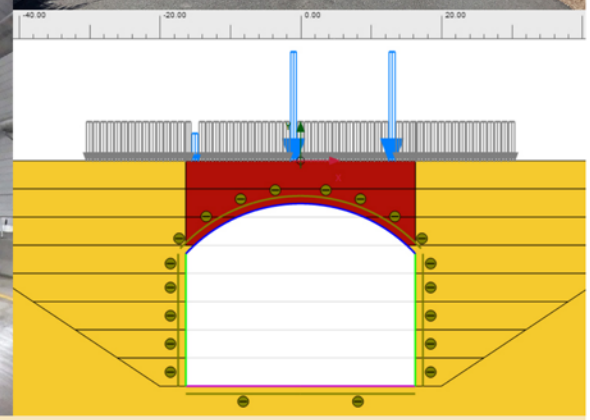

Reference Guide for Load Rating of Tunnel Structures

Publication No. FHWA-HIF-19-010
Infrastructure Office of Bridges and Structures

May 2019

U.S. Department of Transportation
Federal Highway Administration

FOREWORD

The Moving Ahead for Progress in the 21st Century Act (MAP-21, P.L. 112-141), signed into law by President Obama on July 6, 2012, required the Secretary (USDOT), in consultation with the States and Federal agencies with jurisdiction over highway bridges and tunnels to establish the national minimum standards for tunnel inspection, evaluation and inventory. As a result, on August 13, 2015, the Federal Highway Administration published the final rule of the National Tunnel Inspection Standards (NTIS, 23 CFR. §650 Subpart E) for tunnel inspections consistent with the provisions of 23 U.S.C. 144(h).

23 CFR §650.513(g) requires States and Federal agencies to "[R]rate each tunnel's safe vehicular load-carrying capacity in accordance with the Sections 6 or 8, AASHTO Manual for Bridge Evaluation (incorporated by reference, see § 650.517)." Load rating should be performed not later than three months after the completion of the inspection, if a change in condition is identified. If the tunnel under consideration does not have adequate capacity to safely carry the maximum unrestricted legal loads or State routine permit loads, the tunnel shall be appropriately load posted or restricted within 30 days after a load rating that determines the posting/restriction is in need.

However, the AASHTO Manual for Bridge Evaluation (MBE) does not provide information specific to tunnels. The information in Section 5.4 of FHWA's Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual, published 2015 together with the NTIS final rule, is also limited. Therefore, this reference guide document is needed to collect the available information from relevant resources such as the MBE, TOMIE, AASHTO LRFD Bridge Design Specifications, and AASHTO LRFD Road Tunnel Design and Construction Guide Specifications.

This reference guide covers the technical aspects for the load rating of tunnel structures, but also includes practical, representative step-by-step examples. The intended audiences are engineers proficient in the load rating process, but unfamiliar with tunnel structures and tunnel design engineers who are unfamiliar with the load rating process including load rating vehicles, methodology, posting, and permitting. Tunnel design engineers who are familiar with the load rating process will also benefit from this guide.

The subject matter expert technical reviewers for this guide included Dolores Valls (California Department of Transportation), Louis Ruzzi (Pennsylvania Department of Transportation), Bijan Khaleghi (Washington Department of Transportation), and Joseph Rigney (Massachusetts Department of Transportation) as well as many engineering professionals from the highway bridge and tunnel community. Their advice, counsel and contributions during the preparation of this guide are greatly appreciated.

Joseph L. Hartmann, PhD, P.E.
Director, Office of Bridges and Structures
Office of Infrastructure
Federal Highway Administration

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16. Abstract
The purpose of this reference guide is to provide guidelines for load rating the nation's highway tunnels to meet the regulatory requirements and to ensure consistency and uniformity of load rating those tunnels. This reference guide provides technical aspects for the load rating of tunnel structures that meets FHWA's general load rating guidance and provides practical, representative step-by-step examples. This reference guide provides technical details and breadth appropriate for explaining the load rating specifications and guidelines governing United States highway tunnel structures, namely the American AASHTO Manual for Bridge Evaluation and the FHWA Tunnel Operations, Maintenance, Inspection, and Evaluation Manual. This reference guide is applicable to structural components of tunnels that support roadway live loads, including invert slabs, walls, and other supporting members as well as tunnel liners where applicable, that support live load from nearby highways, and provides guidance on evaluating member loads and resistances for the development of load ratings; determining posting ratings or restrictions as necessary; and issuing permits.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
ASR	Allowable Stress Rating
BDS	AASHTO LRFD Bridge Design Specifications, 8th Edition (2017a)
CFR	Code of Federal Regulations
CPT	Cone penetration test
FD	Finite difference
FE	Finite element
FHWA	Federal Highway Administration
GPR	Ground penetrating radar
GSI	Geological strength index
LFR	Load Factor Rating
LRFDTUN	AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, First Edition (2017b)
LRFR	Load and Resistance Factor Rating
MBE	AASHTO Manual for Bridge Evaluation, 3rd Edition (2018)
MUTCD	FHWA Manual on Uniform Traffic Control Devices (2003)
NBI	National Bridge Inventory
NBIS	National Bridge Inspection Standards
NTI	National Tunnel Inventory
NTIS	National Tunnel Inspection Standards
RMR	Rock mass ratings
SASW	Spectral analysis of surface waves
SEM	Sequential excavation method
SNTI	Specifications for National Tunnel Inventory (FHWA 2015b)
SPT	Standard penetration test
Standard Specifications	AASHTO Standard Specifications for Highway Bridges and Interim Specifications, 17th Edition (AASHTO, 2002)
TBM	Tunnel boring machine
TOMIE	Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual (FHWA 2015a)

CHAPTER 1 - INTRODUCTION

1.1 PURPOSE AND SCOPE

This reference guide created by the Federal Highway Administration (FHWA) covers the technical aspects for the load rating of tunnel structures that meets FHWA's general load rating guidance and provides practical, representative step-by-step examples. This reference guide provides technical details and breadth appropriate for explaining the load rating specifications and guidelines governing U.S. highway tunnel structures, namely the American Association of State Highway and Transportation Officials (AASHTO) Manual for Bridge Evaluation (MBE) (AASHTO 2018) and the FHWA Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual (FHWA 2015a). A set of examples illustrate the requirements, procedures, and methods for load rating. Unless otherwise specified, all references refer to MBE 3rd Edition (AASHTO 2018) or TOMIE Manual (FHWA 2015a).

Title 23 of the Code of Federal Regulations (CFR), Part 650 Subpart E – National Tunnel Inspection Standards (NTIS) (23 CFR 650E), legally requires State DOTs, Federal agencies, or tribal governments to:

- Rate each tunnel's safe vehicular load-carrying capacity in accordance with MBE Section 6 or MBE Section 8; and
- Post or restrict the highways in or over the tunnel in accordance with MBE Section 6 when the maximum unrestricted legal or routine permit loads exceed those allowed under the operating rating or equivalent rating factor. Postings are to be made as soon as possible after a load rating determined the need for such a posting.

These NTIS load rating requirements are similar to those stated in 23 CFR Part 650 Subpart C – National Bridge Inspection Standards (NBIS) (23 CFR 650C). However, the current NBIS (as of this publication) does not specify a time in which the load rating is to be accomplished after the completion of the inspection, nor does the NBIS specify a time in which a posting is to be made, relative to the completion of the load rating.

See 1.3.1 for a description of the NTIS standard and specifications it incorporates. As a supplement to the NTIS, FHWA published the TOMIE Manual. The TOMIE Manual states that "[a] load rating is required for all tunnels that: [h]ave a structurally supported roadway system to carry vehicles (not at grade) within the tunnel bore" or that "are subjected to live load force effects from a roadway located above the tunnel" (FHWA 2015a). In the latter case, the tunnel liner can be treated like a culvert where earth pressures and live (truck) loads are distributed through fill, discussed in detail in Section 4.3.2.2. The TOMIE Manual elaborates on the primary purposes of tunnel load rating which include 1) meeting the regulatory requirements; 2) making load posting or restriction decisions; and 3) issuing hauling permits. The MBE was incorporated into the NTIS through reference and provides specifications for load rating, posting, permitting, and fatigue evaluation for bridge and culvert structures. However, the MBE does not provide specific guidance for load rating of tunnels.

In summary, Federal regulation requires States and other tunnel owners to load rate and post/restrict (if necessary) tunnels that are subject to highway vehicular loadings. However, a need exists for national guidance on tunnel load rating. The purpose of this document, referred herein as Guide, is to provide guidelines for load rating the nation's highway tunnels to meet the regulatory requirements and to ensure consistency and uniformity of load rating those tunnels. These guidelines are intended for the evaluation of tunnels constructed using cut-and-cover, shield driven, bored, jacked, drilled and blasted, immersed tube, and sequential excavation construction methods. These construction methods are discussed in detail in Section 2.3.

This reference guide is applicable to structural components of tunnels that support roadway live loads, including invert slabs, walls, and other supporting members as well as tunnel liners where applicable, that support live load from nearby highways. This reference guide is not intended to be used for the design of these members, but as guidance on evaluating member loads and resistances for the development of load ratings; determining posting ratings or restrictions as necessary; and issuing permits. Design of these elements is covered by the AASHTO LRFD Bridge Design Specifications (BDS) (AASHTO 2017a) and the AASHTO LRFD Road Tunnel Design and Construction Guide Specifications (LRFDTUN) (AASHTO 2017b); these specifications are meant to supplement inspection and load rating procedures covered in other manuals, such as the TOMIE Manual and MBE. Where applicable, these specifications, manuals and their relevant articles are referenced to avoid duplication of information. These documents are not intended to supplant proper training and experience or the exercise of judgment by the Engineer. The Owner or the Engineer may require the sophistication of evaluation to be higher than presented within this Guide.

The following is an overview of the Guide's organizational structure.

- Chapter 1 contains introductory information on the development of load rating requirements for highway tunnels, definitions of terms applicable to tunnel load rating, and the applicable standards and specifications including their hierarchy and relevant sections.
- Chapter 2 provides an overview of tunnel fundamentals including types of tunnels, types of construction, and details of tunnel components subject to load rating.
- Chapter 3 covers the collection of data relevant to tunnel load rating including records and inspection.
- Chapter 4 covers the fundamentals of permanent and live load forces applicable to tunnel elements in accordance with the MBE.
- Chapter 5 discusses the structural analysis of applicable tunnel elements subjected to highway live loads including invert slabs with their associated supports, as applicable, and tunnel liners and roof girders. Simple hand calculations, frame analysis, and finite element analysis methods are covered, as well as geotechnical considerations.
- Chapter 6 discusses the load rating methodology and resistance calculations for tunnel elements based on the Load and Resistance Factor Rating (LRFR) method in accordance with the MBE and other specifications as applicable. Limited information on the Load

Factor Rating (LFR) and Allowable Stress Rating (ASR) methodologies are provided as well.

- Chapter 7 covers the procedures and importance of appropriate quality control and quality assurance.
- Chapter 8 provides information on strength evaluation of tunnel components, fatigue evaluation, and other topics.
- Chapter 9 contains four load rating examples that demonstrate the information provided in this Guide using idealized tunnel examples, ranging from slabs and invert girders to frame and finite element analysis of tunnel liners.
- Appendix A provides a flowchart to describe the load rating process, including posting and permitting in accordance with the MBE.
- Appendix B provides a checklist for a generic load rating for use by the reader to help assure that the necessary information is obtained and implemented during the process.

The target audience for this Guide is employees in transportation agencies and their representatives and consultants involved with load rating of tunnel structures. This includes, but is not limited to, engineers and other professionals who are responsible for managing a load rating program, for performing load ratings, for recommending or making load posting decisions, and for conducting load ratings for issuing overweight load permits. *The intended audiences for this Guide are engineers proficient in the load rating process, but unfamiliar with tunnel structures and tunnel design engineers who are unfamiliar with the load rating process including load rating vehicles, methodology, posting, and permitting. Tunnel design engineers who are familiar with the load rating process will also benefit from this Guide.*

The FHWA anticipates in the future publishing two practical Validation Examples based on real, in-service tunnel structures that were load rated in accordance with this Guide.

1.2 DEFINITIONS AND TERMINOLOGY

Bored tunnel. A tunnel constructed using a Tunnel Boring Machine (TBM).

Cut-and-cover. The sequence of construction in which a trench is excavated and the tunnel or culvert section is constructed within the trench and then covered with backfill.

Functional systems. The non-structural systems, such as electrical, mechanical, fire suppression, ventilation, lighting, communications, monitoring, drainage, traffic signals, emergency response (including egress, refuge room spacing, or carbon monoxide detection), or traffic safety components. Functional systems are not structural systems subjected to load rating, but are often necessary to consider as a dead load when load rating certain tunnel components.

Immersed tunnel. A tunnel constructed from prefabricated elements constructed off the tunnel alignment, floated into place over the tunnel alignment, and placed into a prepared trench. Placement is facilitated by the addition of ballast to the elements to cause them to be immersed to the pre-determined depth and then joined to the adjacent element(s) already in place.

Inventory Level Rating (LRFR). Generally corresponds to the rating for HL-93 at the design level of reliability for new structures, but reflects the existing structure and material conditions with regard to deterioration and loss of section.

Invert. On a circular tunnel, the invert is approximately the bottom 90 degrees of the arc of the tunnel; on a square bottom tunnel, it is the bottom of the tunnel.

Invert slab. A structural deck that carries the traffic, and is typically constructed of reinforced concrete with or without invert girders.

Lining. The permanent ground support around the periphery of the tunnel with an inner surface suitable for the end-use of the underground excavation. A lining system may provide limited support if the ground consists of stable rock or continuous support in unstable soils or mixed ground conditions.

Operating Level Rating (LRFR). The load rating for HL-93 at the operating level of reliability.

Legal Load Rating (LRFR). The maximum load level to which a structure may be subjected. Generally corresponds to the rating for legal loads at the operating level of reliability in past load rating practice.

Permit Load Rating (LRFR). Checks the safety and serviceability of structures in the review of permit applications for passage vehicles above the legally established weight limitations.

Portal. The entrance and exit of the tunnel exposed to the environment; portals may include bare rock, constructed tunnel entrance structures, or buildings.

Soil-structure interaction. The process in which the response of the soil influences the motion of the structure and response of the structure influences the motion of the soil.

Tunnel. An enclosed roadway for motor vehicle traffic with vehicle access limited to portals, regardless of type of structure or method of construction, that requires, based on the Owner's determination, special design considerations that may include lighting, ventilation, fire protection systems, and emergency egress capacity. The term tunnel does not include bridges or culverts inspected under the NBIS.

In addition to these definitions the term "*shall*" denotes a requirement for compliance with applicable specifications; the term "*should*" indicate a strong preference for given criterion; and the term "*may*" indicates a criterion that is usable, but other local and suitably document, verified, and approved criteria may also be used in a manner consistent with the applicable specifications.

1.3 SPECIFICATIONS, MANUALS, AND ADDITIONAL RESOURCES

1.3.1 Specifications and Manuals

National Tunnel Inspection Standards (NTIS) (23 CFR 650E). The NTIS establishes the regulation requiring "tunnel owners to establish a program for the inspection of highway tunnels," to maintain an inventory, and to provide inspection findings to FHWA as well as make corrective actions to critical issues found during inspection. This document ultimately sets the rules and regulations regarding tunnel inspection and evaluation. 23 CFR 650.513(g) provides the regulations regarding the load rating of tunnels. The NTIS incorporates, by reference, the Specifications for National Tunnel Inventory (SNTI), TOMIE Manual, and MBE.

Specifications for National Tunnel Inventory (SNTI) (FHWA 2015b). Developed in coordination with the NTIS, the SNTI provides the specifications and coding data required to be submitted to the National Tunnel Inventory (similar to the National Bridge Inventory (NBI)). Details are provided in each chapter of the SNTI about how to log information for each item's code, and the Engineer must decide which codes are applicable for the tunnel at hand. Section 2 provides general information about the tunnel, where Section 2.7 provides the current load rating information. Section 3 discusses the specific tunnel elements, with Section 3.2 (structural) and Section 3.3 (civil) being the most relevant. Section 4 provides an index of inventory items and elements while Section 5 provides an example tunnel coding that helps readers understand the inspection coding process and can be back-referenced to the rest of the SNTI for further understanding. There are no references to any load rating process in the SNTI.

Tunnel Operations, Maintenance, Inspection and Evaluation (TOMIE) Manual (FHWA 2015a). The TOMIE Manual expands upon the inspection requirements to address and serve as a resource for operations, maintenance, inspection, and evaluation in addition to inspection. It provides a wealth of information ranging from initial construction techniques to air handlers, to risk assessment, to basic cost estimating; and shall be considered a primer to this Guide. TOMIE Manual Section 5.4 provides initial discussion of and the requirements for load rating tunnels. Section 5.4.2 introduces the concept of LRFR for tunnels, but is limited in scope.

AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). The MBE provides procedures for determining "the physical condition, maintenance needs, and load carrying capacity" for bridges. MBE Chapter 6 through Chapter 8 are of interest because they present the concepts of load rating for vehicular live loads. In particular, Chapter 6 sets forth the criteria for load rating and posting of bridges and is the basis of several of the load rating concepts presented in this Guide. The most current edition at the time of publication of this Guide is the 3rd Edition. Subsequent references to the MBE in this document are relevant to the MBE 3rd Edition.

1.3.2 Additional Resources

Readers may find the additional resources helpful in obtaining the requisite knowledge for tunnel evaluation and load rating, though the following list is not considered exhaustive. Some are referenced within this Guide, but are not considered foundational as the specifications and manuals previously referenced in Section 1.3.1.

- **AASHTO LRFD Bridge Design Specifications, 8th Edition (BDS) (AASHTO 2017a).** Concepts such as standard loads, load factors, material resistance, and resistance factors that are used in this Guide and the MBE are taken directly from the LRFD BDS. The document represents an agreement among owners for the proper design of highway bridges and is maintained by AASHTO.
- **AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, 1st Edition (LRFDTUN) (AASHTO 2017b).** Similar to the 2017 BDS, the LRFDTUN provides specifications for design, evaluation, and rehabilitation of highway tunnels and is maintained by AASHTO.
- **FHWA Memorandum: Load Rating for the FAST Act's Emergency Vehicles (FHWA 2016).** This is the memorandum that provided explanation of the required ratings for the emergency vehicles enacted by the Fixing America's Surface Transportation Act (FAST Act) law.
- **Technical Manual for Design and Construction of Road Tunnels, Civil Elements (FHWA 2009).** This manual discusses the more up-front planning and design aspects of tunnels. Initial chapters of this technical manual may be useful to understand the chosen tunnel alignment, clearances, and geotechnical and geological mapping information that should be collected. Latter chapters provide an in-depth guide to methods and types of construction.
- **AASHTO Standard Specifications for Highway Bridges (Standard Specifications) (AASHTO 2002).** This publication is a precursor to the 2017 LRFD BDS and may need to be referenced to understand design methodology prior to implementation of LRFD.
- **FHWA Road Tunnel Design Guidelines (FHWA 2004).** This guideline document is a predecessor to the 2009 version of the FHWA Technical Manual for Design and Construction of Road Tunnels, Civil Elements. At this time, it shall be considered as a legacy document.
- **FHWA Highway and Rail Transit Tunnel Inspection Manual (HRTTIM) (FHWA 2005).** This manual covers the specifics and recommended practices used for tunnel inspection but was published prior to the NTIS. The HRTTIM was used in the development of the TOMIE Manual, and has been superseded by the TOMIE Manual for highway tunnels. At this time, it shall be considered as a legacy document for highway tunnels; however, it remains applicable to rail transit tunnels.
- **FHWA Load Rating of Specialized Hauling Vehicles (FHWA 2013).** This memorandum discusses the application and applicability of Specialized Hauling Vehicles (SHV), which includes the Notional Rating Load (NRL), as well as single-unit (SU)4, SU5, SU6, and SU7 loads.

CHAPTER 2 - TUNNEL STRUCTURES

2.1 INTRODUCTION

A general understanding of tunnel structures and construction techniques is presented in this Section to aid and orient the engineer performing tunnel load ratings. A majority of this information is provided in much greater detail in the references listed in Section 1.3.1 and Section 1.3.2, but is briefly summarized within this Section. Basic tunnel types are presented. Construction techniques can have implications for tunnel load rating, and several techniques are mentioned with the potential ramifications for the load ratings. Lastly, the tunnel components subject to load rating are presented, including mention of when they may or may not need to be considered for the tunnel load rating.

2.2 TYPES OF TUNNELS

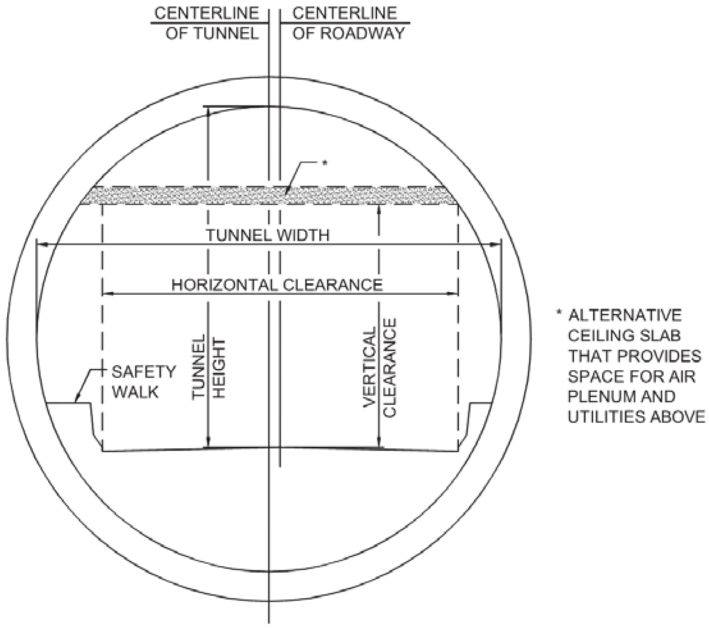
The tunnel structure can be defined by the method used to construct the tunnel. The shape directly affects the load resisting system and load paths through a structure, ultimately affecting the load rating analysis of the tunnel system. This section will briefly describe the various shapes and liner types that are currently used throughout the industry and frequently found in tunnel inventory. This section is not exhaustive, but it is intended to provide a summary of common tunnel types. More information can be found in TOMIE Manual Section 1.4 and LRFDTUN Article 2.4.

2.2.1 Shapes of Tunnels

The shape of a tunnel directly affects the soil-structure interaction response of the tunnel structure. General shapes are presented in TOMIE Manual Section 1.4.2. Shape properties and general load response is briefly described in the following. References to construction types are also described in the following. See Section 2.3 for definitions of types of tunnel construction.

2.2.1.1 Circular Tunnel

A circular shaped tunnel has the greatest cross-sectional area to perimeter ratio (Figure 1). This shape provides a concentric circular load path, which helps distribute loads through compression around the main liner system. This shape includes egg shapes and ellipses, dependent on construction method.

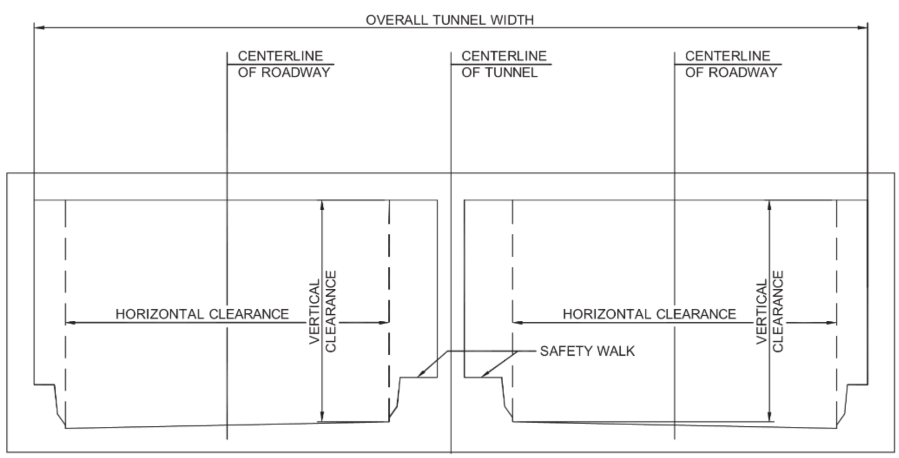


Source: FHWA

Figure 1. Illustration. Example circular tunnel cross section.

2.2.1.2 Rectangular Tunnel

Rectangular shaped tunnels behave very similarly to reinforced concrete box culverts in that a top slab distributes the load to vertical walls (Figure 2). This geometry behaves similarly to other bridge structures with the top slab exhibiting shear and moment responses and interior walls exhibiting primarily axial responses. Exterior walls exhibit shear and moment response due to earth pressure loading in addition to axial response.

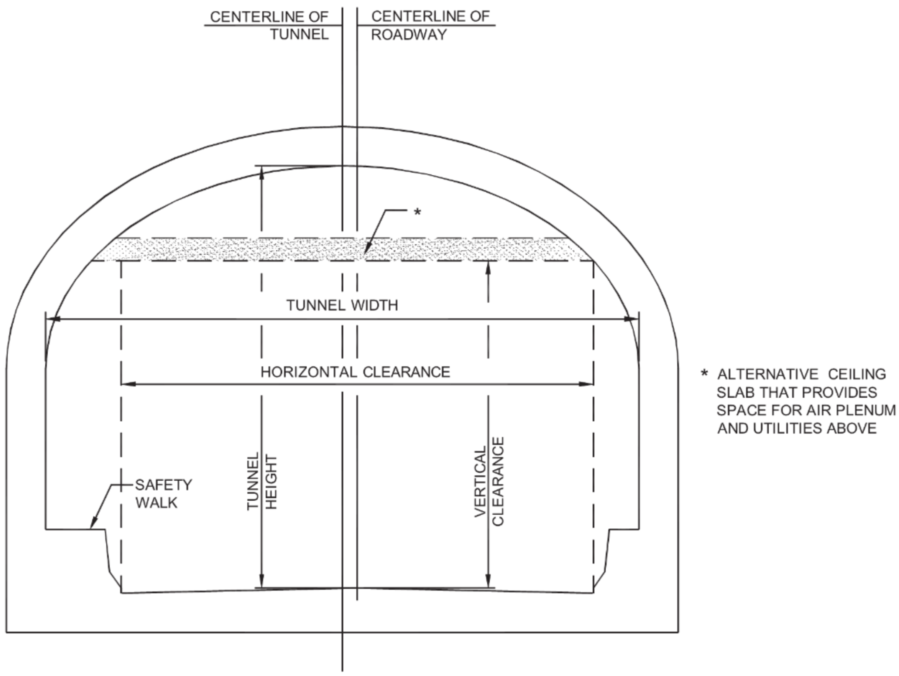


Source: FHWA

Figure 2. Illustration. Example rectangular tunnel cross section.

2.2.1.3 Arch Tunnel

Arch shaped tunnels blend both the circular and rectangular shaped tunnels (Figure 3). This shape generally consists of a flat bottom slab with a circular arch extending above the slab. The addition of a vertical wall system between the floor slab and arch is common, referred to as a horseshoe shape. As with the circular tunnels, this shape provides a concentric circular load path, which helps distribute loads through compression around the main liner system. However, depending on the wall systems and connection to the walls or floor slab, the arch system may develop significantly more moment and shear than a pure circular system.



Source: FHWA

Figure 3. Illustration. Example arch tunnel cross section (horseshoe shape shown).

2.2.2 Liner Types

The tunnel linings, also called ground support elements, provide the main support system, stabilizing the surrounding ground, carrying the surcharges from roadway or structures above, and resisting the infiltration of groundwater. The liner system can consist of a single pass or two pass liner system, with the two pass liner system consisting of an initial liner (or temporary support) and the final liner (or permanent support). Initial liners generally consist of shotcrete and rock bolts, ribbed systems with lagging, or slurry walls. Final liners typically consist of precast or cast-in-place concrete. Liners may or may not be considered part of the load rating system, depending on the load path of other elements or the presence of a roadway above the tunnel structure. When they are considered part of the load rating system, understanding the liner system and the difference between the initial and final liner systems is helpful in properly evaluating the liner components in a load rating. More information can be found in TOMIE Manual Section 1.4.3 and LRFDTUN Article 9.5.

2.2.2.1 Unlined Tunnels in Rock

Tunnels, when constructed through competent rock, may not require a liner because the rock can be self-supporting with the aid of localized or systematic rock reinforcing elements (Figure 4). Unreinforced or reinforced shotcrete may, depending on the situation, be applied to prevent loose rock from falling. Other supports may be needed to control ground or groundwater conditions at localized areas, such as shear zones and faults. Unlined rock tunnels are common in older mountain rail applications but have successfully been converted into local access highway tunnels. This type of tunnel is not common in current design practice.

Source: FHWA

Figure 4. Photo. Unlined rock tunnel.

2.2.2.2 Shotcrete (Sprayed Concrete)

Shotcrete is a pneumatically applied concrete system, conveyed through hoses and projected at high velocity onto the receiving surfaces (Figure 5). Shotcrete is typically reinforced through conventional steel reinforcement, welded wire fabric, steel mesh, fibers, or lattice girders. Typically, shotcrete is reserved for the initial liner, but may be used for the final liner on lightly loaded tunnels. Shotcrete is also used as a local solution to instabilities in rock, especially in conjunction with the sequential excavation method (SEM) of construction. Shotcrete, when used as an initial liner, is typically ignored when considering the effects of the initial liner on the load resisting system.

Source: FHWA

Figure 5. Photo. Shotcrete tunnel liner.

2.2.2.3 Ribbed Systems

Ribbed systems consist of installing timber, steel, or shotcrete ribs at a designed spacing for the encountered ground conditions (Figure 6). Lagging is typically installed between the ribs to provide further support to the existing ground system. The lagging is usually composed of shotcrete or steel decking material, but timber is also used. Ribbed systems are typically reserved for the initial liner with a final liner placed later. Because these are primarily initial liner systems, the effect of the ribs on load resistance is typically ignored because long-term resistance to corrosion or deterioration is not typically considered.

Source: FHWA

Figure 6. Photo. Steel ribbed tunnel liner.

2.2.2.4 Concrete Segmental Linings

Concrete segmental linings typically consist of reinforced precast concrete segments that follow behind the tunnel bore machine, supporting existing ground as the equipment advances (Figure 7). This system is common in soft ground or weak rock applications. Many tunnel boring machines (TBM) will place these segments by thrusting off the completed rings as part of the operations, providing initial and final ground support in a one pass lining process. These segments are designed to resist final loading conditions due to applied loading from the ground, groundwater, and applied surcharge loads from above. Joint systems consisting of gaskets and bolt systems are common to mitigate intrusion of groundwater through the joints.

Source: FHWA

Figure 7. Photo. Concrete segment tunnel lining.

2.2.2.5 Cast-In-Place Concrete

Cast-in-place concrete is typically used as a final liner system (Figure 8). The concrete is typically reinforced with conventional reinforcement or welded wire fabric. This type of tunnel is designed to resist final loading conditions due to applied loading from the ground, groundwater, and applied surcharge loads from above. Joints are typically damp-proofed or waterproofed and a waterproofing membrane is typically used to mitigate intrusion of groundwater and for corrosion protection.

Source: FHWA

Figure 8. Photo. Cast-in-place concrete tunnel.

2.2.2.6 Slurry Walls

Slurry walls are a technique that uses panels filled with slurry placed during primary excavation operations to stabilize earthen sidewalls (Figure 9). After cleaning operations, reinforcement is placed inside the panels and concrete is placed to the bottom of the trench, displacing the slurry as it fills in the panels. Once the wall panels are completed, general excavation can proceed with the slurry wall panels acting as temporary shoring. The effect of the panels on load resistance is typically ignored because they are considered initial (or temporary) liners, and long-term resistance to corrosion or deterioration is not typically considered.

Source: FHWA

Figure 9. Photo. Braced slurry wall tunnel construction.

2.3 TYPES OF CONSTRUCTION

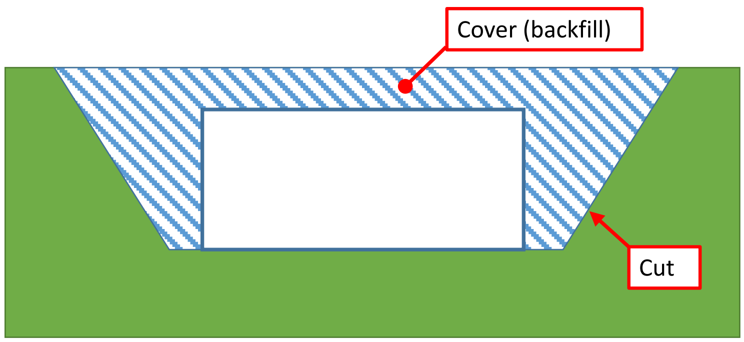
The construction method is important to understand, because it directly affects the analysis decisions required when load rating a tunnel. Similar to the section on tunnel shapes, brief descriptions of common construction methods are provided here, as well as how each method pertains to live load response. Additional information can be found in TOMIE Manual Section 1.4.1. However, for a thorough understanding, each construction type has its own section in the LRFDTUN, including:

- Section 6, Cut-and-Cover Tunnels; Article 6.5 Construction of Cut-and-Cover Tunnel Structures,
- Section 7, Mined and Bored Tunnel Structures; Article 7.5 Construction of Mined and Bored Tunnel Structures, and
- Section 8, Immersed Tunnel Structures; Article 8.5 Construction.

2.3.1 Cut-and-Cover

Cut-and-cover involves excavating through existing ground and building the tunnel within an open excavation or trench (Figure 10). The area around the completed tunnel is then filled in with an engineered backfill material. This type of construction is typical when tunnels are located at shallow depths.

Cut-and-cover tunnel structures are typically cast-in-place or precast concrete structures. Rectangular or arch shapes are common with this method. It is important to note the excavation methods used for these types of structures, because the excavation method can affect the soil loading and interaction around the structure. Excavation can be completed with temporary shoring or open excavation. Backfill soil that is replaced typically has different material properties than the existing ground and needs to be considered in the load rating analysis. Lift thicknesses, compaction requirements, and distance between the completed tunnel and shored systems can all affect this analysis and should be documented by the Owner as part of the tunnel file.



Source: FHWA

Figure 10. Illustration. Cut-and-cover construction.

2.3.2 Shield Driven

Shield driven methods use shields to stabilize surrounding existing soil during excavation. Shield driven methods are sometimes referred to as mechanized excavation methods. This system is used when the ground needs to be controlled during excavation. The shield is advanced one cycle at a time while removing muck and placing ground support concurrent to the face. The process repeats as necessary throughout the construction of the tunnel. Common examples of this construction technique are bored tunnels using a TBM and jacked tunnels (see Section 2.3.2.1). The shape of these tunnels can be circular or rectangular based on the cross section of the shield and the excavation methods used once the shield is in place. This method leaves existing ground in place; therefore, it is important to have accurate boring logs and geotechnical data during a load rating analysis. This information should be documented by the Owner as part of the tunnel file.

2.3.2.1 Bored Tunnels

Bored tunnels are constructed with TBMs, which are shield systems led by drill bits mounted on a rotating cutter head that excavates a circular opening (Figure 11). These machines can be modified for use with varying ground types. The shape of nearly all TBM tunnels is circular. Specific configurations and discussion of TBM use in soft ground and hard rock can be found in TOMIE Manual Section 1.4.1.3.

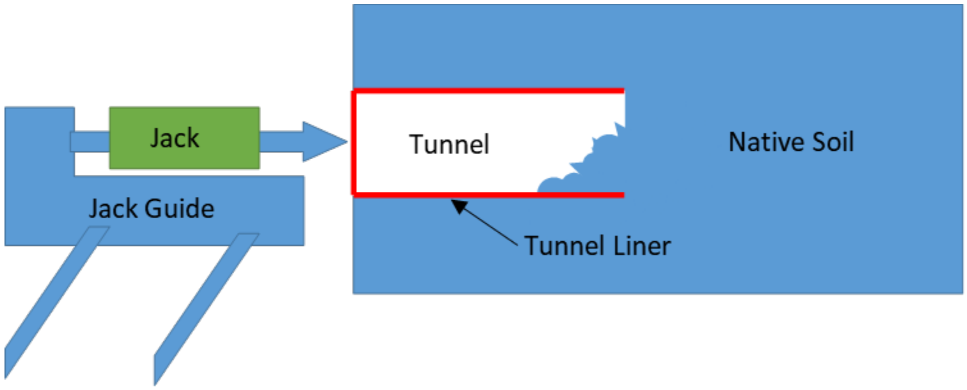
Source: FHWA

Figure 11. Photo. Cross section of pipe drilling machine, similar to a tunnel boring machine.

2.3.2.2 Jacked Tunnels

Tunnel jacking is considered a non-intrusive method of constructing tunnels that can be used when there are conflicts with surrounding highways, buildings, or rail. Headings are advanced by pushing large concrete box sections through the existing soil (Figure 12). Large hydraulic jacks follow the headings, pushing precast tunnel sections into place from a launching pit. TOMIE Manual Section 1.4.1.4 provides additional description and information on this method.

This method of construction creates large friction forces between the existing ground and the precast elements, which may need to be considered in a load rating analysis because this friction force may induce significant stresses into the live load carrying elements. This method leaves the existing ground in place; therefore, it is important to have accurate boring logs and geotechnical data during a load rating analysis. The jacking procedures and equipment used can help assess the frictional forces during construction, when being considered for analysis. This information should be documented by the Owner as part of the tunnel file.



Source: FHWA

Figure 12. Illustration. Tunnel jacking.

2.3.3 Immersed Tube

Immersed tubes are required when a tunnel needs to cross a body of water. Past construction methods used steel shells cast with mass concrete. Modern applications use precast concrete elements that emphasize concrete mix designs to minimize thermal strains and cracking during the fabrication process. Typical construction requires an excavated trench in the waterway bed within which the precast elements are placed (Figure 13). Gaskets are used between segments to ensure a watertight joint. A protective fill is then placed on top of the tunnel elements to protect against ship anchors, scour, and to resist positive buoyancy. Because this is a submerged construction method, a load rating analysis of the liner elements is typically not required unless external parameters have changed. Internal elements supporting live loads may need to be load rated.

Source: FHWA

Figure 13. Photo. Precast immersed tubes in casting yard, prior to in-situ submerging.

2.3.4 Drill and Blast

Drill and blast methods have a long history of success in the U.S. This method is typically limited to hard rock tunnels and are cost effective for shorter applications or when the rock mass is susceptible to variable conditions such as faulting and shear zones. The shapes of these tunnels are typically circular or arched with the existing rock forming the main load resisting system. A final liner can be placed to mitigate concerns due to rock variances and falling debris. Unless there are highly unusual circumstances, this construction type does not require a load rating analysis. Internal elements supporting live loads may need to be load rated.

2.3.5 Sequential Excavation Method

Sequential Excavation Method (SEM) is an effective technique when the existing ground does not have enough strength or stability for a full excavation. This requires the excavation to be completed in stages. The intermediate stages are supported as necessary, and the next portion is incrementally excavated until the full cross-section is completed. With this method, a geotechnical engineer usually monitors the ground conditions at the face while crew members install the initial support needed to stabilize the opening. Typically, this method requires a final liner that ultimately becomes the final load resisting element (Figure 14). The SEM procedures and equipment used can be helpful if considering the effect of several locked-in forces, including frictional forces and settlement during construction. This information should be documented by the Owner as part of the tunnel file. Figure 14 shows SEM construction as well as drill and blast construction.

