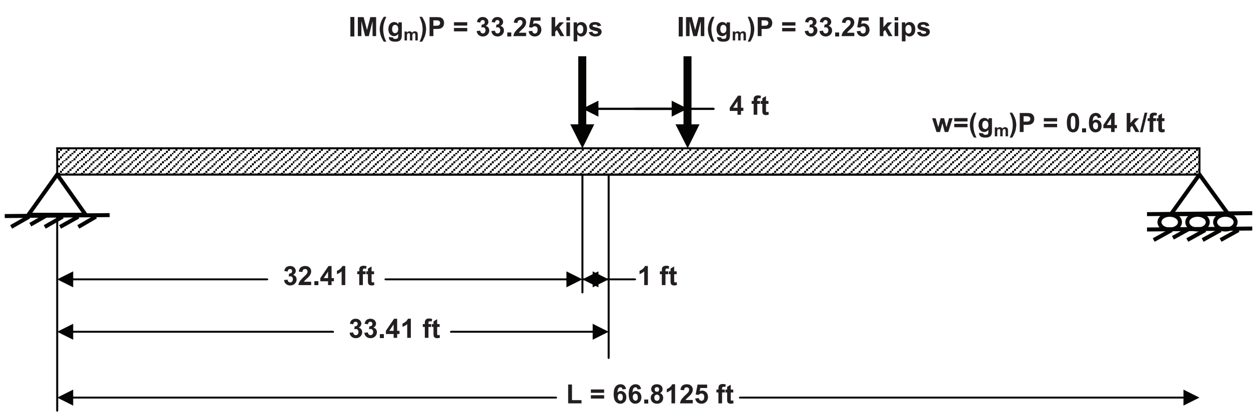


Figure E2-5.2 Maximum Design Tandem Moment

$$DesignTandem(32.41) = 32.3\left(32.41\right) + \left[21.38(32.41) - 0.64\frac{(32.41)^2}{2}\right] = 1,404k-ft$$

$$M_{LL+IM(HL93)} = \max\left(DesignTruck, DesignTandem\right) = 1,589k-ft$$

E2-5.1.4 Maximum dead load moment for an exterior beam (Section E1-5)

$$M_D\left(31.07\right) = 978\ k-ft$$
$$M_D\left(33.4\right) = 986\ k-ft$$

E2-6 Shear Force Analysis

To evaluate Shear in the Prestressed Girders the span was broken up into 20 segments and evaluated at each to determine where the critical shear was encountered. The location of this critical shear was found to be 3.341 ft in from the girder support.

E2-6.1 Interior Beam Shear Analysis

E2-6.1.1 HS20 load:

$$\mathrm{V}_{\mathrm{HS20}}(3.341) = 68.6 kips \text{ (Section E1-6)}$$

E2-6.1.2 HL93 load: (Max of Design Truck and Tandem Truck Load cases)

$$DesignTruck(3.341) = V_{HS20} + g_v w(34.41 - 3.341) = 68.6 + 0.884(0.64)(34.41 - 3.341) = 86.2 \ kips$$

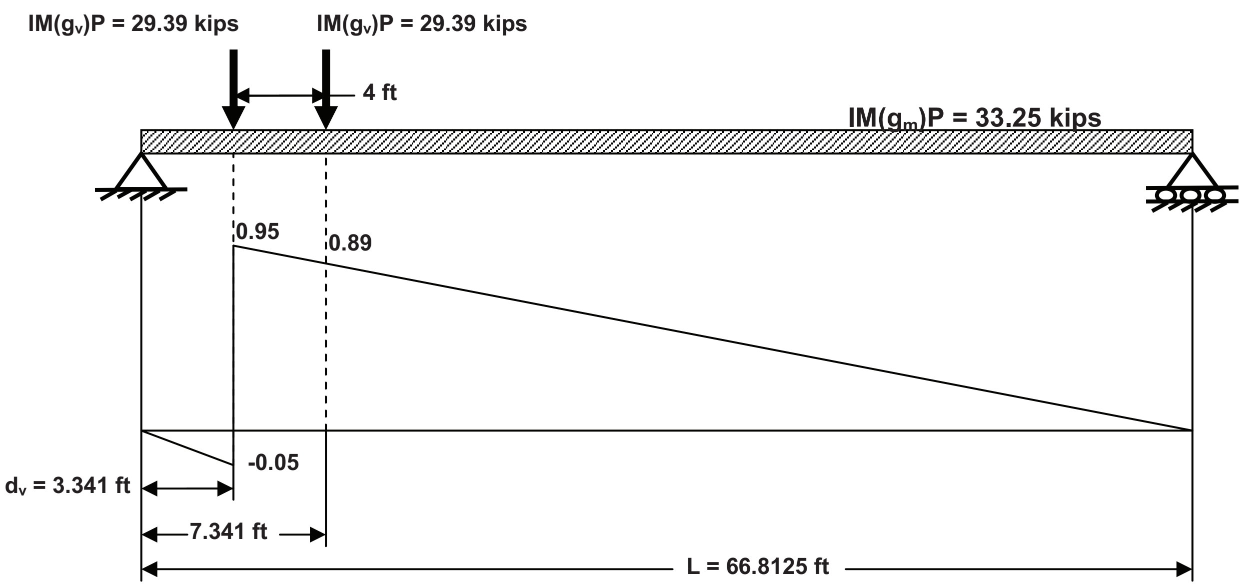


Figure E2-1.1 Interior Beam Shear Design Tandem at 3.341 ft

$$DesignTandem(3.341) = 29.39(.95) + 29.39(.89) + 0.566(34.41 - 3.341) = 71.7 \ kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 86.2 kips$$

E2-6.1.3 Interior beam dead load shear force at 3.341 ft (Section E1-6)

$$\mathrm{V}_{\mathrm{dc}}(3.341) = 54.9\mathrm{kips}$$

E2-6.2 Exterior Beam Shear Analysis

E2-6.1.1 HS20 load:

$$V_{HS20}\left(3.341\right) = 77.6kips \text{ (Section E1-6)}$$

E2-6.1.2 HL93 load: (Max of Design Truck and Tandem Truck Load cases)

$$DesignTruck\left(3.341\right) = V_{HS20} + g_v w\left(34.41-3.341\right) = 77.6 + 1.00\left(0.64\right)\left(34.41-3.341\right) = 96.8\ kips$$
$$DesignTandem\left(3.341\right) = 33.25\left(.95\right) + 33.25\left(.89\right) + 0.64\left(34.41-3.341\right) = 79.2\ kips$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 96.8kips$$

E2-6.2.2 Exterior beam dead load shear force at 3.341 ft (Section E1-6)

$$V_{dc}(3.341)=51.2\text{kips}$$

E2-7 Interior Beam Analysis

E2-7.1 Moment Capacity (Section E1-8)

$$\varphi = 1$$

At 31.07 ft from support	At 33.406 ft from support
φMn(31.07)= 4,542 kip-ft	φMn(33.406) = 4,558 kip-ft

E2-7.2 Prestress Losses

Total Prestressing Losses:

$$\Delta f_s = 45ksi \text{ (Section E1-8)}$$

Stress after Initial Losses: (check for convergence)

$$P_{si} = 874.3kips \text{ (Section E1-8)}$$

Stress after Final Losses:

$$P_s = 736.5kips \text{ (Section E1-8)}$$

E2-7.3 Serviceability Limit State: (allowable stress)

Allowable stress in top flange of prestressed beam: (Article 9.15.2.2 AASHTO, 2002)

$$\mathrm{S_t} = .4\mathrm{f_c'} = 2{,}400 \text{ psi}$$

Allowable stress in bottom flange of prestressed beam: (Article 9.15.2.2 AASHTO, 2002)

$$\mathrm{S_b} = -6\sqrt{\mathrm{f_c'}} = \text{-}464.8 \text{ psi}$$

$$S_{tNC} = 5{,}071 in^3$$
$$S_{tC} = 49{,}990 in^3$$
(Section E1-8)

$$S_{bNC} = 6{,}186 in^3$$
$$S_{bC} = 10{,}907 in^3$$
(Section E1-8)

E2-7.3.1 Stress at top of beam from prestressing and dead load

$$S_{tdl} = 1.758 ksi$$ (Section E1-8)

E2-7.3.2 Stress at top of beam from live load (including impact and distribution factors)

$$S_{tll} = \frac{M_{ll_c}}{S_{tC}} = \frac{16{,}272}{49{,}990} = 0.326 ksi$$

E2-7.3.3 Stress at bottom of beam from prestressing and dead load

$$S_{bdl} = 0.873 ksi$$ (Section E1-8)

E2-7.3.4 Stress at bottom of beam from live load (including impact and distribution factors)

$$S_{bll} = \frac{M_{ll_c}}{S_{bC}} = \frac{16{,}272}{10{,}907} = 1.492 ksi$$

E2-7.4 Shear Capacity (Evaluated 3.341 ft from either support)

$$V_s = 151.5 \text{ kips (Section E1-8)}$$
$$V_c = 133.3\text{kips (Section E1-8)}$$
$$V_P = 3.11 \text{ kips (Section E1-8)}$$

$$V_n = V_c + V_s + V_p = 287.9 \text{ kips}$$
$$\phi V_N = 259 kips$$

E2-8 Exterior Beam Analysis

E2-8.1 Moment Capacity (Section E1-8)

At 31.07 ft from support

$\varphi = 1$

φMn(31.07)= 4,510 kip-ft

At 33.406 ft from support

$\varphi = 1$

φMn(33.406) = 4,526 kip-ft

E2-8.2 Prestress Losses (at mid span)