Source: FHWA

Figure 14. Photo. Example of SEM construction.

2.4 TUNNEL COMPONENTS SUBJECT TO LOAD RATING

The previous sections outlined general tunnel shapes, liner types, and construction methods. These sections are intended to provide a general summary of current tunnel design practice as well as provide references for additional information. There are various aspects of tunnels that need to be considered for load rating dependent on the tunnel type and its internal components.

In general, loading from vehicles and other live loads can be considered to act on a tunnel both internally and externally. Live loads acting internally to the structure include elements that are constructed within the tunnel. This includes roadway systems, support structures, support walls, and other elements that carry live loads from the roadway surface to the tunnel liner and/or supporting ground. Live loads acting externally to the structure are generally transmitted through surcharge loads in the surrounding ground. These are typically applied by vehicle or rail loading above a structure and are typically resisted by the main tunnel liner system. It is important to note that depending on geometry and connection detailing, loads may be transmitted from the liner to internal elements.

The load rating Engineer needs to consider the load paths of the whole tunnel system when performing a live load analysis. The load rating Engineer should evaluate all components that transmit live load and rate each component for the induced live load forces. These components may include, but are not limited to, invert slab, tunnel liner, invert beams or girders, roof girders, ceiling structures, or substructure elements. The live load carrying element with the lowest rating factor is to be the controlling and possible posting element for the tunnel structure.

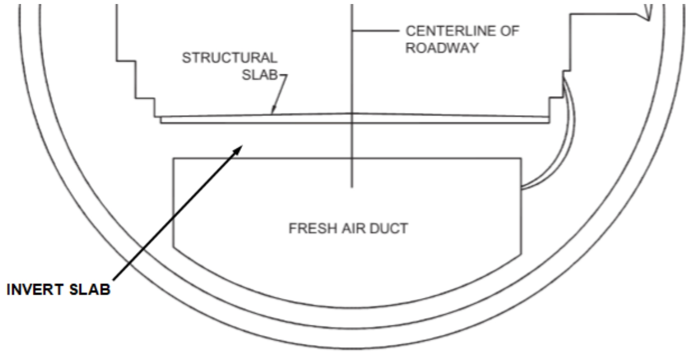
Sample load distributions are shown in Figure 49 and the load rating examples are presented in Chapter 9. Original design documents can greatly help the load rating Engineer fully understand the approach to the original design, while incorporating changing conditions and deterioration. It should be noted that non-vehicular carrying components may need to be evaluated for structural capacity in situations when deterioration or distress is observed during an inspection. See Chapter 8 for more details.

2.4.1 Invert Structures

The invert of a tunnel is the internal component or system that carries live loads through the tunnel. This invert is typically constructed using a slab or slab and girder system that spans transverse to the direction of traffic (Figure 15). For a rectangular or arch shaped tunnel, the invert is typically considered the bottom of the tunnel. This type of invert is typically founded on grade or directly on the tunnel liner system, transmitting loads directly between the structural system and the ground below the tunnel. However, there are instances when a rectangular or arch shaped tunnel will have a plenum below the invert component or system in which the load distribution will be similar to what is described in the following for a circular shaped tunnel.

2.4.1.1 Structural Slabs

For circular shaped tunnels, the invert slab is typically constructed using cast-in-place or precast concrete slabs (Figure 15). The concrete slabs are typically reinforced using conventional reinforcement. For circular tunnels, these slabs provide the main load resisting path for applied dead and live loads. Loading is commonly transmitted through the structural slab to support walls, which in turn transmit the loads to the liner system. This system is considered an internal system to the tunnel. When performing a load rating analysis, this system should be included.



Source: FHWA

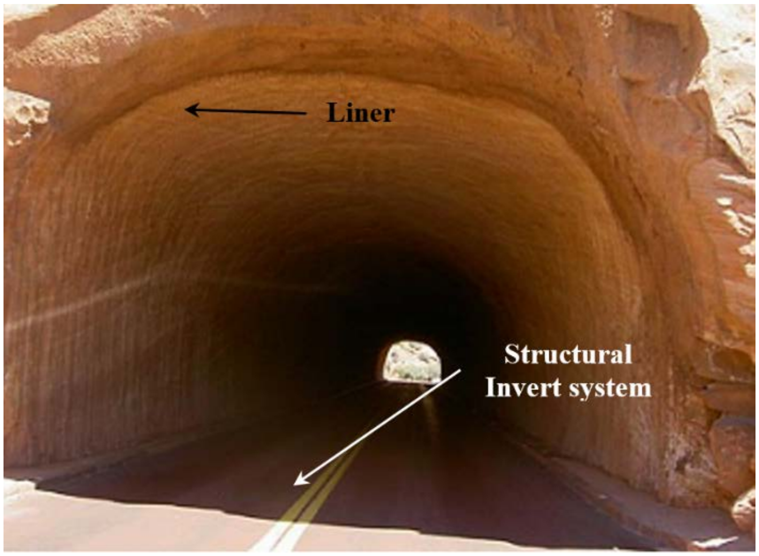
Figure 15. Illustration. Example structural slab in circular shaped tunnel liner.

2.4.1.2 Support Beams and Girders

The use of a slab and girder system is an alternative to using just a structural slab in circular shaped tunnels. This system uses a floor slab that transmits loads to a beam and girder system below the invert. The beam and girder system typically transmits loads to support walls, which in turn transmit the loads to the liner system. This system is considered an internal system to the tunnel. When performing a load rating analysis, this system should be included.

2.4.1.3 Slabs-on-Grade

For rectangular or arch shaped tunnels, the roadway is placed directly on grade (Figure 16) or is supported by a slab-on-grade (Figure 2). Typically, there is a direct transfer of live loads to the ground beneath the invert through the structural slab. While this system is considered an internal system to the tunnel, it is not typically included in the load rating analysis if loads are transmitted directly to the ground below. NTIS (23 CFR 650.513(a)) exempts at-grade roadways in tunnels from load rating.

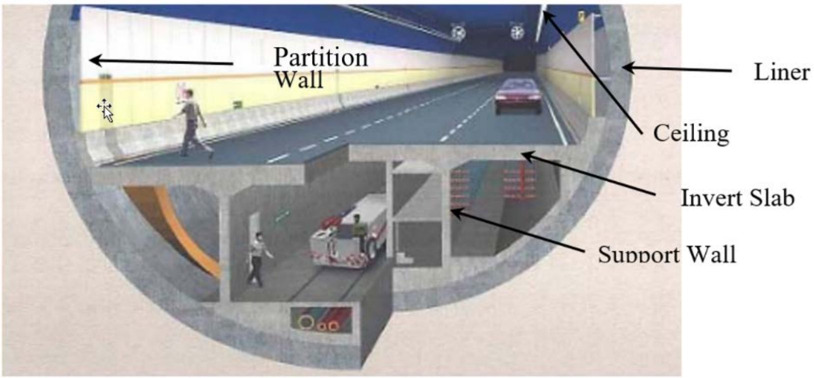


Source: FHWA

Figure 16. Photo. Tunnel with roadway directly supported by rock subgrade.

2.4.2 Internal Columns and Walls

It is common for invert slabs in circular shaped tunnels to have columns or walls that support the invert slab and transfer loads from the invert slab to the foundation system, directly to the liner, or directly to the ground below (Figure 17). The presence of columns and walls creates a frame to resist these loads and is often rigidly connected to the invert slab. This frame system is considered an internal system to the tunnel. When performing a load rating analysis, this system should be included.



Source: FHWA

Figure 17. Illustration. Tunnel with internal columns and walls supporting the invert.

2.4.3 Roof Girders

Roof girders directly support the main horizontal or roof portion of the tunnel liner. The roof girders support the backfill, surcharge. and traffic above. These systems are generally composed of steel (see Figure 18) or concrete girders supporting a decking system. These elements are subject to load rating if the live load above the tunnel is within the zone of influence, which is discussed in Section 4.3.2.

Source: FHWA

Figure 18. Photo. Girder-supported roof.

2.4.4 Ceiling Structures

Similar to roof girders, ceiling structures are common in many different tunnel applications. These structures, typically composed of ceiling slabs or ceiling panels with ceiling girders and hanger rod assemblies, typically are used to form an upper plenum, which may be used for supply and/or exhaust air distribution, to convey tunnel equipment and utilities, and to provide access for tunnel maintenance (Figure 19). Typically, these items do not directly support live loads and are not typically included in the load rating analysis. However, the Owner may require load rating of the ceiling structure if tunnel maintenance traffic is transported across this element. In some instances, dynamic air forces from live loads (piston effect) may have significant effects on the ceiling structural stresses and should therefore be considered. Additionally, this element may act as a strut supporting the outer walls, in which case it may be affected by live loads and would require live load rating. Non-live load carrying elements do not need live load rating; however, they may need strength evaluation if deterioration or distress is observed during inspection. See Section 8.2 for more details. Care shall be taken with the conditions of ceiling elements because they are typically directly above traffic and critical to the safety of the travelling public.

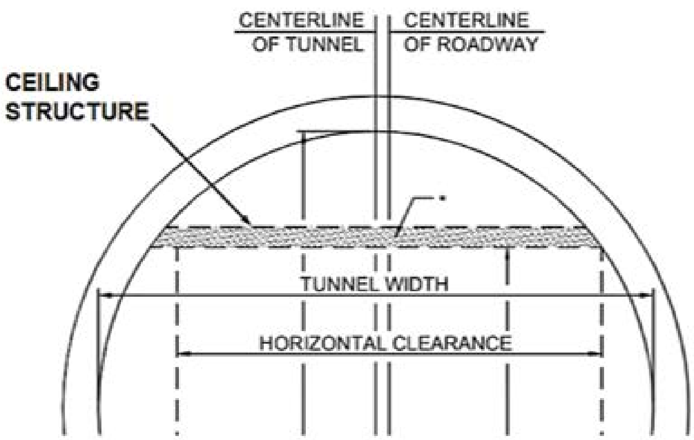


Illustration Source: FHWA, Photo: © 2019 HDR

Figure 19. Illustration and Photo. Tunnel ceiling structure.

2.4.5 Liners

As described previously, liners are the main force resisting system for external forces and are considered an external system. Liners may be considered part of the live load resisting system in two ways. The first is if there is a component of the internal system that is linked to the liner. For example, columns or walls may be framed into the liner. The second is when there is an external live load source that is applying a surcharge through the surrounding ground system. This occurs when there is a roadway or any other live load source above the structure applying these loads, see Section 4.3.2. Evaluation of the live load path through the internal system and in consideration of external forces is required when making a decision whether or not to include the liners in the load rating analysis.

2.4.6 Substructures

Substructures are uncommon in tunnel structures. There may be special circumstances where a substructure element is used to transfer loads from an internal system to the ground below. As noted in the MBE Article 6.1.5.2 (AASHTO 2018):

> *Members of substructures need not be routinely checked for load capacity. Substructure elements such as pier caps and columns should be checked in situations where the Owner has reason to believe that their capacity may govern the load capacity of the entire bridge.*

As such, substructures will not typically be included in a load rating analysis, and will not be discussed further in this Guide. However, the load rating Engineer should carefully exercise their engineering judgment when determining if a substructure should be considered in the load rating analysis. The factors considered may include shapes, types, load paths, redundancy, and deterioration.

CHAPTER 3 - DATA COLLECTION

3.1 INTRODUCTION

The following sections contain information that is used for load rating assessments. Data collected to perform the load rating should be adequately documented. However, when this information is unavailable, it can lead to increased effort to obtain the necessary information, or require engineering assumptions that increase the uncertainty of the load rating. All assumptions should be documented when information is not available and engineering judgment is used.

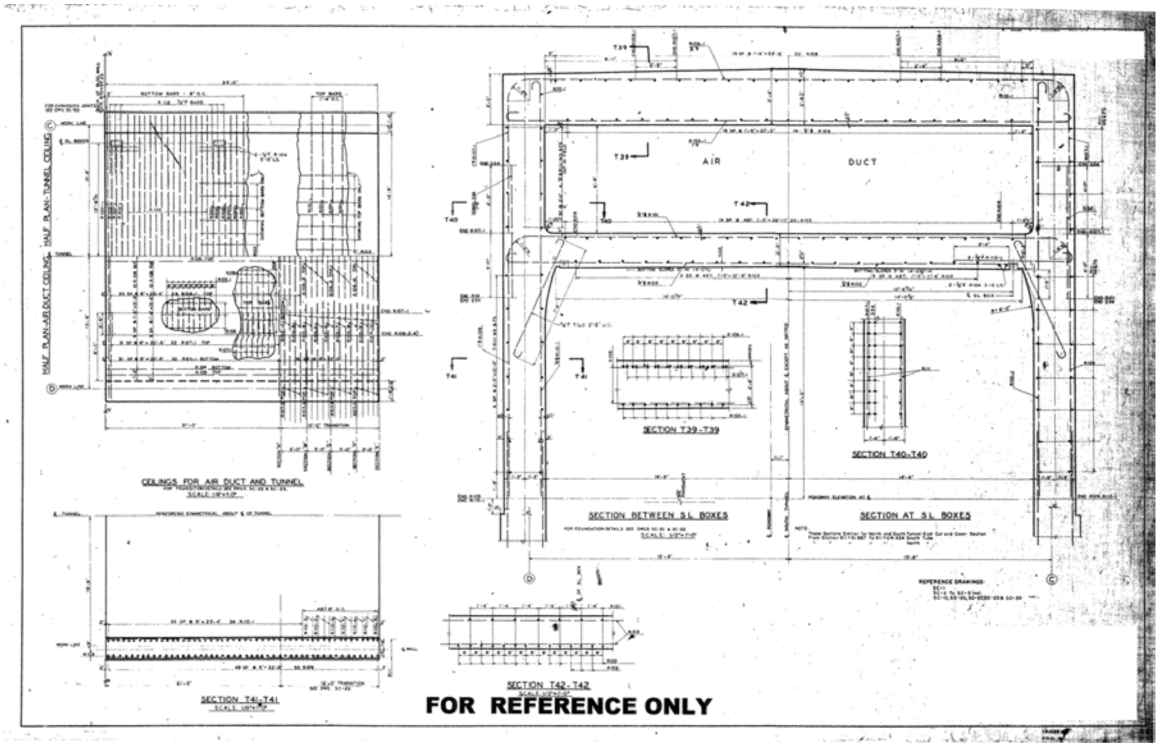
3.2 COMPONENTS OF TUNNEL RECORDS

The NTIS now requires routine inspection of all tunnels on all public roads, similar to the existing NBIS for bridges, including the establishment of the National Tunnel Inventory (NTI), similar to the NBI. Tunnel Records shall be maintained similar to Bridge Files and Documentation as outlined in MBE Section 2. Due to the complexity of soil-structure interaction, component response, and other unique design concerns related to tunnels, Tunnel Records should typically be more extensive than what is found in a typical Bridge Record. Tunnel Records shall maintain all information that may affect the performance or load rating of a tunnel structure.

3.2.1 Plans

3.2.1.1 Construction Plans

Tunnel Records should maintain clear and readable sets of all drawings. Construction plans are critical because they often convey the intent of the design engineer and will indicate requirements and assumptions for shoring, phasing, and other construction issues that may affect load rating assumptions (Figure 20). The construction plans will provide the geometry, material, component cross section, reinforcing patterns, and similar that are needed to determine the resistance (capacity) of individual members to be load rated. Similarly, the geometry of the tunnel, or tunnel components, is necessary to develop an accurate numerical model. In the instance where sufficient construction plans are not available and critical information cannot be collected from field measurement, rating values may be obtained as outlined in MBE Article 6.1.4.



Source: Pennsylvania Department of Transportation

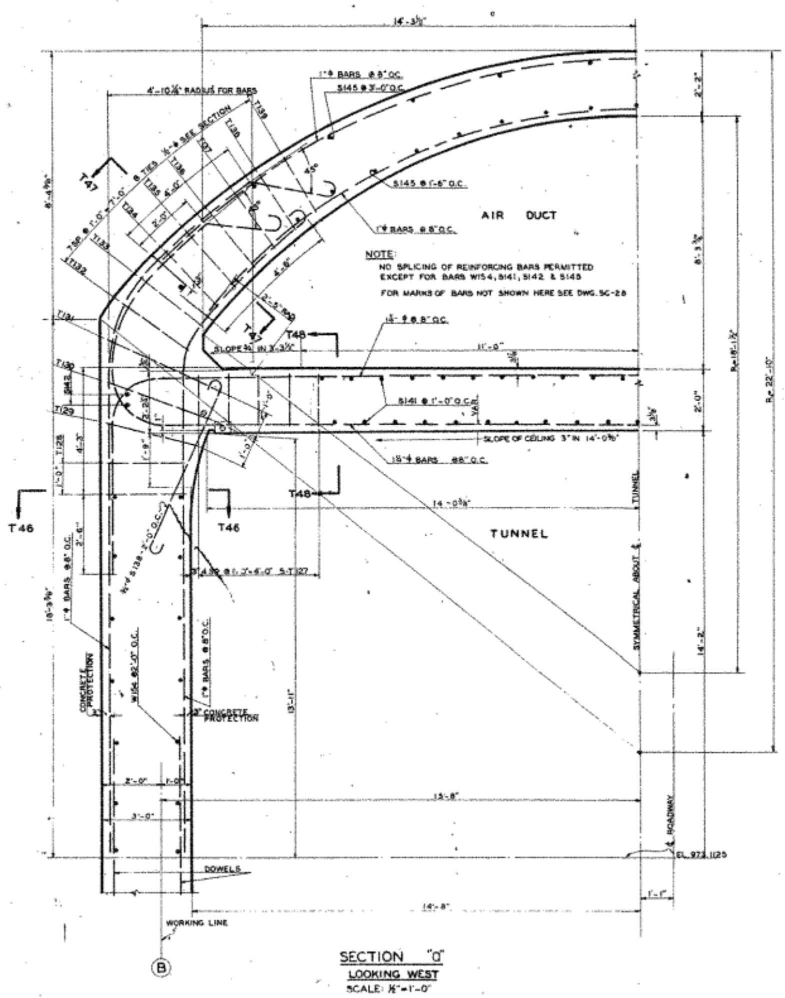
Figure 20. Illustration. Sample tunnel plan sheet (courtesy of Pennsylvania Department of Transportation).

3.2.1.2 Shop and Working Drawings

Tunnel Records should maintain sets of all shop and working drawings developed by the contractor, especially when those drawings indicate means and methods for constructing tunnel elements that will be load rated (for example, Figure 21). In addition to shop and working drawings, logs of excavation work may prove valuable in conveying soil information and resolutions to issues encountered during the construction.

3.2.1.3 As-Built Drawings

As-Built Drawings are important to maintain because they are usually the only record of deviations from the design drawings.



Source: Pennsylvania Department of Transportation

Figure 21. Illustration. A sample detail showing tunnel reinforcing (courtesy of Pennsylvania Department of Transportation).

3.2.2 Other Records

3.2.2.1 Geotechnical Studies

Because many tunnel elements are affected by soil-structure interaction, an understanding of the surrounding soils is necessary to perform an accurate rating analysis. Complete geotechnical reports from the original design should be maintained. If there are borings for nearby projects, the Tunnel Record should be supplemented by this additional information. The geotechnical studies need to provide the load rater with enough information to develop an understanding of the surrounding soil and material properties and design parameters appropriate for analysis. Section 5.6 presents information as well as several parameters that need to be obtained from the geotechnical studies for the load rating analysis.

3.2.2.2 Specifications

Specifications should be maintained in the Tunnel Record. Where a standard specification was used, a reference to that standard with year of publication should be kept to provide a reference to the historical specification. All special provisions should also be kept. Specifications can provide insight into construction methods, including excavation methods, shoring requirements, and soil property information.

3.2.2.3 Materials and Tests

Material testing can provide critical information when performing a load rating analysis. Reports of nondestructive and laboratory tests of materials should be maintained for reference. Other reports, such as compaction results for backfill, should also be maintained where the information is pertinent to the proper evaluation of a tunnel structure. Material certification records, when required, should also be maintained.

3.2.2.4 Photographs

Each Tunnel Record shall contain at least a section view of the tunnel portals. Photographs showing major defects or other important features shall also be included. For Tunnel Records, it may be helpful to maintain photographs of major construction activities for future reference. These photographs include excavation operations, tunneling operations, shoring, and other activities that affect the evaluation of the tunnel structure.

3.2.2.5 Maintenance and Repair History

Maintenance and repair history shall be maintained documenting a chronological record of work that has occurred since the initial construction of the tunnel. Details shall include the date of work, description of work performed, the contractor, cost, and related data as mentioned previously in this chapter.

3.2.3 Inspection History

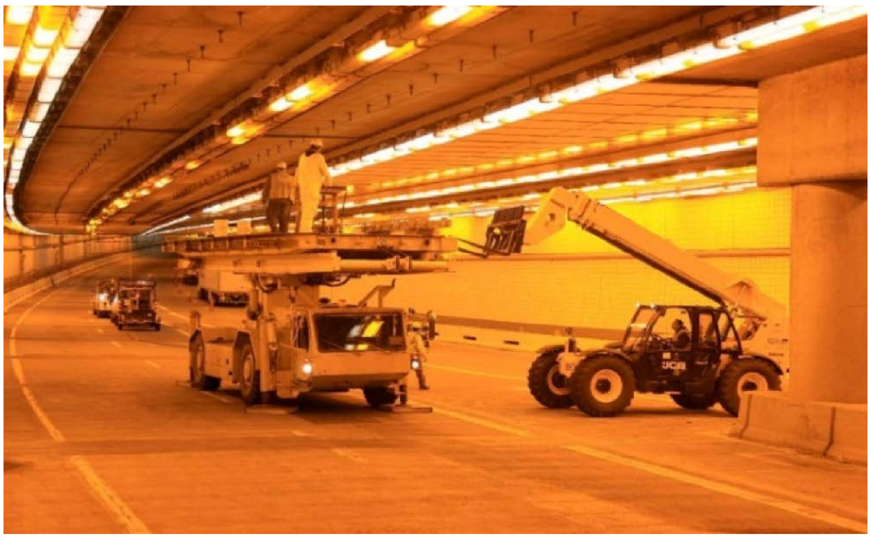
Records of inspection history shall be maintained in chronological order to provide reference to changing conditions within the tunnel structure. These inspections are part of the TOMIE Manual Section 4.6, including the initial inspection, periodic routine inspections, and inspections after an event that could potentially damage the tunnel. The initial inspection shall be in the Tunnel Record. Additionally, any additional evaluations, such as scour and liner integrity, post-seismic, corrosion, and fatigue evaluation studies, shall be maintained with the inspection history.

3.2.4 Load Rating Data

The Tunnel Record shall contain the complete history of all load rating evaluations, including the original load rating performed by the Engineer-of-Record, if available (as it may often be the case that these records have not been retained). The initial load rating can provide insight into the original Engineer's design intent. Subsequent load ratings that account for existing conditions can provide insight into changing conditions, loading, and deterioration that may have been present during subsequent analyses.

3.3 INSPECTION

Inspection is a necessary part of tunnel load rating, just as it is for bridges (Figure 22). The TOMIE Manual was developed to serve as a resource in the areas of tunnel operations, maintenance, inspection, and evaluation. TOMIE Manual Chapter 4 provides information and guidelines for developing a comprehensive tunnel inspection program. This section provides selected discussion regarding adverse effects inspection may have on a load rating analysis. Additional information regarding inspection practices can be found in the TOMIE Manual.



Source: FHWA

Figure 22. Photo. Inspection using a mobile elevated platform.

The following sections identify key information regarding the collection of data for load rating.

3.3.1 Data Collection for Load Rating

Data collection during inspection is necessary to identify changing conditions in the tunnel structure. MBE Article 6.1.1 states that any change in condition, whether due to deterioration or alteration, may require re-evaluation of the member or system. This requirement is up to the discretion of the Owner and rating Engineer. However, it is typical industry practice to re-evaluate the structure when a change occurs that is greater than 5 percent to 10 percent of the previously noted condition. Owners shall establish a policy for when a re-rating is required, and the load rating Engineer shall carefully exercise their judgment considering the change and other factors such as load posting status, and traffic condition when determining the need for a re-rating.

Because of this, items necessary for load rating shall be considered during inspection activities, with enough information gathered when an issue is identified to allow a reevaluation of the load rating to be conducted. Similar to bridge structures, important items generally include deterioration, deformation, section loss of the load supporting components (that is, the tunnel liner and internal structural members) and any change in loads or weights applied to the structure.

Before load rating a tunnel, the current condition and existing loading data for the tunnel shall be reviewed and evaluated. The quality of the data will have a direct influence on the accuracy and reliability of the load rating results.

3.3.1.1 Structural Materials

TOMIE Manual Section 4.9.1.1 identifies the various structural elements required as a part of a routine inspection. Not all of this information is required for rating, and this Guide will present these structural elements with respect to load rating in detail in Section 6.5 of this Guide. The following list describes which structural materials require different evaluations.

- Steel structures are susceptible to section loss due to corrosion or impact damage; cracks due to fatigue stress cycles; and misalignment due to overload or impact damage. Each of these has a different meaning for the structural rating of the tunnel, be it structural capacity or anticipated remaining life.
- Concrete structures are susceptible to scaling, cracking, spalling, and delamination; which can cause deterioration of the concrete and prestressing of strands and reinforcing steel. All of these can be caused by various factors and have different ramifications on the load rating of the tunnel or the expected remaining life.
- Timber structures are subject to section loss due to decay or damage. Fungi, insects, fire, and impact damage can all reduce timber members and thus affect the load rating of the tunnel.
- Masonry structures are subject to deterioration of the masonry units, deterioration of the mortar, or shifting of the masonry relative to its intended position.

Each material type can be a tunnel liner or internal part of the structure such as the invert slab, invert girders or structural walls. Any and all deterioration shall be recorded as a part of Section 3.2 of the SNTI, in the Structural Section.

3.3.1.2 Structural Components

There are several structural components, described in Section 2.3 that each can potentially be affected by deterioration. TOMIE Manual Section 4.9.1 discusses each of these structural components in detail. As such, each is only briefly discussed herein.

- Tunnel liners provide the main support against the surrounding soil, hydrological, or surcharge forces and are all subjected to deterioration no matter the material type.
- Roof girders provide the main support for loads bearing on the tunnel roof and may also experience axial compression from laterally applied loads on the tunnel liner. Roof girders are almost always steel or reinforced concrete.
- Structural columns and walls provide support for the liner and/or the horizontal load carrying members.

- Invert slabs and slabs on girders serve the same function as a bridge superstructure. Invert slabs are most often concrete, and the supporting invert girders, when present, are concrete or steel beams.
- As noted in TOMIE Manual Section 4.10, geotechnical elements can be integral to the behavior of some tunnel types. Changes to a tunnel cover or fill can affect the structural rating. Either a reduction or an increase in cover could affect how loads are distributed to the tunnel liner. Soil or rock conditions subject to scour or other localized changes can also affect how a tunnel responds to load.
- Similar to bridges, other miscellaneous structural members can have an effect on the rating such as interior joints.

As mentioned in the previous section, deterioration from decay, fatigue, impact damage, or other sources shall be recorded in accordance with SNTI Section 3.2—Structural Section. Within those codes are the quantity and extent of the damage or deterioration (Condition State). Ideally, the inspection will include photographs detailing the extent of the damage to assist the rating Engineer in assessing what should be incorporated into the load rating. As will be shown in Chapter 6, these shall be incorporated into the load rating for capacity determination. Severe change in sections or material properties of structural members may result in change in load distribution due to structural stiffness variation. It such instances, structural analysis should be updated in addition to recalculation of sectional resistance.

3.3.2 Data Verification

This section is equivalent to MBE Article 4.3.8.1 and provides some of the basic information that can be collected from the field for the load rating.

3.3.2.1 Geometric Data

Geometric data shall be reviewed using the latest inspection reports and the available as-built plans, working drawings, and construction plans. Changes in tunnel dimensions, members, roadway, and other elements shall be evaluated for accuracy with additional field data gathered if deemed necessary. This data shall be reviewed and evaluated for all elements that may affect the load rating. The majority of these items are required to be collected and logged in accordance with SNTI Section 2.5 Geometric Data Items.

3.3.2.2 Member and Condition Data

The inspection reports shall be reviewed for changes in member and condition data. This includes evaluating changes in original members and data for changes and retrofits to the tunnel system. Material loss, member damage, reconstruction, and fatigue and fracture prone members shall all be evaluated. The majority of this data is required to be collected in accordance with SNTI Section 3.2—Structural Section.

3.3.2.3 Loading and Traffic Data

Changes in loading and applied traffic can affect the load rating. When dead loads change, which may be due to increased fill, wearing surfaces, non-structural aesthetic or utility additions, the load rating will necessarily change. Traffic volumes, speed limits, and surface condition can also affect the load rating. Missing or incomplete data shall be identified and the information supplemented as required with additional field investigations. This information shall be on record in accordance with SNTI Section 2.3—Age and Service Items.

3.3.2.4 Field Tests

Field tests may be used to supplement missing, incomplete, or incorrect data. The need for field tests will be determined by the rating Engineer and Owner, with input from the inspector. Field tests may include non-destructive or destructive testing to determine material properties, reinforcement information, and geotechnical data. A description of available field tests can be found in MBE Article 5.2. Additional field tests, outside of what is described in the MBE, may be required. The application and use of these additional field tests should be undertaken following general procedures and recommendations based on accepted engineering and testing practices.

3.3.3 Sample Forms

For information and examples of sample forms, see the TOMIE Manual, Section 4.12.2, and Table 4.6.

CHAPTER 4 - LOADS FOR EVALUATION

4.1 INTRODUCTION

Chapter 4 of this Guide describes the loads to be used in determining the load effects for use in load rating as described in Chapter 6 of this Guide. As specified in MBE Article 6A.2.1, generally only permanent loads and vehicular loads are considered to be of consequence in load rating. For tunnels and other types of buried structures, earth loads also warrant inclusion. Environmental loads, such as wind, ice, temperature, stream flow, and earthquake load are generally not considered in rating except when specific conditions warrant their inclusion.

4.2 PERMANENT LOADS

Tunnel load rating shall consider all permanent loads that are applied to the structure. Permanent loads include dead loads and locked-in force effects from the construction process. Typically, these loads are limited to the current existing actual loads and do not include future loads, such as future wearing surfaces (see MBE Article 6A.2.2).

4.2.1 Dead Loads: DC and DW

The dead load effects on the structure shall be computed based on conditions that exist at the time of analysis. Dead loads include the weight of all components contributing to the load effects on the structural element being evaluated for rating such as the tunnel liner, ceilings, roadway slabs, appurtenances, mechanical equipment, signs, signals, systems, utilities attached to the structure, or any other temporary or permanent loads applied to the structure. When present, utilities, wearing surfaces, and attachments shall be field verified at the time of inspection. Minimum unit weight of materials shall be in accordance with BDS Table 3.5.1-1, unless more precise information is available.

Loads shall be based on actual field conditions. Past experience has shown there can be significant variations in deck thicknesses, liner thicknesses, and wearing surface thicknesses. Multiple measurements along the structure may be required. MBE Article 6A.2.2.3 allows for a reduction in load factor for wearing surface loads if they are field measured.

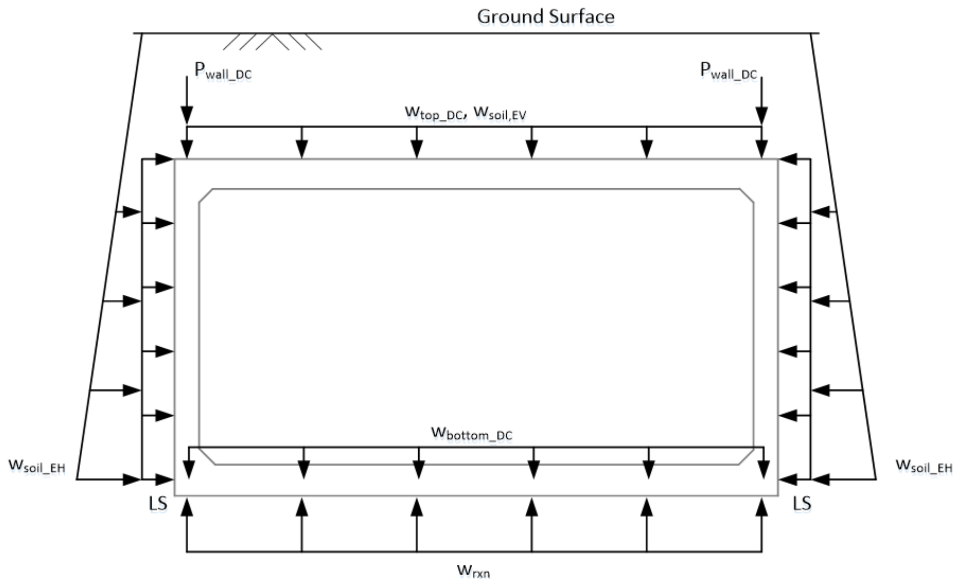
As noted in the LRFDTUN (Article 3.5.2), vertical earth pressure (EV) is generally included in the dead load grouping, which consists of component dead loads (DC) and future wearing surface dead load (DW). The LRFDTUN clarified this condition to include EV loads with the dead load grouping for mined and bored tunnels. For cut-and-cover tunnels, EV shall be treated as a separate load grouping as discussed in the following section. The calculation of the earth loads above the structure is not always the weight of the earth. Soil arching can occur, especially in deep tunnels, where soil stresses are redistributed around the structure as it deforms (Terzaghi et al., 1996), which may affect the earth loads. Active arching can occur when the tunnel is more compressible than the surrounding soil, resulting in lower earth loads on the structure than in the surrounding ground (Tien, 1990). In contrast, passive arching can occur if the ground is more compressible than the structure, which may lead to increased earth loads on the structure. Earth loads should be determined by experienced geotechnical engineers when required for load rating. The design loads for underground structure depend heavily on the characteristics of the material being excavated. Rock and soil conditions can be highly variable and a significant site investigation and characterization program is required to apply the appropriate methods for determining overburden soil and rock loads both vertically and horizontally on the structure. Depending on the characteristics of the material, for example, sound intact rock or highly fractured rock, empirical or advanced numerical methods can be applied to determine the loading conditions on a tunnel liner and ground support systems.

4.2.2 Earth Pressure: EV, EH, and ES

As noted in the LRFDTUN (Article 3.5.2 and Article 3.5.3), earth pressures and surcharge load groups shall be considered for earth loads applied on cut-and-cover and immersed tunnels in accordance with the BDS. MBE Article 6A.5.12.10.1 and Article 6A.5.12.10.2 also provide additional guidance. The load rating shall account for current soil conditions at the time of load rating. If time differential soil conditions affect the loading to the tunnel structure (such as settlement friction), these effects shall be appropriately accounted for.

LRFDTUN Article 3.5.2 provides further discussion of earth loads with respect to tunnels and how these loads shall be evaluated. Generally, EV shall be considered for cut-and-cover tunnels and immersed tunnels for backfill placed directly over the structure. Vertical earth loads for mined (except mined soft ground tunnels) and bored tunnels can generally be considered part of the dead load grouping.

Horizontal earth pressure (EH) and earth surcharge pressure (ES) shall be determined in accordance with the BDS with modifications outlined in the LRFDTUN. As noted in the LRFDTUN, earth surcharges are typically the result of dead load, live load, and other load groups that are associated with load effects other than ES. These loads may be foundation loads, wind loads, soil sliding forces or other loads that are associated with load factors other than ES. The controlling uncertainty in load prediction for these surcharges, however, is based on the transmission of these loads through soil to the tunnel. Therefore, ES shall be applied to these unfactored surcharge loads. These load applications are shown in Figure 23.



Source: FHWA

Figure 23. Illustration. Typical load application on tunnels.

As noted in the LRFDTUN, when applying earth loads, loads should be applied taking into account construction staging and sequencing, when such information is available. If no such information is not available, best estimates should be implemented with engineering judgment. It is important for the Owner to maintain staging and construction sequencing information in the tunnel record, as different construction assumptions can provide widely variable results for loading. For example, cut and cover tunnel structures can be susceptible to load changes due to construction staging and sequencing (for example, the depth of excavation and support of excavation system). Although not directly loading the structure, support of excavation systems can induce lateral loads as well as influence conditions at the base of excavation as the work proceeds to the design depth. Subsequently, loading can be incurred directly on the structure from backfilling and compaction operations as well as the placement of support systems such as ventilation and fire life safety components.

Furthermore, following current LRFD design practice, uncertainty in future load conditions are accounted for in the resistance and load factors applied. However, there are several conditions where loads on a structure may change over time: examples might include loads from weather events such as those that were experienced in New York City following Super Storm Sandy. Other unanticipated loads may be incurred due to adjacent construction or groundwater conditions.

Earth loads should be based on known soil properties, where possible. If soil properties are not known, information from governing construction specifications may be used or field testing may be completed to supplement incomplete or missing soil information.

4.3 LIVE LOADS

Load ratings can be restricted to tunnel components and elements that are subject to highway vehicular loads and/or loads associated with specialized vehicles used for tunnel maintenance and operations. As noted in Chapter 2, live loads can be applied to the external system or to the internal system. An understanding of the tunnel type and configuration is needed to ensure live loads are being applied to all affected elements in a proper manner.

In accordance with MBE Article 6A.2.3, the vehicular live loads used in the evaluation are selected based on the purpose and intended use of the evaluation results. Live loads for load rating include the Design Load (HL-93 for LRFR), Legal Loads, and Permit Loads. Application of these live loads shall be in accordance with MBE Article 6A.2.3.2.

To account for the probability of simultaneous lane occupation by the full HL-93 design live load, a Multiple Presence Factor (MPF) can be applied to the live load vehicle(s). In lieu of site specific data, the MPF values listed in LRFD BDS Table 3.6.1.1.2-1 can be used.

4.3.1 Live Load Application

The number of traffic lanes to be loaded and the transverse placement of wheel lines shall be in conformance with MBE Article 6A.2.3.2 and LRFD BDS Article 3.6.1.3.

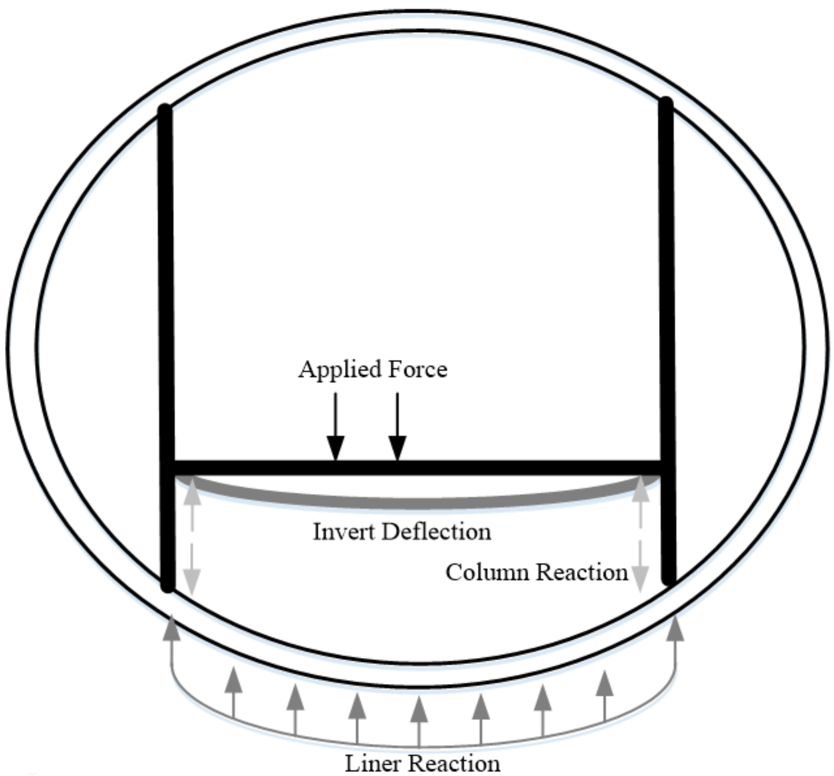
4.3.2 Live Load Distribution

Live load distribution is dependent on the vehicle configuration and the element that is being rated. Approaches to distributing live loads can be found in LRFD BDS Chapter 3 and LRFD BDS Chapter 4 and in LRFDTUN Article 6.9.

4.3.2.1 Internal Systems

For internal systems, live loads are typically applied directly to the main load carrying element, such as an invert slab or slabs spanning between supporting invert girders. Loads are then distributed to the support system and ultimately to the tunnel liner and to the soil outside the tunnel. It is common for internal systems to have the main load resisting element perpendicular to the direction of travel. Although this is not typical for structures where the main force resisting elements are typically parallel to the direction of travel, guidance is given in the LRFD BDS as described below. Alternatively, more refined analysis using other empirical methods or finite element approaches may be used if desired or appropriate.

Invert Concrete Slabs – Invert slabs typically span perpendicular to the direction of traffic. The live loads of each axle/wheel can be distributed over a defined length longitudinally (parallel) to the direction of traffic. The length of longitudinal distribution of the axle/wheel loads should be determined using the approximate equations of LRFD BDS Article 4.6.2.1 for span lengths less than 15.0 feet or LRFD BDS Article 4.6.2.3 for span lengths greater than 15.0 feet (LRFD BDS Article 4.6.2.1.2), or a more refined analysis such as a finite element modeling approach may be used, if desired or appropriate. This longitudinal distribution of the axle loads produces a uniform load which can be multiplied by the width of the transverse strip to be analyzed (typically 1 foot) and the axle load then broken into the individual wheel loads. The individual wheel loads for each type of vehicle are evaluated as point loads that can move along the transverse analysis strip within the limits prescribed in LRFD BDS Article 3.6.1.3 and MBE Article 6A.2.3.2 to produce the extreme force effects for the critical sections being evaluated. In cases where the distribution length is greater than the axle spacing, the axles are considered as a single load distributed over the distribution length plus the axle spacing. Example 1 in Chapter 9 illustrates the distribution of live load for a concrete invert slab and Figure 24 illustrates the load distribution through an internal tunnel roadway system.



Source: FHWA

Figure 24. Illustration. Load distribution of internal system.

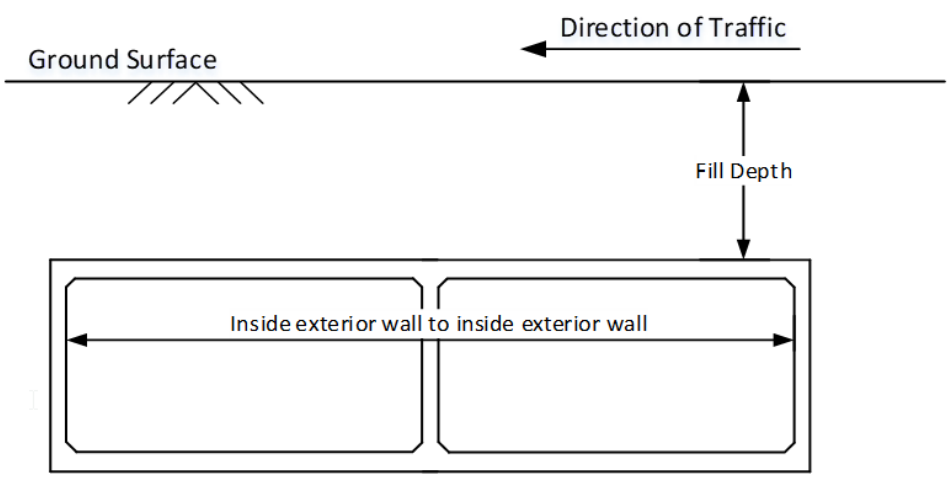
If a one-way slab analysis, described above, produces insufficient rating values and the slab's geometric length and width are such that a two-way slab is deemed appropriate, a two-way slab analysis may be used to re-evaluate the rating factor of the invert slab. This approach typically requires a refined analysis such as FEM to determine the distribution of the axle/wheel loads across the two-way slab system. An alternate source needs to be utilized for analyzing two-way slabs (such as American Concrete Institute (ACI) 318) as the 2017 BDS does not provide design recommendations for two-way slabs.

Concrete Slab on Invert Girders – This type of structural system typically has a concrete deck that spans parallel to traffic and is supported by invert girders that span perpendicular to traffic. The live loads of each axle/wheel can be distributed over a defined length transversely (perpendicular) to the direction of traffic. The length of transverse distribution of the axle/wheel loads may be determined using the approximate equations of LRFD BDS Article 4.6.2.1, or a more refined analysis such as a finite element modeling approach may be used, if desired or appropriate. The transverse distribution of the axle/wheel loads produces a uniform load which can be multiplied by the width of a longitudinal slab strip to be analyzed (typically 1 foot). The slab strip can then be analyzed as a beam element and the extreme force effects determined to perform a rating evaluation of the deck slab. The live load distribution to the invert girders may be determined in accordance with LRFD BDS Article 4.6.2.2.2f. The individual distributed wheel loads for each type of vehicle are evaluated as point loads that can move along the transverse analysis strip within the limits prescribed in LRFD BDS Article 3.6.1.3 and MBE Article 6A.2.3.2 to produce the extreme force effects for the critical sections being evaluated. Example 2 in Chapter 9 illustrates the distribution of live load for this type of structure.

Similar to invert concrete slabs, a two-way slab analysis may be utilized if rating values are insufficient. Refer to Invert Concrete Slabs above for further details.

4.3.2.2 External Systems

For external systems, loading is typically transferred from the roadway above through the soil or rock to the load resisting element, typically a liner or on the top slab for cut-and-cover type tunnels. In accordance with MBE Article 6A.5.12.10.3a, for tunnels with less than 2.0 feet of fill, wheel loads shall be applied as specified in LRFD BDS Article 4.6.2.10. For tunnels with more than 2.0 feet of fill, wheel loads shall be applied as specified in LRFD BDS Article 3.6.1.2.6. MBE Article 6A.5.12.10.3a also states that live loads may be neglected when the fill depth is greater than 8 feet or the clear distance from inside of exterior wall to inside of exterior wall, whichever is greater, as shown in Figure 25. This applies specifically to the load rating of buried reinforced concrete box culverts. The intent is to acknowledge that live load effects become negligible as the amount of fill height between the structure and roadway increases. A similar approach can be considered when applying loads through fill on deep tunnels. This approach should only be considered by engineers familiar with normal response of structures to surcharges through fill.

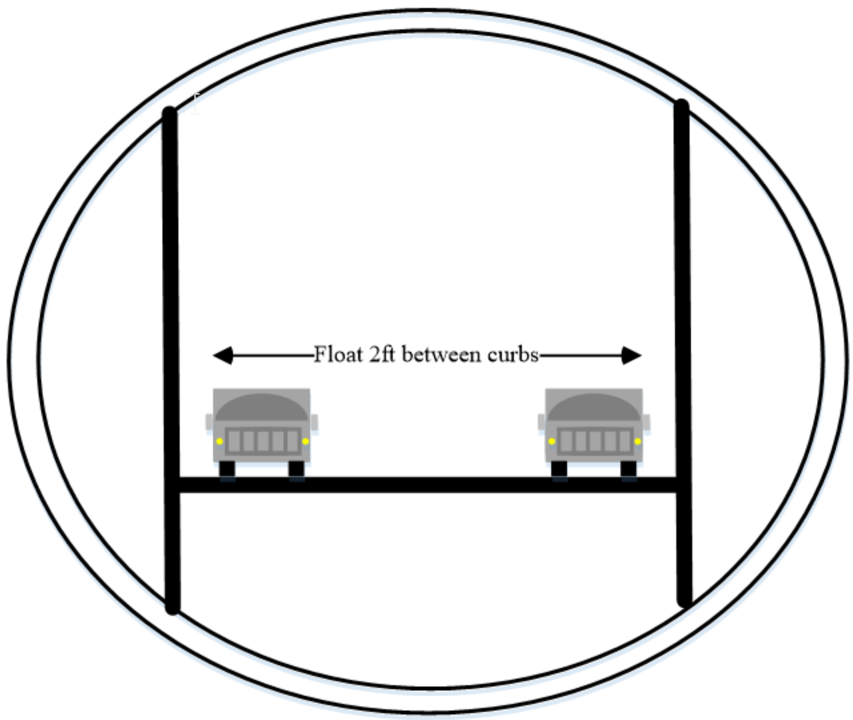


Source: FHWA

Figure 25. Illustration. Fill depth to span length configuration for live load evaluation on the zone of influence.

4.3.2.3 Live Load on Components

The live load is distributed to the system through internal and external systems as described in Sections 4.3.2.1 and 4.3.2.2. The live load path through each component is crucial to determine how to analyze the structure and rate the appropriate components. It is also important to identify where the live load will act on the driving surface. HL-93s are typically considered a floating load and can be placed anywhere within the roadway surface as shown in Figure 26. Contrarily, legal and permit loads can be defined to ride within a striped lane on the roadway surface in accordance with MBE Article C6A.2.3.2.



Source: FHWA

Figure 26. Illustration. Floating truck on roadway.

For an internal system, the live load is carried through the invert structure to the supporting liner below, through the connection points of the invert structure and the tunnel liner, as detailed in Figure 15. It is also important to consider the distribution through the invert structure. In most cases, the live load will be perpendicular to the longitudinal axis of the invert structural strip or girder. In cases where there is a skew between the longitudinal axis of the tunnel and the transverse structural strip, an appropriate skew adjustment factor needs to be accounted for in the distribution factors in accordance with LRFD BDS Article 4.6.2.2.2, Article 4.6.2.2.3 and Article 4.6.2.3.

External systems need to be identified as surface traffic travelling either parallel or perpendicular to the tunnel longitudinal axis. This will identify the soil distribution orientation to transfer the live load from the surface to the buried structure. If there is a skew between the direction of traffic and the longitudinal axis of the tunnel, the live load distribution is either taken as the worst case scenario or interpolated between the live load effects of the transverse and longitudinal live load analysis with respect to the skew angle. It is also important to consider how the load transfers from the top of the liner to the bottom of the liner support surface. In some instances, internal members such as ceilings or invert structures may distribute live load through the tunnel. Additionally, live load surcharge distribution needs to be considered in the structural analysis.

It is also important to note that some systems may contain both internal and external live loads. In these cases, the effects of each internal and external live load needs to be considered as well as the combined internal and external forces simultaneously, which is commonly achieved by enveloping the force effects of the internal system, external system and the internal plus external systems.

4.3.3 Dynamic Load Allowance

The dynamic load allowance shall be applied as specified in MBE Article 6A.4.3.3, Article 6A.4.4.3, and Article 6A.4.5.5. For buried tunnels, cut-and-cover structures, and other buried components, a reduction in the Dynamic Load Allowance may be taken in accordance with the LRFD BDS Article 3.6.2.2 and MBE Article 6A.5.12.10.3b.

4.3.4 Live Load Surcharge

Live load surcharges shall be applied as specified in MBE Article 6A.5.12.10.3c and LRFD BDS Article 3.11.6.4.

4.3.5 Design Live Load

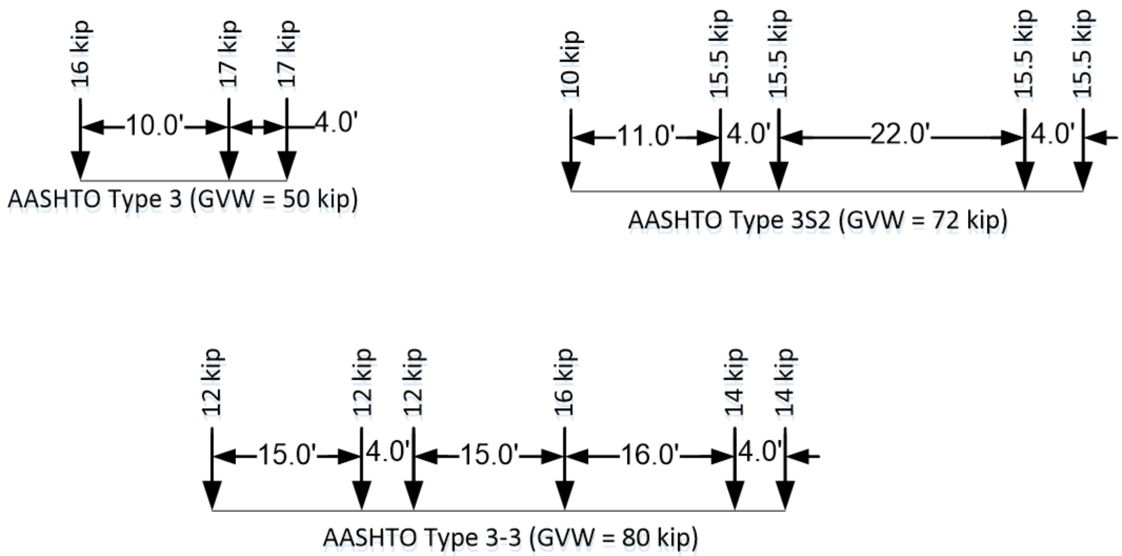
The design live load for LRFR ratings is based on the HL-93 loading from the LRFD BDS Article 3.6.1.2. Diagrams for the HL-93 truck, tandem and lane loadings can also be found in the MBE Appendix C6A.

4.3.6 Legal and Permit Loads

Legal loads used in load rating typically include the AASHTO Routine Commercial Traffic legal trucks, Specialized Hauling Vehicles (SHV) and State-specific legal trucks. Depending on location of the tunnel, Emergency Vehicles may also be required to be evaluated at the legal rating level in accordance with the FHWA FAST Act memo from 2016 (FHWA, 2016).

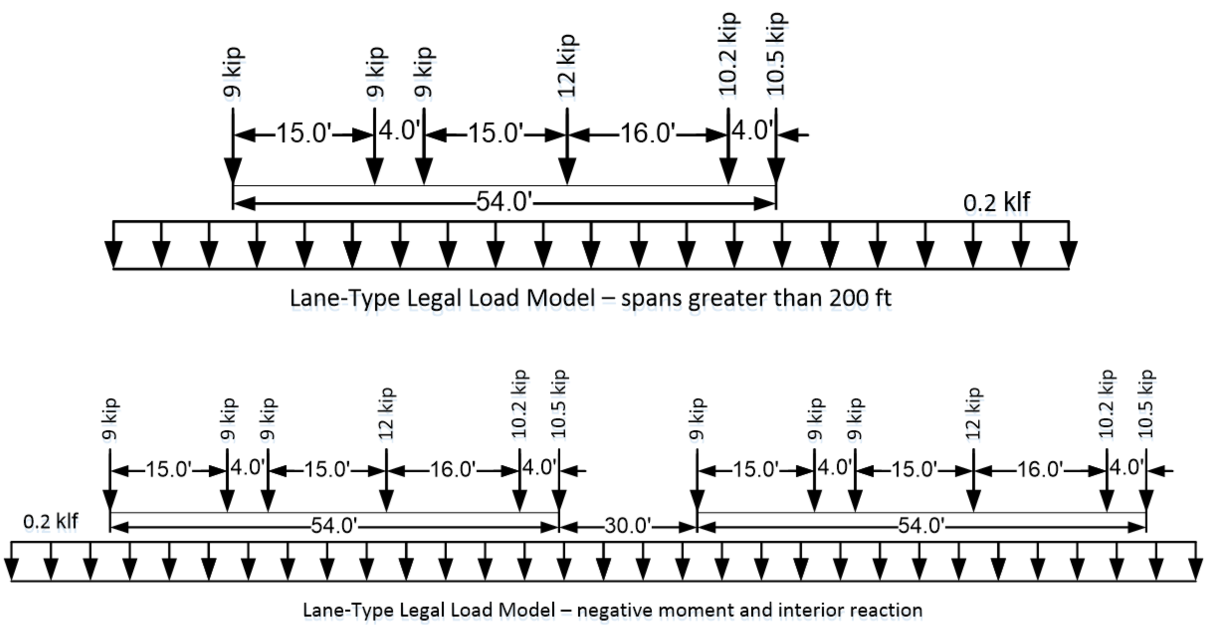
4.3.6.1 AASHTO Legal Loads

AASHTO Legal Loads represent legal vehicles that are within the federal truck weight limitations (including Federal Bridge Formula B) and are allowed to operate on the interstate and/or some highway system without a permit. The AASHTO Legal Loads are defined in the MBE and consist of three Legal Trucks (Figure 27 and Figure 28) for routine commercial traffic, and the Notional Rating Load (NRL) (Figure 29),and the Single Unit Trucks (SU4, SU5, SU6 and SU7) (Figure 30) for Specialized Hauling Vehicles (SHV). The NRL is itself a notional load which acts as a screening load for the Single Unit SHVs (SU4 – SU7).



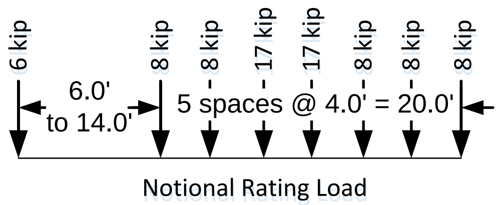
Source: FHWA

Figure 27. Illustration. AASHTO routine commercial traffic legal loads (Type 3, Type 3S2, and Type 3-3).



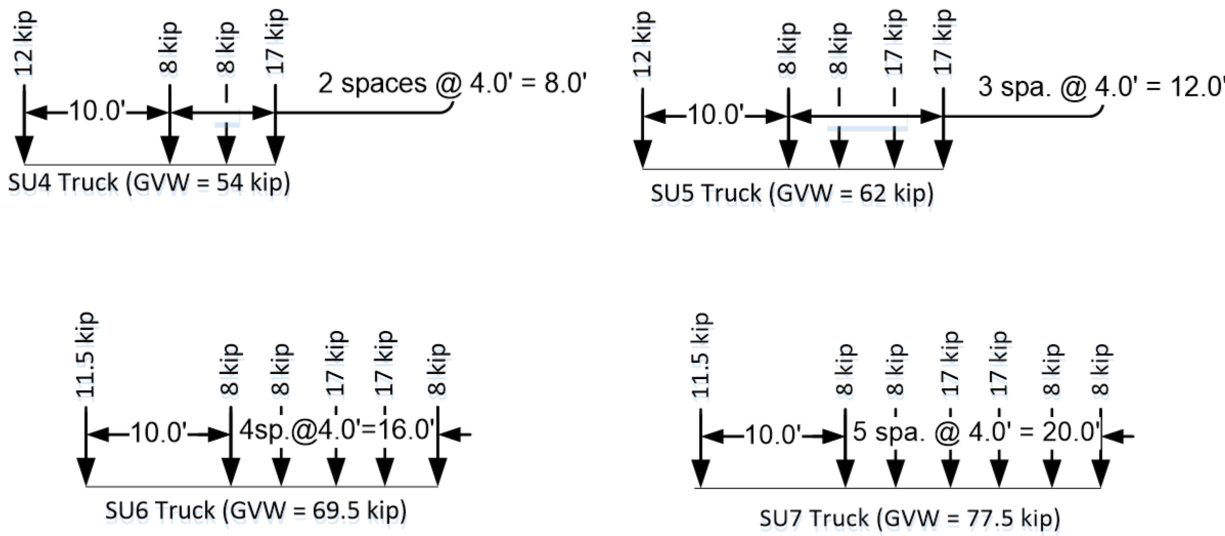
Source: FHWA

Figure 28. Illustration. AASHTO lane-type legal loads for routine commercial traffic.



Source: FHWA

Figure 29. Illustration. AASHTO Notional Rating Load (NRL) for Specialized Hauling Vehicles.

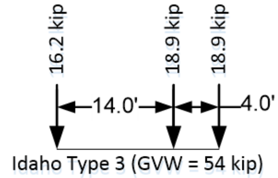


Source: FHWA

Figure 30. Illustration. AASHTO Single Unit (SU) truck rating loads for Specialized Hauling Vehicles.

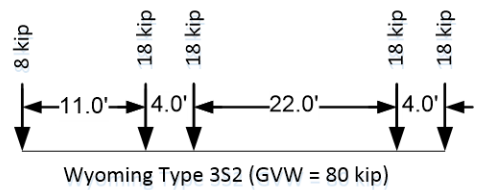
4.3.6.2 State Legal Loads

States may specify unique legal live loads to represent specific traffic common to that area. These loads are typically a variation to the standard AASHTO legal loads. State specific legal loads typically will supplement the AASHTO legal loads and are therefore typically used as the posting standard. The screening criteria in MBE Article C6A.4.1 and Figure A6A-1 may not be valid for State Legal Loads. A few examples of State specific legal loads are shown below (Figure 31 and Figure 32).



Source: FHWA

Figure 31. Illustration. Idaho Type 3 Single Unit legal load.



Source: FHWA

Figure 32. Illustration. Wyoming Type 3S2 legal load.

4.3.6.3 Emergency Vehicles

Emergency Vehicles were introduced as part of the FAST Act in 2015. FHWA followed this Act with a memorandum on November 3, 2016. This memorandum provided guidance on maintaining compliance with the load rating and posting requirements of structures by defining the Type EV2 and Type EV3 rating load models (Figure 33). By this memorandum, these vehicles need to be considered for load rating on the Interstate highway system and for all structures within reasonable access to and from the Interstate highway system, and for structures on other highways as directed by the Owner.

The Type EV2 vehicle screens for single rear axle emergency vehicles and consists of a 24 kip front single axle followed by a 33.5 kip rear single axle that trails at a distance of 15 feet. The Type EV3 vehicle screens for tandem rear axle emergency vehicles and consists of a 24 kip front single axle followed by a 62 kip rear tandem axle (wheels spaced at 4 feet) that trails at a distance of 17 feet (distance from front axle to the centerline of the rear tandem axle).

Source: FHWA

Figure 33. Illustration. Emergency Vehicle legal loads.

4.3.6.4 Permit Loads

Permit loads are actual trucks exceeding one or more of the legal load limits in federal law or state statute. NTIS (23 CFR 650.505) defines a routine permit load as "*a vehicular load that has a gross weight, axle weight, or distance between axles not conforming with State laws or legally configured vehicles, and is authorized for unlimited trips over an extended period of time to move alongside other heavy vehicles on a regular basis.*" Permit load ratings check the safety and serviceability of tunnels in the review of permit applications for the passage of vehicles above the legally established weight limitations. Permit loads are done at a third level rating that should normally be applied to structures having sufficient capacity for legal loads. Permits are issued on a single trip, multiple trip, or annual basis. Axle configurations are specific to the permitted vehicle and may be required to be escorted to control their speed, lane position, the presence of other vehicles in adjacent lanes or on the structure at the same time, or a combination thereof (MBE Article 6A.4.5.1). An example of a single trip permit is shown below (Figure 34).

Figure 34. Photo. Example of a single trip permit load.

4.4 OTHER LOADS: WL, WS, TG, TU, WA, AP, EQ, CR, SH AND SE

In general, only permanent loads and vehicular loads are considered in a load rating analysis. Environmental loads, such as wind, ice, temperature, stream flow, and earthquake are usually not considered in rating except when unusual conditions or specialized structures may warrant their inclusion (MBE Article 6A.2.3.5 through Article 6A.2.3.8).

4.4.1 Wind Loads: WL and WS

Tunnels are not exposed to wind loads. Therefore, wind loads (WL and WS) need not be considered unless special circumstances justify otherwise (MBE Article 6A.2.3.5).

4.4.2 Temperature Loads: TG and TU

For tunnels, there are certain components that may be subject to uniform temperature (TU) change. For example, suspended ceilings or floor slabs that enclose ventilation ducts may experience temperature changes. Temperature gradients (TG) may also occur with certain elements. For example, temperature gradients commonly occur between the inside face and outside face of the tunnel liner. However, temperature effects generally need not be considered in calculating load ratings for concrete components that contain well-distributed reinforcement to control thermal cracking. When temperature effects are considered, MBE Article 6A.2.3.6 indicates that a reduced long-term modulus of elasticity for concrete may be used in structural analysis.

4.4.3 Water Loads: WA (Hydrostatic Loads)

Loads due to water will occur with immersed tunnels and when the groundwater level is at or above the level of the tunnel. Groundwater elevations should be determined as part of the original subsurface investigation program. If this information is not known, historical data or field investigations may be undertaken to determine the groundwater elevation.

Water (WA), or hydrostatic loads applied to the tunnel element being analyzed should be the actual hydrostatic pressure applied to the tunnel as determined by the subsurface investigation. Special considerations for aquifers and active drainage systems are provided in LRFDTUN Article 3.7. Hydrostatic loads also need to be considered for immersed tunnels. Additional information regarding transient loads is provided in LRFDTUN Article 8.7.1.2.1. In some instances, WA may reduce total loading on the tunnel element. When this occurs, the Engineer shall take care to ensure that this condition is typical and not likely to change, as this may increase the load rating in certain conditions.

4.4.4 Air Pressure: AP

Air pressure (AP) loads are a unique loading applied to tunnel structures. AP is developed from two sources, the tunnel ventilation system and the vehicles passing through the tunnel (Piston Effect). LRFDTUN Article 3.8 provides guidance on when AP should be considered and which elements are affected by this load. Typically, AP is applied to ceiling structures and attachments when generated by passing vehicles and need not be considered for load rating elements unless special circumstances justify otherwise. Strength evaluation of those structural components and connections affected by this type of load requires the consideration of AP loads.

4.4.5 Earthquake: EQ

Earthquake effects need not be considered in calculating load ratings (MBE Articles 6A.2.3.7).

4.4.6 Superimposed Deformations: CR and SH

Creep and shrinkage effects do not need to be considered in calculating load ratings where there is well-distributed reinforcement to control cracking in non-segmental, non-prestressed concrete components. For creep effects in concrete, LRFDTUN Article 3.10.4 directs the Engineer to the LRFD BDS Section 5. LRFD BDS Article 5.4.2.3 provides direction with regard to creep and shrinkage in concrete components.

4.4.7 Settlement: SE

Ground settlement can be caused by long-term movements of the surrounding soil and should be considered for tunnel structures that exhibit signs of distress due to settlement. If settlement has occurred, especially differential settlement, the impacts and changes to the tunnel structure and analyzed elements shall be included in the analysis.

4.4.8 Railroad and Airport Loading

A structural analysis to validate the adequacy of the structure shall be performed when a tunnel is subjected to overhead railroad and/or airport loading. Railroad and airport loading are special cases where freight/transit rail or aircraft loads apply live loads on the tunnel structure through the surrounding soil structure as described in Section 4.3.2.2. Loading is typically transferred from the vehicle above through the soil or rock to the load resisting element, typically a liner or on the top slab for cut-and-cover type tunnels. When this situation occurs, the railroad and airport load should be considered in a similar manner as a live load. When analyzing for this type of loading, consideration should be given to lowering the live load factor for the Strength-I limit state as determined by the Owner.

CHAPTER 5 - STRUCTURAL ANALYSIS

5.1 INTRODUCTION

This section focuses on the structural analysis concepts associated with load rating of tunnel structural components. Analysis of the structure to obtain the forces acting on each structural member are analogous to the design analysis procedures used to obtain design forces. It is assumed that the rating Engineer is experienced and knowledgeable in the design of such structures, and this section references applicable articles of the MBE and LRFD BDS to point the rating Engineer to appropriate sections to consider when load rating. Every structure is different and engineering judgment is required to discern necessary methodologies and components for analysis, along with the needed limit states for rating.

5.2 ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS

As noted in LRFD BDS Article 4.4, acceptable methods of analysis include any method of analysis that satisfies the requirements of equilibrium and compatibility and utilizes stress-strain relationships for the proposed materials. Refined analysis, as described in LRFD BDS Article 4.6.3, may be used if the approximate methods listed below are not within the code requirements or if desired by the Owner and/or the rating Engineer if the structure being rated does not exhibit sufficient load capacity (MBE Article 6A.3.3).

5.3 APPROXIMATE METHODS OF STRUCTURAL ANALYSIS

Except as specified herein, approximate methods of distribution analysis as described in LRFD BDS Article 4.6.2 may be used for evaluating existing structures.

5.3.1 Deck Slab Approximate Methods

Stringer supported concrete slabs that carry normal traffic satisfactorily do not need to be routinely evaluated for load capacity in accordance with MBE Article 6.1.5.1. However, floor beam spacing not satisfying MBE Article 6.1.5.1 or heavily spalled and deteriorated concrete decks and/or decks subjected to State legal or permit loads with axle loads notably larger than design values may need to be checked for punching shear under wheel loads. Periodic inspection of the decks shall be performed to verify satisfactory performance. For situations where decks are evaluated, or for frames, such as invert slabs or cut-and-cover tunnels where the slab is integral with the supporting walls, the following references provide approximate methods for live load distribution on deck slabs for scenarios with and without fill.

5.3.1.1 Deck Slab Approximate Methods (No Fill)

For deck slabs that have no fill, LRFD BDS Article 4.6.2 contains approximate methods to assess live load distribution on slabs in which the deck is subdivided into strips perpendicular to the supporting components. For slabs that span more than 15.0 feet and which span primarily in the direction parallel to traffic, LRFD BDS Article 4.6.2.3 would apply. Otherwise, LRFD BDS Article 4.6.2.1 applies within listed constraints of the article.

5.3.1.2 Deck Slab Approximate Methods (With Fill < 2.0 feet)

For concrete slabs that have less than 2.0 feet of fill over them, such as cut-and-cover structures, LRFD BDS Article 4.6.2.10.2 applies for traffic traveling parallel to the span. LRFD BDS Article 4.6.2.10.3 applies for traffic traveling perpendicular to the span and refers to the use of LRFD BDS Article 4.6.2.1.

5.3.1.3 Deck Slab Approximate Methods (With Fill ≥ 2.0 feet)

For concrete slabs that have fill of 2.0 feet or greater over them, such as cut-and-cover structures, LRFD BDS Article 3.6.1.2.6 applies, which describes the live load distribution of wheel loads through earth fills.

5.3.2 Beam-Slab Structures

LRFD BDS Article 4.6.2.2 contains several cross-section types with approximate live load distribution equations/methods that should be used if the structure falls within the applicable range or limits of the equations/methods. LRFD BDS Article 4.6.2.2.2f contains guidance on live load distribution for transverse invert girders, similar to those depicted in Example 2 of this document.

5.3.2.1 Composite Sections

The calculation of elastic stresses at a section shall consider the sequence of loading in accordance with MBE Article 6A.6.9.2 and LRFD BDS Article 6.10.1.1.1. Assume unshored construction unless indicated otherwise in the structure documents. Refer to MBE Article 6A.6.9.2.

5.3.2.2 Non-Composite Sections

In certain situations, the compression flange of non-composite positive flexure sections may be considered to be adequately braced by the concrete deck. Refer to MBE Article 6A.6.9.3.

5.3.2.3 Encased I-Sections

In certain situations, a concrete encased steel I-section may be assumed to act as a composite section at the service and fatigue limit states. It may also be considered composite at the strength limit state if sufficient shear transfer is verified by calculations. Refer to MBE Article 6A.6.9.4.

5.4 FRAME ANALYSIS

Analysis of frames, such as for cut-and-cover or invert slab structures, can be accomplished by use of standard structural analysis methods as noted in LRFD BDS Article 4.4.

5.4.1 Frame Modeling

Typically a 1 foot wide strip of a frame, or other unit width, may be used as a typical analysis section for evaluation. Proper modeling of the support conditions, lateral support (or lack thereof) for side sway, joint interaction, and other modeling constraints are necessary to obtain accurate analysis results.

5.4.2 Live Load Distribution

Proper load distribution based on the unit width of the frame is essential. For live loads, use of the approximate deck strip methods listed above in Section 5.3.1 may be utilized provided the structure falls within the approximate method's limitations. Otherwise, a refined methodology may be required.

5.4.3 Combined Flexural and Axial Effects

Walls in cut-and-cover and invert slab structures shall be evaluated for flexural and axial force effects (MBE Article 6A.5.7) that include slenderness effects (LRFD BDS Article 5.6.4.3) and biaxial flexure requirements (LRFD BDS Article 5.6.4.5). Roof systems may also require combined flexural and axial effects if the lateral load produces high enough axial stress in accordance with LRFD BDS Article 5.6.4.5. Use of interaction diagrams as capacity evaluation aids is also recommended (see MBE Appendix G6A).

5.5 NUMERICAL MODELING

Simplified analysis methods, such as those discussed in the preceding sections, may not be sufficient for evaluating certain unique or otherwise challenging tunnel and ground configurations. Numerical methods, such as Finite Element (FE), Finite Difference (FD), or Discrete Element (DE) analysis, may be used in the absence of suitable simplified methods, or to perform a more refined structural evaluation.

Numerical modeling is particularly useful when the tunnel is embedded in soil and the load effects on the tunnel liner itself are relevant to the load rating. Structural load effects (that is, axial, moment, and shear forces) on the tunnel lining can be heavily influenced by soil-structure interaction. Determination of axial, moment, and shear load effects throughout the lining of circular or other irregularly shaped structures is not always straightforward. Moment and shear forces in the lining are related to the lining deformation, which is influenced by the movements of the surrounding soil. As a result, a force-based analysis procedure utilizing simplified earth pressure and surcharge diagrams may not be sufficient for identifying the maximum load effects for load rating purposes. To overcome these challenges, the tunnel lining and structural elements can be modeled inside a continuum representing the ground using commercial software packages such as, but not limited to, the FE program Plaxis (2018), the FD program FLAC (Itasca, 2016), or the DE program UDEC (Itasca, 2014).

Some examples where numerical modeling may be required or helpful when performing a load rating are as follows:

- Unique tunnel geometry or loading that does not match the assumptions for a simplified method.
- For cut-and-cover tunnels with large dimensions and/or challenging soil conditions where soil loading and SSI effects become dominant.
- If deterioration is observed or anticipated in the tunnel liner and SSI is expected to affect the liner performance as it relates to the tunnel load rating.
- When structural load effects on the roadway support system are affected by the response of the tunnel liner (for example, invert slab structurally connected to the tunnel liner).
- When other nearby structures contribute to dead or live load effects on the tunnel lining or roadway support system.
- When varied surface grade conditions exist which affect the demand or capacity of the structure and do not match the assumptions for a simplified method (for example, irregular grade conditions above a shallow tunnel).
- When changes occur in the ground stress conditions around the tunnel during the life of the structure that may affect the load effects or capacity (for example, changes in live load conditions at the surface above a shallow tunnel, changes to existing grade level above a shallow tunnel).
- When cuts or excavations are made near the tunnel that may affect the capacity or load effects on the liner and roadway support system.
- When construction sequencing may influence structure load effects in a way that is not accounted for by simplified methods.

General considerations and guidelines for numerical modeling of tunnels are provided as follows.

5.5.1 Model Type

A numerical model of a tunnel typically includes structural elements (for example, beams, plates, or shells) to represent the tunnel lining and interior structural systems. In addition, soil-structure interaction is simulated using either soil springs, solid (continuum) elements, or discrete elements. Example illustrations of a spring model and a continuum model are illustrated in Figure 35 and Figure 36, respectively.

5.5.1.1 Spring Models

It is sometimes desirable to develop a structural model where the soil is represented by springs, as illustrated in Figure 35. The structural lining can be represented by elastic beam elements and the soil springs can be linear or nonlinear. The soil springs for such a model are typically developed by the geotechnical engineer, and provided to the structural engineer. The structural engineer can then use the model to quickly consider various load cases relevant to the load rating. Since spring stiffness is dependent on the stress level in the soil, iteration between the geotechnical and structural engineer to refine the springs may be required.

For some problems, such as where ground displacements are expected to be relatively small and ground stresses are not expected to approach a yielding condition, linear elastic soil springs may be sufficient. For instance, the ground response around a deep tunnel that is embedded in (and in intimate contact with) competent soil or rock could be reasonable approximated by linear elastic springs. The linear soil spring stiffness can be estimated similar to the theoretical assumption of a flexible beam foundation on an elastic medium. For this formulation, Vesic (1961) proposed a "Winkler" soil spring of stiffness k based on the following relation:

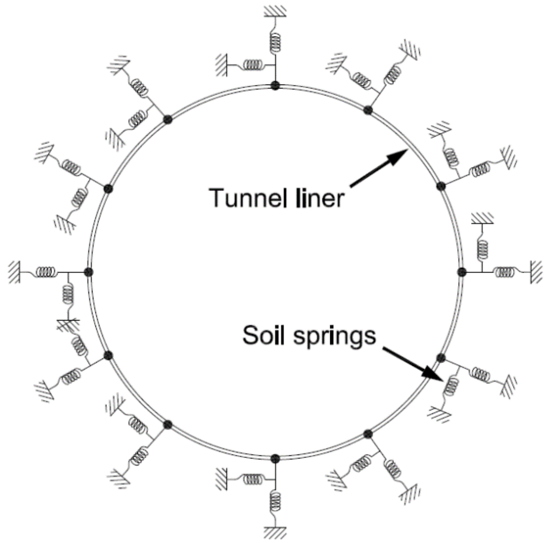
$$k = \frac{0.65E_s}{1-\nu^2}\sqrt[12]{\frac{E_s B^4}{EI}} \qquad \text{(Eq. 5.5.1.1-1)}$$

where E_s and ν are the Young's modulus (secant modulus relevant to the stress/strain range of interest) and Poisson's ratio of the foundation soil, respectively. *B* is the beam width, and *EI* is the flexural stiffness of the beam. Typically, *B* can be taken as a unit length of lining in the out-of-plane direction (for example, 1 foot), similar to a plane strain model as discussed in the next section. Similarly, *EI* can be taken as the flexural stiffness of the same unit length section of the lining.

For tunnels embedded in softer ground and/or shallow overburden conditions, soil (or weak rock) nonlinearity can significantly influence the response of the tunnel structure. In such cases, nonlinear soil springs will lead to more realistic results because they are able to address potential gapping between the soil and liner and account for soil yielding.

Nonlinear soil springs can be developed based on a separate numerical model analysis performed by the geotechnical engineer. A typical tunnel cross section is modeled inside a soil domain represented by solid (continuum) elements. Soil behavior is controlled in the continuum by nonlinear constitutive models. Solid element soil-structure modeling is discussed in greater detail in Section 5.5.1.2.