Total non-composite dead load:

$$M_{dl_{nc}}(33.406) = 10{,}749 \text{ kip-in (Section E1-9)}$$

Total composite dead load: (same as interior)

$$M_{dl_c}(33.406) = 1{,}085\ k-in \quad \text{(Section E1-9)}$$

Dead load moment due to self weight of the girder: (same as interior)

$$M_g(33.406) = 3{,}904\ k-in \quad \text{(Section E1-9)}$$

Stress after Initial Losses: (check for convergence)

$$P_{si} = 874.3 kips \quad \text{(Section E1-9)}$$

Stress after Final Losses:

$$P_s = 736.5 kips \quad \text{(Section E1-9)}$$

E2-8.3 Serviceability Limit State: (allowable stress)

$$S_{tNC} = 5,071 in^3$$
$$S_{tC} = 44,374 in^3$$
(Section E1-9)

$$S_{bNC} = 6,186 in^3$$
$$S_{bC} = 10,784 in^3$$
(Section E1-9)

E2-8.3.1 Stress at top of beam from prestressing and dead load

$$S_{tdl} = 1.631 ksi \quad \text{(Section E1-9)}$$

E2-8.3.2 Stress at top of beam from live load (including impact and distribution factors)

$$S_{tll} = \frac{M_{ll_c}}{S_{tC}} = \frac{19,068}{44,374} = 0.430 ksi$$

E2-8.3.3 Stress at bottom of beam from prestressing and dead load

$$S_{bdl} = 0.978 ksi \quad \text{(Section E1-9)}$$

E2-8.3.4 Stress at bottom of beam from live load (including impact and distribution factors)

$$S_{bll} = \frac{M_{ll_c}}{S_{bC}} = \frac{19,068}{10,784} = 1.768 ksi$$

E2-8.4 Shear Capacity (Evaluated 3.341 ft from either support)

$$\mathrm{V_s} = 151.5 \text{ kips (Section E1-9)}$$
$$\mathrm{V_c} = 113.3 \text{kips (Section E1-9)}$$
$$\mathrm{V_P} = 3.11 \text{ kips (Section E1-9)}$$
$$V_n = V_c + V_s + V_p = 267.9 \text{ kips}$$
$$\phi V_N = 241 kips$$

E2-9 Rating Calculation (LFR)

Inventory Level	Operating Level	Legal Level
$\gamma_{DC} = 1.25$	$\gamma_{DC} = 1.25$	$\gamma_{DC} = 1.25$
$\gamma_{LL} = 1.75$	$\gamma_{LL} = 1.35$	$\gamma_{LL} = 1.55(ADTT = 655)$

Table E2-9.1 Load and Resistance Factor Rating (LFR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007))

Flexure (Interior girder) $\phi = 1$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level**	$RF = \dfrac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \dfrac{(1)4,542 - 1.25(1,032)}{1.75(1,356)} = 1.37$	49.3 tons
	Operating Level	$RF = \dfrac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \dfrac{(1)4,542 - 1.25(1,032)}{1.35(1,356)} = 1.78$	64.1 tons
	Legal Level	$RF = \dfrac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HS20}} = \dfrac{(1)4,542 - 1.25(1,032)}{1.55(1,053)} = 1.99$	71.6 tons
Flexure (Exterior girder) $\phi = 1$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level**	$RF = \dfrac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \dfrac{(1)4,510 - 1.25(978)}{1.75(1,589)} = 1.18$	42.5 tons
	Operating Level	$RF = \dfrac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \dfrac{(1)4,510 - 1.25(978)}{1.35(1,589)} = 1.53$	55.1 tons
	Legal Level	$RF = \dfrac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HS20}} = \dfrac{(1)4,510 - 1.25(978)}{1.55(1,234)} = 1.72$	61.9 tons
Shear (Interior Girder at 3.341 ft) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level**	$RF = \dfrac{\phi\phi_c\phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}} = \dfrac{(0.9)287.9 - 1.25(54.9)}{1.75(86.2)} = 1.26$	45.4 tons
	Operating Level	$RF = \dfrac{\phi\phi_c\phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}} = \dfrac{(0.9)287.9 - 1.25(54.9)}{1.35(86.2)} = 1.64$	59.0 tons
	Legal Level	$RF = \dfrac{\phi\phi_c\phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \dfrac{(0.9)287.9 - 1.25(54.9)}{1.55(68.6)} = 1.79$	64.4 tons

Shear (Exterior Girder at 3.341 ft) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level**	$RF = \frac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}} = \frac{(0.9)267.9 - 1.25(51.2)}{1.75(96.8)} = 1.05$	37.8 tons
	Operating Level	$RF = \frac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}} = \frac{(0.9)267.9 - 1.25(51.2)}{1.35(96.8)} = 1.36$	**49.0 tons**
	Legal Level	$RF = \frac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HS20}} = \frac{(0.9)267.9 - 1.25(51.2)}{1.55(77.6)} = 1.47$	**52.9 tons**
Interior Beam Serviceability (at midspan)	**Top of Beam Inventory Level**	$RF_t = \frac{S_t - S_{tdl}}{S_{tll}} = \frac{2.4 - 1.758}{0.326} = 1.97$	70.9 tons
	Bottom of Beam Inventory Level	$RF_b = \frac{S_b - S_{bdl}}{-S_{bll}} = \frac{-.4648 - 0.873}{-1.492} = 0.90$	32.4 tons
Exterior Beam Serviceability (at midspan)	**Top of Beam Inventory Level**	$RF_t = \frac{S_t - S_{tdl}}{S_{tll}} = \frac{2.4 - 1.631}{0.430} = 1.79$	64.4 tons
	Bottom of Beam Inventory Level	$RF_b = \frac{S_b - S_{bdl}}{-S_{bll}} = \frac{-.4648 - 0.978}{-1.768} = 0.82$	**29.5 tons**

Rating of a Steel Girder Bridge (GDOT BRIDGE ID # 085-0018)

Example F1:

Rating by the Allowable Stress Method (ASR) Using Load Distribution and Dynamic Allowance Factors Stipulated by AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002).

F1-1 Basic Geometry and Bridge Information

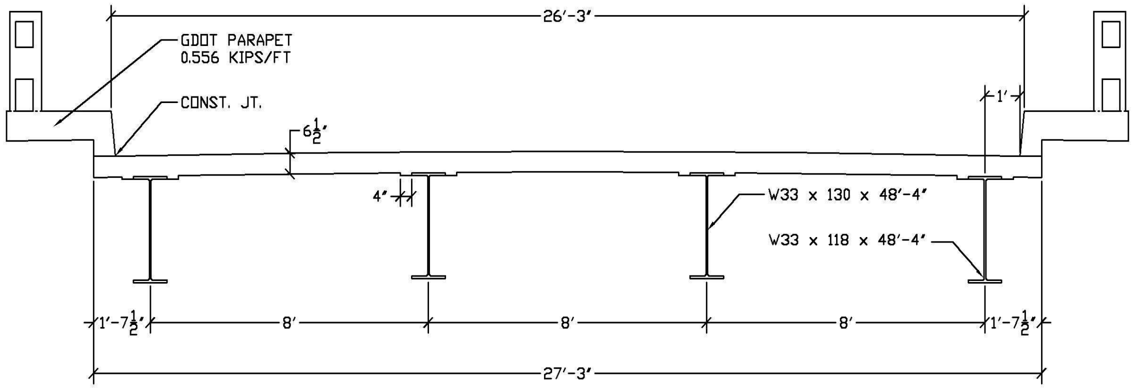


Figure F1-1.1 Bridge Cross Section at Mid-span

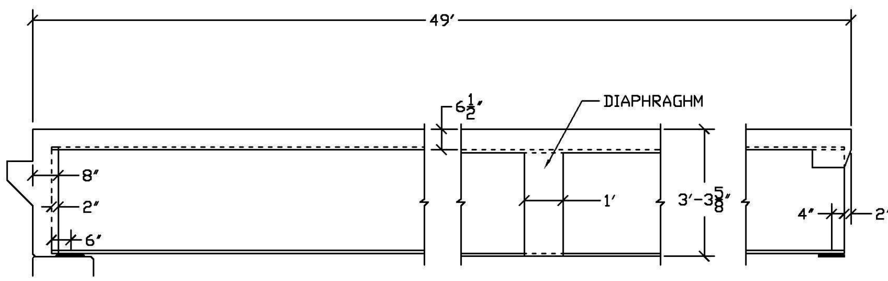


Figure F1-1.2 Girder Details

Unit weight of concrete:

$$w_c = .15\, kips/ft^3$$

Weight of steel beam per foot of length:

Interior Girder: $w_s = .13\, kips/ft$

Exterior Girder: $w_s = .118\, kips/ft$

Weight per ft of standard GDoT rail and sidewalk: $w_{pr} = .556\, kips/ft$

F1-2 LOADS

F1-2.1 Permanent Loads

As per Article 3.23.2.3.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002) the dead load supported by the outside stringers or beams shall be the portion of the floor slab carried by the stringer or beam. Curbs, railings, and wearing surfaces if placed after the slab has cured, may be distributed equally to all roadway girders.