The solid element soil-structure model is used to develop nonlinear soil springs are developed by prescribing translational displacements to each liner node (one at a time) and computing the corresponding reaction force between the liner and surrounding soil. The relationship between the prescribed displacements and the reaction forces can be taken as the soil spring at each respective node. Coupling effects can typically be ignored for simplicity. This procedure can be carried out to develop soil springs for both the radial and tangential directions (see Figure 35).



Source: FHWA

Figure 35. Illustration. Example of a numerical model using soil spring elements to represent the surrounding ground.

5.5.1.2 Solid Element (Continuum) Models

Instead of using springs to model the soil-structure interaction, the soil or rock surrounding a tunnel can be modeled using solid continuum elements, as illustrated in Figure 36. This method is typically appropriate for soil or rock conditions which are fairly continuous and do not contain predominant weak planes that would affect the tunnel. In such models, structural elements are used to represent the tunnel lining and other structural components. Interface elements can be applied at the soil-structure interface to allow additional control of the interaction.

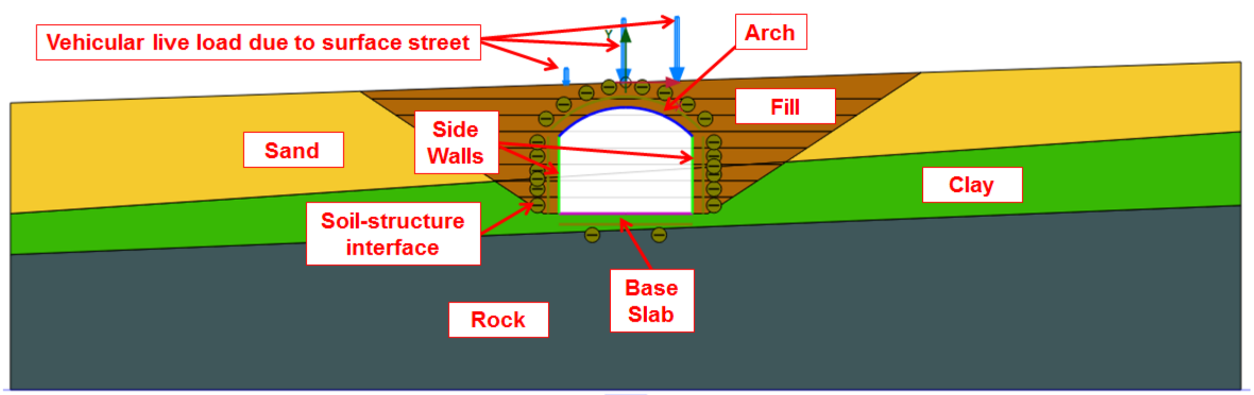
For illustration purposes, Figure 36 presents a model of a tunnel structure that is similar to the one used for Example Problem 4 (see Section 9.5). The model represents a cut-and-cover reinforced concrete box with a circular arch top, with the tunnel roadway supported by a slab on grade. Four soil layers are included in the illustration, including native sand, clay and rock layers, and a zone of fill material replacing a sloped tunnel excavation. A vehicular live load is included on the ground surface, representing traffic on an at-grade roadway running transverse to the tunnel longitudinal axis. Live loading can also be applied inside the tunnel (not shown), if deemed to contribute to the maximum load effects for load rating purposes.

In this example, the arch, side walls, and base slab are modeled using plate elements. Plates elements are structural objects that are used to model slender structures in the ground with a significant flexural rigidity, *EI,* and normal stiffness, *EA*, such as a tunnel lining (Plaxis, 2018). For many static tunnel problems, linear elastic structure properties can be assumed in the FE analysis, and verified to be compatible with the computed liner deformations based on the results of the analysis. In some instances where larger deformations are expected, nonlinear properties may need to be considered, such as by implementing the full moment-curvature relationship of a reinforced concrete tunnel liner.

The soil and rock layers are modeled as a continuum using solid elements, with soil behavior controlled based on the assigned constitutive (stress-strain) models. A range of sophistication in constitutive models is typically available in commercial software packages, starting with a linear elastic model at the low end, progressing to an elastic-perfectly plastic model and then towards a variety of fully non-linear inelastic models, some of which account for pressure-dependent soil stiffness and strength. When relying upon numerical models to perform a load rating analysis, it is important for the rater to have a firm understanding of the stress-strain behavior assigned in the model, and the associated limitations to the model results. Results should only be relied upon for load rating analysis when model convergence can be confirmed (particularly for nonlinear models) and simulations are numerically stable.

As illustrated in Figure 36, a soil-structure interaction interface can be modeled along the outside of the tunnel lining. The interface is typically assigned a reduced strength (for example, 0.5 to 0.75), as compared to the adjacent soil. It represents the interface friction or adhesion between the soil and the tunnel liner (or other buried structural element). If an interface element is not included, the stress transfer between the soil and the tunnel can become exaggerated. In addition, if the soil is modeled as linear elastic, infinite strength is unrealistically implied for the soil and the soil-structure bond can be even more exaggerated. As a result, including a soil-structure interaction interface and nonlinear soil model (with a defined non-infinite strength, yielding or failure criterion) typically leads to more realistic results.

Continuum models are versatile in that the ground conditions can be assigned to match any desired geometry, such as the sloping ground surface and soil strata illustrated in Figure 36. Earth pressures are automatically generated, based on the assigned soil properties and constitutive models. Structural elements (for example, beams, plates, and shells) can be used to represent a wide range of structural configurations, including the tunnel liner, and any additional roadway support or other interior or exterior structural systems. Traffic or other dead or live loads can also be applied as point or distributed loads or displacements at any location throughout the model.



Source: FHWA

Figure 36. Illustration. Example of a numerical model using solid elements to represent the surrounding ground.

5.5.1.3 Discrete Element Models

A discrete element (DE) method may be more appropriate when the tunnel is surrounded by rock that contains predominant weak planes that are oriented unfavorably to the excavation in accordance with FHWA *Technical Manual for Design and Construction of Road Tunnels – Civil Elements*, Table 6-12 (FHWA, 2009). In DE modeling, the ground surrounding the tunnel is represented by discrete blocks representing jointed rocks, with discontinuities located in between the blocks. Blocks can rotate and large displacements can occur along the discontinuities, representing a more realistic simulation of jointed rock behavior. While DE modeling is more commonly used during the design and construction phases, there may be some instances where it is required for evaluating existing tunnels for load rating purposes.

5.5.2 Model Dimensions and Boundary Conditions

For practical purposes, many tunnel problems can be sufficiently approximated by two-dimensional (2D) plane strain models, similar to the one shown in Figure 36. Plane strain models represent a slice of unit thickness (for example, one foot) along a cross section perpendicular to the tunnel. 2D plane strain modeling is appropriate when the problem geometry (tunnel, soil/rock layering, loading, etc.) is more or less uniform along the length of the tunnel section of interest. In such models, displacements (and strains) are assumed to be zero in the direction parallel to the longitudinal axis of the tunnel. When the conditions for plane strain modeling are not met, a three-dimensional (3D) model is required.

Model dimensions should be assigned so as not to influence the results of interest.

It is typically desirable to use the minimum possible dimensions that do not influence the load effects in order to minimize computation time and storage requirements. Minimum acceptable model dimensions depend on many factors, such as the diameter (or width) of the tunnel, depth below grade, properties of the surrounding soil or rock, topography and soil layering, and type of loading applied to the tunnel. As a result, there is no definitive guideline for choosing the minimum model dimensions, and significant prior experience and/or a trial-and-error type approach may be required to verify appropriate model dimensions.

For example in Figure 36, the overall model width of 300 feet is approximately 9 times the tunnel width of 33 feet (with 4 tunnel widths on each side of the tunnel). These dimensions were found to be sufficient by starting with a narrower model (for example 5 times the tunnel width), and incrementally increasing the width while comparing the tunnel load effects after each run. The load effects on the tunnel were found to stabilize by the time the model width reached 300 feet, confirming that a width of 300 feet is sufficient for load rating analysis. A similar exercise can be performed in order to determine an acceptable model depth for each load rating analysis.

Model boundary conditions should also be selected to avoid influencing the results. Typical boundary conditions used in geotechnical modeling consist of horizontal fixity along the vertical boundaries at the edges of the model, with free vertical movements. Horizontal and vertical fixities are applied to the base of the model. These boundaries conditions will be appropriate for most cases.

5.5.3 Mesh Type, Size and Sensitivity

Numerical model results can be sensitive to the discretization of elements, or mesh type and size. It is typically desirable to minimize the number of elements in a model without influencing the load effects in order to minimize computation time and storage requirements. Determining an appropriate element size can depend on several factors including the problem geometry, element type, and results of interest. It is often desirable to use finer (that is, smaller) elements in soil-structure interaction regions such as around the perimeter of the tunnel, and coarser (that is, larger) elements in the far field away from the tunnel.

Elements types with several nodes along each boundary can accommodate more complicated deformation patterns than elements with few nodes, allowing for coarser discretization. As a result, a mesh made up of 15-node triangular elements may not need to be as fine as a mesh of 4-node rectangular elements, for example. Aspect ratio is also an important consideration when developing a model mesh. The aspect ratio of an element describes its deviation from having all sides of equal length. An ideal aspect ratio is one, but that is not possible to achieve for all elements in models with complicated geometry. Long thin elements have a high aspect ratio, which is undesirable as it can cause convergence problems or produce inaccurate results.

Due to the complexity of identifying an appropriate mesh for load rating analysis, a trial-and-error approach may be useful for verification. For example, the problem illustrated in Figure 36 was simulated using a range of different 15-node triangular element sizes using Plaxis 2D (2018). Each mesh was generated automatically by Plaxis, including the following coarseness settings: Very Coarse (720 elements); Coarse (997 elements); Medium (1453 elements); and Very Fine (2268 elements). The Very Coarse mesh is illustrated in Figure 37, and the Very Fine mesh is illustrated in Figure 38. An example of the mesh size-dependence of tunnel load effects is illustrated in Figure 39. Moment demands on the left and right tunnel walls are compared as computed for each model mesh coarseness setting. The difference in results is most noticeable between the Very Coarse and Medium mesh settings, and becomes less noticeable (as it begins to converge) between the Medium, Fine, and Very Fine mesh. As a result, the Very Coarse mesh is not sufficient in this example, and a Fine or Very Fine mesh should be adopted for load rating analysis.

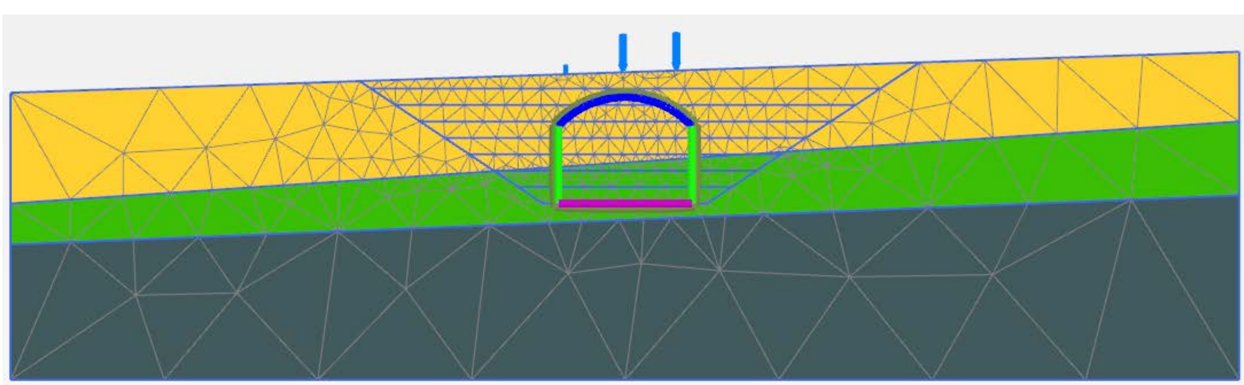


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Figure 37. Illustration. "Very Coarse" mesh (Plaxis, 2018).

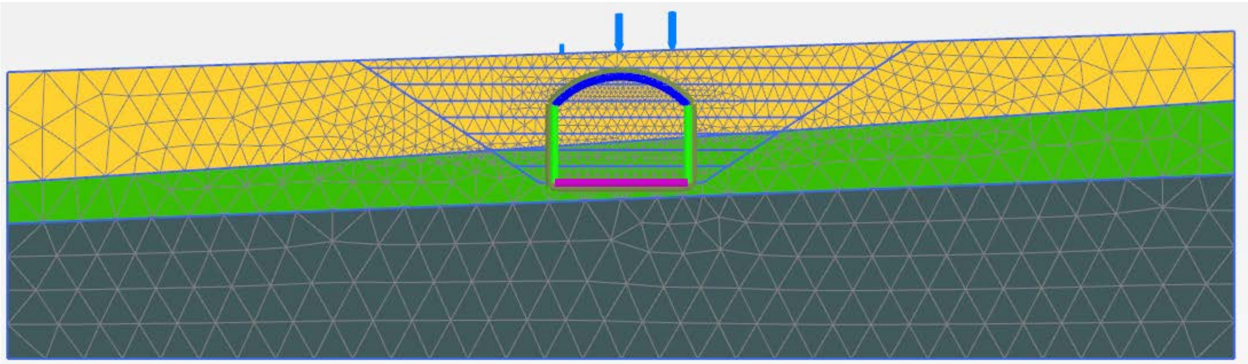

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Figure 38. Illustration. "Very Fine" mesh (Plaxis, 2018).

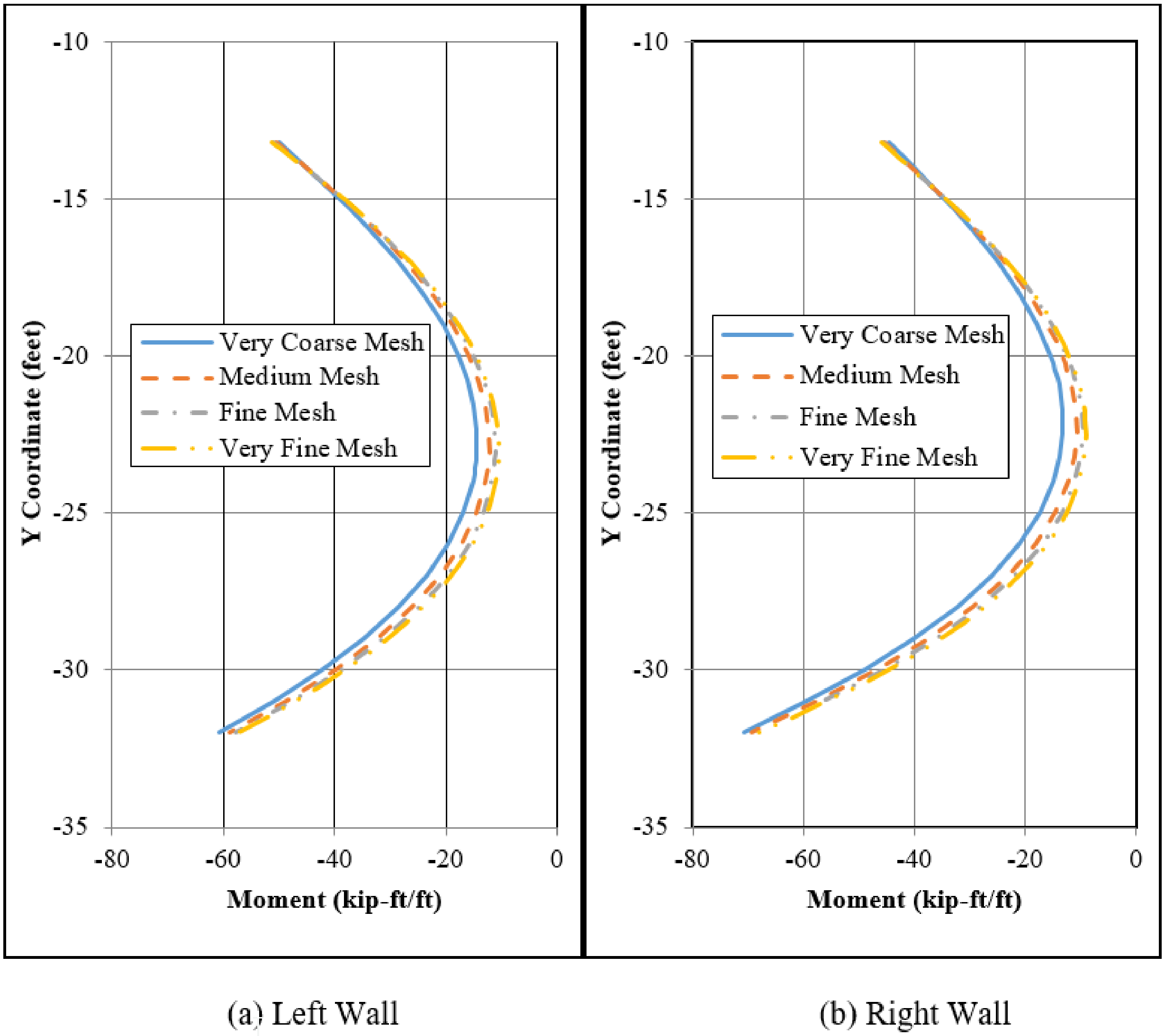

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Figure 39. Illustration. FE model results: comparison of moment on left and right tunnel walls considering a range of mesh coarseness.

5.5.4 Construction Sequencing

Load effects in underground structures can be significantly influenced by the sequence of installation or construction. When using numerical models to develop structure load effects for use in load rating, the analysis should be performed in stages to simulate a realistic sequence of construction. In each analysis stage, the model is allowed to reach equilibrium under the current conditions. Soil stresses and associated strength and stiffness values are updated based on the current stress conditions (when a nonlinear constitutive soil model is used).

Conditions may also change during the life of a structure, which could influence the load rating. Examples include changes to the grade level above the tunnel or addition of nearby structures which impose surcharge earth pressure loading on the tunnel.

Staged construction analysis is illustrated in Figure 40 through Figure 44 for the cut-and-cover example model of Section 9.5. In the example, a sloped excavation is performed and then the tunnel is constructed and backfilled. A live load is applied at the ground surface to simulated traffic at-grade crossing over the shallow tunnel. No live load is applied inside the tunnel, because a live load inside the tunnel decreases the load effects on structural components for this example. The example can be simulated in a numerical analysis as follows:

- Initial soil stresses are computed in the original ground condition without any excavations, structures, or surcharge loading (note that grey lines in the figure are not activated in the current stage).
- In Stage 1, a sloped excavation is performed to allow for tunnel construction.
- In Stage 2, the tunnel is activated, including all of the plate elements and soil-structure interfaces.
- In Stages 3 through 10, the backfill is added in lifts (one lift per analysis stage). At the end of Stage 10, backfilling is complete (not shown).
- In Stage 11, live loads are activated to represent truck wheel loads crossing above the tunnel at the ground surface.

Note that the grey lines in the illustrations are model elements that are not activated in the current stage.

Once the model comes to equilibrium after each analysis stage, the corresponding results can be evaluated.

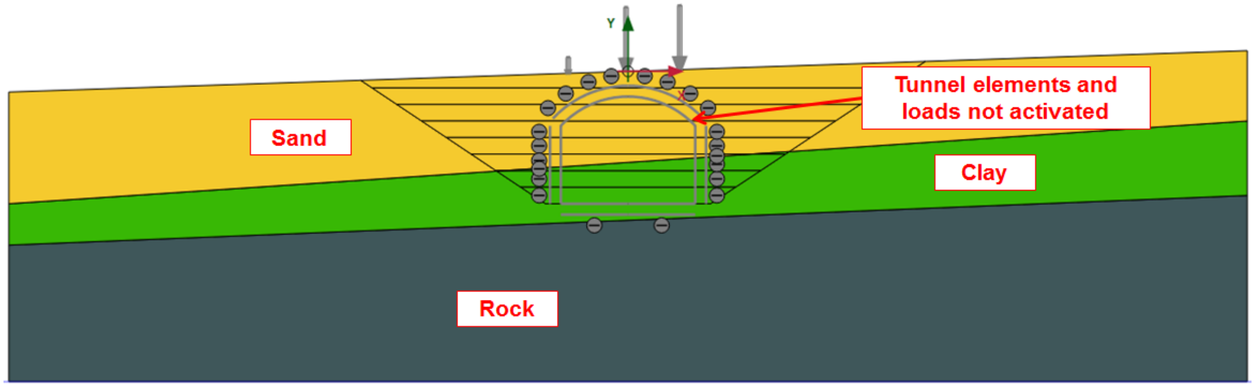

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Figure 40. Illustration. Example of staged construction analysis phases. Initial condition – Calculate initial stresses in soils without structures or loading.

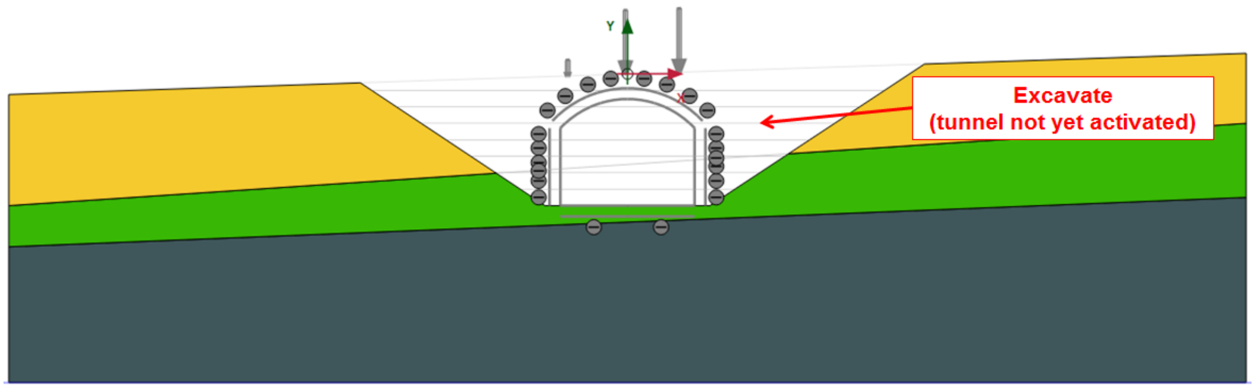

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Figure 41. Illustration. Example staged construction analysis phases. Stage 1 – Excavate.

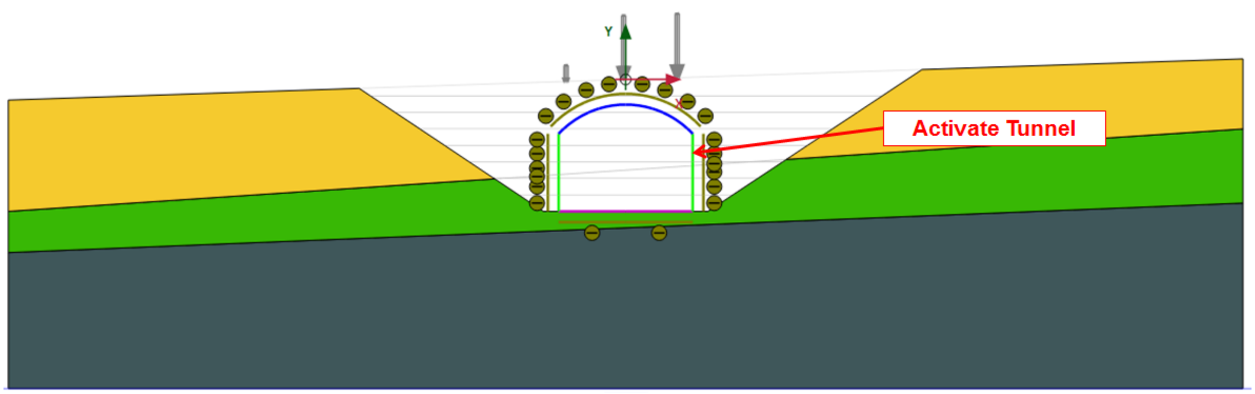

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Figure 42. Illustration. Example staged construction analysis phases. Stage 2 – Activate tunnel.

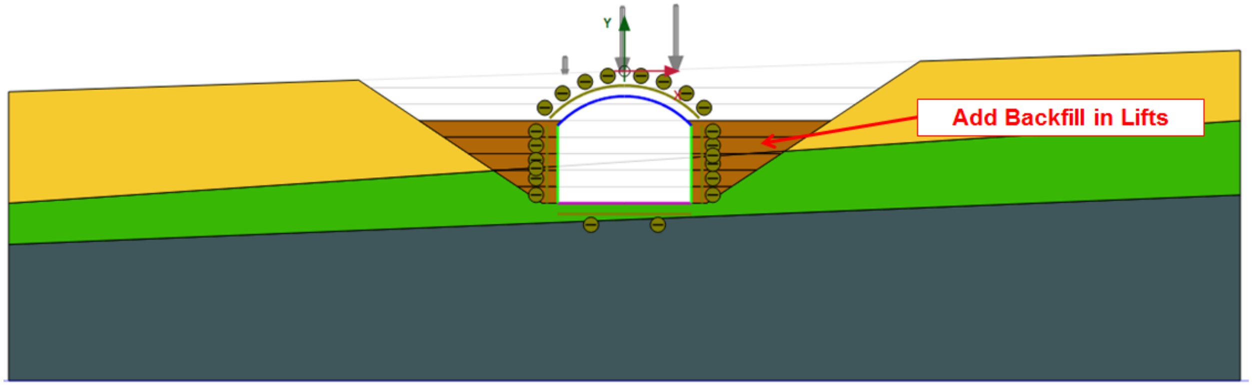

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Figure 43. Illustration. Example staged construction analysis phases. Stages 3 through 10 – Add fill in lifts (one lift per analysis stage).

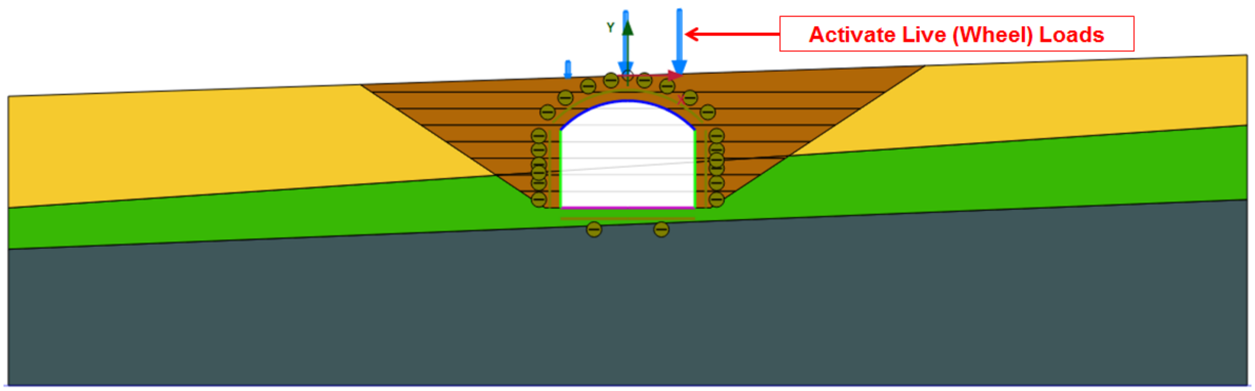

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Figure 44. Illustration. Example staged construction analysis phases. Stage 11 – Activate live loads representing truck wheel loads crossing above tunnel at surface.

5.5.5 Sample Results

Numerical model results can be used to illustrate the tunnel behavior and determine load effects on relevant structural elements for use in load rating. Deformed mesh plots can be exaggerated to illustrate ground and tunnel deformations. Moment, shear, and axial forces can be evaluated after each analysis phase to track the changes in load effects during various phases of construction, and due to changes that may take place during the life of the structure. For instance, Figure 45 shows the deformed mesh, with deformations exaggerated 100 times, after the completion of backfilling (after Stage 10). Figure 46 illustrates the moment diagram along the arch after application of the surface live load (after Stage 11).

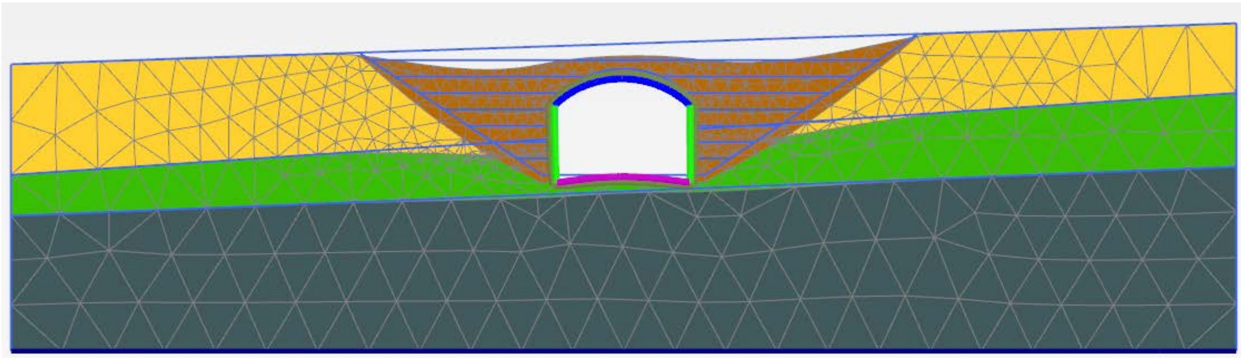

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Figure 45. Illustration. Example numerical modeling results: deformed mesh after backfilling (exaggerated 100 times).

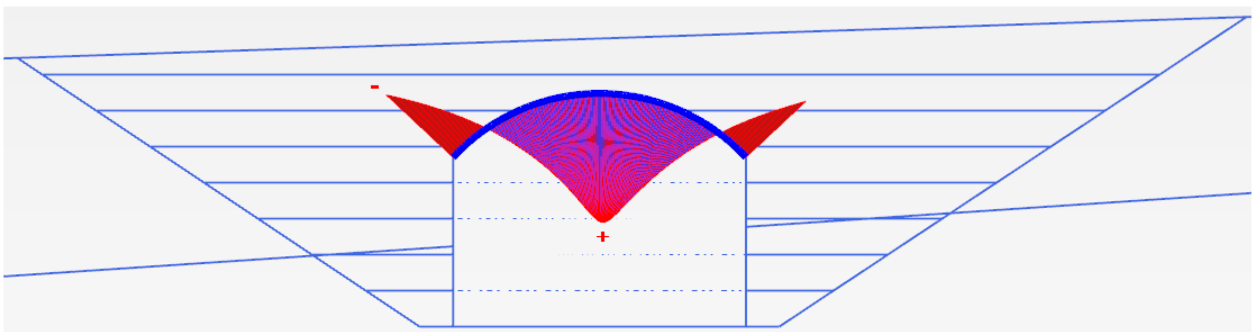

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Figure 46. Illustration. Example numerical modeling results: moment load effects on the arch after live load application.

5.5.6 Model Validation

Numerical models and their results are only as good as the input parameters and assumptions applied by the engineer when using FE or FD software. In order to facilitate careful model validation, it is good practice to start with as simple a model as possible, and gradually increase the level of complexity to match the problem of interest. Individual aspects of the model can be validated based on problems with known solutions. For instance, structural elements can be subjected to related but simplified loading configurations and verified with theoretical moment and shear diagrams before progressing to the actual problem loading conditions. Similarly, soil behavior can be tested in the model and compared with theoretical results such as limit equilibrium earth pressures to verify that the behavior is as intended by the engineer.

Numerical model verification and validation is crucial and requires the engineer to have a strong fundamental knowledge base. Model validation should be performed and documented when using numerical models to guide engineering decisions. Justification should be provided for the selection of all model parameters and assumptions, including soil and structure properties and constitutive models, interface properties, geometric simplifications, and magnitude and distribution of applied loading.

5.6 GEOTECHNICAL CONSIDERATIONS

Subsurface conditions significantly influence tunnel planning, design, and construction, requiring detailed geotechnical investigations to be performed prior to construction for most tunnels. Load rating typically involves evaluations of existing tunnel conditions, which may or may not involve significant geotechnical considerations. For example deep tunnel structures embedded in rock may not require specific geotechnical input for load rating, particularly if the roadway support structure is not influenced significantly by the tunnel liner (see Example 1 and Example 2 in Section 9.2 and Section 9.3 in this Guide, respectively). However, if deterioration of the tunnel liner is observed at the time of load rating or anticipated to occur in the future, then geotechnical information may be required to assess the liner performance as it relates to the tunnel load rating. In addition, shallow cut-and-cover tunnels typically require geotechnical input in the form of horizontal and vertical earth loads, at a minimum (see Example 3 in Section 9.4). As cut-and-cover tunnels become larger and soil loads become more dominant, or involve more complex geometry and soil conditions, detailed geotechnical input may be required. Such situations may require analysis using numerical models as discussed in Section 5.5 (see Example 4 in Section 9.5). For all tunnels, the rating Engineer should be aware of the ground conditions surrounding the tunnel, and be able to assess whether or not these conditions would affect the load rating.

When geotechnical input is required for tunnel load ratings, existing information can typically be used. Geotechnical Data, Design, and/or Baseline Reports are often prepared prior to tunnel construction, and may be available to the rating Engineer. Regional and local geologic information can be found in the literature or in geologic maps. Geotechnical boring logs from the subject tunnel or other nearby structures may provide information that is useful to load rating analysis.

If geotechnical considerations are required for a load rating analysis and sufficient information is unavailable, geotechnical site investigations may be required. Site investigations should collect sufficient information for geotechnical characterization including:

- Surface topography overlying the tunnel.
- Regional and site specific geology including faults and fracture zones.
- Subsurface stratigraphy.
- Engineering properties of soil and rock layers.
- Hydrological conditions including groundwater depths along the alignment, locations of perched water, groundwater recharge, locations of aquifers/confined aquifers, artesian conditions and any other unusual hydrostatic pressures.
- Any unusual or challenging subsurface conditions that may affect the tunnel load rating.

Typical subsurface investigations include vertical exploratory borings (for example, hollow stem auger or mud rotary) and an associated laboratory test program. To provide detailed characterizations, more extensive subsurface investigations may be performed such as cone penetration testing (CPT), Ground Penetrating Radar (GPR) as illustrated in Figure 47, horizontal drilling (Figure 48), Spectral Analysis of Surface Waves (SASW), and downhole suspension P&S wave logging. In-situ field tests may also be useful for subsurface characterization, such as pressuremeter or dilatometer tests to determine ground stiffness parameters, pump tests to measure water recharge rates, or falling head tests to measure infiltration rates.

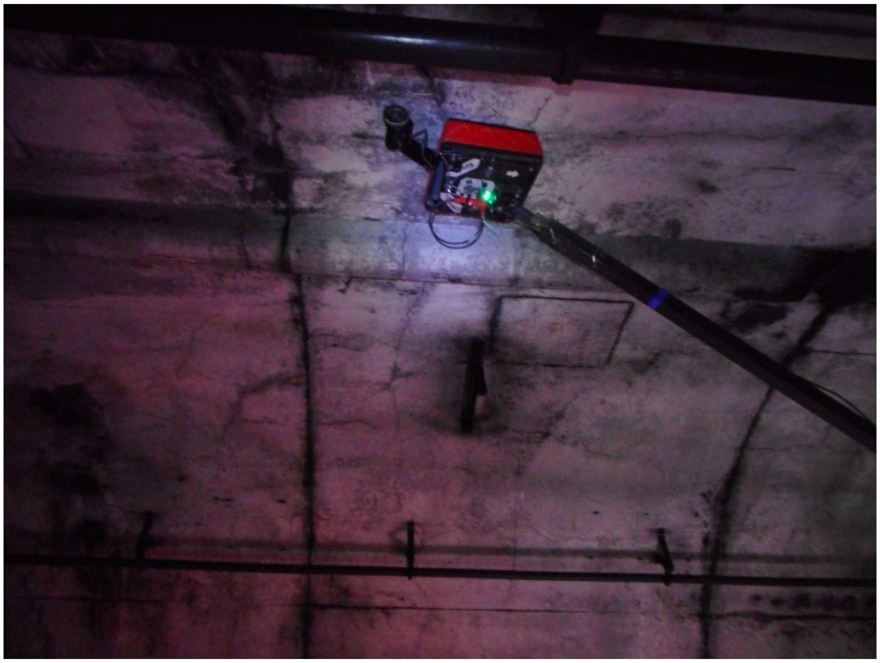

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Figure 47. Photo. Ground penetrating radar being performed inside a San Francisco Bay Area Rapid Transit (BART) tunnel.

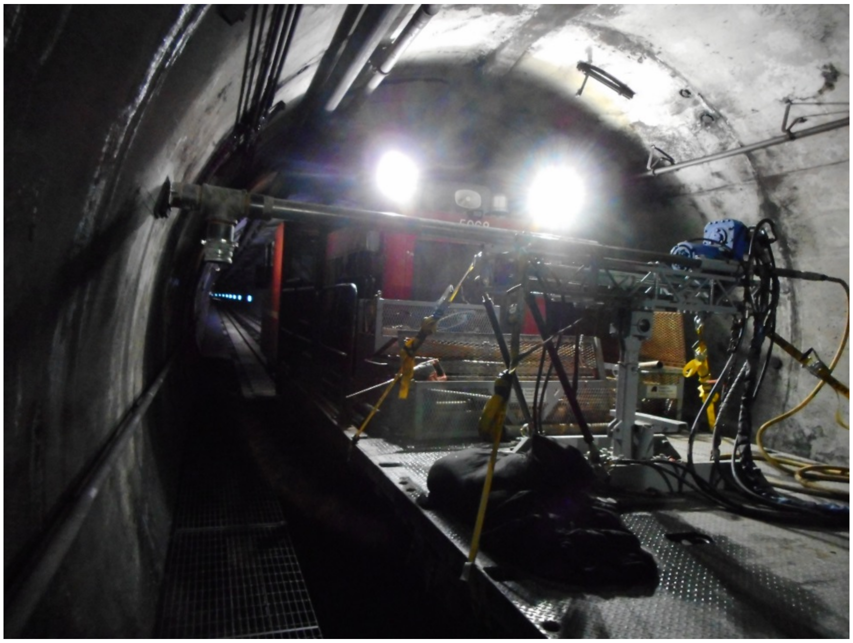

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Figure 48. Photo. Horizontal drilling inside a San Francisco Bay Area Rapid Transit (BART) tunnel.

5.6.1 Rock Parameters

This section provides background information on engineering rock parameters and behavior as related to tunnel structures in general. Much of this information is required to perform tunnel planning, design, and construction. For evaluation of existing tunnel conditions for load rating purposes, detailed characterization of rock parameters is not typically required, but may be necessary in some circumstances, such as when deterioration of the liner is observed or anticipated.

On a massive scale, rock is typically broken up between various seams, joints, infills and other discontinuities. As a result, tunnel behavior in rock is heavily influenced by characteristics of the overall rock mass, in addition to the intact properties of individual rock particles. Geotechnical studies for tunnels in rock should include geologic mapping, rock coring, laboratory rock testing, and geophysical soundings in order to collect parameters needed in the development of the stiffness and strength properties of rock mass surrounding the tunnel. The following information is often gathered for engineering design and may be useful for load rating of some tunnels:

- Rock lithology
- Mineral composition
- Weathering
- Rock Quality Designation (RQD)
- Type of fracturing and joints
- Hardness and abrasivity
- Strength of intact rock
- Number of joint sets
- Joint roughness
- Joint spacing and persistence
- Joint surface weathering
- Joint infilling
- Presence of water in joints
- Rock mass permeability

5.6.1.1 Strength

Rock mass strength is an important parameter for tunnel evaluations which cannot be directly measured either in the field or in the laboratory. As a result, the rock mass strength is typically assumed based on an interpretation of other identifiable rock parameters such as those listed above. Several methods are used to define empirical failure criteria. For example, Hoek and Brown (1980) developed the Rock Mass Rating (RMR) using rock mass characterization to derive strength parameters for failure criteria. The RMR system uses the following six parameters to classify a rock mass:

- Uniaxial compressive strength of rock material
- Rock quality designation (RQD)
- Spacing of discontinuities
- Condition of discontinuities
- Groundwater conditions
- Orientation of discontinuities

Each of the six parameters is assigned a value corresponding to the characteristics of the rock. These values are derived from field surveys and laboratory tests. The sum of the six parameters is the RMR value, which lies between 0 and 100. Table 1through Table 11 presents the methods for developing the standard RMR value for each of the six categories. The assignment of various RMR rating values for every type of rock mass requires considerable judgment and experience. The RMR value can then be used to estimate the failure envelopes of the rock mass following the generalized Hoek and Brown's (1980) empirical failure criterion as follows:

$$\sigma_{1f} = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^2} \qquad \text{(Eq. 5.6.1.1-1)}$$

where:

σ_{1f} = Major principal stress at failure (ksi)
σ_c = Unconfined compressive strength (ksi)
σ_3 = Confining pressure (ksi)
m, s, = Emperical constants given by the following equations:

$$m = m_i e^{\left(\frac{RMR-100}{28}\right)} \qquad \text{(Eq. 5.6.1.1-2)}$$

$$s = e^{\left(\frac{RMR-100}{9}\right)} \qquad \text{(Eq. 5.6.1.1-3)}$$

Where m_i is the Hoek-Brown parameter *m* for intact rock. There is much history and development associated with the Hoek-Brown parameter, which is beyond the scope of this document. The above relationships for *m* and *s* represent one recommendation out of several variations that are available. Refer to Hoek and Brown (1980) or Wood (1991) for detailed discussions.

Certain computer programs have the option of selecting the Hoek and Brown failure criterion, which resembles a hyperbolic envelope. Alternatively, traditional strength parameters *c* (cohesion coefficient) and ϕ (friction angle) can be estimated from the Hoek and Brown failure envelope based on the anticipated overburden pressure for use with the Mohr-Coulomb failure criterion.

Similar to the RMR system, Barton and Lien (1974) of the Norwegian Geotechnical Institute (NGI) developed a Tunneling Quality Index (Q) for determining rock mass characteristics and tunnel support requirements. The numerical value of the Index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000. Many of the rock parameters considered in the Q system are very similar to the RMR system but are combined differently.

Another available rock mass index is the Geological Strength Index (GSI), which concentrates on the description of two factors, rock structure and block surface conditions (Hoek, 2007). The value of GSI ranges from 10 for extremely poor rock mass to 100 for intact rock and the method is much simpler. Correlations of Q and GSI values to rock mass strength are available in the literature (Barton and Lien, 1974; Hoek, 2007).

5.6.1.2 Deformation

The failure criteria discussed above address limit states of stress conditions beyond which yielding will initiate. In conventional elastic-plastic modeling, the rock mass will behave elastically within the failure envelope, and its deformation is largely governed by the modulus of deformation, which is generally described by a shear modulus and Poisson's ratio. Outside of the failure envelope, plastic deformation will take place.

Since the shear modulus of rock mass is a composite behavior of rock and joint movements upon loading, laboratory testing of intact rock alone does not provide the necessary information. Field physical measurements are the most appropriate methods for determining the deformation characteristics of rock mass. Among them, pressuremeter testing and shear wave velocity measurements are practical options for in-situ methods because the test area generally covers a few feet of ground comprising several rock joints.

Alternatively, researchers have proposed estimating rock mass deformation modulus using rock mass classification from the back analyses of a number of case histories (FHWA, 2009). The back analyses yielded correlations of rock mass deformation modulus with RMR, Q and GSI values. In the absence of physical field measurements, the correlations in FHWA (2009) may be used to select rock mass deformation modulus, if it is required for a load rating application.

Table 1. Rock Mass Rating (RMR) Classification, Part A, Classification Parameters and Their Ratings, Parameter 1, Strength of Intact Rock for Point-Load Strength Index (Adapted from Hoek and Brown, 1980).

Criteria Category	1	2	3	4	5
Point-Load Strength Index	greater than 1.450 ksi	0.580 to 1.450 ksi	0.290 to 0.580 ksi	0.145 to 0.290 ksi	less than 0.145 ksi
Rating	**15**	**12**	**7**	**4**	**Uniaxial Compresion Strength is preferred**

Table 2. Rock Mass Rating (RMR) Classification, Part A, Classification Parameters and Their Ratings, Parameter 1, Strength of Intact Rock for Uniaxial Compressive Strength (Adapted from Hoek and Brown, 1980).

Criteria Category	1	2	3	4	5	6	7
Uniaxial Compressive Strength	greater than 36.3 ksi	14.5 to 36.3 ksi	7.25 to 14.5 ksi	3.63 to 7.25 ksi	0.725 to 3.63 ksi	0.145 to 0.725 ksi	less than 0.145 ksi
Rating	**15**	**12**	**7**	**4**	**2**	**1**	**0**

Table 3. Rock Mass Rating (RMR) Classification, Part A, Classification Parameters and Their Ratings, Parameter 2, Drill Core Quality RQD (Adapted from Hoek and Brown, 1980).

Criteria Category	1	2	3	4	5
Drill Core Quality RQD	90% to 100%	75% to 95%	50% to 75%	25% to 50%	0% to 25%
Rating	**20**	**17**	**13**	**8**	**5**

Table 4. Rock Mass Rating (RMR) Classification, Part A, Classification Parameters and Their Ratings, Parameter 3, Spacing of Discontinuities (Adapted from Hoek and Brown, 1980).

Criteria Category	1	2	3	4	5
Spacing of Discontinuities	greater than 78.7 in.	23.6 in. to 78.7 in.	7.87 in. to 23.6 in.	2.36 in. to 7.87 in.	less than 2.36 in.
Rating	**20**	**15**	**10**	**8**	**5**

Table 5. Rock Mass Rating (RMR) Classification, Part A, Classification Parameters and Their Ratings, Parameter 4, Condition of Discontinuities (Adapted from Hoek and Brown, 1980).

5A. Length Persistence.

Criteria Category	1	2	3	4	5
Length, Persistence	less than 3.3 ft	3.3 ft to 9.8 ft	9.8 ft to 32.8 ft	32.8 ft to 65.6 ft	greater than 65.6 ft
Rating	6	4	2	1	0

5B. Separation.

Criteria Category	1	2	3	4	5
Seperation	none	less than 0.004 in.	0.004 to 0.039 in.	0.039 to 0.197 in.	greater than 0.197 in.
Rating	6	5	4	1	0

5C. Roughness.

Criteria Category	1	2	3	4	5
Roughness	very rough	rough	slightly rough	smooth	slickensided
Rating	6	5	3	1	0

5D. Infilling (gouge).

Criteria Category	1	2	3	4	5
Infilling (gouge)	none	hard filling and less than 0.197 in.	hard filling and greater than 0.197 in.	soft filling and less than 0.197 in.	soft filling and greater than 0.197 in.
Rating	6	4	2	2	0

5E. Weathering.

Criteria Category	1	2	3	4	5
Weathering	unweatherd	slightly wet	moderately wet	highly wet	decomposed
Rating	6	5	3	1	0

Table 6. Rock Mass Rating (RMR) Classification, Part A, Classification Parameters and Their Ratings, Parameter 5, Ground Water (Adapted from Hoek and Brown, 1980).

Criteria Category	1	2	3	4	5
Inflow per 32.8 ft tunnel length	none	less than 2.64 gal/min	2.64 to 6.60 gal/min	6.60 to 33.0 ga;/min	greater than 33.0 gal/min
p_w / $\sigma 1$	0	0 to 0.1	0.1 to 0.2	0.2 to 0.5	greater than 0.5
General Conditions	completely dry	damp	Wet	dripping	flowing
Rating	**15**	**10**	**7**	**4**	**0**

p_w = joint water pressure; $\sigma 1$ = major principle stress.

Table 7. Rock Mass Rating (RMR) Classification, Part B, Rating Adjustment for Discontinuity Orientations (Adapted from Hoek and Brown, 1980).

Classification*	very favorable	favorable	fair	unfavorable	very unfavorable
Ratings for Tunnels	0	-2	-5	-10	-12
Ratings for Foundations	0	-2	-7	-15	-25
Ratings for Slopes	0	-5	-25	-50	-60

* Use Table 8 and Table 9 to determine Classification.

Table 8. Rock Mass Rating (RMR) Classification, Part F, Guidelines for Classification of Discontinuity Orientations with Discontinuity Strike Perpendicular to Tunnel Axis (Adapted from Hoek and Brown, 1980).

Strike Perpendicular to Tunnel Axis	Classification
Drive with dip – Dip 45 to 90 degrees	very favorable
Drive with dip – Dip 20 to 45 degrees	favorable
Drive against dip – Dip 45 to 90 degrees	fair
Drive against dip – Dip 20 to 45 degrees	unfavorable

Table 9. Rock Mass Rating (RMR) Classification, Part F, Guidelines for Classification of Discontinuity Orientations with Discontinuity Strike Parallel to Tunnel Axis (Adapted from Hoek and Brown, 1980).

Strike Parallel to Tunnel Axis	Classification
Dip 45 to 90 degrees	very unfavorable
Dip 20 to 45 degrees	fair
Dip 0 to 20 degrees – Irrespective of strike	fair

Table 10. Rock Mass Rating (RMR) Classification, Part C, Rock Mass Classes Determined From Total Ratings (Adapted from Hoek and Brown, 1980).

Rating	100 to 81	80 to 61	60 to 41	40 to 21	20 to 0
Class No.	I	II	III	IV	V
Description	VERY GOOD	GOOD	FAIR	POOR	VERY POOR

Table 11. Rock Mass Rating (RMR) Classification, Part D, Meaning of Rock Mass Classes (Adapted from Hoek and Brown, 1980).

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 49.2ft span	6 months for 26.2ft span	1 week for 16.4ft span	10 hours for 8.2ft span	30 minutes for 3.3ft span
Cohesion of the rock mass	greater than 58.0 psi	43.5 to 58.0 psi	29.0 ksi to 43.5 psi	14.5 to 29.0 psi	less than 14.5 psi
Friction angle of the rock mass	less than 45 degrees	35 to 45 degrees	25 to 35 degrees	15 to 25 degrees	less than 15 degrees

5.6.2 Soil Parameters

There is typically a high level of uncertainty and variability in soil parameters throughout the subsurface area of interest for a tunnel. Soil parameters for use in engineering analysis can be determined based on conventional soil borings, laboratory testing, field testing, back analysis of case histories experiments, and engineering judgment. On most projects, soil borings are typically performed by hollow stem auger or mud rotary drill rigs. Soil samples are collected at selected depths by driving a split-spoon sampler (for example, Standard Penetration Test, or SPT) with repeated hammer blows over a specified length. Engineering properties of the soil are often correlated based on the SPT blow counts, or determined based on laboratory tests performed on the collected samples. Laboratory tests provide information on soil classification, grain size distribution, plasticity, density, moisture content, stiffness, strength, and other relevant behavior characteristics for modeling and evaluation purposes. Cone Penetration Testing (CPT) and geophysical testing can also be performed to estimate in-situ soil properties.

For load rating, if soil properties are needed in analysis, the load rating Engineer should start by searching for original tunnel design information, including geotechnical investigations and reports and construction plans. If the necessary information is not available, in-situ tests and geotechnical exploration may be required.

Soil unit weight, strength, and deformation characteristics are the most likely engineering parameters to be required for load rating purposes. Horizontal and vertical earth pressures are determined using the unit weight and strength parameters. More detailed load rating analyses (such as those performed by numerical modeling as discussed in Section 5.5) will also require information on soil deformability or stress-strain behavior.

Unit weight is typically determined by performing moisture and density tests on samples collected during field investigations, or estimated based on soil descriptions and typical values.

Strength parameters and deformation characteristics of soils are discussed in the following sections.

5.6.2.1 Strength

Tunnel load rating evaluations may require an estimate of soil strength. Strengths can be evaluated based on laboratory tests on samples where disturbance has been minimized as much as possible. Triaxial and direct-shear testing are among the most common laboratory strength tests. However, it is difficult to achieve truly undisturbed samples, and actual field conditions are difficult to mimic in the laboratory.

In-situ strength properties can also be correlated from field data. Such correlations are often used for granular soils (that is, sand), whose strength is highly susceptible to influence from sample disturbance. Correlations for effective friction angle are available for SPT blow counts (for example, FHWA, 1986) as well as CPT data (for example, Robertson and Cabal, 2014). For cohesive soil, the undrained shear strength can also be correlated based on CPT data (Robertson and Cabal, 2014). In-situ testing such as vane shear can be performed to provide an index to strength and sensitivity (Terzaghi et al., 1996).

Ultimately, sound engineering judgment is needed when adopting strengths for engineering applications. It is good practice to review as much information as possible (for example, from field testing, laboratory testing, geology, and correlations) before selecting strength properties for use in engineering analyses. Since it is not possible to identify a precise value of each material parameter, it is also wise to perform sensitivity analyses to understand a range of analysis outcomes and address uncertainty (see Example 4 in Section 9.5 in this Guide).

5.6.2.2 Deformation

The deformation characteristics of soils surrounding a tunnel can affect the distribution of stresses in the structural members when subjected to external loads. As a result, soil deformations and soil-structure interaction may need to be considered in some load rating analyses.

Soil behavior is highly nonlinear even at very small strain levels, and the stress-strain behavior of soil varies widely and is difficult to predict. At the most basic level, soil deformation is typically considered by adopting elastic stiffness parameters such as Young's modulus or shear modulus and Poisson's ratio. Due to the high level of soil nonlinearity, secant modulus values should be used to approximate stiffness relevant to strain levels of interest when assuming linear elastic behavior. A more accurate representation of soil stress-strain behavior can be achieved through numerical modeling with nonlinear constitutive models, as discussed in Section 5.5.

A large amount of information is available in the literature for estimating soil stiffness parameters based on soil descriptions, laboratory test data, and empirical correlations (for example, AASHTO 2017, FHWA 1986, and Kulhawy and Mayne 1990). However, there is considerable scatter in the recommendations and data, so engineers must exercise great care in selecting stiffness parameters from generic recommendations in the literature. Alternatively, in-situ testing such as pressuremeter or dilatometer testing, or downhole shear wave velocity measurements can provide a more direct measurement of soil stiffness and stress-strain behavior.

It is good practice to estimate soil deformation characteristics using a range of methods (for example, recommendations in the literature, correlations, and in-situ testing), and use judgment to select parameters for analysis while accounting for uncertainty. For tunnels that are particularly sensitive to soil stiffness, a range of possible parameters (for example, upper-estimate and lower-estimate) can be considered in the analysis to determine the worst-case load effects for load rating purposes (see Example 4 in Section 9.5 in this Guide).

5.7 SELECTION OF ANALYSIS METHOD

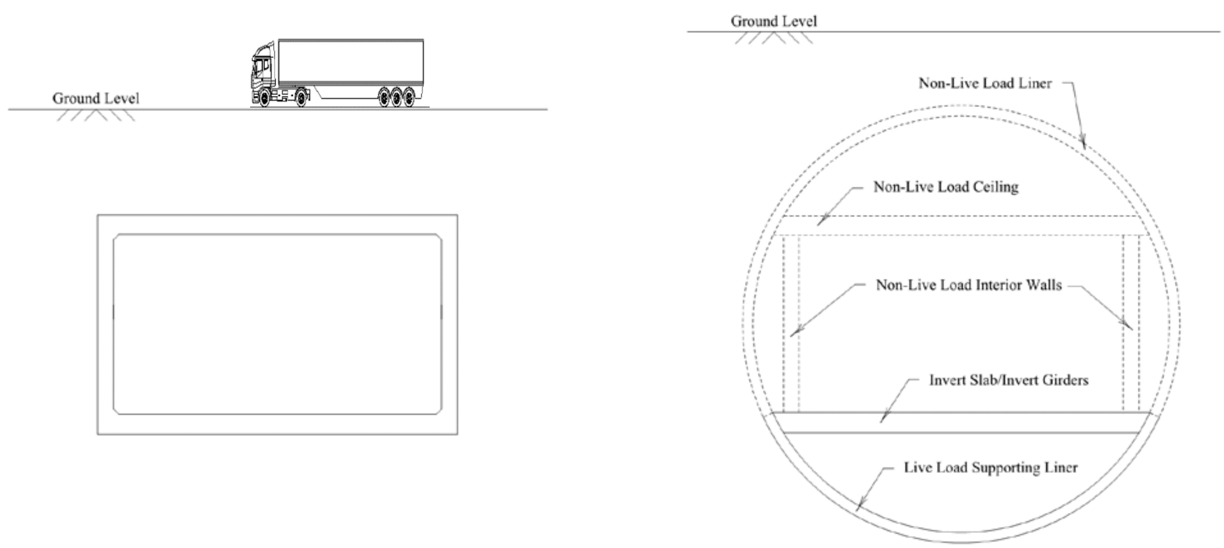
The selection of an appropriate analysis method depends on the details of each particular tunnel structure. For a tunnel that matches the assumptions of an established simplified method, that method should be used to perform the initial load rating analysis. If the simplified analysis method results in a sufficient load rating and is believed to be conservative, a refined analysis may not be needed. If the simplified analysis method results in an insufficient load rating and is believed to be overly-conservative, then a more complex analysis procedure such as numerical modeling may be considered if justified appropriate and necessary. In addition, for problems which fall outside the range of applicability for a simplified analysis method, a sufficiently realistic and sophisticated method of analysis should be adopted.

CHAPTER 6 - LOAD RATING PROCEDURE

6.1 INTRODUCTION

Load rating provides a basis for determining the safe load capacity of tunnel components. Load rating typically requires engineering computations to determine the loads and resistances, as well as engineering judgment regarding the existing condition of the structure in order to determine a rating value that is applicable to maintaining the safe use of the tunnel components. Load ratings can also be used to identify the need for load posting or component strengthening, as well as make decisions regarding overweight-vehicle permits.

This section details how to load rate two types of tunnel systems, as discussed in Chapter 2.This includes external and internal loading. Samples of these scenarios are depicted in Figure 49 where solid lines are live load carrying members and dashed lines are non-live load carrying members. Note, these are samples and each structure shall be individually evaluated to identify the live load carrying members. Evaluation of the live load caring elements are discussed in more detail in Chapter 5



A. Cut and cover (external system).   B. Bored tunnel (internal system).

Source: FHWA

Figure 49. Illustrations. Live load carrying members.

MBE Article 6A.4.1 describes the three load rating procedures or levels for evaluation consistent with Load and Resistance Factor Rating (LRFR). These are:

- Design load rating (1st level evaluation)
- Legal load rating (2nd level evaluation)
- Permit load rating (3rd level evaluation)

The above levels are structured to be performed in a sequential manner, as needed, starting with the design load rating. MBE Appendix A6A contains a flowchart that visualizes the process for LRFR ratings. The above levels are described in more detail later in this chapter.

6.2 LOAD RATING EQUATION

Load Ratings are generally expressed as a rating factor for a particular live load model, using the general LRFR rating equation shown below, taken from MBE Article 6A.4.2.1 and modified to include tunnel structures similar to the Load Rating Equation from TOMIE Manual Section 5.4.2.4.

$$RF = \frac{C \pm (\gamma_{DC})(DC) \pm (\gamma_{DW})(DW) \pm (\gamma_{EV})(EV) \pm (\gamma_{EH})(EH) \pm (\gamma_{ES})(ES) \pm (\gamma_{P})(P)}{(\gamma_{LL})(LL + IM) \pm (\gamma_{LS})(LS)}$$

MBE Article 6A.4.2.1 defines the terms of the equation in depth.

For the Strength Limit States:

$$C = \varphi_c \varphi_s \varphi R_n \qquad \text{(MBE 6A.4.2.1-2)}$$

Where the following lower limit shall apply:

$$\varphi_c \varphi_s \geq 0.85 \qquad \text{(MBE 6A.4.2.1-3)}$$

For the Service Limit States:

$$C = f_R \qquad \text{(MBE 6A.4.2.1-4)}$$

where:

RF	=	Rating factor
C	=	Capacity
f_R	=	Allowable stress specified in the LRFD BDS code
R_n	=	Nominal member resistance (as inspected)
DC	=	Dead load effect due to structural components and attachments
DW	=	Dead load effect due to wearing surface and utilities
EV	=	Vertical earth pressure
EH	=	Horizontal earth pressure
ES	=	Uniform earth surcharge
LS	=	Live load surcharge
P	=	Permanent loads other than dead loads
LL	=	Live load effect
IM	=	Dynamic load allowance
γ_{DC}	=	LRFD BDS load factor for structural components and attachments
γ_{DW}	=	LRFD BDS load factor for wearing surfaces and utilities
γ_{EV}	=	LRFD BDS load factor for vertical earth pressure
γ_{EH}	=	LRFD BDS load factor for horizontal earth pressure
γ_{ES}	=	LRFD BDS load factor for uniform earth surcharge
γ_{LS}	=	LRFD BDS load factor for live load surcharge
γ_{P}	=	LRFD BDS load factor for permanent loads other than dead loads = 1.0

γ_{LL} = Evaluation live load factor
φ = LRFD resistance factor
φ_c = Condition factor
φ_s = System factor

Note that unlike the general limit state equation in the LRFD BDS, ductility, redundancy, and operational importance load modifiers, η, are not included in the general load rating equation. For load ratings, ductility is considered in conjunction with redundancy and incorporated in the system factor, φ_s. Operational importance is not explicitly considered in the LRFR load rating provisions; however, the live load factors for legal and permit loads are dependent on the average daily truck traffic (ADTT) at the tunnel site.

6.3 LOAD RATING TYPES

6.3.1 Design Load Rating (1st Level Evaluation)

The design load rating assesses the performance of the existing (or a newly designed/constructed) tunnel utilizing the LRFD HL-93 live load. The design load rating of tunnels may be performed at the same design level (Inventory level) reliability adopted for new bridges by the LRFD BDS or at a second, lower-level reliability, comparable to the Operating level reliability inherent in past load rating practice. The design load rating produces Inventory and Operating level rating factors for the HL-93 loading. The design load rating serves as a screening criterion to identify tunnels and tunnel components that should be load rated for legal loads in accordance with MBE Article 6A.4.3.

When the Rating Factor at the Inventory Level Rating is greater than 1.0 for design load rating (HL-93 live load), no further evaluation for legal loads under the envelope of grandfathered provisions (exclusion trucks) is necessary for those tunnel components. Tunnel components that pass the HL-93 live load screening only at the Operating Level Rating may not have adequate capacity for certain legal loads heavier than the AASHTO legal loads. Existing structures that do not pass a design load rating at the Inventory Level Rating, but pass a design load rating at the Operating Level Rating shall be evaluated by load rating for applicable legal loads heavier than the AASHTO legal loads. Existing structures that do not pass a design load rating at the Operating Level Rating shall be evaluated by load rating for all applicable legal loads.

6.3.2 Legal Load Rating (2nd Level Evaluation)

Tunnel components that do not pass a design load rating at the Operating Level Rating (RF < 1.0) or that the State has specific legal loads exceeding federal weight limits defined in 23 United States Code 127(a)(1) and (2) or 23 CFR 658.17 (b) to (e) shall be evaluated by load rating for legal loads to establish the need for load posting or strengthening. Legal loads include the AASHTO legal loads and State legal loads. FHWA emergency vehicles do NOT need to be evaluated for rating if the Operating Rating factor of AASHTO Type 3 Legal Load is greater than 1.85, the LFR Inventory Rating Factor for an HS-20 is greater than 1.0, the LRFR Inventory Rating Factor for an HL-93 is greater than 0.9. Refer to MBE Article 6A.4.4 and its associated commentary C6a.4.4.1 for additional background on legal loads for LRFR ratings and FHWA

Memorandum “Load Rating for FAST Act’s Emergency Vehicles” published Nov. 3, 2016 for Emergency Vehicles requirement details.

Vehicles that do not surpass a load rating factor of 1.0 need to be posted. Postings typically include legal loads and unrestricted routine permit loads that do not meet rating requirements in accordance with 23 CFR 650.313c and 23 CFR 650.513(g). Posting procedures are further discussed in Section 6.7.

Live load factors shall correspond to the live load designation and Average Daily Truck Traffic (ADTT) as indicated in MBE, Appendix B6A, or as otherwise specified by the governing agency. Additionally, State specific legal loads having only minor variations from the AASHTO legal loads shall be load rated using load factors specified for the routine commercial AASHTO legal trucks. Those that are significantly heavier than the AASHTO legal loads may be load rated using load factors specified for routine permits, if the structural component has sufficient capacity for AASHTO legal loads (MBE 6A.2.3).

6.3.3 Permit Load Rating (3rd Level Evaluation)

Permit Load Rating checks the safety and serviceability of structures in the review of permit applications for the passage of vehicles above legally established weight limitations. This is considered a third level rating and is typically only applied to structures that have sufficient capacity for the Legal Loads described previously (depending on State statute, regulation and permitting policy). Permit loads evaluation is primarily strength; however, service limit states are strongly recommended or required by some owners. These evaluations are generally performed for both moment and shear evaluations and may include connections or alternative strength evaluations if requested by the owner.

6.3.3.1 Routine Permit Loads

Routine permits are usually valid for unlimited trips over a period of time, not to exceed one year. Typically, routine permit vehicles are allowed to mix with the normal traffic stream and move at normal speeds without restrictions, unless restrictions are specified by the Owner on the hauling permits.

6.3.3.2 Special Permit Loads

Special permits are usually valid for a single trip or a set number of trips only due to these vehicles typically being significantly heavier than the Legal Loads. It is not uncommon that special permits have restrictions imposed on the movement of the vehicle, including the use of escorts, speed limitations, and traffic control requirements, as specified by the Owner. Special permit vehicles are normally route-specific or operate on the designated routes written on the issued permits.

6.4 RATING REQUIREMENTS

6.4.1 Limit States & Load Factors

For structures that are similar to typical bridge type structures (cut-and-cover, invert slab, invert girders with or without stringers and a deck slab), the MBE limit states and load factors may be used. Strength, service and fatigue are the primary limit states for these types of structures; however service and fatigue limit states may selectively applied as specified by the owner. MBE Article 6A.4.2.2 and MBE Table 6A.4.2.2-1 contain load factors applied to typical structure types at the three different levels of rating. MBE Table 6A.4.2.2-1 also references additional load factors for specified limit states that are included under MBE Article 6A.4.4 and Article 6A.4.5 for Legal and Permit load rating.

6.4.2 Condition Factors

Use of Condition Factors as presented below may be considered optional based on an Agency's load-rating practice, but are in addition to element level data. The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles. If condition information is collected and recorded in the form of NBI condition ratings only, then the first column of the following table may be referenced. Table 12 is based on MBE Table 6A.4.2.3-1 & Table C6A.4.2.3-1 as well as the SNTI Condition States. Also refer to MBE Article 6A.4.2.3.

Table 12. Condition Factors (MBE Table 6A.4.2.3-1).

Condition Rating from "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges"	Condition State from "Specifications for the National Tunnel Inventory"	Structural Condition of Member	Condition Factor, φ_c
6 or higher	Condition State 1	Good or Satisfactory	1.00
5	Condition State 2	Fair	0.95
4 or lower	Condition States 3 or 4	Poor	0.85

6.4.3 System Factors

MBE Article 6A.4.2.4 indicates the use of system factors (φ_s). MBE defines system factors as "*System factors are multipliers applied to the nominal resistance in the general load rating equation and are intended to reflect the level of redundancy of the given structural system*". Structures that are less redundant will have their factored member capacities reduced, and, accordingly, will have lower ratings. Note that in accordance with MBE Article C6A.4.2.1, ductility is considered in conjunction with redundancy and is incorporated in the system factor, φ_s. MBE Table 6A.4.2.4-1 identifies system factors for flexural and axial effects on a variety of structural systems. All other force effects have a system factor of 1.0. System factors that correspond to the load modifiers (η) for ductility and redundancy in LRFD BDS Article 1.3.3 and Article 1.3.4 shall be used in the evaluation. Note that the system factor in the MBE is the inverse of the load modifier in the LRFD BDS, as the load modifiers in the LRFD BDS are applied on the load side of the general limit state equation while system factors are applied to the resistance of the member in the MBE.

The system factors listed in MBE Table 6A.4.2.4-1 are generally more conservative than the corresponding load modifiers in the LRFD BDS, and can be applied at the discretion of the Owner. MBE Table 6A.4.2.4-1 is based on typical bridge superstructure types, but can also be applied to similar structure types inside of tunnels. For example, an invert slab or cut-and-cover type structure would likely fall under the “All Other Girder Bridges and Slab Bridges” category as their redundancy would be similar to a slab type bridge.

For tunnel structural systems, LRFDTUN Article 1.3.3 and Article 1.3.4 provide load modifiers for ductility and redundancy, respectively. These shall be considered with corresponding system factors as deemed necessary in the load rating evaluation.

6.5 SPECIFIC TYPES OF STRUCTURES OR COMPONENTS CONDITION

6.5.1 Concrete Structures

MBE Article 6A.5 contains specific provisions for load rating of concrete structures. Guidance is provided pertaining to unknown materials, resistance factors, applicable limit states, and items that relate to determining the structural capacity. LRFDTUN Article 4.5.2 provides provisions for the design of tunnel structural components constructed of normal weight concrete or lightweight concrete and reinforced with steel bars or welded wire fabric, and/or prestressed with strands, bars, or wires. Concrete structural components that possibly need to be evaluated through a load rating include a structural roadway slab or the tunnel lining if subjected to live load.

6.5.1.1 Material Properties

For newer structures, LRFDTUN Article 4.5.2.1 refers designers to LRFD BDS Section 5 for reinforced or prestressed concrete material properties.

When the compressive strength of concrete is unknown and the concrete is in satisfactory condition, MBE Table 6A.5.2.1-1 may be used. If initial load capacity is not adequate, cores may be taken to determine actual strength (see MBE C6A.5.2.1).

When the reinforcing steel yield strength is unknown, MBE Table 6A.5.2.2-1 may be used.

When the prestressing steel tensile strength is unknown, MBE Table 6A.5.2.3-1 may be used. If the strand type is unknown, assume stress-relieved (see MBE Article C6A.5.2.3).

6.5.1.2 Resistance Factors for Concrete

Resistance factors, φ, for concrete for the strength limit state shall be taken as specified in LRFD BDS Article 5.5.4.2 and MBE Article 6A.5.3.

6.5.1.3 Flexural Members

The flexural capacity of members shall be evaluated for all required rating levels. The LRFD BDS provides the necessary provisions for computation of the nominal flexural resistance, M_n. Load raters or engineers shall note that MBE Article 6A.5.6 requires that a reduction factor, K, be applied to the flexural resistance of concrete members that do not satisfy the minimum

flexural reinforcement provisions in the LRFD BDS. Furthermore, for components constructed of normal weight concrete reinforced with steel fibers, LRFDTUN Article 4.5.4.3 provides additional requirements that need to be considered for the computation of the nominal flexural resistance.

6.5.1.4 Bearing

For components constructed of normal weight concrete reinforced with steel fibers, LRFDTUN Article 4.5.4.3.4 requires an additional computation for the factored splitting resistance. When necessary, this splitting resistance should be considered in the rating analysis.

Stringer-supported concrete decks supported on invert girders and subjected to normal traffic and in satisfactory condition need not be load rated in accordance with MBE Article 6.1.5.1unless the slab is showing signs of deterioration or distress. Concrete slabs shall be regularly inspected for deterioration and distress to determine the adequacy of the system.

6.5.1.5 Evaluation for Shear

In accordance with LRFDTUN Article 4.5.2.2, reinforced and prestressed concrete structural components shall be evaluated for the force effects due to shear at the service and strength limit states. Although a shear check for non-posttensioned non-segmental concrete members may be waived in the design load or the legal load rating in accordance with MBE Article 6A.5.8 in special instances, it is prudent and strongly recommended that shear be always included in all load rating analyses for the design, legal and permit loads unless the load rating Engineer can conservatively justify that shear does not pose any safety concerns in those instances.

The longitudinal reinforcement shall be evaluated for increased tension when using the Modified Compression Field Theory to evaluate the concrete shear resistance. As noted in MBE Article C6A.5.8, in lieu of the more detailed analysis outlined in the LRFD BDS for MCFT, a simplified analysis which assumes $\beta = 2.0$ and $\theta = 45°$ may be first attempted for the shear resistance of reinforced concrete sections and standard prestressed concrete sections with transverse reinforcement, as a conservative shear resistance for load rating will typically be obtained. The Engineer can make further refinements to β and θ as necessary, should the first attempt not yield a shear resistance that provides for a sufficient load rating.

6.5.1.6 Evaluation for Flexure and Axial Force

In accordance with MBE Article 6A.5.7, tunnel components that are subjected to a combination of axial force and moment shall be evaluated by considering the effect on load capacity of the interaction of axial and bending load effects. MBE Appendix G6A provides a method that is often used for evaluating load capacity of components subjected to flexural and axial effects (see MBE commentary C6A.5.7).

In accordance with LRFDTUN Article 4.5.2.2, cast-in-place concrete liners for mined and bored tunnels and precast segmental concrete liners for bored tunnels need to consider the combined effects of moment and axial force. Therefore, if these liner components shall be load rated (subjected to live load), the interaction of axial and bending load effects needs to be considered.

6.5.2 Steel Structures

MBE Article 6A.6 contains specific provisions for load rating of permanent steel structures and components. Guidance is provided pertaining to unknown materials, resistance factors, applicable limit states, and items that relate to determining the structural capacity. Structural steel components may include the systems supporting the roadway surface, such as invert girders or a steel frame system.

6.5.2.1 Material Properties

When the mechanical properties of structural steel are unknown, the properties given in MBE Table 6A.6.2.1-1 may be used based upon the year of construction. If initial load capacity is not adequate or if there is doubt about the nature and quality of materials, the mechanical properties can be verified by testing (see MBE Article C6A.6.2.1). For material properties of newer structural steel components, LRFD BDS Section 6 shall be consulted by the Engineer.

6.5.2.2 Resistance Factors for Steel

Resistance factors, ϕ, for structural steel for the strength limit state shall be taken as specified in LRFD BDS Article 6.5.4.2 and MBE Article 6A.6.3. LRFDTUN Article 4.6 does not specify any adjustments for resistance factors for structural steel tunnel components.

6.5.2.3 Tension Members

In accordance with MBE Article 6A.6.6, tunnel components subjected to axial tension shall be evaluated with consideration of yielding on the gross section and fracture on the net section. The nominal resistance for yielding on the gross section and for fracture on the net section shall be computed in accordance with LRFD BDS Article 6.8.2.

6.5.2.4 Non-composite Compression Members

The nominal compressive resistance of non-composite compression members is discussed in MBE Article 6A.6.7, and in detail in LRFD BDS Article 6.9.4.1 and Article 6.9.4.2. In order to compute the nominal compressive resistance on non-composite compression members, the Engineer must determine if the section is non-slender or slender. A non-composite compression member is considered non-slender when LRFD BDS Equation 6.9.4.2.1-1 is satisfied for elements in the member cross-section. If the member is non-slender, the nominal compressive resistance shall be determined in accordance with LRFD BDS Article 6.9.4.1. Members that have elements in the cross-section that do not satisfy LRFD BDS Equation 6.9.4.2.1-1 are considered as slender, and their nominal compressive resistance can be determined in accordance with LRFD BDS Article 6.9.4.2.2 . Note that MBE Article 6A.6.7 refers the Engineer to the AISC Steel Construction Manual, 13th Edition for slender elements; however, that design methodology has since been adopted by AASHTO in LRFD BDS Article 6.9.4.2.2 in the release of the 8th Edition.

6.5.2.5 Combined Axial Compression and Flexure

MBE Article 6A.6.8 notes that the load rating of steel members subjected to axial compression and concurrent moments is to be determined using the interaction equations specified in LRFD BDS Article 6.9.2.2. The load rating of these members should also consider second-order effects. Generally, for tunnel components (because of their short height), these second-order effects can be approximated with the moment magnification method given in LRFD BDS Article 4.5.3.2.2b. Furthermore, the load rating of compression members with interaction equations may require an iterative approach to determine the governing rating. MBE Appendix H6A provides a load rating approach with interaction equations.

6.5.2.6 I-Sections in Flexure

I-Sections subjected to flexure are to have their nominal flexural resistance computed in accordance with MBE Article 6A.6.9, which points the Engineer to LRFD BDS Article 6.10.6.2. Generally, for tunnel components, flange lateral bending does not need to be considered in the load rating equation. Also, as noted in MBE Article 6A.6.9, the constructability requirements of LRFD BDS Article 6.10.3 and the fatigue requirements of webs specified in LRFD BDS Article 6.10.5.3 do not need to be considered in the load rating.

LRFD BDS Article 6.10.7 provides provisions for computing the nominal flexural resistance of composite sections in positive flexure. When computing member flexural stresses, permanent dead loads applied to the composite section may be assumed to be carried by the long-term composite section, as defined in LRFD BDS Article 6.10.1.1.1b.

LRFD BDS Article 6.10.8 provides provisions for computing the nominal flexural resistance of composite sections in negative flexure as well as non-composite sections. In accordance with MBE Article 6A.6.9.3, the top flanges in compression in positive flexure can be assumed to be continuously braced by the concrete deck when there are no shear connectors present, as long as the top flange is in full contact with the concrete deck and there are no signs of cracking, corrosion, or separation along the deck-girder interface.

6.5.2.7 Shear

As noted in MBE Article 6A.6.10, shear resistance for I-girders at the strength limit state is to be computed in accordance with LRFD BDS Article 6.10.9. In this article, provisions are provided for computing the nominal shear resistance of both unstiffened and stiffened webs.

6.5.2.8 Critical Connections

MBE Article 6A.6.12 provides guidance with regard to the load rating evaluation of critical connections. The MBE points out that external connections of non-redundant members shall be evaluated during a load rating analysis in situations where the evaluator has reason to believe that their capacity may govern the load rating. For bridges, external connections are considered as connections that transfer calculated load effects at support points of a member, while non-redundant members are those that do not have an alternate load path and whose failure is expected to cause the collapse of the structure. In the case of tunnel components, there may be external connections, in a floor system for example, but there is probably little likelihood that non-redundant members are used as structural steel tunnel components. Additionally, connections and splices of structural steel components are often designed to have equal or greater capacity than the members they adjoin. However, in the case of tunnel components, the evaluator may want to consider any connection that is deemed critical to the live load functionality of the tunnel system, and not necessarily just those that are considered external or non-redundant.