F1-2.1.1 Interior girder loads

Weight per linear foot of the reinforced concrete slab = $\left(\frac{6.5}{12}\right)(8)(0.150)$ = 0.650 k/ft

Weight per linear foot of cast-in-place beams = (0.130) = 0.130 k/ft

Weight of parapet, rail and sidewalk assembly = $\frac{(2)(0.556)}{4}$ = 0.278 k/ft

Weight per linear foot of diaphragm = $\frac{(1)(24)\left(\frac{33.125}{12}\right)(0.15)}{4(48)}$ = 0.052 k/ft

Since there is no wearing surface present on the bridge, DW = 0

Total dead load for interior beam = 1.11 k/ft

F1-2.1.2 Exterior girder loads

Weight per linear foot of the reinforced concrete slab = $\left(\frac{6.5}{12}\right)(5.625)(0.150)$ = 0.457 k/ft

Weight per linear foot of cast-in-place beams = (0.118) = 0.118 k/ft

Weight of parapet, rail and sidewalk assembly = $\frac{(2)(0.556)}{4}$ = 0.278 k/ft

Weight per linear foot of diaphragm = $\frac{(1)(24)\left(\frac{33.125}{12}\right)(0.15)}{4(48)}$ = 0.052 k/ft

Since there is no wearing surface present on the bridge, DW = 0

Total dead load for exterior beam = 0.91 k/ft

F1-2.2 Vehicular Live Load

The design vehicular live load on the bridge consists of AASHTO HS20 truck with the spacing between the two 32-kip rear-axle loads to be varied from 14 ft to 30 ft to produce extreme force effects. The HS 20 truck is shown below.

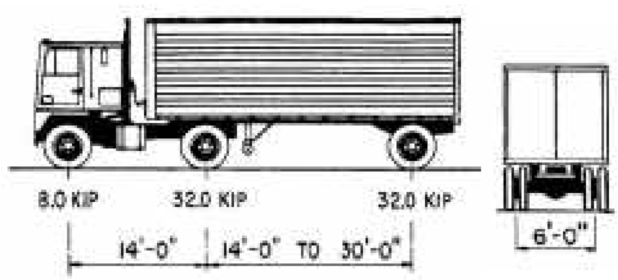


Figure B-2.1 AASHTO HS20 Truck

F1-3 Dynamic Load Allowance

Article 3.8.2.1 of the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002), the dynamic load allowance is taken as:

$$I = \frac{50}{L+125} = \frac{50}{48+125} = 0.29 \leq 0.3$$

Where:
I = impact factor (maximum 30 percent)
L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member

F1-4 Live Load Distribution Factors

F1-4.1 Interior Beam Moment distribution factors

As per Table 3.23.1 of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002), the distribution factors for moment in interior and exterior beams are computed as:

$$DFM = \frac{S}{5.5} = \frac{8}{5.5} = 1.45$$

F1-4.2 Exterior Beam Moment distribution factors

Using the structure shown in Figure F1.4.1 the moment distribution factor is calculated by computing the reaction at R_B, but not taken less than (Article 3.23.2.3.1 AASHTO 2002):

$$\frac{S}{4.0+0.25S} = \frac{8}{4+0.25(8)} = 1.33$$

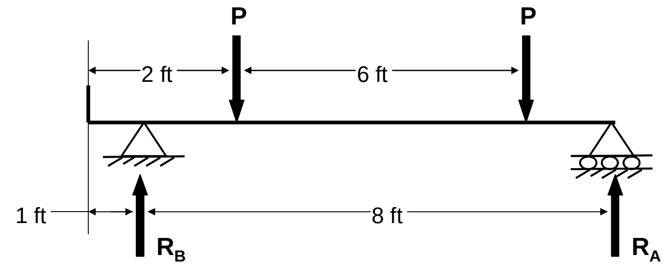


Figure F1-4.1 Exterior Girder Shear Distribution Factor

$$\sum M_A = 0$$
$$R_B(8) - P(7) - P(1) = 0\,.$$
$$R_B = 1.0P$$

Thus, the distribution factor for shear in an exterior beam is:

$$DFM = 1.33$$

F1-4.3 Summary of distribution factors

Table F1-4.1 Distribution Factors

Action	Interior Beam	Exterior Beam
Bending Moment	1.45	1.33

F1-5 Flexural Analysis

F1-5.1 Maximum live load bending moment

A rudimentary structural analysis of a simply supported beam subjected to a vehicular load having two rear axles and one front axle as shown in Figure B1-5.1 shows that the absolute maximum moment occurs under the middle axle when such an axle is positioned at a distance of 2.33 ft to the left of the beam centerline. By applying the dynamic allowance factor and the distribution factor for moment of interior beams, we can then compute the maximum live load under the following loads:

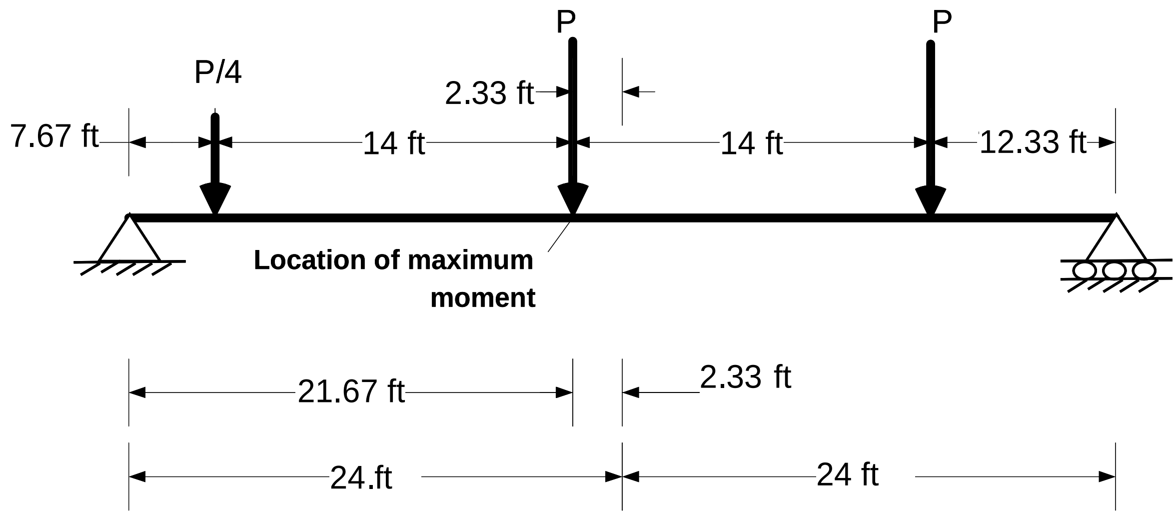


Figure F1-5.1 Max Live Load Moment

F1-5.1.1 Maximum live load moment for an interior beam

With P = 16 kips, $DFM = 1.45$, and IM =1.29, M_{LL+IM} can be computed at the location of the maximum live load moment (at 21.67 ft from the left support) to be:

$$M_{LL+IM} = 553.8k - ft$$

Live load moment (at midspan):

$$M_{LL+IM} = 537.5k - ft$$

F1-5.1.2 Maximum live load moment for an exterior beam

With P = 16 kips, $DFM = 1.33$, and IM =1.29, M_{LL+IM} can be computed at the location of the maximum live load moment (at 21.67 ft from the left support) to be:

$$M_{LL+IM} = 508.0k - ft$$

Live load moment (at midspan):

$$M_{LL+IM} = 493.0k - ft$$

F1-5.2 Maximum dead load moment

F1-5.2.1 Maximum dead load moment for an interior beam

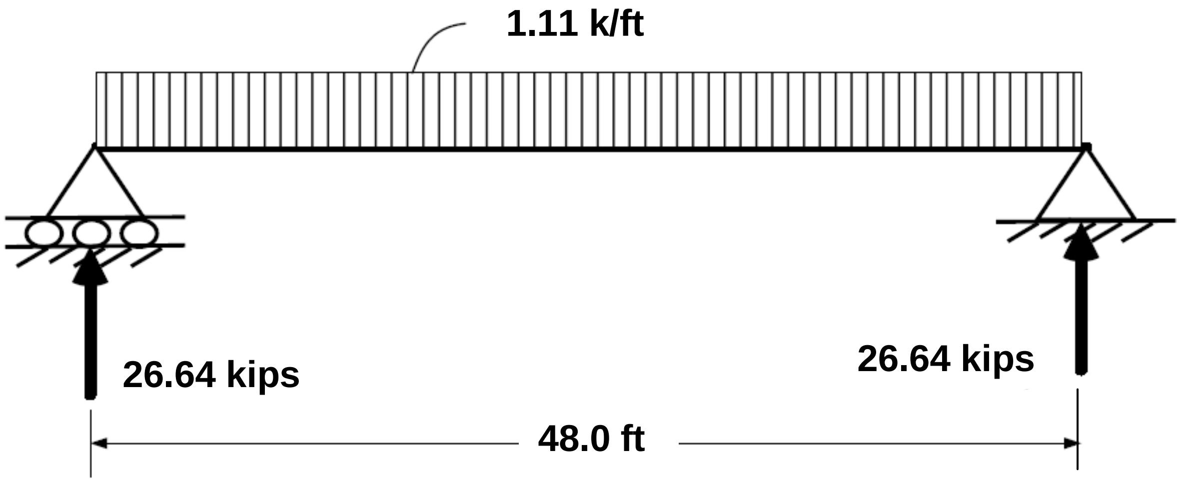


Figure F1-5.3 Interior Girder Dead Load Moment

The dead load moment at the location where the live load produces the maximum effect is

$$M_D = 26.64(21.67) - 1.11\frac{(21.67)^2}{2} = 316.7k - ft$$

The dead load moment at midspan is:

$$M_{D(m)} = 1.11\frac{(48)^2}{8} = 319.7k - ft$$

F1-5.2.2 Maximum dead load moment for an exterior beam

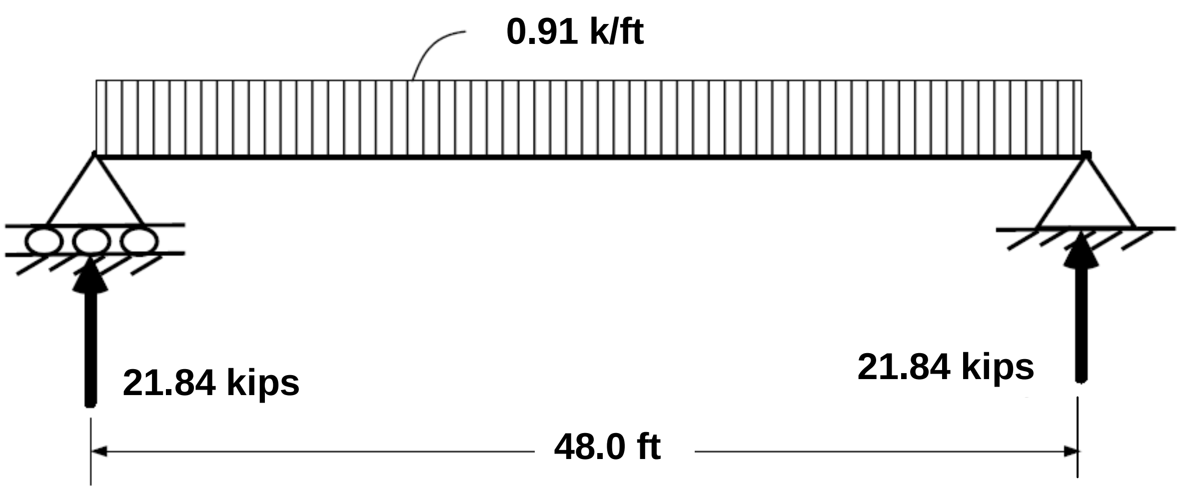


Figure F1-5.4 Exterior Girder Dead Load Moment

The dead load moment at the location where the live load produces the maximum effect is

$$M_D = 21.84(21.67) - 0.91\frac{(21.67)^2}{2} = 259.6k - ft$$

The dead load moment at midspan is:

$$M_{D(m)} = 0.91\frac{(48)^2}{8} = 262.1k - ft$$

F1-6 Load Combination

F1-6.1 Interior Girder

Table B1-6.1 shows that the governing moment loading case occurs at the maximum live load moment location (21.67 ft from the support).

Table F1-6.1 Interior Girder Load Combinations

	Moment at 24 ft from support	**Moment at 21.67 ft from support**
M_{LL+IM}	537.5 k-ft	553.8 k-ft
M_{DL}	319.7 k-ft	316.7 k-ft
M_{DL}+ M_{LL+IM}	857.2 k-ft	870.5 k-ft

F1-6.2 Exterior Girder

Table B1-7.2 shows that the governing moment loading case occurs at the maximum live load moment location (21.67 ft from the support).

Table F1-6.2 Exterior Girder Load Combinations

	Moment at 24 ft from support	**Moment at 21.67 ft from support**
M_{LL+IM}	493.0 k-ft	508.0 k-ft
M_{DL}	262.1 k-ft	259.6 k-ft
M_{DL}+ M_{LL+IM}	755.1 k-ft	767.6 k-ft

F1-7 Member Capacity

Table D.7.1 Girder Capacity Calculations

Flexure (Interior Girder)	**Inventory Level**	$f_{y_{INV}} = (0.55) f_y = (0.55)(36) = 19.8$ ksi $M_{INV} = f_{y_{INV}} Z = (19.8)(467) = 770.6$ kip-ft	AASHTO MCE 2000 D.6.6.2.3
	Operating Level	$f_{y_{OPR}} = (0.75) f_y = (0.75)(36) = 27$ ksi $M_{OPR} = f_{y_{OPR}} Z = (27)(467) = 1,050.8$ kip-ft	
Flexure (Exterior Girder)	**Inventory Level**	$f_{y_{INV}} = (0.55) f_y = (0.55)(36) = 19.8$ ksi $M_{INV} = f_{y_{inv}} Z = 19.8(415) = 684.8$ kip-ft	
	Operating Level	$f_{y_{OPR}} = (0.75) f_y = (0.75)(36) = 27$ ksi $M_{OPR} = f_{y_{inv}} Z = 27(415) = 933.8$ kip-ft	

F1-8 Rating Calculation (ASR)

Table F1-8.1 Allowable Stress Rating (ASR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002))

Flexure (Interior girder)	**Inventory Level**	$RF = \frac{M_{INV} - M_D}{M_{LL+IM}} = \frac{770.6 - 316.7}{553.8} = 0.82$	29.5 tons
	Operating Level	$RF = \frac{M_{OPR} - M_D}{M_{LL+IM}} = \frac{1,050.8 - 316.7}{553.8} = 1.33$	47.9 tons
Flexure (Exterior girder)	**Inventory Level**	$RF = \frac{M_{INV} - M_D}{M_{LL+IM}} = \frac{684.8 - 259.6}{508.0} = 0.84$	30.2 tons
	Operating Level	$RF = \frac{M_{OPR} - M_D}{M_{LL+IM}} = \frac{933.8 - 259.6}{508.0} = 1.33$	47.9 tons

Example F2:

Rating by the Load Factor Method (LFR) Using Load Distribution and Dynamic Allowance Factors Stipulated by the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002).

F2-1 Analysis

F2-1.1 Maximum live load Bending Moment

F2-1.1.1 Interior beam live load Bending Moment

Table F2-1.1 Interior Beam Load Combinations (computed in Section F1-6)

	Moment at 24 ft from support	**Moment at 21.67 ft from support**
M_{LL+IM}	537.5 k-ft	553.8 k-ft
M_{DL}	319.7 k-ft	316.7 k-ft
$M_{DL}+M_{LL+IM}$	857.2 k-ft	870.5 k-ft

F2-1.1.2 Exterior beam live load Bending Moment

Table B3-1.2 Exterior Beam Load Combinations (computed in Section F1-6)

	Moment at 24 ft from support	**Moment at 21.67 ft from support**
M_{LL+IM}	493.0 k-ft	508.0 k-ft
M_{DL}	262.1 k-ft	259.6 k-ft
$M_{DL}+M_{LL+IM}$	755.1 k-ft	767.6 k-ft

F2-2 Member Capacity

Table F2-2.1 Beam capacity calculation

Flexure (Interior Girder)	$M_{INV} = f_y Z = 36(467) = 1{,}401$ kip-ft	AASHTO MCE 2000 D.6.6.2.3
Flexure (Exterior Girder)	$M_{INV} = f_y Z = 36(415) = 1{,}245$ kip-ft	

F2-3 Rating Calculation (LFR)

Table F2-3.1 Load Factor Rating (LFR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002))

Flexure (Interior girder)	**Inventory Level**	$RF=\frac{\varphi M_{INV}-\gamma_{DC}M_D}{\gamma_{LL}M_{LL+IM}}=\frac{0.9(1,401)-1.3(316.7)}{2.17(553.8)}=0.71$	25.6 tons
	Operating Level	$RF=\frac{\varphi M_{INV}-\gamma_{DC}M_D}{\gamma_{LL}M_{LL+IM}}=\frac{0.9(1,401)-1.3(316.7)}{1.3(553.8)}=1.18$	42.5 tons
Flexure (Exterior girder)	**Inventory Level**	$RF=\frac{\varphi M_{INV}-\gamma_{DC}M_D}{\gamma_{LL}M_{LL+IM}}=\frac{0.9(1,245)-1.3(259.6)}{2.17(508.0)}=0.71$	25.6 tons
	Operating Level	$RF=\frac{\varphi M_{INV}-\gamma_{DC}M_D}{\gamma_{LL}M_{LL+IM}}=\frac{0.9(1,245)-1.3(259.6)}{1.3(508.0)}=1.19$	42.8 tons

Example F3:

Rating by the Load and Resistance Factor Method (LRFR) Using Load Distribution and Dynamic Allowance Factors Stipulated by the *AASHTO LRFD Specifications* (2007).

F3-1 Dynamic Load Allowance

From Table 3.6.2.1 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007), the dynamic load allowance is taken as 33%. Thus, the dynamic load factor to be applied to the static load is:

$$\left(1+\frac{IM}{100}\right)=1.33$$

F3-2 Live Load Distribution Factors

F3-2.1 Interior Beams Distribution Factor for *moment*

As per Table 4.6.2.2.2b-1 of AASHTO LRFD Bridge Design Specifications (AASHTO, 2007), the distribution factor for moment in interior beams, g_m , is specified as follows

When one lane is loaded: $$g_{m1}=0.06+\left(\frac{s}{14}\right)^{0.4}+\left(\frac{s}{L}\right)^{0.3}\left(\frac{k_g}{12Lt_s^3}\right)^{0.1}$$

When two or more lane are loaded: $$g_{m2}=0.075+\left(\frac{s}{9.5}\right)^{0.6}+\left(\frac{s}{L}\right)^{0.2}\left(\frac{k_g}{12Lt_s^3}\right)^{0.1}$$

The distribution factor for moment in exterior beams, g_m, is specified as follows

In the case of one lane loaded: g_m is computed by the lever rule

In the case of two or more lane loaded:

The longitudinal stiffness parameter:

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

In which $n=\frac{E_B}{E_D}$ where

E_B = modulus of elasticity of the beam material

E_D = modulus of elasticity of the deck material

e_g = the distance between the centers of gravity of the beams and deck

I = moment of inertia of the beam

A = area of beam

$$n=\frac{E_B}{E_D}=9.189$$

$$e_g=\frac{33.1}{2}+\frac{6.5}{2}=19.8\,in$$

$$I=6,710\,in^4$$

$$A=38.3\,in^2$$

$$K_g = n\left(I + Ae_g^2\right) = 9.189\left[6,710 + (38.3)(19.8)^2\right] = 196,600\ in^4$$