For tunnel components, structural steel connections are likely to be either bearing type connections or slip-critical connections. MBE Article 6A.6.12.2 notes that bearing type connections are to be evaluated at the strength limit state at the Operating Level for HL-93 live loading. LRFD BDS Article 6.13.2.7 provides provisions for computing the nominal shear resistance of high strength bolts, while LRFD BDS Article 6.13.2.9 provides provisions for determining the nominal bearing resistance at bolt holes. MBE Article 6A.6.12.3 notes that slip-critical connections are to be evaluated for slip at the Service II load combination at the Operating Level for HL-93 live loading, as well as for bearing, shear and tensile resistance at the strength limit state. LRFD BDS Article 6.13.2.9 provides provisions for determining the nominal slip resistance of a slip-critical connection.

6.5.3 Timber Structures

This document does not provide specific guidelines for evaluation of timber structures and components. Refer to MBE Article 6A.7 for load rating procedures.

6.5.4 Masonry Structures

This document does not provide specific guidelines for evaluation of masonry structures and components. Refer to MBE Article 6A.9.1 for load rating procedures.

6.6 CONSIDERATION OF STRUCTURAL DEFICIENCIES

Current conditions of a tunnel may influence the behavior and capacity of the structure. Deficiencies and deteriorations may influence the dead loads as well as the load distribution of the structure. Additionally, deficiencies may influence the resulting capacities of individual members, which may adversely affect the rating factor as well as the structural health and safety of the tunnel. These deficiencies should be accounted for in not only live load carrying members but non-live load carrying members. If non-live load carrying members are found to have questionable deficiencies, these members should be evaluated for structural health and safety as discussed in Chapter 8. The following sections discuss common deficiencies found in tunnel components. Additional deficiencies may exist and may require further investigation.

6.6.1 Typical Deficiencies

Live load and non-live load carrying tunnel components need to be considered when assessing the structural health and safety of tunnel structures. Deterioration and damage to these structural elements can create a safety and longevity issue for a tunnel structure. Structural elements including, but not limited to, invert structures, ceilings and roofs, liners, columns and piles typically carry structural loads and are critical to the structural safety of a tunnel. Each of these structural elements inherently have common deficiency characteristics that should be considered when inspecting tunnels. However, care should be taken to consider possible critical deficiencies, not specifically detailed in this document, but which could cause structural health issues.

Deterioration markers such as cracks and efflorescence in concrete may indicate deficiencies in the member's structural integrity that is not visibly notable without further, in-depth inspection techniques. These may be an indication of the general deterioration and in some cases warrant further advanced inspection techniques (NDT, etc.) to ascertain the condition of the structure.

6.6.1.1 Invert Structures

Invert structures can consist of slab elements or invert girder and deck elements, as detailed in Section 2.4.1. Common defects of these structures depend on the material used to construct these systems as well as the use of an invert slab or invert girder system. These elements and defects are discussed in further detail in SNTI Section 3.2.

Invert slabs, typically using concrete construction, shall be inspected for delaminations, spalls and patched areas. Exposed rebar also needs to be documented along with efflorescence, rust staining and cracking, which may indicate shallow and corroding reinforcing steel. These invert slabs shall be inspected for defects on the bottom as well as the top of the slab surface. The top of the wearing surface shall be inspected in the instance when a wearing surface is impeding the visibility to the top side of the invert slab. These defects are discussed in further detailed in SNTI Section 3.2.

Invert girders may consist of concrete construction or a combination of steel and concrete construction. Invert girders may be made of either steel or concrete where the supporting deck is typically concrete, but may be steel.

- Common defects of steel elements, either girder or deck, are corrosion, cracking, connection deterioration and distortion.
- Concrete elements need to be inspected for delaminations, spalls and patched areas. Exposed rebar also needs to be documented along with efflorescence, rust staining and cracking, which may indicate shallow and corroding reinforcing steel.
- Prestressed concrete elements also need to be inspected for delaminations, spalls and patched areas. Exposed rebar or prestressing steel also needs to be documented along with efflorescence, rust staining and cracking, which may indicate shallow and corroding reinforcing steel or prestressing steel.
- Other invert girder types are discussed in SNTI Section 3.2.

6.6.1.2 Roof and Ceiling Elements

Roof and ceiling elements, discussed in more detail in Section 2.4.3 and 2.4.4 respectively, can consist of roof girders, ceiling slabs or ceiling girders and ceiling panels.

- Roof girders typically support the tunnel liner or exposed rock.

  A ceiling slab separates the upper plenum space in the tunnel from the main tunnel roadway and is used to convey exhaust and/or fresh air within the tunnel. Ceiling slabs can be comprised on concrete structural slabs, a slab with an architectural finish on the underside such as ceramic tiles or concrete filled metal pans, or steel composite metal pans. The ceiling slab is designed for loads from personnel accessing the plenum area, from ventilation pressures and special loads such as earthquakes.
- Ceiling panels separate the upper plenum from the space above the tunnel roadway. These systems consist of hangers and anchorages which in turn are attached to the tunnel roof /liner at the top and the ceiling girders/grid system at the bottom that support the inlaid ceiling panels (analogous to a “drop ceiling” arrangement in buildings).

Roof and ceiling girders are typically constructed from either concrete, steel or prestressed elements where ceiling structures are typically constructed from concrete. Common defects are similar between these two structural systems and shall be inspected for similar traits. Similar to invert structures, these elements shall be inspected for typical defects, dependent on the material type. Steel elements shall be inspected for corrosion, cracking, connection deterioration and distortions. Concrete or prestressed elements shall be inspected for delaminations, spalls and patched areas. Exposed rebar or prestressing steel also needs to be documented along with efflorescence, rust staining and cracking, which may indicate shallow and corroding reinforcing steel or prestressing steel. Other roof and ceiling girder types are discussed in SNTI Section 3.2.

Hanger and anchorage defects shall be inspected for deteriorations detailed above, along with bowing and elongation, creep and anchorage area deterioration.

6.6.1.3 Liner/Column/Pile Elements

Liner, column and pile elements, detailed in Section 2.4.2, Section 2.4.5 and Section 2.4.6 in this Guide, can be critical load carrying members and need to be inspected for deterioration and structural adequacy.

Liner defects are dependent on the type of material used to construct the tunnel liner. Common defects, detailed in SNTI Section 3.2, are listed below:

- Steel liners need to be inspected for corrosion, cracking, connection deterioration, distortion, and leakage.
- Concrete liners shall be inspected for delaminations, spalls and patched areas, exposed rebar, efflorescence/rust staining, cracking, distortion and leakage.
- Timber shall be inspected for decay or rot, voids, cracks/splits/checks, timber distortion, insect infestation, loose or missing connectors and leakage.

- Masonry liners shall be inspected for efflorescence/rust staining, mortar breakdown, split/spall, patched area, masonry displacement, distortion and leakage.
- Unlined rock shall be inspected for rockfall, patched areas and leakage.
- Rock bolt/dowel liners shall be inspected for loose bolt/dowel misalignment, deformation or cracking
- Other tunnel liners shall be inspected for cracking, distortion, patched areas and leakage.

Column and piles typically consist of steel or concrete construction. Similar to defects mentioned above, these elements shall be inspected for common defects relevant to the specific material type.

- Common defects of steel elements are corrosion, cracking, connection deterioration and distortion.
- Concrete elements need to be inspected for delaminations, spalls and patched areas. Exposed rebar also needs to be documented along with efflorescence, rust staining and cracking, which may indicate shallow and corroding reinforcing steel.
- Prestressed concrete elements also need to be inspected for delaminations, spalls and patched areas. Exposed rebar or prestressing steel also needs to be documented along with efflorescence, rust staining, and cracking, which may indicate shallow and corroding reinforcing steel or prestressing steel.

6.6.2 Influence of Deficiencies

Deficiencies and deteriorations may influence the structural behavior of a tunnel in multiple aspects and each aspect needs to be considered individually. Deficiencies in tunnel elements need to be considered in the load evaluation, structural evaluation, as well as member capacity calculations.

Member deterioration, including spalling and corrosion, may influence the structural dead loads acting on the system. Observed member geometric discrepancies from the original construction plans may also affect the structural loads impacting the structure and should therefore be accounted for in the load evaluation and structural modelling of the tunnel.

Measured material properties and section properties may influence structural behavior. These properties govern the load distribution of the structure resulting in a change in force effects from the original condition. Therefore, section loss, deformations, or effective geometry should be accounted for in the structural model. This may include a reduced section, common for steel and timber members, or an effective cracked section, common for concrete and masonry members. Additionally, reinforced concrete area and reinforced rebar area need to be evaluated when considering cracked concrete members.

Finally, member deteriorations and deficiencies are critical when evaluating the member capacities. Field measured member geometric properties should be used when evaluating the member capacities as well as reduced section properties due to corrosion and cracking. The capacity geometries typically coincide with the geometries used in the load evaluation and structural analysis. Altered material strength should also be considered when evaluating the capacity of critical members.

Members subjected to material loss or damage need to be accounted for in the geometric properties of said members. For steel members, MBE Article 6A.6.5 discusses the need for steel deterioration to be accounted for as part of the load rating analysis. For steel section loss, the location and dimension of the damage and loss of section needs to be documented accurately and then accounted for in the model to properly reflect the remaining section and to model any changes in the structure behavior and loss of capacity. Documentation of the deterioration should occur as part of the data collection and inspection activities discussed in Chapter 3 of this document. If insufficient data is provided in the latest inspection to properly account for a known deteriorated member, then a follow-up special inspection should be performed to obtain the needed data prior to finalizing the load rating analysis.

MBE Article C6A.6.5 provides discussion for how section loss due to corrosion on tension members, compression members, and flexural members shall be considered in the load rating. In some cases, an all-encompassing assumption of a certain percentage of section loss may not be sufficient, and localized effects of section loss may need to be considered.

6.7 POSTING OF TUNNELS

MBE Article 6A.8.1 provides guidance on posting. Posting of structures should not be confused with load rating. Inspection and load rating are engineering-related activities, whereas posting is a policy decision. If applicable legal loads exceed the calculated load capacity of the structure, the structure must be posted; however, the structure may be posted at a lower level. Structures not capable of carrying a minimum gross live load weight of three tons (3 T) must be closed.

6.7.1 Posting Loads and Regulatory Signs

MBE Article 6A.8.2 provides guidance on posting loads. But in general, when the maximum legal load under federal law or State statute exceeds the safe load capacity of a structure, restrictive load posting shall be required. Though there is variation among the States with respect to the type of signs preferred for posting structures, the posting signs used shall conform to the FHWA Manual on Uniform Traffic Control Devices (MUTCD) or other governing regulations, and shall be established in accordance with the requirements of the agency having authority over the highway (FHWA, 2003). Guidance on regulatory signage is provided in MBE Article 6A.8.4.

6.7.2 Posting Analysis

If the rating factor (RF) for any of the AASHTO or State Legal trucks should fall between 0.3 and 1.0, then a posting analysis as described in MBE Article 6A.8.3 shall be performed to establish the safe posting load for that vehicle. When the RF is less than 0.3, then that legal vehicle should not be allowed on the structure. When the RF is greater than 1.0, then that legal vehicle need not be restricted.

6.7.3 Speed Limits

Lower speed limits will reduce impact load to the extent that lowering the weight limit may not be required, but should only be considered in certain situations and after careful consideration. See MBE Article 6A.8.5.

6.8 OVERWEIGHT LOAD PERMITTING

Permit load ratings are generally performed if the tunnel and all of its components has a rating factor greater than 1.0 when evaluated for legal loads, since States normally restrict overweight vehicles from crossing load posted structures. Overweight permitting is governed by States' statute, regulation or operating policy and procedures. Permits are issued by States on a single trip, multiple trip, or annual basis. Refer to MBE Article 6A.4.5 for requirements and procedures for checking structures for overweight permitting.

6.8.1 Routine (Annual) Permits

Routine or annual permits are usually valid for unlimited trips over a period of time, not to exceed one year, for vehicles of a given configuration within specified gross and axle weight limits. Refer to MBE Article 6A.4.5.3.1.

6.8.2 Special (Limited Crossing) Permits

Special permits are usually valid for a single trip only, for a limited number of trips, or for a vehicle of specified configuration, axle weights, and gross weight. Special permit vehicles are usually heavier than those vehicles issued annual permits, and special restrictions may apply to reduce the load effects such as restricting other traffic, reduced speeds, and limiting vehicular travel path to a certain position on the structure. Refer to MBE Article 6A.4.5.3.2.

6.8.3 Live Load and Dynamic Load Allowance (IM)

The live load to be used in the evaluation for permit decisions shall be the actual permit truck or the vehicle producing the highest load effect in a class of permit vehicles operating under a single permit. Refer to MBE Article 6A.4.5.4.1 for additional requirements.

Impact load to be applied for permit load rating may be eliminated for slow moving vehicles traveling at ≤ 10 mph in accordance with MBE Article 6A.4.5.5.

6.8.4 Live Load Factors

The live load factors used for permit load rating may be based on MBE Table 6A.4.5.4.2a-1. Refer to MBE Article 6A.4.5.4.2a and Article 6A.4.5.4.2b.

6.9 OTHER LOAD RATING METHODS

MBE Chapter 6B contains a choice of either Allowable Stress Rating (ASR) or Load Factor Rating (LFR) methods at Inventory and Operating levels. These rating methods may be useful for comparison when rating older structures designed to past standards. For these rating methods, the AASHTO Standard Specifications for Highway Bridges shall be used for items not covered in this chapter or MBE Chapter 6B. Refer to MBE Article 6B.1.1.

6.9.1 Allowable Stress Rating (ASR) Method

The allowable or working stress rating method is described in MBE Article 6B.3.1. Load rating of buried structures, similar to box culverts, was not introduced into the MBE until 2013, which only included LRFR rating procedures. ASR rating procedures will require interpretation of the rating of a concrete member and the ASD culvert design specifications. It is generally considered that ASR is only acceptable for rating masonry or timber structures and is therefore not generally recommended for concrete or steel structures.

6.9.2 Load Factor Rating (LFR) Method

The load factor rating method is described in MBE Article 6B.3.2. Load rating of buried structures, similar to box culverts, was not introduced into the MBE until 2013, which only included LRFR rating procedures. LFR rating procedures will require interpretation of the rating of a concrete member and the LFD culvert design specifications.

6.9.3 Rating Levels

Each structure rated using ASR or LFR shall be load rated at two levels, Inventory and Operating levels. Inventory and Operating Rating Levels are described in MBE Article 6B.2.1 and Article 6B.2.2.

6.9.4 Rating Equation

Both the ASR and LFR methods use the same basic equation, MBE Equation 6B.4.1-1, which is shown below.

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

where:

RF	=	The rating factor for the live load carrying capacity. The rating factor multiplied by the rating vehicle in tons gives the rating of the structure.
C	=	The capacity of the member (see MBE Article 6B.5)
D	=	The dead load effect on the member (see MBE Article 6B.6.1). For composite members, the dead load effect on the non-composite section and the dead load effect on the composite section need to be evaluated when the Allowable Stress method is used
L	=	The live load effect on the member (see MBE Article 6B.6.2)
I	=	The impact factor to be used with the live load effect (see MBE Article 6B.6.4)
A_1	=	Factor for dead loads. For ASR, A_1 = 1.0. For LFR, A_1 = 1.3. See MBE Article 6B.4.2 and Article 6B.4.3.
A_2	=	Factor for live load. For ASR, A_2 = 1.0. For LFR A_2 = 2.17 for Inventory level and A_2 = 1.3 for Operating level. See MBE Article 6B.4.2 and Article 6B.4.3.

CHAPTER 7 - QUALITY ASSURANCE AND QUALITY CONTROL

7.1 INTRODUCTION

Tunnel inspection and load rating procedures are to be reviewed to ensure the work is adequate, complete, and consistent. This review process, referred to as Quality Assurance / Quality Control, requires defined roles for the person(s) performing the work as well as the person(s) reviewing the work.

Quality Assurance (QA) is defined by the MBE as "procedures used to verify the adequacy of the quality control procedures to meet or exceed the standards established by the program manager [or may be established by an organization or national guidance in some instances]." Quality Control (QC) is likewise defined by the MBE as "procedures to maintain the quality of the bridge inspections, bridge data, scour evaluations, and load ratings, and are usually continuously performed within the bridge inspection teams or units performing these functions."

The requirements of the QA/QC procedures are typically defined by the governing agency, or by the organization performing the tunnel inspection and load rating, and approved by the governing agency. This procedure is thoroughly discussed in MBE Section 1.4. The procedures outlined in the following sections are complimentary and parallel to the requirements set forth in the MBE.

7.2 ROLES AND RESPONSIBILITIES

The QC/QA process is a twofold procedure that must work in harmony with each other. The QC process typically requires some form of a checklist and/or detailed review process by a qualified person other than the originators to ensure uniformity and completeness of the work performed.

The QA procedures are typically submitted to the governing agency's program manager prior to the work being performed. The QA procedure occurs upon completion of the QC, as the QA is used to verify the adequacy of the QC. The QA shall be performed by a qualified individual who is independent of the tunnel inspection and load rating. The QA documentation is typically submitted to the governing agency's program manager after all work is complete.

This QC process may vary depending on the type and complexity of the work performed, as well as the requirements of the governing agency. This QC process is required for all work performed on the tunnel structure, including but not limited to inspection procedures, work safety, and load rating calculations. The organization performing this QC work typically submits the complete quality control procedures and all supporting documentation such as qualifications, QC plans and checklists to the governing agency's program manager prior to the work being performed. These QC procedures are conducted throughout the work performance and completed documentation is submitted to the governing agency's program manager after the work is complete.

7.3 QUALIFICATIONS

Each team member involved in the inspection and load rating of a tunnel must satisfy the qualifications as set forth or approved by the governing agency's program manager. These qualifications may vary depending on the type and complexity of the work being performed, and must therefore be coordinated with the governing agency's program manager. All qualification documentation typically must be submitted to the governing agency's program manager prior to work being performed.

7.4 DOCUMENTATION REQUIREMENTS

QC and QA documentation requirements may consist of a number of items for the work being performed and vary with each step in the process. Documentation requirements and the procedures are discussed in detail in the MBE Section 1.4.2 and are outlined here.

Typical QC documentation may include, but is not limited to (MBE 3rd. Ed. Section 1.4.2):

- Defined QC roles and responsibilities;
- Qualifications for the program manager, tunnel inspection personnel, and load rating personnel, including:
    - Education,
    - Certification or registration,
    - Training, and
    - Years and type of experience;
- Procedures for review and validation of inspection reports and data;
- Procedures for documenting important tunnel inspection information;
- Procedures for validation of load rating data; and
- Procedures for identification and resolution of data issues including errors, omissions, compatibility between items, changes, or any combination thereof.

Typical QA documentation may include but is not limited to (MBE 3rd. Ed. Section 1.4.2):

- Defined QA roles and responsibilities;
- Frequency parameters for review;
- Reasons and justifications for chosen analysis procedures; and
- Procedures and sampling parameters for selecting tunnels to conduct independent review and check of results, including:
    - Condition rating of elements or change in condition rating,
    - Load rating,
    - Posting status,
    - Deficiency status,
    - Critical findings and the status of any follow-up action, and
    - Location of tunnel.

7.5 LOAD RATING QC/QA CHECKLIST

A full checklist is provided in Appendix B. This checklist may be modified by an owner, in which case the modified checklist shall be used.

CHAPTER 8 - OTHER TOPICS

8.1 INTRODUCTION

The functional life of the tunnel structure and the safety of the traveling public relies on more than the live load demand versus the capacity of the live load carrying members. Most tunnel structures consist of non-live load carrying members, such as non-live load bearing liners, which are structural components and should have their structural adequacy verified as part of a load rating. Therefore, inspection and evaluation of these non-live load carrying members also needs to be considered when evaluating the life and safety of tunnel structures, or the overall safety of the tunnel structure.

Additionally, fatigue of steel members needs to be considered when evaluating the life and safety of a tunnel structure. Discussed herein are several types of fatigue including load-induced and distortion-induced fatigue. Fracture control is an integral part of fatigue analysis and this combined with older structures needs to be considered and evaluated. Finally, alternative analysis methods will be discussed in relation to fatigue life and conditions.

8.2 STRENGTH EVALUATION OF COMPONENTS UNDER NON-VEHICULAR LOADING

Most tunnels have non-live load carrying members that carry substantial dead and earth loads and should therefore be considered when evaluating the life and safety of a tunnel structure. Though these members do not carry live loads, their adequacy to carry their applied loads is critical to the well-being of the structure and is essential to ensure public safety. This section will discuss non-live load carrying members. These discussions will include the following:

1. Critical Non-Live Load Carrying Members and Connections: Why non-live load carrying members need to be evaluated and the impact this can have on a tunnel’s service life.

2. Determining When to Evaluate Non-Live Load Carrying Members and Connections: When does a load rater needs to consider non-live load carrying members.

3. Evaluation Procedures of Non-Live Load Carrying Members and Connections: How are non-live load carrying members evaluated and how to draw conclusions of critical members.

4. Strategies for Remediation of Critical Members and Connections: What procedures are followed when critical non-live load members are identified.

8.2.1 Critical Non-Live Load Carrying Members and Connections

Structural members and non-structural members of tunnel structures need to be inspected and evaluated when assessing the life and safety of tunnels. These members do not carry live load but are an integral part to the system adequacy of the structure. These members may include tunnel liners, conduit supports or fascia walls. These non-live load carrying members may still carry a significant level of stress and are therefore critical to the structural adequacy of the structural system.

The classification of these members fall into two types of members, structural and non-structural. Structural members are members that carry measurable load and are essential to the structural adequacy and stability of the tunnel. A non-structural member is a member that can be removed from the system and the structural integrity and stability of the tunnel will remain adequate and perform as designed, such as non-load bearing walls separating duct work from the travel ways.

Non-structural components also need to be fully assessed in addition to the structural elements and connections. Failure of any of these non-live load carrying structural elements and connections may result in a partial or complete failure of the system. Falling and deteriorated elements can cause hazards to vehicular traffic, pedestrian traffic, as well as maintenance workers. These elements may include architectural enhancements or partition walls, anchor bolts, and connections. Similarly, driving surfaces, safety walkways, and access paths should be evaluated for safety, structural adequacy, and deterioration to ensure a safe structure.

Connections often exhibit a critical element in the safety of a structure as they often transfer force effects from one element to another. Often, these connections may not transfer live load but still need to be inspected and evaluated for structural safety. Load path through the connection needs to be carefully considered.

In the case where non-live load carrying members or connections present a situation where safety of the structure is at risk, appropriate actions shall be taken and addressed as approved by the owner and the Engineer.

8.2.2 Determining When to Evaluate Non-Live Load Carrying Members and Connections

Structural and non-structural, non-live load elements need to be evaluated for strength, stability and serviceability once they have been identified as critical elements via structural analysis. The evaluation process is different if the critical element is structural or non-structural. After inspections, load rating Engineers will determine if strength evaluation of these members or connections are warranted due to condition changes. Structural elements will require strength, stability, and serviceability evaluations while non-structural elements will only need serviceability evaluation.

Structural and non-structural members needs to be inspected on a regular interval to measure changing conditions of the member and determine if further investigation is necessary. Additional inspections should be conducted when changed conditions are observed. These conditions may include, but are not limited to, the following:

- Extreme events such as vehicular crashes, fires or floods;
- Deterioration of the elements including falling concrete, spalls, delamination, corrosion or leaking;
- Change in stress condition such as:
    - Fill height alterations,
    - Change in internal or external load applications,
    - Adjacent structure alterations,

- Internal or external roadway overlays,
- System modifications, or
- Element deteriorations.

8.2.3 Evaluation Procedures of Non-Live Load Carrying Members and Connections

Strength evaluation of non-live load carrying members requires application of a strength evaluation, typically either Load and Resistance Factor Design (LRFD) or Load Factor Design (LFD), in compliance with appropriate AASHTO design specifications. This evaluation approach typically requires that the structural demand R_u is less than or equal to the capacity of the element, ϕR_n, where the system and condition factors need to be included as detailed in BDS Section 1.3.2 and MBE Section 6A.5.12.7 and 6A.5.12.8.

Stability evaluation of member buckling is typically embedded within the strength evaluation of that member. However, a global stability analysis may be required to ensure structural stability through an advanced analysis which may include an eigenvalue analysis or the Direct Analysis Method, for members such as structural wall panels supporting utility ducts or load bearing liners.

Serviceability requirements consider both stress evaluations as well as functionality requirements. Stress evaluations include ensuring service stresses are acting within the geometric and materialistic linear elastic behavior. This requires evaluation of the loads as well as the element capacities. This process may also include evaluation of the material properties to determine if they are within the parameters specified in the original construction plans. Functionality must also be considered to ensure that the element is sufficient to provide safety to the travelling public and maintenance personnel. This may include evaluation of the roadway surface, ADA requirements for pedestrian access, as well as verification that member deterioration conditions are stable including steel corrosion, leakage, concrete delamination, and concrete spalling.

8.2.4 Strategies for Remediation of Critical Members and Connections

Remediation measures are required when critical members are found to be inadequate, including but not limited to rehabilitation, preservation, or replacement. These decisions typically involve coordination with the Owner, contracting services, tunnel structure inspection team, and the tunnel structure rating team. This may vary from element strengthening, element replacement to structure replacement. In the event that the structure is deemed inadequate to provide traveling public safety, tunnel closure may be appropriate until the critical issues are resolved.

8.3 FATIGUE EVALUATION OF STEEL MEMBERS

Steel members subject to cyclic loads, primarily consisting of live load, need to be evaluated for fatigue to ensure the system is structurally adequate to carry the prescribed loads over the remaining structural life. Fatigue damage has been traditionally categorized into two types: Load-Induced and Distortion-Induced. As defined by MBE Section 7.1, these fatigue types are defined as:

- Load-Induced fatigue is due to the in-place stresses in the steel plates that comprise bridge member cross-sections. These in-place stresses are those typically calculated by the Engineer during bridge design or evaluation.

- Distortion-Induced fatigue is due to secondary stresses in the steel plates that comprise bridge member cross-sections. These stresses, which are typically caused by out-of-plane forces, can only be calculated with very refined methods of analysis, far beyond the scope of a typical bridge design or evaluation. These secondary stresses are minimized through proper detailing.

Each of these fatigue types are discussed in detail in the MBE 3rd Ed., Section 7. These provisions should be used when evaluating the fatigue condition of steel tunnel members that are subjected to a cyclic load such as live-load. Below is a summary of the steps and considerations required when evaluating steel members in a tunnel.

8.3.1 Load-Induced Fatigue Damage Evaluation

Load-induced fatigue is comprised of regular stress variations due to the cyclic application of a load, commonly the live load. This fatigue stress range is the differential stress between the maximum and minimum stress at a given critical location. The fatigue live load used for evaluation of steel elements in tunnels is defined in BDS Section 3.6.1.4. Fatigue is categorized into two levels, infinite life and finite life. Only members that fail the infinite life check are subject to more complex finite life evaluations. Critical, net area sections are to be evaluated for fatigue evaluations by applying the existing element condition including corrosion, deformations, and cracks. The evaluation procedure, detailed in MBE Section 7, is as follows:

- Estimating Stress Ranges;

- Determining Fatigue-Prone Details;

- Infinite-Life Check

- Estimating Finite Fatigue Life;

- Fatigue Serviceability Index; and

- Strategies to Increase Fatigue Serviceability Index.

8.3.2 Distortion-Induced Fatigue Damage Evaluation

Distortion-induced fatigue, as outlined in MBE Section 7.3, "is typically caused by the out-of-plane deformation of the web plate, which causes fatigue crack formation on details that are prone to such cracking under cyclic load."

Procedures and considerations for distortion-induced fatigue is discussed in more detail in MBE Section 7.3. This evaluation requires advanced modeling and evaluation, and it is recommended that this process be supervised by a qualified engineer.

8.3.3 Fracture-Control for Older Tunnel Steel Members

As noted in MBE 3rd, Edition, Section 7.4 for bridges, but applicable for tunnel steel members, "bridges fabricated prior to the adoption of *AASHTO's Guide Specifications for Fracture-Critical Nonredundant Steel Bridge Members* (1978) may have lower fracture toughness levels than are currently deemed acceptable. Destructive material testing of bridges fabricated prior to 1978 to ascertain actual toughness levels may be justified. Decisions on fatigue evaluations of a bridge can be made based upon the information from these tests."

8.3.4 Alternate Analysis Methods

As noted in MBE Section 7.5, alternative analysis techniques, such as fracture mechanics and hot-spot stress analysis, may be used to predict the finite fatigue life of a detail. The estimate for finite life obtained from these methods should be used in place of *Y* in MBE Section 7.2.6 to determine the fatigue serviceability index.

8.4 NON-DESTRUCTIVE LOAD TESTING FOR LOAD RATING

Existing tunnel structures frequently have limited data and/or show significant deterioration to which the rating is questionable, cannot be calculated (no existing plans and structure is showing distress) or is deficient. It often takes time and resources to replace these structures but economic reasons require these structures to remain in service. In such instances, it is permissible to perform a non-destructive load testing to provide or increase the existing load rating values.

Non-destructive load testing typically evaluates the live load rating factor but can extend to non-live load stresses under the right circumstances. However, non-live load stresses are typically hard or impossible to observe since these stresses are locked into place unless the conditions are changed. Live load rating typically consists of subjecting the monitored structure to a live load and observing the structural response. Load testing application involves instrumentation of the main live load members and/or deteriorated members or members of interest. Instrumentation is most commonly a strain measurement of the member in the main moment direction but may also involve rosette strain measurements (typically on gusset plates or girder webs) or accelerometers to measure dynamic response spectra.

Service level live loads are generally applied to the structure during the load testing to ensure the structure is within its elastic response range. Loading is configured to encompass the critical live load scenario for the main flexural members or the members under evaluation. The structure response measurements can be used to validate a specific load rating scenario or can be used to calibrate a Finite Element (FE) model which can then be used to extend the rating to encompass a wider range of loads. The calibrated FE model will also be used to analyze the load effects (that is, stresses, moments, and shears) due to other loads, such as dead loads and earth loads. MBE 3rd Edition, Section 8 provides general guidance for using nondestructive load testing (including diagnostic and proof load tests) for load rating.

CHAPTER 9 - LOAD RATING EXAMPLES

9.1 INTRODUCTION

The following examples show sample rating calculations for four different conditions including:

1. Reinforced concrete invert slab.
2. Steel invert girder.
3. Reinforced concrete box tunnel.
4. Reinforced concrete box with circular top.

These examples are fictitious examples assembled for demonstration purposes only. Actual tunnel rating calculations may require more evaluations than those shown depending on the type and conditions of the tunnel. For the purpose of space and complexity, liberties have been taken to omit certain aspects of an actual rating including:

- Checking reinforcement development lengths in accordance with BDS to ensure adequate development is obtained. If inadequate development is observed, the reinforcing area should be proportioned to the length of provided development to the length of required development within the development length. Beyond the development length, full measured reinforcing area may be used.

- Critical sections were evaluated at points of maximum and minimum shear and moment forces, as applicable to the limit state being evaluated. Additional critical sections may be deemed necessary based on structural condition and structure complexity.

- Fatigue was not performed for the steel invert girder. Fatigue checks shall be checked as specified by the MBE and BDS for all steel elements within the parameters for fatigue evaluation. For advanced fatigue evaluation, refer to MBE and BDS to all appropriate sections.

- Live load selections shall conform to MBE Section 6A.4 or the governing agency of the tunnel structure. For demonstration purposes, Example 9.1 includes Design Load HL-93, AASHTO Legal Loads and Emergency Vehicle loads. Example 9.2 and 9.3 include Design Load HL-93 and AASHTO Legal Loads. Example 9.4 only included Design Load HL-93.

The scope of these examples is to show the process required to perform a load rating of a live-load carrying member under various scenarios. Actual rating calculations shall consider all appropriate conditions including reinforcing development, section variations, deteriorations, corrosion, and section loss. The provided examples also use assumed soil parameters. A thorough evaluation of the soil conditions should be performed for actual load rating calculations.

9.2 EXAMPLE 1 – REINFORCED CONCRETE INVERT SLAB

Example 1 presents a circular tunnel (Figure 50) with a structural invert slab rigidly supported by walls. The fill depth above the tunnel is such that surface traffic will not impact the tunnel liner and is therefore not rated for this example, see Section 4.3.2.2 for further details. The invert slab and support walls are rated with the LRFR and LFR methods for the design vehicles (HL-93 and HS-20, respectively), the AASHTO SHVs (Notional Rating Load), and required emergency vehicles (EV2 and EV3) at the legal load rating level.

This example will perform the following steps to rate this reinforced concrete invert slab:

1. Structure data
2. Dead load, DC, calculations
3. Wearing surface, DW, calculations
4. Live load application
5. Structural analysis
6. Resistance calculations
7. LRFR rating calculations
8. LFR rating calculations

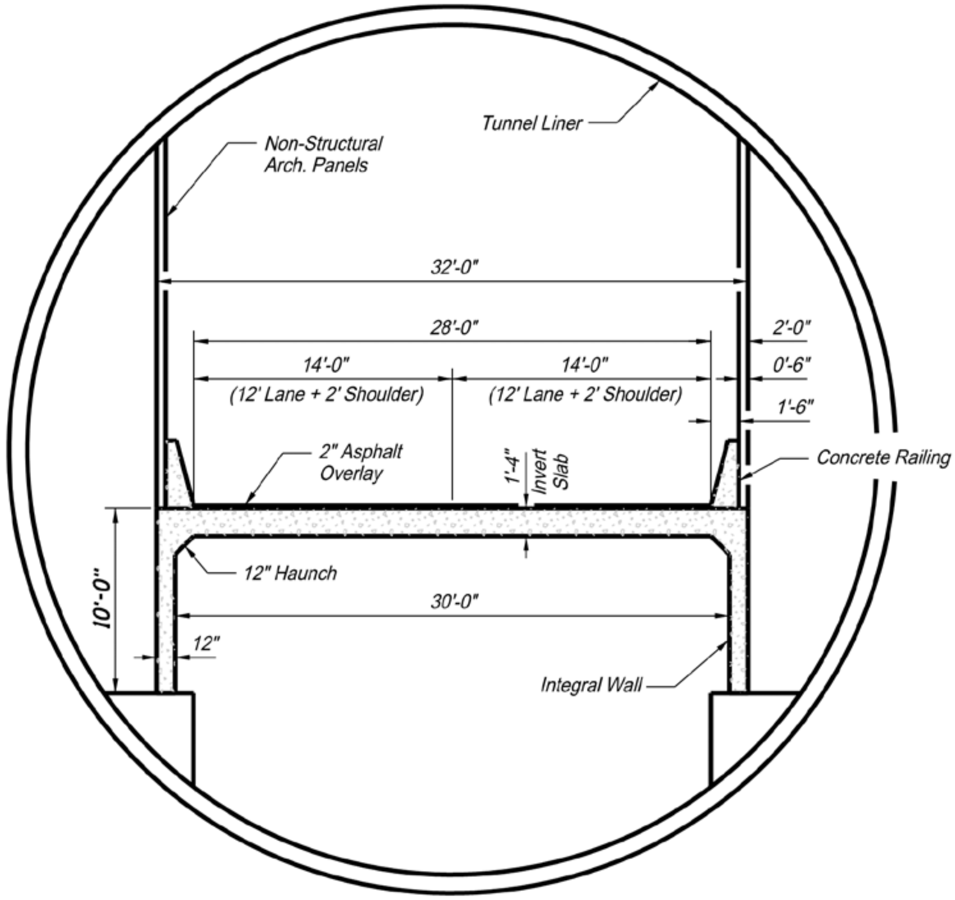
9.2.1 Structure Data

9.2.1.1 Materials

Materials are known, otherwise use MBE Article 6A5.2.1 and 6A5.2.2.

Concrete:	f'_c	=	4.0 ksi
Reinforcing Steel:	f_y	=	60.0 ksi
	E_s	=	29,000 ksi

9.2.1.2 Dimensions



Source: FHWA

Figure 50. Illustration. Cross-section showing invert slab and integral structure.

Top slab thickness:	t_{ts}	=	16 inches
Wall thickness:	t_{wall}	=	12 inches
Haunch size:	$\dim_{haunch}$	=	12 inches
Width out-to-out of slab:	w_{out}	=	32.00 ft
Width of roadway:	w_{rdwy}	=	28.00 ft
Height of wall:	h_{wall}	=	10.00 ft

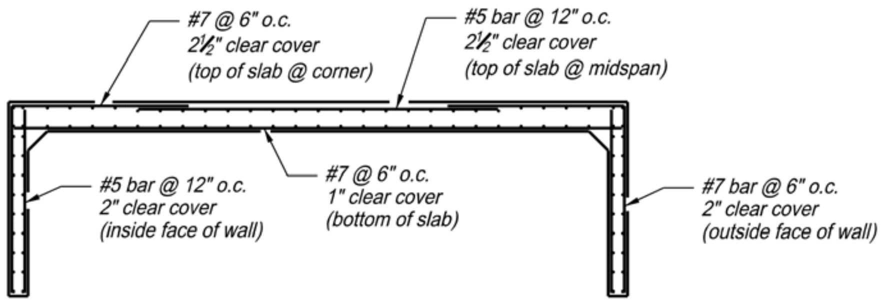
9.2.1.3 Example Notes:

- This cross-section is fictitious and only for the purpose of demonstrating the principles of rating invert slabs with integral walls.
- Live loading is only applicable to slab and support walls. No live load above tunnel within zone of influence; therefore, tunnel liner is not rated.
- If the upper walls were structural, they would need to be included in the structural analysis and rated as well.

- While not assumed here, any other mechanical (including dynamic equipment loads) and architectural systems supported by the invert slab, should be included in the dead load.
- Focus of example is LRFR. Abbreviated section is provided at the end of this example to discuss differences for LFR and Rating Factors based on an HS-20 Truck.
- This example focuses on an interior section of the invert slab. A section taken near the ends of the slab would need to account for increases in load effects due to reduced live load distribution near the edge.
- The rating Engineer should evaluate the existing record drawings to determine if the rebar lap lengths and development lengths meet current design code requirements. This example is based on the assumption that rebar lap lengths and development lengths meet current design code requirements (Figure 51).
- The rating Engineer should review the record drawings and inspection reports carefully to properly identify the support condition (pinned, expansion, fixed).
- Rating is performed only at maximum shear and moment locations. Typical ratings include evaluation at all tenth points and any other locations of interest such as section changes.

9.2.1.4 Rating Approach/Assumptions:

- LRFR evaluation is performed for a 1-foot wide strip (perpendicular to direction of traffic).
- 2 inch asphalt overlay was placed at time of construction, but was not field measured during inspections.
- LRFR live load ratings are evaluated for HL-93 design loading (Truck or Tandem) Notional Rating Load (NRL) and the EV3 emergency vehicle (FHWA Memo).
- Invert slab is considered a frame that transmits moment through the slab/wall corner.
- Support condition at bottom of frame is considered a pin type connection. For this example, the supports are not evaluated as part of the load rating.
- There are no signs of distress or deterioration; therefore, the invert slab is considered to be in satisfactory physical condition.
- There are no post/pre-tensioning forces; therefore, MBE Article 6A.2.2.2 does not apply.



Source: FHWA

Figure 51. Illustration. Reinforced concrete invert slab reinforcing steel details.

9.2.2 Dead Load, DC

Normal weight of reinforced concrete:

$$w_c = 0.145 \text{ kcf} + 0.005 \text{ kcf} = 0.150 \text{ kcf}$$ LRFD BDS Table 3.5.1-1 and Table C3.5.1

The following weights are calculated for the analysis strip width (Figure 52):

$$w_{strip} = 1.00 \text{ ft}$$

Top slab weight:

$$q_{top\_DC} = t_{ts} w_c w_{strip} = \left(16/12\right)(0.150)(1.00) = 0.200 \text{ klf}$$

Concrete barrier rail weight (one side only, assuming weight is 0.500 klf per side):

$$P_{rail\_DC} = w_{barrier} w_{strip} = 0.500(1.00) = 0.500 \text{ kip}$$

Non-structural architectural panel weight (one side only, assuming panels have thickness, t_{panel}, of 6 inches and height, h_{panel}, of 20 ft):

$$P_{ns\_wall} = t_{panel} h_{panel} w_{strip} w_c = \left(6/12\right)(20)(1.00)(0.150) = 1.500 \text{ kip}$$

Invert slab wall and haunch weight (one side only):

$$P_{wall} = t_{wall}(h_{wall} - t_{ls}) w_c w_{slip} = \left(16/12\right)\left(10 - 16/12\right)(0.150)(1.00) = 1.300 \text{ kip}$$

$$P_{fillet\_DC} = \frac{\dim_{haunch}^2}{2} w_c w_{strip} = \left(\frac{\left(12/12\right)^2}{2}\right)(0.150)(1.00) = 0.075 \text{ kip}$$

The haunch weight is assumed to be part of wall load in the structural model.

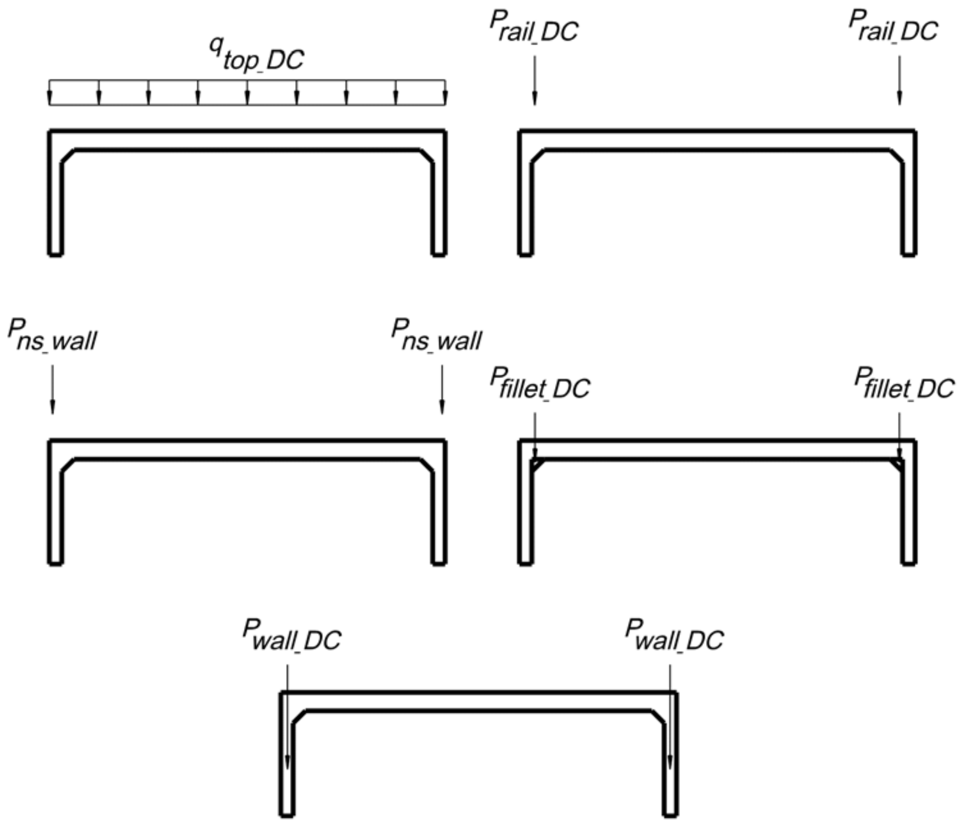


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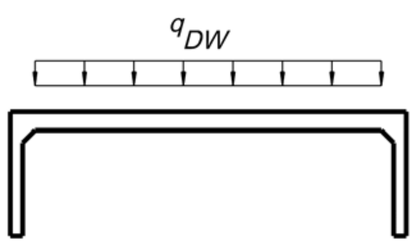
Figure 52. Illustration. Locations of applied dead loads.

9.2.3 Wearing Surface, DW

Bituminous wearing surface weight over roadway width with a density, w_{DW}, of 0.140 kcf in accordance with LRFD BDS Table 3.5.1-1 and an estimated thickness, t_{ws}, of 2 inches (Figure 53):

$$q_{DW} = t_{ws} w_{DW} w_{strip} = \left(2/12\right)(0.140)(1.00) = 0.023 \text{ klf}$$



Source: FHWA

Figure 53. Illustration. Location of applied wearing surface load.

9.2.4 Live Load Application

9.2.4.1 Live Load Distribution

The invert slab spans in the direction perpendicular (transverse) to the direction of traffic. Since this structure is similar to a box culvert type structure with less than 2 feet of fill, MBE Article 6A.5.12.10.3a and LRFD BDS Article 4.6.2.10.3 indicate that live load may be distributed to the top slab using the equations specified in LRFD BDS Article 4.6.2.1 for concrete decks with primary strips perpendicular to the direction of traffic. LRFD BDS Article 4.6.2.1.3 specifies the following:

- The equivalent strips for decks that span primarily in the transverse direction shall not be subject to width limits.
- *S* is the spacing of support components (ft):

$$S = w_{out} - 2\left(\frac{t_{wall}}{2}\right) = 32.00 - 2\left(\frac{(12/12)}{2}\right) = 31.00 \text{ ft}$$

- For positive moment, width of equivalent strip in inches:

$$w_{pos} = 26.0 + 6.6S$$ LRFD BDS Table 4.6.2.1.3-1

- For negative moment, width of equivalent strip in inches:

$$w_{neg} = 48.0 + 3.0S$$ LRFD BDS Table 4.6.2.1.3-1

Widths of equivalent interior strip for one axle are calculated as follows:

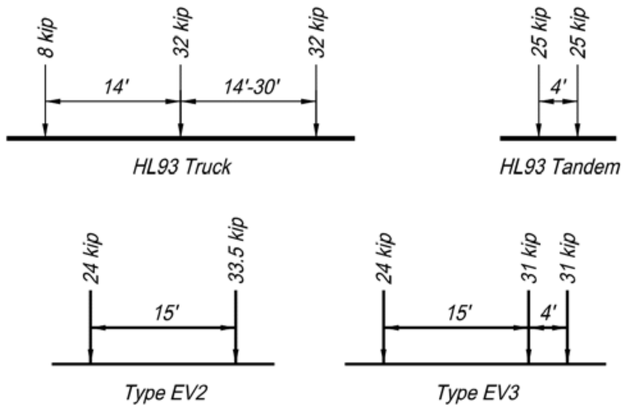
Width of equivalent interior strip in positive moment regions:

$$w_{pos} = 26.0 + 6.6(31.00) = 230.6 \text{ inches} = 19.22 \text{ ft}$$

Width of equivalent interior strip in negative moment regions:

$$w_{neg} = 48.0 + 3.0(31.00) = 141.0 \text{ inches} = 11.75 \text{ ft}$$

For this example, the HL-93 Design Truck/Tandem (MBE Article 6A.2.3), AASHTO NRL for Specialized Hauling Vehicles, and the EV3 emergency vehicle in accordance with the FHWA *Memorandum: Load Rating for the FAST Act's Emergency Vehicles* (FHWA, 2016) will be evaluated (Figure 54). The lane portion of the HL-93 design truck will not be applied in accordance with LRFD BDS Article 3.6.1.3.3. Axle configurations are shown below. *Note, the EV2 emergency vehicle was not rated in this example as EV3 will control by inspection as the EV3 has an additional axle that will be placed within the same span.* In this example, the EV3 vehicle will be evaluated with 1 lane and 2 lane situations utilizing the appropriate multiple presence factors. The FHWA Memorandum does allow the emergency vehicles to be rated for a single lane if necessary.



Source: FHWA

Figure 54. Illustration. Rating vehicles for example.

Axle loads for each vehicle can be distributed longitudinally over the equivalent interior strip width, but not transversely. For instances where the axle spacings are less than the equivalent interior strip widths calculated above, the strip widths need to be modified to account for the overlap of adjacent axles. The axle load(s) are then divided by the distribution width to give an equivalent load to be analyzed in the 1-foot wide transverse section used for structural analysis.

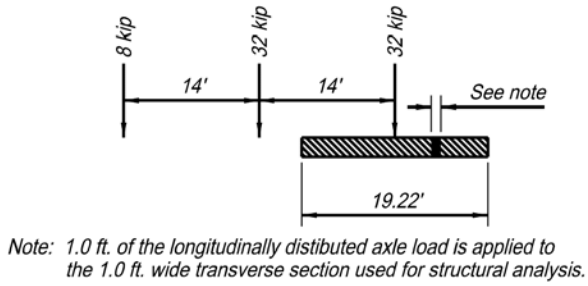
The equations from LRFD BDS Article 4.6.2.1.3, as utilized for this example, are an approximate method considered acceptable for this type of structure. However, a more refined structural model (MBE Article 6A.3.3) and analysis, such as a full 3D analysis considering the distribution of wheel loads using two-way slab action, may be used to determine model live loads if desired.

9.2.4.2 HL-93 Live Load

The positive moment strip width is calculated as 19.22 feet, from w_{pos} equation in Section 9.2.4.1, which is more than the controlling 14-foot axle spacing of the HL-93 truck and the 4-foot axle spacing of the tandem. The maximum equivalent load applied on a 1-foot-wide strip will be determined by checking the following combinations of axle(s) below. The note in Figure 55 stating that 1-foot of the calculated longitudinally distributed axle load is applied to the 1-foot-wide transverse section used for the structural analysis applies to live load calculations that follow.

HL-93 truck positive moment equivalent load on 1-foot-wide analysis strip:

For (1) 32 kip HL-93 truck axle (Figure 55):

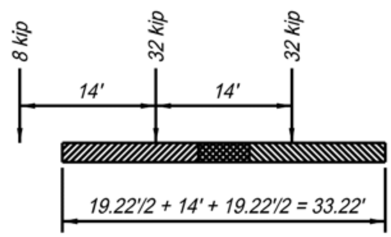


Source: FHWA

Figure 55. Illustration. HL-93 truck, (1) 32-kip axle over equivalent strip width.

$$Axle_{HL93\_trk1\_pos} = \left(\frac{32 \text{ kips}}{w_{pos}}\right) w_{strip} = \left(\frac{32}{19.22}\right) 1.00 = 1.67 \text{ kip}$$

For (2) 32 kip HL-93 truck (Figure 56):

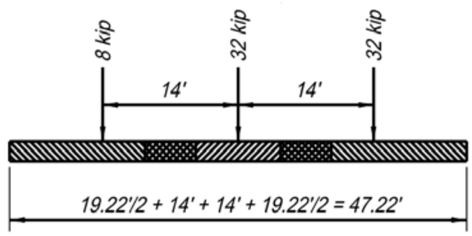


Source: FHWA

Figure 56. Illustration. HL-93 truck, (2) 32-kip axles over overlapping equivalent strip width.

$$\begin{aligned} Axle_{HL93\_trk2\_pos} &= \left(\frac{(32{+}32) \text{ kip}}{\left(\frac{w_{pos}}{2}\right) + 14 \text{ ft} + \left(\frac{w_{pos}}{2}\right)} \right) w_{strip} \\ &= \left(\frac{32+32}{\left(\frac{19.22}{2}\right) + 14 + \left(\frac{19.22}{2}\right)} \right) 1.00 \\ &= 1.93 \text{ kip} \end{aligned}$$

For (3) HL-93 truck axles (Figure 57):



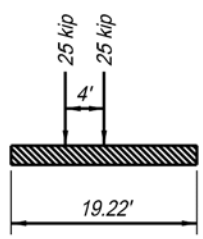
Source: FHWA

Figure 57. Illustration. HL-93 truck, (3) axles over overlapping equivalent strip width.

$$
\begin{aligned}
Axle_{HL93\_trk3\_pos} &= \left(\frac{(8{+}32{+}32)\text{ kip}}{\left(\frac{w_{pos}}{2}\right) + 14\text{ ft} + 14\text{ ft} + \left(\frac{w_{pos}}{2}\right)} \right) w_{strip} \\
&= \left(\frac{8+32+32}{\left(\frac{19.22}{2}\right) + 14 + 14 + \left(\frac{19.22}{2}\right)} \right) 1.00 \\
&= 1.52\text{ kip}
\end{aligned}
$$

HL-93 tandem positive moment equivalent load on 1-foot-wide analysis strip:

For (1) 25 kip HL-93 tandem axle (Figure 58):

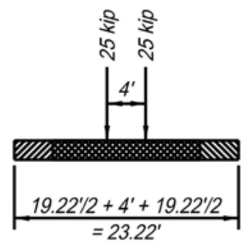


Source: FHWA

Figure 58. Illustration. HL-93 tandem, (1) 25-kip axle over equivalent strip width.

$$
Axle_{HL93\_tdm1\_pos} = \left(\frac{25\text{ kips}}{w_{pos}} \right) w_{strip} = \left(\frac{25}{19.22} \right) 1.00 = 1.30\text{ kip}
$$

For (2) 25 kip HL-93 tandem axles (Figure 59):



Source: FHWA

Figure 59. Illustration. HL-93 tandem, (2) 25-kip axles over equivalent strip width.

$$Axle_{HL93\_tdm2\_pos} = \left(\frac{(25+25) \text{ kip}}{\left(\frac{w_{pos}}{2}\right) + 4 \text{ ft} + \left(\frac{w_{pos}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{25+25}{\left(\frac{19.22}{2}\right) + 4 + \left(\frac{19.22}{2}\right)} \right) 1.00$$

$$= 2.15 \text{ kip}$$

The negative moment strip width is calculated as 11.75 feet, from w_{neg} equation in Section 9.2.4.1, which is less than the 14-foot axle spacing of the HL-93 truck, but not the tandem. The maximum equivalent load applied on a 1-foot-wide strip will be determined by checking the following combinations of axle(s):

HL-93 truck and tandem negative moment equivalent load on 1-foot-wide analysis strip:

For (1) 32 kip HL-93 truck axle:

$$Axle_{HL93\_trk\_neg} = \left(\frac{32 \text{ kips}}{w_{neg}} \right) w_{strip} = \left(\frac{32}{11.75} \right) 1.00 = 2.72 \text{ kip}$$

For (1) 25 kip HL-93 tandem axle:

$$Axle_{HL93\_tdm1\_neg} = \left(\frac{25 \text{ kips}}{w_{neg}} \right) w_{strip} = \left(\frac{25}{11.75} \right) 1.00 = 2.13 \text{ kip}$$

For (2) 25 kip HL-93 tandem axles:

$$Axle_{HL93\_tdm2\_neg} = \left(\frac{(25+25)\text{ kip}}{\left(\frac{w_{neg}}{2}\right) + 4\text{ ft} + \left(\frac{w_{neg}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{25+25}{\left(\frac{11.75}{2}\right) + 4 + \left(\frac{11.75}{2}\right)} \right) 1.00$$

$$= 3.17\text{ kip}$$

The maximum axle load for the HL-93 truck or tandem for positive and negative moment situations per 1-foot-wide strip are therefore:

Positive Moment:	$Axle_{HL93\_pos}$	=	2.15 kip
Negative Moment:	$Axle_{HL93\_neg}$	=	3.17 kip

9.2.4.3 Notional Rating Load (NRL) Live Load

The positive strip width is calculated as 19.22 feet, from w_{pos} equation in Section 9.2.4.1, which is more than the controlling NRL constant axle spacing of 4 feet between the rear axles and the 6 feet minimum variable spacing between the front and second axle. The maximum equivalent load applied on a 1-foot-wide strip will be determine by checking combinations of axles from 2 axles through all 8 axles.

For (2) 17 kip axles:

$$Axle_{NRL\_2\_pos} = \left(\frac{(17+17)\text{ kip}}{\left(\frac{w_{pos}}{2}\right) + 4\text{ ft} + \left(\frac{w_{pos}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{17+17}{\left(\frac{19.22}{2}\right) + 4 + \left(\frac{19.22}{2}\right)} \right) 1.00$$

$$= 1.46\text{ kip}$$

For (2) 17 kip axles and (1) 8 kip axle:

$$Axle_{NRL\_3\_pos} = \left(\frac{(8+17+17)\text{ kip}}{\left(\frac{w_{pos}}{2}\right)+4+4+\left(\frac{w_{pos}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{8+17+17}{\left(\frac{19.22}{2}\right)+4+4+\left(\frac{19.22}{2}\right)} \right) 1.00$$

$$= 1.54\text{ kip}$$

For (2) 17 kip axles and (2) 8 kip axles:

$$Axle_{NRL\_4\_pos} = \left(\frac{(8+17+17+8)\text{ kip}}{\left(\frac{w_{pos}}{2}\right)+4+4+4+\left(\frac{w_{pos}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{8+17+17+8}{\left(\frac{19.22}{2}\right)+4+4+4+\left(\frac{19.22}{2}\right)} \right) 1.00$$

$$= 1.60\text{ kip}$$

For (2) 17 kip axles and (3) 8 kip axles:

$$Axle_{NRL\_5\_pos} = \left(\frac{(8+17+17+8+8)\text{ kip}}{\left(\frac{w_{pos}}{2}\right)+4+4+4+4+\left(\frac{w_{pos}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{8+17+17+8+8}{\left(\frac{19.22}{2}\right)+4+4+4+4+\left(\frac{19.22}{2}\right)} \right) 1.00$$

$$= 1.65\text{ kip}$$

For (2) 17 kip axles and (4) 8 kip axles:

$$Axle_{NRL\_6\_pos} = \left(\frac{(8+8+17+17+8+8)\text{ kip}}{\left(\frac{w_{pos}}{2}\right)+4+4+4+4+4+\left(\frac{w_{pos}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{8+8+17+17+8+8}{\left(\frac{19.22}{2}\right)+4+4+4+4+4+\left(\frac{19.22}{2}\right)} \right) 1.00$$

$$= 1.68\text{ kip}$$

For (2) 17 kip axles and (5) 8 kip axles:

$$Axle_{NRL\_7\_pos} = \left(\frac{(8+8+17+17+8+8+8)\text{ kip}}{\left(\frac{w_{pos}}{2}\right)+4+4+4+4+4+4+\left(\frac{w_{pos}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{8+8+17+17+8+8+8}{\left(\frac{19.22}{2}\right)+4+4+4+4+4+4+\left(\frac{19.22}{2}\right)} \right) 1.00$$

$$= 1.71\text{ kip}$$

For (2) 17 kip axles, (5) 8 kip axles and (1) 6 kip axle:

$$Axle_{NRL\_8\_pos} = \left(\frac{(6+8+8+17+17+8+8+8)\text{ kip}}{\left(\frac{w_{pos}}{2}\right)+6+4+4+4+4+4+4+\left(\frac{w_{pos}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{6+8+8+17+17+8+8+8}{\left(\frac{19.22}{2}\right)+6+4+4+4+4+4+4+\left(\frac{19.22}{2}\right)} \right) 1.00$$

$$= 1.63\text{ kip}$$

A similar approach is used to calculate the negative moment since the negative moment strip width is calculated as 11.75 feet, from w_{neg} equation in Section 9.2.4.1, which is greater than the maximum 6 foot axle spacing.

For (2) 17 kip axles:

$$Axle_{NRL\_2\_neg} = \left(\frac{(17+17)\text{ kip}}{\left(\frac{w_{neg}}{2}\right) + 4\text{ ft} + \left(\frac{w_{neg}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{17+17}{\left(\frac{11.75}{2}\right) + 4 + \left(\frac{11.75}{2}\right)} \right) 1.00$$

$$= 2.16\text{ kip}$$

For (2) 17 kip axles and (1) 8 kip axle:

$$Axle_{NRL\_3\_neg} = \left(\frac{(8+17+17)\text{ kip}}{\left(\frac{w_{neg}}{2}\right) + 4 + 4 + \left(\frac{w_{neg}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{8+17+17}{\left(\frac{11.75}{2}\right) + 4 + 4 + \left(\frac{11.75}{2}\right)} \right) 1.00$$

$$= 2.13\text{ kip}$$

For (2) 17 kip axles and (2) 8 kip axles:

$$Axle_{NRL\_4\_neg} = \left(\frac{(8+17+17+8)\text{ kip}}{\left(\frac{w_{neg}}{2}\right) + 4 + 4 + 4 + \left(\frac{w_{neg}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{8+17+17+8}{\left(\frac{11.75}{2}\right) + 4 + 4 + 4 + \left(\frac{11.75}{2}\right)} \right) 1.00$$

$$= 2.11\text{ kip}$$

For (2) 17 kip axles and (3) 8 kip axles:

$$Axle_{NRL\_5\_neg} = \left(\frac{(8+17+17+8+8)\text{ kip}}{\left(\frac{w_{neg}}{2}\right)+4+4+4+4+\left(\frac{w_{neg}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{8+17+17+8+8}{\left(\frac{11.75}{2}\right)+4+4+4+4+\left(\frac{11.75}{2}\right)} \right) 1.00$$

$$= 2.09\text{ kip}$$

For (2) 17 kip axles and (4) 8 kip axles:

$$Axle_{NRL\_6\_neg} = \left(\frac{(8+8+17+17+8+8)\text{ kip}}{\left(\frac{w_{neg}}{2}\right)+4+4+4+4+4+\left(\frac{w_{neg}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{8+8+17+17+8+8}{\left(\frac{11.75}{2}\right)+4+4+4+4+4+\left(\frac{11.75}{2}\right)} \right) 1.00$$

$$= 2.08\text{ kip}$$

For (2) 17 kip axles and (5) 8 kip axles:

$$Axle_{NRL\_7\_neg} = \left(\frac{(8+8+17+17+8+8+8)\text{ kip}}{\left(\frac{w_{neg}}{2}\right)+4+4+4+4+4+4+\left(\frac{w_{neg}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{8+8+17+17+8+8+8}{\left(\frac{11.75}{2}\right)+4+4+4+4+4+4+\left(\frac{11.75}{2}\right)} \right) 1.00$$

$$= 2.07\text{ kip}$$

For (2) 17 kip axles, (5) 8 kip axles and (1) 6 kip axle:

$$Axle_{NRL\_8\_neg} = \left(\frac{(6+8+8+17+17+8+8+8)\text{ kip}}{\left(\frac{w_{neg}}{2}\right) + 6+4+4+4+4+4+4+4 + \left(\frac{w_{neg}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{6+8+8+17+17+8+8+8}{\left(\frac{11.75}{2}\right) + 6+4+4+4+4+4+4+4 + \left(\frac{11.75}{2}\right)} \right) 1.00$$

$$= 1.92\text{ kip}$$

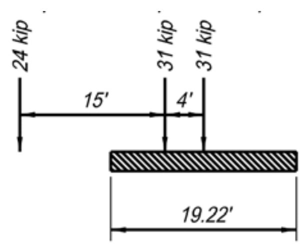
The largest positive and negative axle load is used as the point load acting on the unit width evaluation strip. From the analysis above, the maximum positive axle load is 1.71 kip per evaluation strip while the maximum negative axle load is 2.16 kip per evaluation strip.

9.2.4.4 EV3 Live Load

EV3 loads are calculated similar to HL-93. The same positive moment strip width of 19.22 feet, from w_{pos} equation in Section 9.2.4.1, which is more than the 4-foot or 15-foot axle spacing of the EV3 truck. The maximum equivalent load applied on a 1-foot-wide strip will be determined by checking the following combinations of axle(s):

EV3 truck positive moment equivalent load on 1-foot-wide analysis strip:

For (1) 31 kip EV3 truck axle (Figure 60):

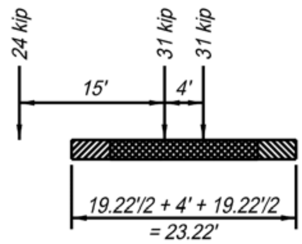


Source: FHWA

Figure 60. Illustration. EV3 truck, (1) 31-kip axle over equivalent strip width.

$$Axle_{EV3\_1\_pos} = \left(\frac{31\text{ kips}}{w_{pos}} \right) w_{strip} = \left(\frac{31}{19.22} \right) 1.00 = 1.61\text{ kip}$$

For (2) 31 kip EV3 truck axles (Figure 61):



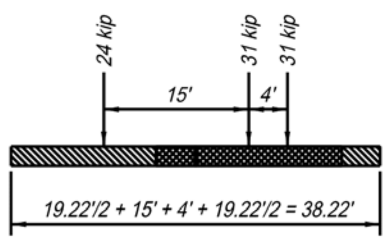
Source: FHWA

Figure 61. Illustration. EV3 truck, (2) 31-kip axles over equivalent strip width.

$$Axle_{EV3\_2\_pos} = \left(\frac{(31+31)\text{ kip}}{\left(\frac{w_{pos}}{2}\right) + 4\text{ ft} + \left(\frac{w_{pos}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{31+31}{\left(\frac{19.22}{2}\right) + 4 + \left(\frac{19.22}{2}\right)} \right) 1.00 = 2.67\text{ kip}$$

For (3) EV3 truck axles (Figure 62):



Source: FHWA

Figure 62. Illustration. EV3 truck, (3) axles over equivalent strip width.

$$Axle_{EV3\_3\_pos} = \left(\frac{(24+31+31)\text{ kip}}{\left(\frac{w_{pos}}{2}\right) + 4\text{ ft} + 15\text{ ft} + \left(\frac{w_{pos}}{2}\right)} \right) w_{strip}$$

$$= \left(\frac{24+31+31}{\left(\frac{19.22}{2}\right) + 4 + 15 + \left(\frac{19.22}{2}\right)} \right) 1.00 = 2.25\text{ kip}$$

The negative moment strip width is calculated as 11.75 feet, from w_{neg} equation in Section 9.2.4.1, which is less than the 15 foot axle spacing between the front and first rear axle, but not the 4 ft spacing of the rear axles. The maximum equivalent load applied on a 1-foot-wide strip will be determined by checking the following combinations of axle(s):

EV3 truck negative moment equivalent load on 1-foot-wide analysis strip:

For (1) 31 kip EV3 truck axle:

$$Axle_{EV3\_1\_neg} = \left(\frac{31\text{ kips}}{w_{neg}}\right) w_{strip} = \left(\frac{31}{11.75}\right) 1.00 = 2.64\text{ kip}$$

For (2) 31 kip EV3 truck axles:

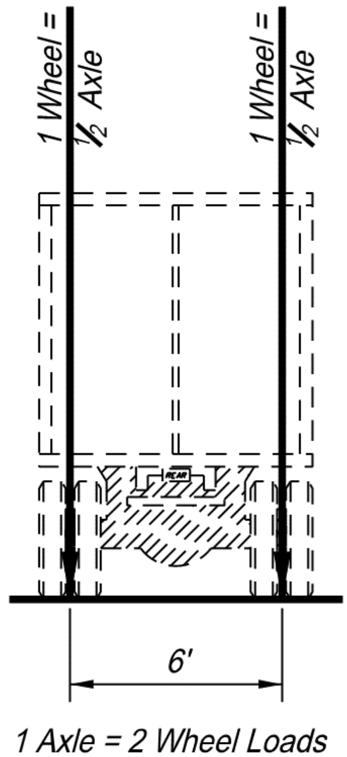
$$\begin{aligned} Axle_{EV3\_2\_neg} &= \left(\frac{(31+31)\text{ kip}}{\left(\frac{w_{neg}}{2}\right) + 4\text{ ft} + \left(\frac{w_{neg}}{2}\right)}\right) w_{strip} \\ &= \left(\frac{31+31}{\left(\frac{11.75}{2}\right) + 4 + \left(\frac{11.75}{2}\right)}\right) 1.00 \\ &= 3.94\text{ kip} \end{aligned}$$

The maximum axle load for the EV3 truck for positive and negative moment situations per 1-foot-wide strip are therefore:

Positive Moment:	$Axle_{EV3\_pos}$ =	2.67 kip
Negative Moment:	$Axle_{EV3\_neg}$ =	3.94 kip

9.2.4.5 Wheel Loads

Now that the live load applied to a 1-foot-wide strip has been determined; the axle load can simply be divided into each of the two wheels for the wheel point loads applied to the analysis strip (Figure 63):



Source: FHWA

Figure 63. Illustration. Standard truck, axle load vs. wheel load.

(1) wheel load for HL-93 on evaluation strip for positive moment:

$$P_{HL93\_pos} = \frac{Axle_{HL93\_pos}}{2 \text{ wheels}} = \frac{2.15}{2} = 1.08 \text{ kip}$$

(1) wheel load for HL-93 on evaluation strip for negative moment:

$$P_{HL93\_neg} = \frac{Axle_{HL93\_neg}}{2 \text{ wheels}} = \frac{3.17}{2} = 1.59 \text{ kip}$$

(1) wheel load for NRL on evaluation strip for positive moment:

$$P_{NRL\_pos} = \frac{Axle_{NRL\_pos}}{2 \text{ wheels}} = \frac{1.71}{2} = 0.86 \text{ kip}$$

(1) wheel load for NRL on evaluation strip for negative moment:

$$P_{NRL\_neg} = \frac{Axle_{NRL\_neg}}{2 \text{ wheels}} = \frac{2.16}{2} = 1.08 \text{ kip}$$

(1) wheel load for EV3 on evaluation strip for positive moment:

$$P_{EV3\_pos} = \frac{Axle_{EV3\_pos}}{2 \text{ wheels}} = \frac{2.67}{2} = 1.34 \text{ kip}$$

(1) wheel load for EV3 on evaluation strip for negative moment:

$$P_{EV3\_neg} = \frac{Axle_{EV3\_neg}}{2 \text{ wheels}} = \frac{3.94}{2} = 1.97 \text{ kip}$$

9.2.4.6 Impact

The live load impact factor is in accordance with MBE Article 6A.4.4.3 and LRFD BDS Table 3.6.2.1-1:

$$IM = 33\%$$

9.2.5 Structural Analysis

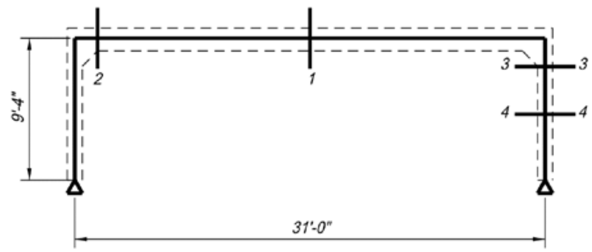
A frame analysis procedure is applied for the invert slab model with beam-column elements based on gross section properties. The widths of the slab and sidewalls are considered as a unit length of 1 foot for the two-dimensional representation of the continuous structure. All the load effects and member resistances are calculated using this 1-foot-wide strip representation. Structural analysis of the invert slab is based on a frame model with moment resisting connections between the slab and the wall joints. The frame is assumed to have a pinned connection at the bottom of each wall, and free to side sway at the slab to wall joints. A P-Delta analysis was used to capture secondary effects from eccentric loading. Although not shown in this example, consideration for slenderness in the walls may be required in accordance with LRFD BDS Article 5.6.4.3.

In accordance with the reinforcing steel details in Figure 51, bars are continuous between the slab and sidewalls; therefore, moment transfer across the joint is assumed.

Wheel loads for the HL-93, NRL and EV3 were placed within the lanes to produce the extreme force effects in accordance with LRFD BDS Article 3.6.1.3. Both 1 lane loaded (Multiple Presence Factor (MPF = 1.2) and 2 lanes loaded (MPF = 1.0) were considered in accordance with LRFD BDS Article 3.6.1.1.2. If a single EV3 were to control, a MPF=1.0 would be used. However, if a single NRL would to control, a MPF=1.2 would be used.

The critical failure modes for the invert slab model are due to flexure, shear, and axial forces. The shear demand is checked at the critical shear section located at a distance d_v, away from the sidewall surface (LRFD BDS Article 5.7.2.8, Article 5.7.3.2, and Article 5.12.8.6). The negative moment demand is checked at the intersection of the haunch and the uniform depth slab (LRFD BDS Article 12.11.5.2 and Article 12.14.5.5) and for positive moment at the mid-span of the slab. For four critical sections (see Figure 64), the unfactored ultimate section forces are listed in the following. The factored loads are determined by applying the maximum and minimum load factors to construct an envelope of ultimate force occurrences. Axial thrust was observed to be negligible compared to the magnitude of the moment and therefore it was conservative to neglect the axial effects.

Table 13 through Table 15 presents the analysis results for unfactored moment, shear, and axial thrust for the HL-93, NRL and EV3 live loads. The unfactored live load force effects incorporate the multiple presence factors (MPF) and the dynamic load allowance (IM = 33 percent). For sections 2, 3 and 4, negative moment (flexural tension at top of slab and outside face of walls) controls where positive moment controls section 1. It should also be noted that the moment is evaluated at the wall to slab interface while shear is evaluated at dv from the face of the slab to wall interface, applied to sections 2 and 3.



Source: FHWA

Figure 64. Illustration. Critical sections evaluated for invert slab.

Table 13. Analysis results for moment (kip-inch) at critical sections.

Section	**1**	**2**	**3**	**4**
Description	Top Slab Center	Top Slab Corner	Sidewall Top	Sidewall Center
DC	174.3	61.0	93.7	53.0
DW	20.2	7.2	10.8	6.1
LL+IM (HL-93)	225.9	124.3	164.6	93.1
LL+IM (NRL)	179.9	84.4	111.8	63.2
LL+IM (EV3)	280.3	153.9	204.0	115.3

Table 14. Analysis results for shear (kip) at critical sections.

Section	2	3
Description	Top Slab Corner	Sidewall Top
DC	2.8	1.0
DW	0.3	0.1
LL+IM (HL-93)	4.8	1.8
LL+IM (NRL)	3.2	1.2
LL+IM (EV3)	5.9	2.2

Table 15. Analysis results for thrust (kip) at critical sections.

Section	3	4
Description	Sidewall Top	Sidewall Center
DC	5.4	5.9
DW	0.3	0.3
LL+IM (HL-93)	4.8	4.8
LL+IM (NRL)	3.2	3.2
LL+IM (EV3)	5.9	5.9

9.2.6 Resistance Calculations

9.2.6.1 Nominal Flexural Resistance

The depth, analysis section width, and reinforcement for each of the critical sections are tabulated below in Table 16.

Table 16. Critical section data.

Section	Depth (inch)	Width (inch)	Bar Number	Bar Spacing (inch)	Cover (inch)
1 - Top	16.000	12.000	5	12.000	2.500
1 - Bottom	16.000	12.000	7	6.000	1.000
2 - Top	16.000	12.000	7	6.000	2.500
2 - Bottom	16.000	12.000	7	6.000	1.000
3 - Outside	12.000	12.000	7	6.000	2.000
3 - Inside	12.000	12.000	5	12.000	2.000
4 - Outside	12.000	12.000	7	6.000	2.000
4 - Inside	12.000	12.000	5	12.000	2.000

Concrete Properties:

f'_c = 4.0 ksi	
$\propto_1$ = 0.85	LRFD BDS 5.6.2.2
β_1 = 0.85 if $f'_c \leq 4.0$ ksi	LRFD BDS 5.6.2.2
λ = 1.0 (normal weight concrete)	LRFD BDS 5.4.2.8
γ_3 = 0.67	LRFD BDS 5.6.3.3
γ_1 = 1.6	LRFD BDS 5.6.3.3

Modulus of rupture:

$$f_r = 0.24\lambda\sqrt{f'_c} = 0.24(1.0)\sqrt{4.0} = 0.480 \text{ ksi}$$ LRFD BDS 5.4.2.6

Compression reinforcement in flexural capacity calculations is conservatively ignored.
Calculated results are based on a per foot analysis width.

9.2.6.1.1 Section 1 – Top Slab Center (Positive Moment)

Rectangular section height:	h	=	16.00 inches
Rectangular section width:	b	=	12.00 inches
Clear distance to rebar from tension face:	clr	=	1.00 inches
Area of single rebar:	A_{s_bar}	=	0.60 inch^2
Diameter of rebar:	dia_{bar}	=	0.875 inches
Spacing of rebar:	s	=	6 inches

Determine equivalent area of reinforcing bar:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.60(12.00)}{6} = 1.20 \text{ inch}^2$$

Determine distance from the extreme compression fiber to the centroid of the reinforcement:

$$d_s = h - \frac{dia_{bar}}{2} - clr = 16.00 - \frac{0.875}{2} - 1.00 = 14.563 \text{ inches}$$

Determine distance from the extreme compression fiber to the neutral axis for non-prestressed tension reinforcement only:

$$c = \frac{A_s f_y}{\alpha_1 \beta_1 f'_c b} = \frac{1.20(60)}{0.85(0.85)(4.0)(12.00)} = 2.08 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4

Determine the equivalent stress block:

$$a = \beta_1 c = 0.85(2.08) = 1.76 \text{ inches}$$ LRFD BDS Eq. 5.6.2.2

Calculate nominal moment resistance:

$$M_{n\_bott1} = A_s f_y\left(d_s - \frac{a}{2}\right) = 1.20(60)\left(14.563 - \frac{1.76}{2}\right) \qquad \text{LRFD BDS Eq. 5.6.3.2.2-1}$$
$$= 985.0 \text{ kip-inch}$$

Note that the nominal moment resistance based on top reinforcement is not critical at mid-span and need not be checked.