With one lane loaded:

$$g_{m1} = 0.06 + \left(\frac{8}{14}\right)^{0.4}\left(\frac{8}{47.5}\right)^{0.3}\left(\frac{196,600}{12(47.5)(6.5)^3}\right)^{0.1} = 0.06 + (0.799)(0.586)(1.025) = 0.54$$

With two or more lane loaded:

$$g_{m2} = 0.075 + \left(\frac{8}{9.5}\right)^{0.6}\left(\frac{8}{47.5}\right)^{0.2}\left(\frac{196,600}{12(47.5)(6.5)^3}\right)^{0.1} = 0.075 + (0.902)(0.7)(1.025) = 0.72$$

$$g_m = \max\left(g_{m1}, g_{m2}\right) = \max\left(0.54, 0.72\right) = 0.72$$

F3-2.2 Exterior Beams Distribution Factor for *moment*

The distribution factor for moment in exterior beams is specified in Table 4.6.2.2.2d-1 of AASHTO LRFD Bridge Design Specifications (2007) as follows:

- When one design lane is loaded, the lever rule is used to determine the distribution factor, g_m
- When two or more lanes are loaded, the distribution factor is computed by multiplying the distribution factor for moment in interior beam by a correction factor, e, defined as

$$e = 0.77 + \frac{d_e}{9.1}$$

Where d_e is the distance from the exterior web of the exterior beam to the interior edge of the curb of traffic barrier.

For computing the distribution factor by the lever rule, a simple structural member such as the one shown below is analyzed

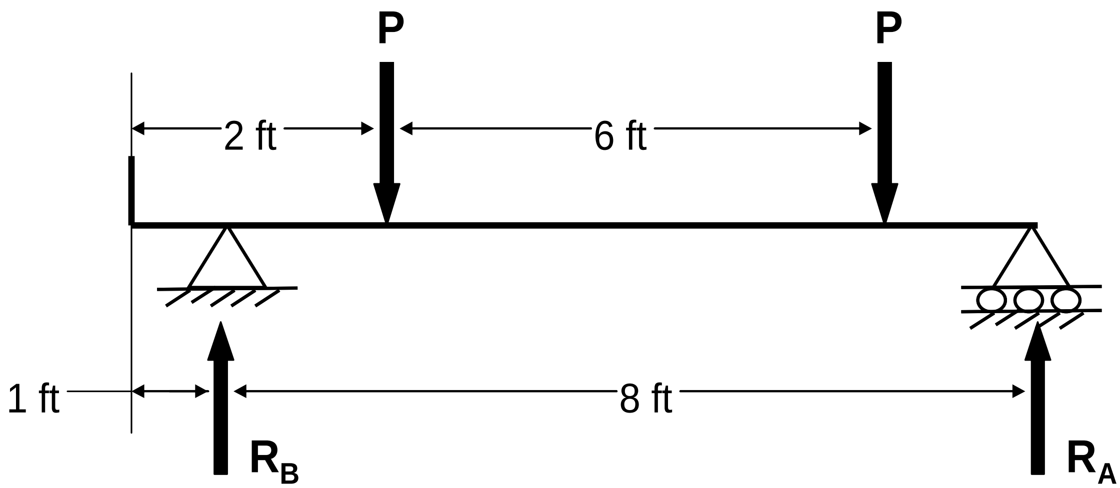


Figure F3-2.1 Exterior Girder Shear Distribution Factor

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Article 3.6.1.1.2 of AASHTO LRFD Bridge Design Specification (AASHTO, 2007) states that a multiple presence factor m = 1.20 must be used when computing girder distribution factors by the lever rule. Thus, when one lane is loaded the distribution factor for the moment in exterior beams is:

$$g_{m1(exterior)} = m\frac{\left(\frac{1}{8}+\frac{7}{8}\right)}{2} = 1.2(0.5) = 0.60$$

When two or more lanes are loaded:

$$e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{(0.9375)}{9.1} = 0.87$$

$$g_{m2(exterior)} = e\ g_{m(interior)} = (0.87)(0.72) = 0.63$$

$$g_m = \max(g_{m1(exterior)}.g_{m1(exterior)}) = 0.63$$

F3-2.3 Summary Results of Load Distribution Factors

The following table summarizes the results of calculations concerning the live load distribution factors:

Table F3-2.1 Girder Distribution Factors

Action	Interior Beam	Exterior Beam
Bending Moment	0.72	0.63

F3-3 Analysis

With the LRFR method the HL93 load case in considered at the inventory and operating load level. The HS20 load case is considered for ratings at the legal load level. The HL93 load consists of the HS20 load plus an additional lane load ($w = 0.64 kips / ft$) that is not subjected to an impact factor

F3-3.1 Maximum Bending Moment

F3-3.1.1 Maximum live load moment for an interior beam

With P = 32 kips, $g_m = 0.72$, and IM =1.33, M_{LL+IM} can be computed at the location of the maximum live load moment (at 21.42 ft from the left support) to be:

$$M_{LL+IM} = 558\, k - ft$$

The HL93 load in this case is:

$$DesignTruck(17.17) = M_{LL+IM} + g_m \left[15.2(21.42) - w \frac{(21.42)^2}{2} \right]$$

$$= 558 + 0.72 \left[15.2(21.42) - 0.64 \frac{(21.42)^2}{2} \right] = 687\, k - ft$$

Tandem Truck: P = 25 kips

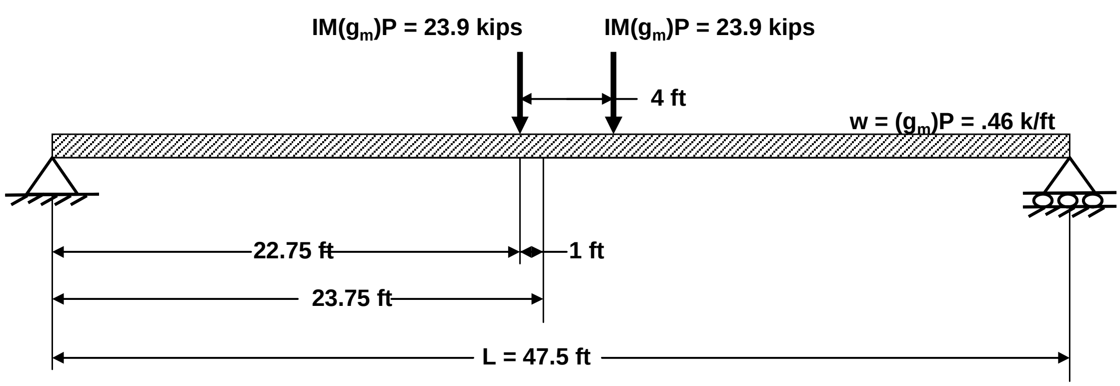


Figure F3-3.1 Maximum Design Tandem Moment

$$DesignTandem(22.75) = 22.9\left(22.75\right) + \left[10.94(22.75) - 0.46 \frac{(22.75)^2}{2} \right] = 651\, k - ft$$

$$M_{LL+IM\,(HL93)} = \max\left(DesignTruck, DesignTandem\right) = 687 k - ft$$

F3-3.1.2 Maximum live load moment for an exterior beam

With P = 32 kips, $g_{m(exterior)} = 0.63$, and IM =1.33, M_{LL+IM} can be computed at the location of the maximum live load moment (at 21.42ft from the left support) to be:

$$M_{LL+IM} = 489.0\ k - ft$$

The HL93 load in this case is:

$$DesignTruck(21.42) = M_{LL+IM} + g_m\left[9.58(21.42) - w\frac{(21.42)^2}{2}\right]$$
$$= 489 + 0.63\left[15.2(21.42) - 0.64\frac{(21.42)^2}{2}\right] = 602k - ft$$

Tandem Truck: P = 25 kips

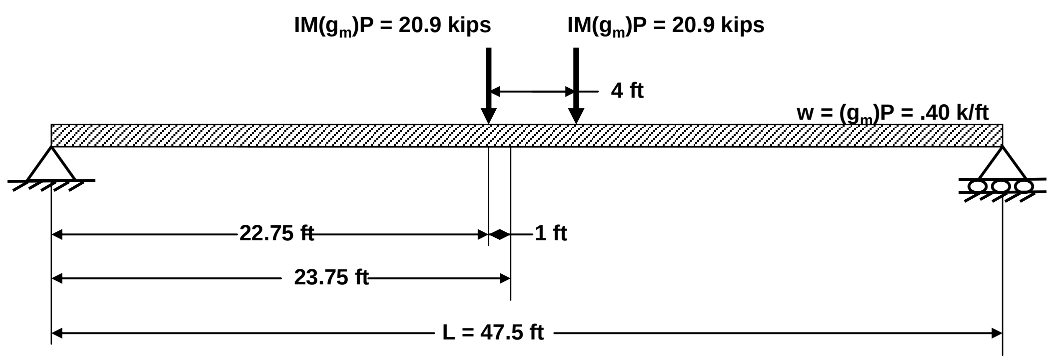


Figure F3-3.2 Maximum Design Tandem Moment

$$DesignTandem(22.75) = 20.0\left(22.75\right) + \left[9.58(22.75) - 0.40\frac{(22.75)^2}{2}\right] = 570\ k - ft$$

$$M_{LL+IM\,(HL93)} = \max\left(DesignTruck, DesignTandem\right) = 602k - ft$$

F3-4 Member Capacity

Table F3-4.1 Beam capacity calculation

Flexure (Interior Girder)	$M = f_y Z = 36\left(467\right) = 1{,}401$ kip-ft	AASHTO MCE 2000 D.6.6.2.3
Flexure (Exterior Girder)	$M = f_y Z = 36\left(415\right) = 1{,}245$ kip-ft	

F3-5 Rating Calculation (LRFR)

The following factors are defined thus:

Live load factor for legal load level $\gamma_{LL} = 1.43$ (LRFR Table 6-5 based on ADTT of 210)

Resistance Factor (for shear and flexure) φ = .9

Condition Factor (related to NBI Item 59) $\phi_c = 1$

System Factor (related to structural redundancy) $\phi_s = 1$

Table F3-5.1 Load and Resistance Factor Rating (LRFR) Calculation for HL93 load at Inventory and Operating Levels and HS20 load at the Legal Level (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO LRFD Specifications (2007))

Flexure (Interior girder) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(1,401) - 1.25(316.7)}{1.75(687)} = 0.72$	<u>25.9 tons</u>
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(1,401) - 1.25(316.7)}{1.35(687)} = 0.93$	<u>33.5 tons</u>
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.5$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HS20}} = \frac{0.9(1)(1)(1,401) - 1.25(316.7)}{1.43(558)} = 1.08$	<u>38.9 tons</u>
Flexure (Exterior girder) $\phi = 0.9$ $\phi_c = 1$ $\phi_s = 1$	**Inventory Level** $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(1,245) - 1.25(259.6)}{1.75(602)} = 0.76$	27.4 tons
	Operating Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.35$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1)(1,245) - 1.25(259.6)}{1.35(602)} = 0.98$	35.3 tons
	Legal Level $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.5$	$RF = \frac{\phi\phi_c\phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HS20}} = \frac{0.9(1)(1)(1,245) - 1.25(259.6)}{1.43(489)} = 1.14$	41.0 tons

Rating of a Steel Girder Bridge Pier Cap (GDOT BRIDGE ID # 085-0018)

Example F4:

Rating by the Allowable Stress Method (ASR) Using Load Distribution and Dynamic Allowance Factors Stipulated by the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002).

F4-1 Basic Geometry and Bridge Information:

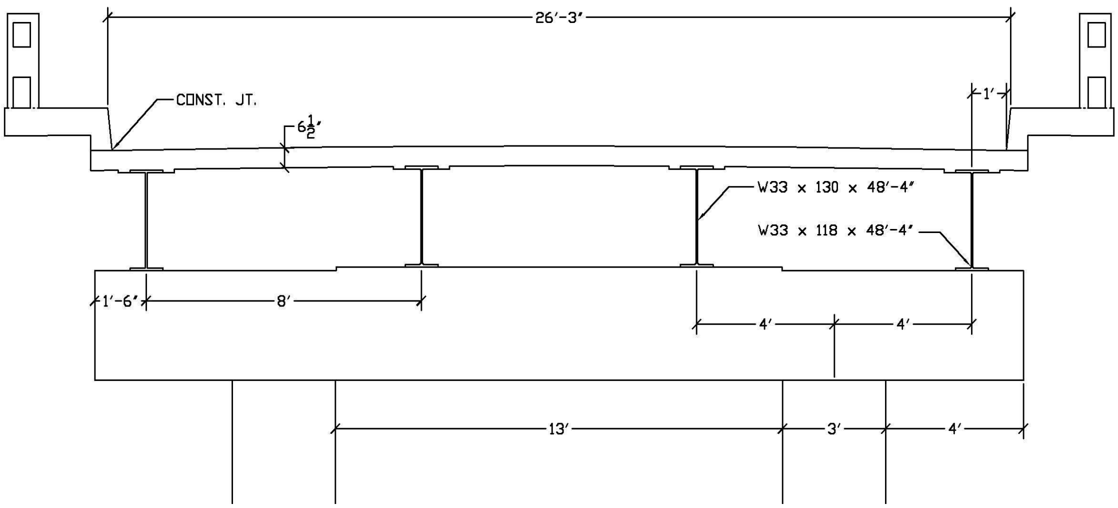


Figure F4-1.1 Bridge Cross Section at Mid-span

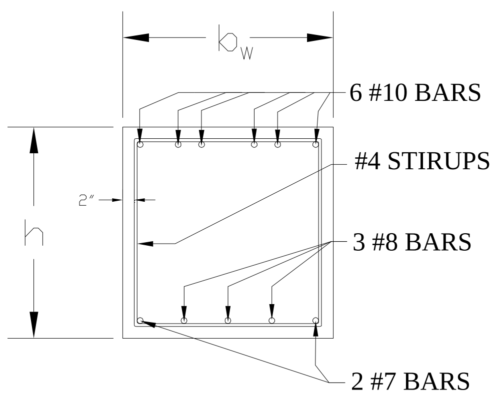


Figure F4-1.2 Pier Cap Details

Unit weight of concrete:

$$w_c = .15\,kips/ft^3$$

Weight of steel beam per foot of length:

Interior Girder: $w_s = .13\,kips/ft$

Exterior Girder: $w_s = .118\,kips/ft$

Weight per ft of standard GDoT rail and sidewalk: $w_{pr} = .556\,kips/ft$

F4-2 LOADS

F4-2.1 Permanent Loads

As per AASHTO Article 3.23.2.3.1 the dead load supported by the outside stringers or beams shall be the portion of the floor slab carried by the stringer or beam. Curbs, railings, and wearing surfaces if placed after the slab has cured, may be distributed equally to all roadway girders.

F4-2.1.1 Interior girder loads

Weight per linear foot of the reinforced concrete slab = $\left(\frac{6.5}{12}\right)(8)(0.150)$ = 0.650 k/ft

Weight per linear foot of cast-in-place beams = (0.130) = 0.130 k/ft

Weight of parapet, rail and sidewalk assembly = $\frac{(2)(0.556)}{4}$ = 0.278 k/ft

Weight per linear foot of diaphragm = $\frac{(1)(24)\left(\frac{33.125}{12}\right)(0.15)}{4(48)}$ = 0.052 k/ft

Since there is no wearing surface present on the bridge, DW = 0

Total dead load reaction for interior beam = 1.11 k/ft

F4-2.1.2 Exterior girder loads

Weight per linear foot of the reinforced concrete slab = $\left(\frac{6.5}{12}\right)(5.625)(0.150)$ = 0.457 k/ft

Weight per linear foot of cast-in-place beams = (0.118) = 0.118 k/ft

Weight of parapet, rail and sidewalk assembly = $\frac{(2)(0.556)}{4}$ = 0.278 k/ft

Weight per linear foot of diaphragm = $\frac{(1)(24)\left(\frac{33.125}{12}\right)(0.15)}{4(48)}$ = 0.052 k/ft

Since there is no wearing surface present on the bridge, DW = 0

Total dead load reaction for exterior beam = 0.91 k/ft

Interior Girder

$$P_{DL} = \frac{48}{2}[1.11] = 26.6 kips$$

Exterior Girder

$$P_{DL} = \frac{48}{2}[0.91] = 21.8 kips$$

Dead Load of the substructure:

$$\mathrm{W_{dl}} = \mathrm{b(h)(w_c)} = 3(3)(0.15) = 1.35 \frac{kips}{ft}$$

Dead Load Shear:

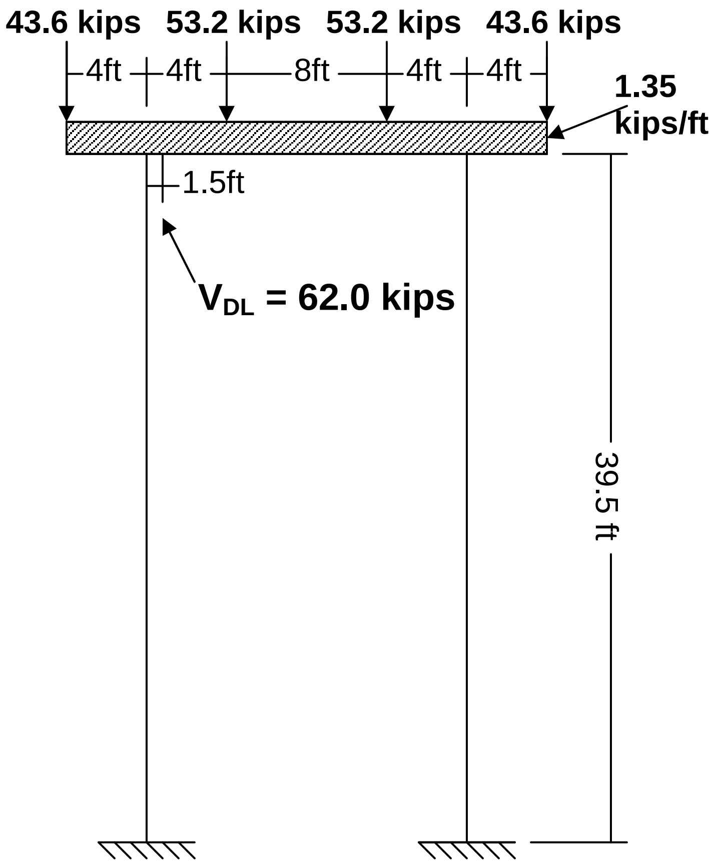


Figure F4-2.1 Pier Dead Load 2D Frame Model

F4-2.2 Vehicular Live Load

The design vehicular live load on the bridge consists of AASHTO HS20 truck with the spacing between the two 32-kip rear-axle loads to be varied from 14 ft to 30 ft to produce extreme force effects. The HS 20 truck is shown below.

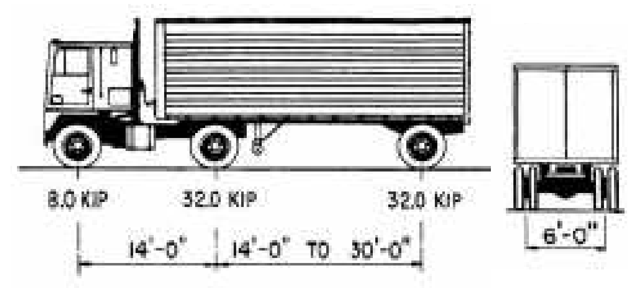


Figure F4-2.1 AASHTO HS20 Truck

F4-3 Dynamic Load Allowance

From Section 3.8.2.1 of the AASHTO Standard Specifications (2002), the dynamic load allowance is taken as:

$$I = 1 + \frac{50}{L + 125} = 1 + \frac{50}{49 + 125} = 1.29$$

With a specified maximum value of 30%

F4-4 Analysis

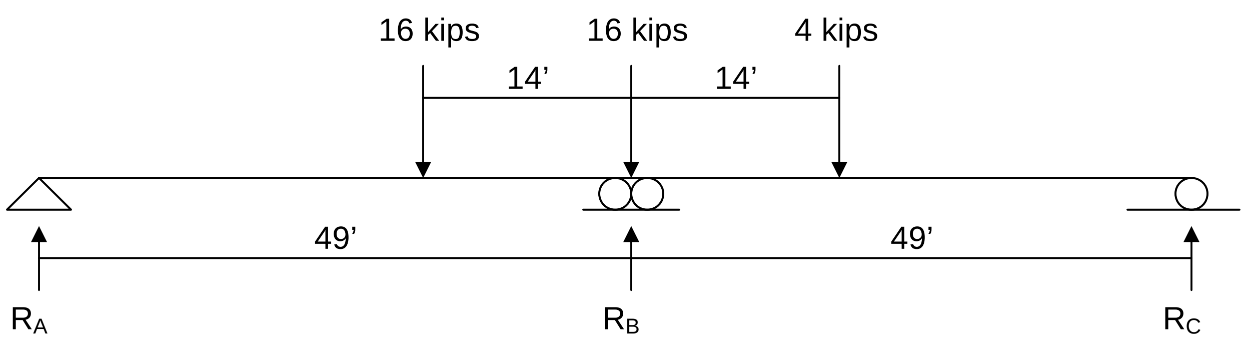
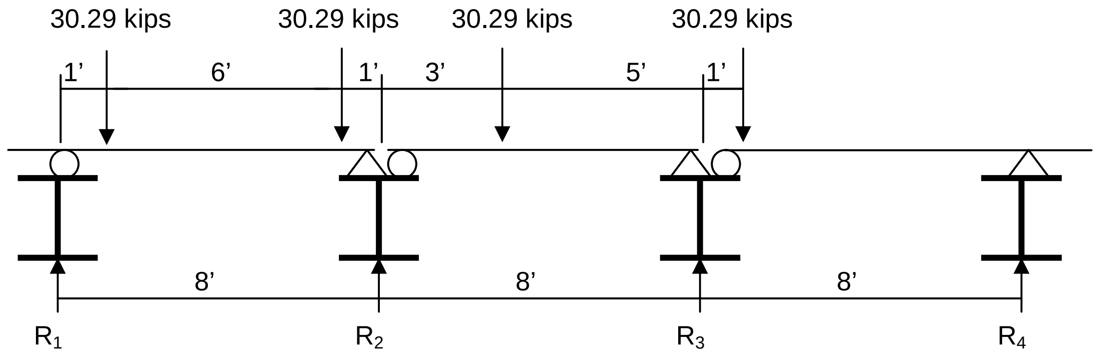


Figure F4-4.1 Wheel Line Load

$$R_B = 16 + 16\left(\frac{49-14}{49}\right) + 4\left(\frac{49-14}{49}\right) = 30.29 kips$$



$R_1 = 30.29$ $\quad$ $R_2 = 49.22$ $\quad$ $R_3 = 37.86$ $\quad$ $R_4 = 3.79$

Figure F4-4.2 Girder Live Load Reactions

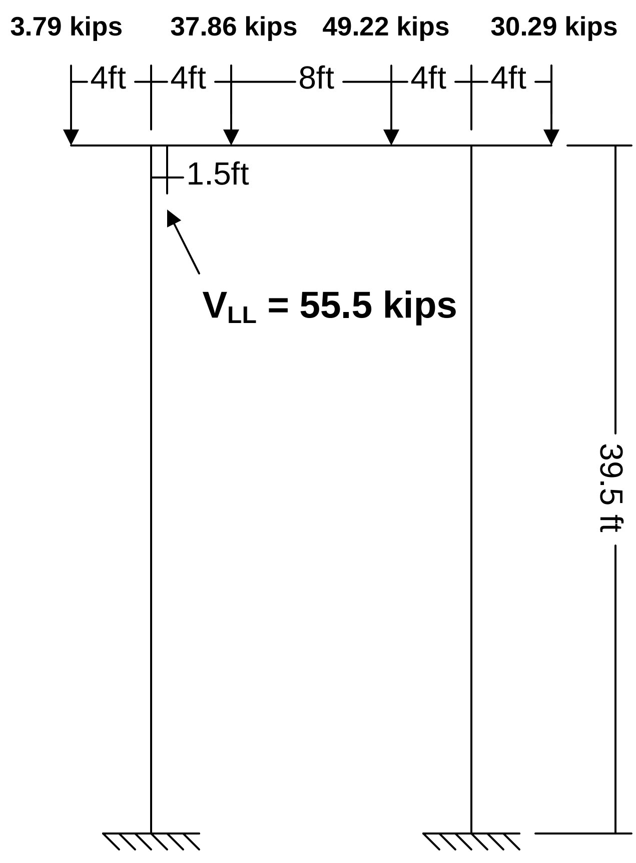


Figure F4-4.3 Pier Live Load 2D Frame Model

$$V_{LL} = 55.5 kips$$
$$V_{LL+IM} = 55.5(1.29) = 71.6 kips$$

F4-5 Member Capacity

$h = 36in$

$d = 36 - 2 - .5 - .5 = 33in$

$b_w = 36in$

$s = 12in$

$A_s = 0.4in^2$

Table F4-5.1 Pier Cap Capacity Calculations

Pier Cap Shear Capacity	**Inventory Level** f_y = 20 ksi f_c' = 3000 psi	Shear Capacity: Concrete $V_c = .9\sqrt{f_c'}\, b_w\, d/1000 = 58.6$ kips Shear Capacity: Steel $V_s = A_s f_y\, d/s = 22$ kips	AASHTO MCE 2000 D.6.6.2.3
	Operating Level f_y = 28 ksi f_c' = 3000 psi	Shear Capacity: Concrete $V_c = 1.3\sqrt{f_c'}\, b_w\, d/1000 = 84.6$ kips Shear Capacity: Steel $V_s = A_s f_y\, d/s = 30.8$ kips	

F4-6 Rating Calculation (ASR)

Table F4-6.1 Allowable Stress Rating (ASR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002))

Shear	**Inventory Level**	$RF = \frac{V_N - V_{DL}}{V_{LL+IM}} = \frac{80.6 - 62}{71.6} = 0.26$	9.36 tons
	Operating Level	$RF = \frac{V_N - V_{DL}}{V_{LL+IM}} = \frac{115.4 - 62}{71.6} = 0.75$	27.0 tons

Rating of a Steel Girder Bridge Pier Cap (GDOT BRIDGE ID # 085-0018)

Example F5:

Rating by the Load Factor Method (LFR) Using Load Distribution and Dynamic Allowance Factors Stipulated by the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002).

F5-1 LOADS

F5-1.1 Permanent Loads (section F4-2)

$$\mathrm{V}_{DL} = 62.0 \text{ kip}$$

F5-1.2 Vehicular Live Load

The design vehicular live load on the bridge consists of AASHTO HS20 truck with the spacing between the two 32-kip rear-axle loads to be varied from 14 ft to 30 ft to produce extreme force effects. The HS 20 truck is shown below.

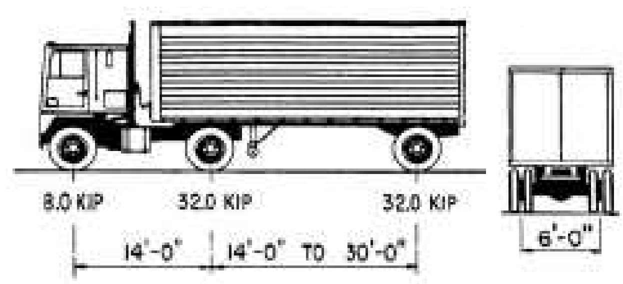


Figure F5-1.1 AASHTO HS20 Truck

F5-2 Dynamic Load Allowance

From Section 3.8.2.1 of the AASHTO Standard Specifications (2002), the dynamic load allowance is taken as:

$$\mathrm{I} = 1 + \frac{50}{\mathrm{L}+125} = 1 + \frac{50}{49+125} = 1.29$$

With a specified maximum value of 30%

F5-3 Analysis

$$V_{LL} = 55.5 kips$$
$$V_{LL+IM} = 55.5(1.29) = 71.6 kips$$

F5-4 Member Capacity

$$h = 36 in$$
$$d = 36 - 2 - .5 - .5 = 33 in$$

$$b_w = 36 in$$
$$f_c^{'} = 3000 \, psi$$

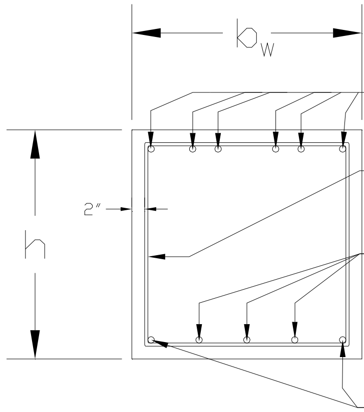


Figure F5-4.1 Pier Cap Detail

Table F5-4.1 Pier Cap Capacity Calculations

Pier Cap Shear Capacity	Shear Capacity: Concrete $V_c = 2\sqrt{f'_c}\, b_w\, d/1000 = 130 \text{ kips}$ Shear Capacity: Steel $V_s = A_s f_y\, d/s = 44 \text{ kips}$	AASHTO MCE 2000 D.6.6.2.3

F5-5 Rating Calculation (LFR)

Inventory $\gamma_{DC} = 1.3$ $\gamma_{LL} = 2.17$ $\phi = .85$

Operating $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.3$ $\phi = .85$

Table F5-6.1 Load Factor Rating (LFR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002))

Shear	**Inventory Level**	$RF = \frac{\varphi V_N - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{LL}} = \frac{0.85(174) - 1.3(62)}{2.17(71.6)} = 0.43$	15.5 tons
	Operating Level	$RF = \frac{\varphi V_N - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{LL}} = \frac{0.85(174) - 1.3(62)}{1.3(71.6)} = 0.72$	25.9 tons

Rating of a Steel Girder Bridge
Pier Cap
(GDOT BRIDGE ID # 085-0018)

Example F6:

Rating by the Load and Resistance Factor Rating Method (LRFR) Using Load Distribution and Dynamic Allowance Factors Stipulated by the *AASHTO* LRFD Bridge Design Specifications (AASHTO, 2007).

F6-1 Permanent LOADS (section F4-2)

$$V_{DL} = 62.0 \text{ kip}$$

F6-2 Dynamic Load Allowance

From Table 3.6.2.1 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007), the dynamic load allowance is taken as 33%. Thus, the dynamic load factor to be applied to the static load is:

$$\left(1+\frac{IM}{100}\right)=1.33$$

F6-3 Analysis

F6-3.1 Wheel line load

Computed in Section B8-4: $R_B = 16+16\left(\frac{49-14}{49}\right)+4\left(\frac{49-14}{49}\right)=30.29 kips$

$$DesignTrucks = 2(30.29)+(0.64)(39)=85.5 \text{ kips}$$

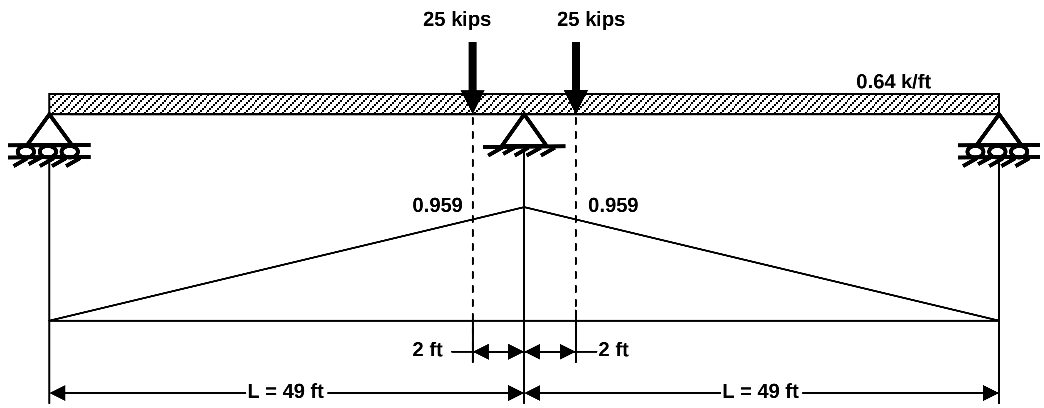


Figure F6-3.1 Interior Beam Shear Design Tandem

$$DesignTandem = 25(0.959)+25(0.959)+(0.64)(49)=79.3 \text{ kips}$$

$$V_{HL93} = \max(DesignTruck, DesignTandem) = 85.5 \text{ kips}$$

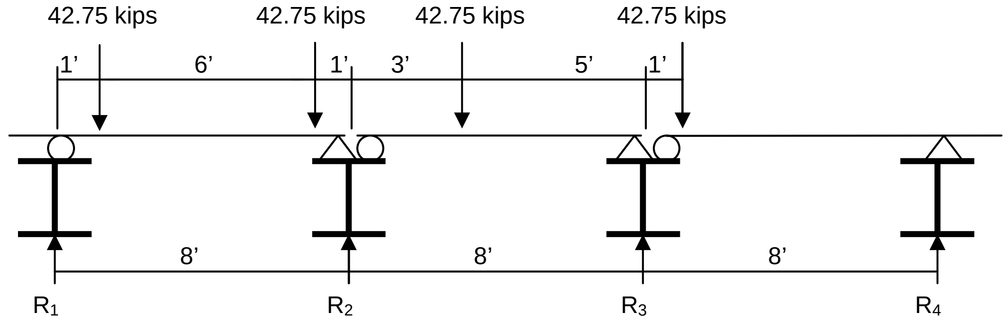
F6-3.2 Girder live load reactions

<u>F6-3.2.1 HS20 girder live load reactions (section F4-4)</u>

$$V_{LL} = 55.5 kips$$

$$V_{LL+IM} = 55.5(1.33) = 73.8 kips$$

F6-3.2.1 HL93 girder live load reactions



$R_1 = 42.75$ $R_2 = 69.47$ $R_3 = 53.44$ $R_4 = 5.34$

Figure F4-3.2 Girder Live Load Reactions

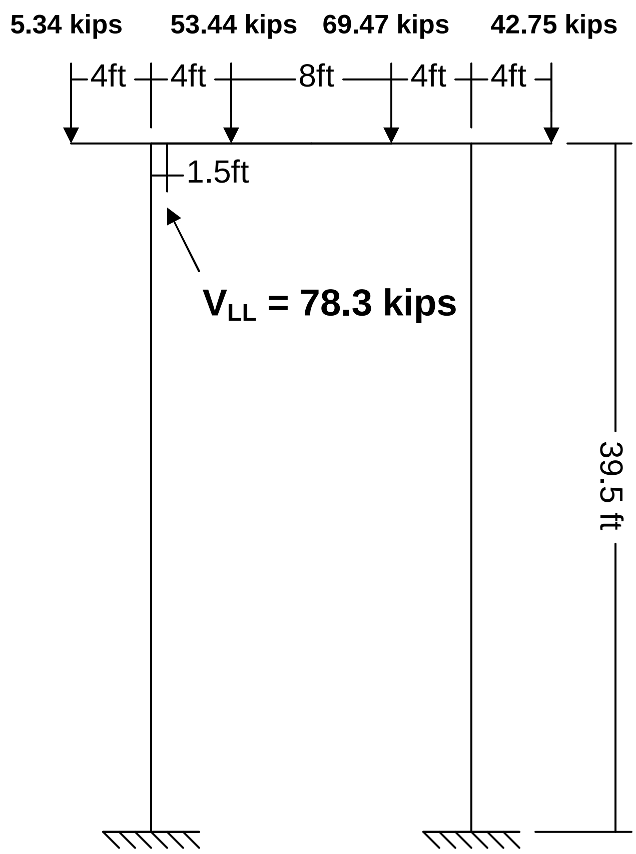


Figure F4-3.3 Pier Live Load 2D Frame Model

$$V_{LL} = 78.3 kips$$

$$V_{LL+IM} = 78.3(1.29) = 104 kips$$

F6-4 Member Capacity

Table F6-4.1 Pier Cap Capacity Calculations

Pier Cap Shear Capacity	Shear Capacity: Concrete $V_c = 2\sqrt{f_c^{'}}b_w\, d/1000 = 130\ \text{kips}$ Shear Capacity: Steel $V_s = A_s f_y\, d/s = 44\ \text{kips}$	AASHTO MCE 2000 D.6.6.2.3

F6-5 Rating Calculation (LRFR)

Inventory $\gamma_{DC} = 1.25$ $\gamma_{LL} = 1.75$

Operating $\gamma_{DC} = 1.3$ $\gamma_{LL} = 1.35$

Live load factor for legal load level $\gamma_{LL} = 1.43$ (LRFR Table 6-5 based on ADTT of 210)

Resistance Factor (for shear and flexure) $\varphi = .9$

Condition Factor (related to NBI Item 59) $\phi_c = 1$

System Factor (related to structural redundancy) $\phi_s = 1$

Table F6-5.1 Load and Resistance Factor Rating (LRFR) Calculation for HL93 load at Inventory and Operating Levels and HS20 load at the Legal Level (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO LRFD Specifications (2007))

Shear (Interior Girder)	**Inventory Level**	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(174) - 1.25(62)}{1.75(104)} = 0.43$	15.5 tons
	Operating Level	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HL93}} = \frac{0.9(1)(1)(174) - 1.25(62)}{1.35(104)} = 0.56$	20.2 tons
	Legal Level	$RF = \frac{\phi\phi_c\phi_s V_n - \gamma_{DC}V_{DL}}{\gamma_{LL}V_{HS20}} = \frac{0.9(1)(1)(174) - 1.25(62)}{1.43(73.8)} = 0.75$	27 tons