Determine resistance factor:

$$\varepsilon_{cl} = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.0021 \le 0.002 \;\therefore\; \varepsilon_{cl} = 0.002 \qquad \text{LRFD BDS 5.6.2.1}$$

$$\varepsilon_{tl} = 0.005 \qquad \text{LRFD BDS 5.6.2.1}$$

The net tensile strain is determined using similar triangles based on a strain limit of 0.003 in the extreme compression fiber, see LRFD BDS Article C5.6.2.1.

$$\varepsilon_t = \frac{0.003}{c}(d_s - c) = \frac{0.003}{2.08}(14.563 - 2.08) = 0.018$$

Since $\varepsilon_t \geq \varepsilon_{tl}$, the section is tension controlled and 0.90 is used for the resistance factor in accordance with LRFD BDS Article 5.5.4.2. If $\varepsilon_t \leq \varepsilon_{cl}$, it would be compression controlled and the resistance factor would be 0.75 in accordance with LRFD BDS Article 5.5.4.2 and MBE Article 6A.5.5. If ε_t was between ε_{cl} and ε_{tl}, then LRFD BDS Eq. 5.5.4.2-2 would be used to interpolate a resistance factor.

$$\phi_f = 0.90 \qquad \text{LRFD BDS 5.5.4.2}$$

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(16.00)^2}{6} = 512.00 \text{ inch}^3 \qquad \text{LRFD BDS 5.6.3.3}$$

The cracking moment equation simplifies to the following (only monolithic sections and no prestress forces):

$$M_{cr} = \gamma_3\gamma_1 f_r S_c = 0.67(1.6)(0.480)(512.0) = 263.5 \text{ kip-in} \qquad \text{LRFD BDS Eq. 5.6.3.3-1}$$

$$\phi_f M_{n\_bott1} = 0.90(985.0) = 886.5 \text{ kip-inch}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

9.2.6.1.2 Section 2 – Top Slab Corner (Negative Moment)

Rectangular section height:	h	=	16.00 inches
Rectangular section width:	b	=	12.00 inches
Clear distance to rebar from tension face:	clr	=	2.50 inches
Area of single rebar:	A_{s_bar}	=	0.60 inch^2

Diameter of rebar: dia_{bar} = 0.875 inches
Spacing of rebar: s = 6 inches

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.60(12.00)}{6} = 1.20 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 16.00 - \frac{0.875}{2} - 2.50 = 13.062 \text{ inches}$$

$$c = \frac{A_s f_y}{\alpha_1 \beta_1 f'_c b} = \frac{1.20(60)}{0.85(0.85)(4.0)(12.00)} = 2.08 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4

$$a = \beta_1 c = 0.85(2.08) = 1.76 \text{ inches}$$ LRFD BDS Eq. 5.6.2.2

Calculate nominal moment resistance (the resistance for tension on the bottom face is the same as calculated in Section 9.2.6.1.1):

$$M_{n\_top2} = A_s f_y \left(d_s - \frac{a}{2} \right) = 1.20(60)\left(13.062 - \frac{1.76}{2} \right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 877.0 \text{ kip-in}$$

$$M_{n\_bott2} = M_{n\_bott1} = 985.0 \text{ kip-inch}$$

Note that the nominal moment resistance based on bottom reinforcement is not critical at the corner. The nominal resistance is shown for example purposes, but will not be evaluated for rating factor results.

Determine resistance factor, where ε_{cl} (0.002) and ε_{tl} (0.005) have been determine previously in Section 9.2.6.1.1:

$$\varepsilon_t = \frac{0.003}{c}(d_s - c) = \frac{0.003}{2.08}(13.062 - 2.08) = 0.016 \geq \varepsilon_{tl} \therefore$$

$$\phi_f = 0.90$$ LRFD BDS 5.5.4.2

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(16.00)^2}{6} = 512.00 \text{ inch}^3$$ LRFD BDS 5.6.3.3

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(512.0) = 263.5 \text{ kip-inch}$$ LRFD BDS Eq. 5.6.3.3-1

$$\phi_f M_{n\_bott2} = 0.90(985.0) = 886.5 \text{ kip-inch}$$

$$\phi_f M_{n\_top2} = 0.90(877.0) = 789.3 \text{ kip-inch}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

9.2.6.1.3 Sections 3 and 4 – Sidewall Interior at Top and Center

Since the sidewall is symmetric, the negative and positive moment capacities are identical.

Rectangular section height:	h	=	12.00 inches
Rectangular section width:	b	=	12.00 inches
Clear distance to rebar from tension face:	clr	=	2.00 inches
Area of single rebar:	A_{s_bar}	=	0.31 inch^2
Diameter of rebar:	dia_{bar}	=	0.625 inches
Spacing of rebar:	s	=	12 inches

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.31(12.00)}{12} = 0.31 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 12.00 - \frac{0.625}{2} - 2.00 = 9.687 \text{ inches}$$

$$c = \frac{A_s f_y}{\alpha_1 \beta_1 f'_c b} = \frac{0.31(60)}{0.85(0.85)(4.0)(12.00)} = 0.54 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4

$$a = \beta_1 c = 0.85(0.54) = 0.46 \text{ inches}$$ LRFD BDS Eq. 5.6.2.2

Calculate nominal moment resistance:

$$M_{n\_wall\_\text{int}} = A_s f_y \left(d_s - \frac{a}{2} \right) = 0.31(60)\left(9.687 - \frac{0.46}{2} \right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 175.9 \text{ kip-inch}$$

$$M_{n\_wall4\_\text{int}} = M_{n\_wall3\_\text{int}} = 175.9 \text{ kip-inch}$$

Note, the nominal moment resistance for tension on the interior of the wall is not critical based on loadings and lack of moment causing tension in the interior face of the walls. Nominal resistance is shown for example purposes and for showing that minimum reinforcement requirements are met, but will not be evaluated for rating factor results.

Determine resistance factor, where ε_{cl} (0.002) and ε_{tl} (0.005) have been determine previously in Section 9.2.6.1.1:

$$\varepsilon_t = \frac{0.003}{c}(d_s - c) = \frac{0.003}{0.54}(9.687 - 0.54) = 0.051 \geq \varepsilon_{tl} \therefore$$

$$\phi_f = 0.90$$ LRFD BDS 5.5.4.2

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(12.00)^2}{6} = 288.00 \text{ inch}^3$$ LRFD BDS 5.6.3.3

$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(288.0) = 148.2$ kip-inch LRFD BDS Eq. 5.6.3.3-1

$$\phi_f M_{n\_wall3\_int} = 0.90(175.9) = 158.4 \text{ kip-inch}$$

$$\phi_f M_{n\_wall4\_int} = 0.90(175.9) = 158.4 \text{ kip-inch}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

9.2.6.1.4 Sections 3 and 4 – Sidewall Exterior at Top and Center

Rectangular section height:	h	=	12.00 inches
Rectangular section width:	b	=	12.00 inches
Clear distance to rebar from tension face:	clr	=	2.00 inches
Area of single rebar:	A_{s_bar}	=	0.60 inch^2
Diameter of rebar:	dia_{bar}	=	0.875 inches
Spacing of rebar:	s	=	6 inches

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.60(12.00)}{6} = 1.20 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 12.00 - \frac{0.875}{2} - 2.00 = 9.562 \text{ inches}$$

$$c = \frac{A_s f_y}{\alpha_1 \beta_1 f'_c b} = \frac{1.20(60)}{0.85(0.85)(4.0)(12.00)} = 2.08 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4

$$a = \beta_1 c = 0.85(2.08) = 1.76 \text{ inches}$$ LRFD BDS Eq. 5.6.2.2

Calculate nominal moment resistance:

$$M_{n\_wall3\_ext} = A_s f_y \left(d_s - \frac{a}{2} \right) = 1.20(60)\left(9.562 - \frac{1.76}{2} \right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 625.0 \text{ kip-inch}$$

$$M_{n\_wall4\_ext} = M_{n\_wall3\_ext} = 625.0 \text{ kip-inch}$$

Determine resistance factor, where ε_{cl} (0.002) and ε_{tl} (0.005) have been determine previously in Section 9.2.6.1.1:

$$\varepsilon_t = \frac{0.003}{c}(d_s - c) = \frac{0.003}{2.08}(9.562 - 2.08) = 0.011 \geq \varepsilon_{tl} \therefore$$

$$\phi_f = 0.90$$ LRFD BDS 5.5.4.2

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(12.00)^2}{6} = 288.00 \text{ inch}^3$$ LRFD BDS 5.6.3.3

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(288.0) = 148.2 \text{ kip-inch}$$ LRFD BDS Eq. 5.6.3.3-1

$$\phi_f M_{n\_wall3\_ext} = 0.90(625.0) = 562.5 \text{ kip-inch}$$

$$\phi_f M_{n\_wall4\_ext} = 0.90(625.0) = 562.5 \text{ kip-inch}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

9.2.6.2 Nominal Shear Resistance

Shear resistance will be determined by using the sectional design model in accordance with LRFD BDS Article 5.7.3.

9.2.6.2.1 Section 1 – Top Slab Corner

Rectangular section height:	h	=	16.00 inches
Rectangular section width:	b	=	12.00 inches
Clear distance to rebar from tension face:	clr	=	2.50 inches
Area of single rebar:	A_{s_bar}	=	0.60 inch^2
Diameter of rebar:	dia_{bar}	=	0.875 inches
Spacing of rebar:	s	=	6 inches

LRFD BDS Article 5.7.2.3 indicates a minimum amount of transverse reinforcement is not needed in slabs, footings, or culvert type sections. For this example, there is no shear reinforcement included in the slab, therefore:

$$A_v = 0.00 \text{ inch}^2$$

Determine equivalent area of flexural reinforcing bar per strip width:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.60(12.00)}{6} = 1.20 \text{ inch}^2$$

Determine distance from the extreme compression fiber to the centroid of the reinforcement:

$$d_e = h - \frac{dia_{bar}}{2} - clr = 16.00 - \frac{0.875}{2} - 2.50 = 13.062 \text{ inches}$$

For the top slab section near the corner, the reaction forces in the direction of the applied shear will introduce compression into the region, so the critical section for shear can be taken at a distance, d_v, from the face of the wall in accordance with LRFD BDS Article 5.7.3.2:

$$d_v = \max(0.72h,\ 0.9d_e) = \max\left[0.72(16.00),\ 0.9(13.062)\right]$$
$$= 11.76 \text{ inches}$$ LRFD BDS 5.7.2.8

As there is no required transverse reinforcement and no applied axial tension force, β can be assumed to be 2.0:

$$\beta = 2.0$$ LRFD BDS 5.7.3.4.1

There is no prestress force applied, therefore:

$$V_p = 0 \text{ kip}$$

Calculate the shear resistance of the section:

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}bd_v$$
$$= 0.0316(2.0)(1.0)\sqrt{4.0}(12.00)(11.76) = 17.8 \text{ kip}$$ LRFD BDS Eq. 5.7.3.3-3

$$V_s = 0 \text{ kip}$$ LRFD BDS Eq. 5.7.3.3-4

$$V_{n1} = 0.25f'_c bd_v + V_p$$
$$= 0.25(4.0)(12.00)(11.76) + 0 = 141.1 \text{ kip}$$ LRFD BDS Eq. 5.7.3.3-2

$$V_{n2} = V_c + V_s + V_p = 17.8 + 0 + 0 = 17.8 \text{ kip}$$ LRFD BDS Eq. 5.7.3.3-1

$$V_{n\_sec2} = \min(V_{n1}, V_{n2}) = \min(141.1, 17.8) = 17.8 \text{ kip}$$ LRFD BDS 5.7.3.3

9.2.6.2.2 Section 3 – Sidewall Top

Rectangular section height:	*h*	=	12.00 inches
Rectangular section width:	*b*	=	12.00 inches
Clear distance to rebar from tension face:	*clr*	=	2.00 inches
Area of single rebar:	A_{s_bar}	=	0.60 inch^2
Diameter of rebar:	dia_{bar}	=	0.875 inches
Spacing of rebar:	*s*	=	6 inches

$$A_v = 0.00 \text{ inch}^2$$

$$A_s = \frac{A_{s\_bar}b}{s} = \frac{0.60(12.00)}{6} = 1.20 \text{ inch}^2$$

$$d_e = h - \frac{dia_{bar}}{2} - clr = 12.00 - \frac{0.875}{2} - 2.00 = 9.562 \text{ inches}$$

The section at the bottom of the haunch is assumed to control for this example as this section has the largest shear force below the haunch and the haunch region immediately above has more resistance due to the thickened section; however, all sections cases must be checked for a complete rating analysis.

$$d_v = \max(0.72h,\ 0.9d_e) = \max\left[0.72(12.00),\ 0.9(9.562)\right]$$
$$= 8.64 \text{ inches}$$ LRFD BDS 5.7.2.8

As wall thickness is less than 16 inches, β can be assumed to be 2.0:

$$\beta = 2.0$$ LRFD BDS 5.7.3.4.1

Calculate the shear resistance of the section:

$$V_p = 0 \text{ kip}$$

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}bd_v$$
$$= 0.0316(2.0)(1.0)\sqrt{4.0}(12.00)(8.64) = 13.1 \text{ kip}$$ LRFD BDS Eq. 5.7.3.3-3

$$V_s = 0 \text{ kip}$$ LRFD BDS Eq. 5.7.3.3-4

$$V_{n1} = 0.25 f'_c bd_v + V_p$$
$$= 0.25(4.0)(12.00)(8.64) + 0 = 103.7 \text{ kip}$$ LRFD BDS Eq. 5.7.3.3-2

$$V_{n2} = V_c + V_s + V_p = 13.1 + 0 + 0 = 13.1 \text{ kip}$$ LRFD BDS Eq. 5.7.3.3-1

$$V_{n\_sec3} = \min(V_{n1}, V_{n2}) = \min(103.7, 13.1) = 13.1 \text{ kip}$$ LRFD BDS 5.7.3.3

9.2.6.3 Axial Thrust Resistance

For this example, there are no external horizontal loads applied to the invert slab so axial demand is not anticipated to be significant and will not be evaluated for the slab section. Axial capacity of the walls will be evaluated and is calculated in accordance with LRFD BDS Article 5.6.4.5. The maximum factored demand load is checked for 10 percent of the nominal compressive capacity of the wall sections. LRFD BDS Article 5.6.4.5 states that if the factored axial demand is lower than 10 percent of the factored nominal compressive strength, then the factored bending moment can be checked for the factored flexural resistance without considering the axial load. For the case where the rating factor is lower than 1.0, an additional load rating would need to be performed in accordance with MBE Article 6A.5.7 and LRFD BDS Equation 5.6.4.5-1.

Rectangular section height:	h	=	12.00 inches
Rectangular section width:	b	=	12.00 inches

$$\phi_t = 0.70 \text{ (no spirals or ties)}$$

$$A_g = bh = (12.00)(12.00) = 144.00 \text{ inch}^2$$

$$P_{wall} = 0.1A_g f'_c = 0.1(144.00)(4.0) = 57.6 \text{ kip}$$

If P_u exceeds $0.10A_gf'_c$, then LRFD BDS Equation 5.6.4.5-1 would be utilized, otherwise, Equation 5.6.4.5-3 can be utilized. Section 4 is the critical section. Based on the unfactored axial loads in Table 15 and LRFR design (HL-93) inventory load factors (discussed later), check the factored axial load, P_u, versus the interaction threshold, P_{wall}:

$$P_u = 16.3 \text{ kips} < P_{wall} = 57.6 \text{ kip}$$

Therefore, the rating can be based on flexure without considering effect of axial load on resistance.

9.2.7 LRFR Rating Calculations

The structural condition of the invert slab is satisfactory and the system factor falls under the category for "All Other Girder Bridges and Slab Bridges." Therefore:

Condition factor:	ϕ_c	= 1.00	MBE Table 6A.4.2.3-1
System factor:	ϕ_s	= 1.00	MBE Table 6A.4.2.4-1

Since the invert slab is not buried, resistance factors are not based on LRFD BDS Table 12.5.5-1, but instead as follows (based on previous calculations, flexure resistance is tension controlled):

Shear resistance factor:	ϕ_v	= 0.90	LRFD BDS 5.5.4.2
Flexure resistance factor:	ϕ_f	= 0.90	LRFD BDS Eq. 5.5.4.2-2
Thrust resistance factor:	ϕ_t	= 0.70	LRFD BDS Eq. 5.5.4.2-2

Resistance factors for compression, in LRFD BDS editions prior to 2005, were based on the type of loading. Resistance factors are now determined by strain conditions. Based on the previous calculations in Section 9.2.6.3 above, axial effects can be ignored, but a check to determine if the rating factor for thrust is greater than 1.0 is needed. A value of 0.70 is selected from LRFD BDS Article 5.5.4.2 as it matched the resistance factor from the older specifications for a compression member without spirals or ties and is comparable to the value for compression in a strut-and-tie system.

The equation for calculating the rating factor is based on MBE Equation 6A.4.2.1-1, which has been simplified for the load types being applied. The component for permanent loads other than dead loads, P, has been removed.

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

For the Strength Limit State:

$$C = \phi_c \phi_s \phi R_n$$ MBE Eq. 6A.4.2.1-2

Table 17 though Table 19 shows the results of the overall capacity based on MBE Equation 6A.4.2.1-2

Table 17. Moment capacity, $\phi_c\phi_s\phi_f M_n$..

Section	Section 1	Section 2	Section 3	Section 4
Description	Top Slab Center	Top Slab Corner	Sidewall Top	Sidewall Center
Moment (kip-inch)	886.5	789.3	562.5	562.5

Table 18. Shear capacity, $\phi_c\phi_s\phi_v M_n$.

Section	Section 2	Section 3
Description	Top Slab Corner	Sidewall Top
Shear (kip)	16.1	11.8

Table 19. Compression/thrust capacity, $\phi_c\phi_s\phi_t T_n$.

Section	Section 3	Section 4
Description	Sidewall Top	Sidewall Center
Compression/Thrust (kip)	40.3	40.3

Load factors are taken from MBE Table 6A.4.2.2-1 (for reinforced concrete, only Strength-I applies) and the FHWA emergency vehicle memo.

DC	γ_{DC}	=	1.25	MBE Table B6A-1
DW	γ_{DW}	=	1.50	MBE Table B6A-1
HL-93 LL (Inventory)	$\gamma_{LL,inv}$	=	1.75	MBE Table B6A-1
HL-93 LL (Operating)	$\gamma_{LL,opr}$	=	1.35	MBE Table B6A-1
NRL (AADT<1000) & EV3 LL	$\gamma_{LL,legal}$	=	1.30	MBE Table 6A.4.4.2.3b-1 and FHWA Memo

Table 20 through Table 22 shows the results for rating factors based on MBE Equation 6A.4.2.1-1 as discussed above:

Table 20. LRFR rating factors for moment at critical sections.

Rating Vehicle	Rating Level	Section 1	Section 2	Section 3	Section 4
HL-93	Inventory	1.62	3.23	1.49	2.99
HL-93	Operating	2.09	4.19	1.93	3.88
NRL	Legal	2.73	6.40	2.95	5.93
EV3	Legal	1.75	3.51	1.62	3.25

Table 21. LRFR rating factors for shear at critical sections.

Rating Vehicle	Rating Level	Section 2	Section 3
HL-93	Inventory	1.44	3.31
HL-93	Operating	1.87	4.29
NRL	Legal	2.86	6.56
EV3	Legal	1.57	3.59

Table 22. LRFR rating factors for compression/thrust at critical sections.

Rating Vehicle	Rating Level	Section 3	Section 4
HL-93	Inventory	3.95	3.88
HL-93	Operating	5.13	5.03
NRL	Legal	7.84	7.69
EV3	Legal	4.30	4.22

9.2.8 LFR Rating Calculations

LFR rating methodology is presented below based on the following:

- Dead load analysis and unfactored loads would be determine using the same process as for LRFR. DW loads are included in the overall dead loads.
- The design truck for live load is designated as HS-20 truck for LFR instead of the HL-93 used for LRFR, but the truck axle configurations are the same.
- The AASHTO Standard Specifications Article 3.24.3.1 for main reinforcement perpendicular to traffic is only for spans of 2 to 24 feet. Therefore, live load distribution for LFR would need to be performed by a more refined structural model and analysis, such as a full 3D analysis, or the updated LRFD distribution methods accounting for changes in multiple presence factors between specifications could be used. For this example, the unfactored live loads from the LRFR analysis will be utilized for this example to show the procedure for LFR ratings.
- Multiple presence factor for LRFR is replaced by Standard Specifications Article 3.12. For one or two lanes, 100 percent of live load effect is used.
- Strength reduction factors are used instead of resistance factors. Standard Specifications Article 8.16.1.2.2 contains the strength reduction factors for concrete.
- Dynamic load (impact) is also different in LFR and uses Standard Specifications Equation 3-1 in Article 3.8.2.1, where the span length, *S*, is taken as 30 feet:

$$I = \min\left(\frac{50}{S+125}, 30\%\right) = \min\left(\frac{50}{30+125}, 30\%\right) = 30\% \quad \text{Standard Specifications Eq. 3-1}$$

- LFR nominal moment capacities are calculated with equations similar to those of LRFR. Standard Specifications Article 8.16.3.2 contains Equations 8-15, 8-16, and 8-17 for determining nominal moment capacity.
- Standard Specifications Article 8.16.3.1 along with Equation 8-18 can be used for maximum reinforcement checks for flexural members.
- Standard Specifications Article 8.17.1 contains the provisions for minimum reinforcement of flexural members.
- Standard Specifications Article 8.16.6.2 contains the provisions for shear strength in concrete.
- Standard Specifications Article 8.16.4.3 contains provisions for biaxial loading, which are similar to the provisions in LRFD BDS Article 5.6.4.5.
- LFR has no equivalent to the LRFR condition or system factors; therefore, capacity is based on the strength requirements in Standard Specifications Chapter 8 (see MBE Article 6B.4.3.2).

The following example goes through an LFR rating for moment at Section 1 for the EV3 truck. It uses the unfactored loads from the previous LRFR analysis. For example purposes, this example shows the calculations for only an EV3.

Table 23. Analysis results for moment (kip-inch) at Section 1 with LFR live load.

Section	**1**
Description	Top Slab Center
DC	174.3
DW	20.2
LL+I (EV3)	280.3

Compute the nominal flexural capacity at Section 1:

Rectangular section height:	h	=	16.00 inches
Rectangular section width:	b	=	12.00 inches
Rebar clearance to tension face:	clr	=	1.00 inches
Area of single rebar:	A_{s_bar}	=	0.60 inch^2
Diameter of rebar:	dia_{bar}	=	0.875 inches
Spacing of rebar:	s	=	6 inches

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.60(12.00)}{6} = 1.20 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 16.00 - \frac{0.875}{2} - 1.00 = 14.563 \text{ inches}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.20(60)}{0.85(4.0)(12.00)} = 1.76 \text{ inches}$$ Standard Spec. Eq. 8-17

$$\phi_f = 0.90$$ Standard Spec. 8.16.1.2.2

$$\phi M_{n\_bott1} = \phi_f A_s f_y \left(d - \frac{a}{2} \right)$$

$$= 0.90(1.20)(60)\left(14.563 - \frac{1.76}{2} \right) = 886.5 \text{ kip-inch}$$ Standard Spec. Eq. 8-16

Note, the nominal moment resistance on the top reinforcement is not critical at mid-span and need not be checked.

Check maximum reinforcement requirement:

$$\beta_1 = 0.85$$ Standard Spec. 8.16.2.7

$$\rho_b = \frac{0.85 \beta_1 f'_c}{f_y} \left(\frac{87,000}{87,000 + f_y} \right)$$

$$= \frac{0.85(0.85)(4,000)}{60,000} \left(\frac{87,000}{87,000 + 60,000} \right) = 0.0285$$ Standard Spec. Eq. 8-18

$$\rho = \frac{A_s}{bd_s} = \frac{1.20}{12.00(14.563)} = 0.0069$$ Standard Spec. 8.1.2

$$\rho = 0.0069 < 75\% \rho_b = 0.0214 \therefore \text{OK}$$ Standard Spec. 8.16.3.1.1

Maximum reinforcement requirements at this section are satisfied.

Check minimum reinforcement requirement:

Gross moment of inertia:

$$I_g = \frac{bh^3}{12} = \frac{12.00(16.00)^3}{12} = 4096 \text{ inch}^4$$ Standard Spec. 8.1.2

Distance from centroidal axis of gross section to extreme fiber in tension:

$$y_t = \frac{h}{2} = \frac{16.00}{2} = 8.00 \text{ inches}$$ Standard Spec. 8.1.2

$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{4,000} = 474 \text{ psi}$$ Standard Spec. 8.15.2.1.1

Cracking moment:

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{474(4096)}{8.00} = 242.9 \text{ kip-inch}$$ Standard Spec. Eq. 8-2

$$\phi M_{n\_bott1} = 886.5 \text{ kip-inch} > 1.2M_{cr} = 1.2(242.9) = 291.5 \text{ kip-inch} \therefore \text{ OK}$$

Minimum reinforcement requirements at this section are satisfied.

The equation for calculating the rating factor is taken from MBE Article 6B.4.1:

$$RF = \frac{C - A_1 D}{A_2 L(1+I)}$$ MBE Eq. 6B.4.1-1

The capacity at this section is taken as:

$$C = \phi M_{n\_bott1} = 886.5 \text{ kip-in}$$

The load factors are taken from MBE Article 6B.4.3:

Dead load	A_1	=	1.3
Live load for EV3 at inventory	$A_{2,inv}$	=	2.17
Live load for EV3 at operating	$A_{2,opr}$	=	1.3

The unfactored loads from Table 23:

$$D = DC + DW = 174.3 + 20.2 = 194.5 \text{ kip-inch}$$

$$L(1+I) = LL + I_{EV3} = 273.9 \text{ kip-inch}$$

Calculate the LFR rating factors for the EV3 truck at both inventory and operating levels:

$$RF_{inv} = \frac{C - A_1 D}{A_{2,inv} L(1+I)} = \frac{886.5 - 1.3(194.5)}{2.17(273.9)} = 1.07$$

$$RF_{opr} = \frac{C - A_1 D}{A_{2,opr} L(1+I)} = \frac{886.5 - 1.3(194.5)}{1.3(273.9)} = 1.78$$

Note that EV3 only requires an operating rating factor. However, for example purposes, an inventory rating factor was provided.

9.3 EXAMPLE 2 – STEEL INVERT GIRDER

Example 2 presents a circular tunnel with a structural invert slab supported on steel invert girders (Figure 65). The tunnel is below the influence of live load from above; therefore, the tunnel liner is not rated for this example. The invert slab and steel invert girder are rated with the LRFR and LFR methods for the design vehicles (HL-93 and HS-20, respectively) and the AASHTO legal loads Type 3, 3S2 and 3-3. A posting analysis is performed as well. Note that for the sake of brevity, this example does not consider all legal loads; see Example 1 for inclusion of all legal loads.

This example will perform the following steps to rate this steel invert girder:

1. Structure data
2. Dead load, DC, calculations
3. Wearing surface, DW, calculations
4. Live load application
5. Structural analysis
6. Resistance calculations
7. LRFR rating calculations
8. Posting evaluation
9. LFR rating calculations

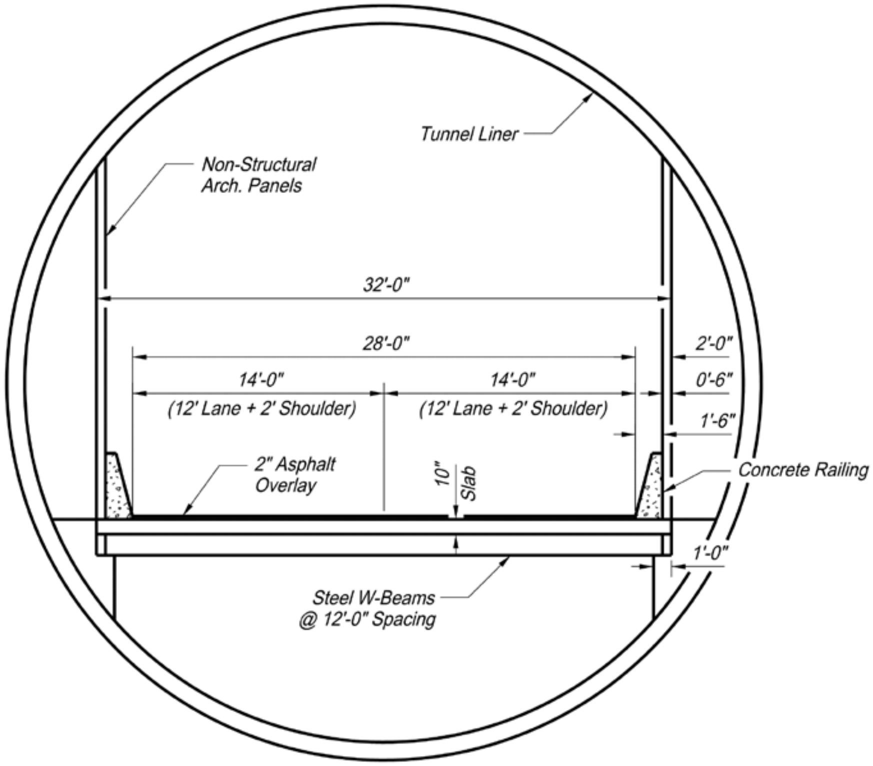
9.3.1 Structure Data

9.3.1.1 Materials

Materials are known, otherwise use MBE Article 6A5.2.1 and Article 6A5.2.2.

Concrete:	f'_c	=	4.0 ksi
Reinforcing Steel:	f_y	=	60.0 ksi
ASTM A709 Grade 50 Steel:	F_y	=	50.0 ksi

9.3.1.2 Dimensions



Source: FHWA

Figure 65. Illustration. Cross-section showing invert slab structure and steel invert girder.

9.3.1.3 Example Notes

- This cross-section is fictitious and only for the purpose of demonstrating the principles of rating a concrete slab on steel invert girders.
- Live loading is only applicable to slab and invert girders. No live load above tunnel within zone of influence; therefore, tunnel liner is not rated.
- If the upper architectural walls were structural, they would need to be included in the structural analysis and rated as well.
- While not assumed here, any other mechanical (including dynamic equipment loads) and architectural systems supported by the slab and invert girder, should be included in the dead load.
- This example focuses on an interior section of the slab and an interior invert girder. A section taken near the ends of the slab and an exterior invert girder would need to account for increases in load effects due to reduced live load distribution near the edge, similar to an edge beam.
- The rating Engineer should evaluate the existing record drawings to determine if the mild reinforcement lap lengths and development lengths meet current design code

requirements. This example is based on the assumption that mild reinforcement lap lengths and development lengths meet current design code requirements.

- The rating Engineer should review the record drawings and inspection reports to properly identify the support condition (pinned, expansion, fixed).
- ADTT > 5000
- The connection of the diaphragm connection plates to the steel invert girder flanges is a Category C' detail. In accordance with MBE Article 6A.6.4.1, details known to be fatigue-prone (Category C and lower in accordance with MBE), should be checked for infinite fatigue life in accordance with MBE Article 7.2.4. Details that do not have infinite fatigue life may be optionally checked for remaining fatigue life. Fatigue is not checked for the Category C' detail in this example and is assumed to have infinite fatigue life. It should also be noted that these types of connections, detailed to current LRFD BDS requirements, have not exhibited fatigue problems in existing structures.
- The W-shape rolled beam (invert girder) is a single piece with no splices.
- Rating is performed only at maximum shear and moment locations. Typical ratings include evaluation at all tenth points and all section change locations.

9.3.1.4 Rating Approach/Assumptions:

- LRFR evaluation of the slab is performed using a 1-foot-wide strip of the deck (parallel to the direction of traffic).
- 2 inch asphalt overlay (not measured during inspection) was placed at time of construction; therefore, the full deck thickness was treated as structural and was not reduced to account for sacrificial wearing surface on the concrete.
- LRFR live load ratings are evaluated for HL-93 design loading (truck or tandem), and the AASHTO legal loads Type 3, 3S2, 3-3 for this example. For a complete rating, all applicable legal loads, including State legal loads, the EV2 and EV3 emergency vehicles should be rated as well, similar to Example 1.
- Slab and steel invert girders are not considered to act compositely for this example. However, the top flange of the steel beam is embedded into the concrete slab; therefore, the top flange is continuously laterally braced (MBE Article 6A.6.9.3).
- It is assumed the slab was formed with temporary forms and is subjected to its own weight for dead load.
- Support condition at invert girder ends are assumed to be pin/roller and the invert girder acts as a simply supported beam. Concrete slab is assumed to be continuous over the invert girders.
- Diaphragms between invert girders are connected to the bearing stiffeners and are considered sufficient to provide end bracing. There are also diaphragms at the 1/3 points of the invert girder. The bearing stiffeners and diaphragms are not rated in this example as they are not primary members. However, a rating of the bearing stiffener may be warranted in a full rating analysis.

- There are no signs of distress or deterioration; therefore, the slab and invert girders are considered to be in satisfactory physical condition.
- There are no post/pre-tensioning forces; therefore, MBE Article 6A.2.2.2 does not apply.

9.3.2 Dead Load, DC

9.3.2.1 Concrete Slab

Normal weight of reinforced concrete:

$$w_c = 0.145 \text{ kcf} + 0.005 \text{ kcf} = 0.150 \text{ kcf}$$ LRFD BDS Table 3.5.1-1 & C3.5.1

The following weights are calculated for the analysis strip width (Figure 66):

$$w_{strip} = 1.00 \text{ ft}$$

Top slab weight:

$$t_{slab} = 10 \text{ inches}$$

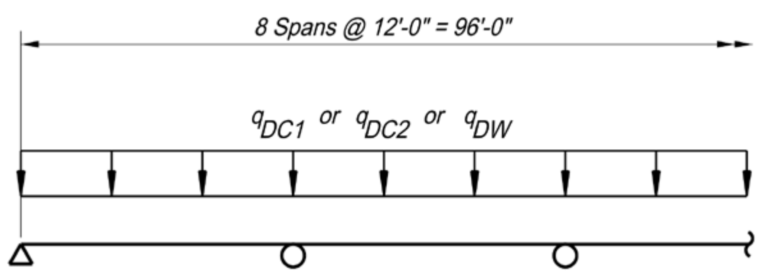
$$q_{slab} = t_{slab} w_c w_{strip} = \left(10/12\right)(0.150)(1.00) = 0.125 \text{ klf}$$

Concrete barrier rail weight (assuming weight, $q_{barrier}$, is 0.500 klf per side distributed across the width of the slab, w_{slab}, which is 32 feet wide):

$$q_{rail\_DC2} = \frac{2q_{slab} w_{strip}}{w_{slab}} = \frac{2(0.500)(1.00)}{32} = 0.094 \text{ klf}$$

Non-structural architectural panel weight (assuming panels have thickness, t_{panel}, of 6 inches and height, h_{panel}, of 20 feet distributed across the width of the slab):

$$q_{panel\_DC2} = \frac{2t_{panel} h_{panel} w_c w_{strip}}{w_{slab}} = \frac{2\left(6/12\right)(20)(0.150)(1.00)}{32} = 0.020 \text{ klf}$$



Source: FHWA

Figure 66. Illustration. Locations of applied concrete slab dead loads.

9.3.2.2 Steel Invert Girder

Invert girder spacing, span length and weight:

$$S_{FB\_spa} = 12.00 \text{ ft}$$

$$S_{FB\_span} = 31.00 \text{ ft}$$

$$w_{grdr} = 84 \text{ plf}$$

Tributary slab weight (Figure 67):

$$q_{trib\_slab\_DC1} = S_{FB\_spa} t_{slab} w_c = (12)\left(10/12\right)0.150 = 1.500 \text{ klf}$$

Invert girder self-weight (add additional 10 percent to invert girder's weight to account for intermediate diaphragms. This is a conservative assumption for this example. Ideally load rater should account for the actual diaphragm weights calculated from the plans.):

$$q_{grdr\_DC1} = w_{grdr}\ (1.10) = \left(84/100\right)1.10 = 0.092 \text{ klf}$$

Invert girder bearing stiffener and end diaphragm weight (assumed weight at each bearing location):

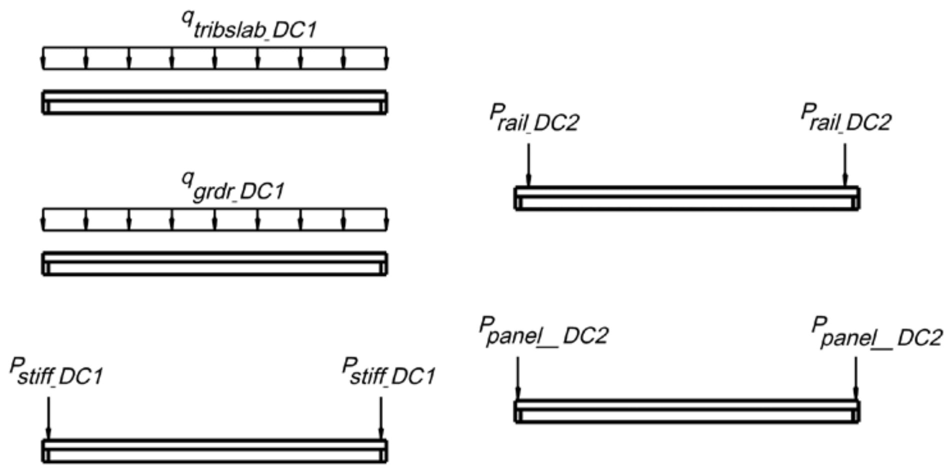
$$P_{stiff\_DC1} = 0.175 \text{ kip}$$

Concrete barrier rail weight (per side at bearing location):

$$P_{rail\_DC2} = q_{barrier} S_{FB\_spa} = 0.500(12) = 6 \text{ kip}$$

Non-structural architectural panel weight (per side at bearing location):

$$P_{panel\_DC2} = t_{panel} h_{panel} S_{FB\_spa} w_c = \left(6/12\right)(20)(12)(0.150) = 18 \text{ kip}$$



Source: FHWA

Figure 67. Illustration. Locations of applied invert girder dead loads.

9.3.3 Wearing Surface, DW

9.3.3.1 Concrete Slab

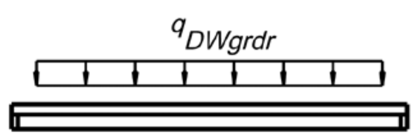
Bituminous wearing surface weight over roadway width (w_{rdwy}) with a density, w_{DW}, of 0.140 kcf in accordance with LRFD BDS Table 3.5.1-1 and an estimated thickness, t_{ws}, of 2 inches:

$$q_{DW} = \frac{t_{ws} w_{DW} w_{rdwy} w_{strip}}{w_{slab}} = \frac{\left(2/12\right)(0.140)(28)(1.00)}{32} = 0.020 \text{ klf}$$

9.3.3.2 Steel Invert Girder

Bituminous wearing surface weight over roadway width (Figure 68):

$$q_{DW} = t_{ws} w_{DW} S_{FB\_spa} = \left(2/12\right)(0.140)(28) = 0.280 \text{ klf}$$



Source: FHWA

Figure 68. Illustration. Location of applied wearing surface load to invert girder.

9.3.4 Live Load Application

9.3.4.1 Concrete Slab

9.3.4.1.1 Live Load Distribution

The concrete slab spans in the direction parallel to the direction of traffic. For slabs spanning less than 15 feet, live load can be distributed to the top slab using the equations specified in LRFD BDS Article 4.6.2.1 for concrete decks with primary strips parallel to the direction of traffic. For spans larger than 15 feet, the provisions of LRFD BDS Article 4.6.2.3 should be used. LRFD BDS Article 4.6.2.1.3 specifies the following:

- The equivalent strips for decks that span primarily in the direction parallel to traffic are subject to a maximum width of 144 in in accordance with LRFD BDS Article 4.6.2.1.3.
- S is the spacing of support components (invert girders, ft):

  $S = 12$ ft

- For positive moment, width of equivalent strip in inches:

  $w_{pos} = 26.0 + 6.6S$ LRFD BDS Table 4.6.2.1.3-1

- For negative moment, width of equivalent strip in inches:

  $w_{neg} = 48.0 + 3.0S$ LRFD BDS Table 4.6.2.1.3-1

Widths of equivalent interior strip for one axle are calculated as follows:

Width of equivalent interior strip in positive moment regions:

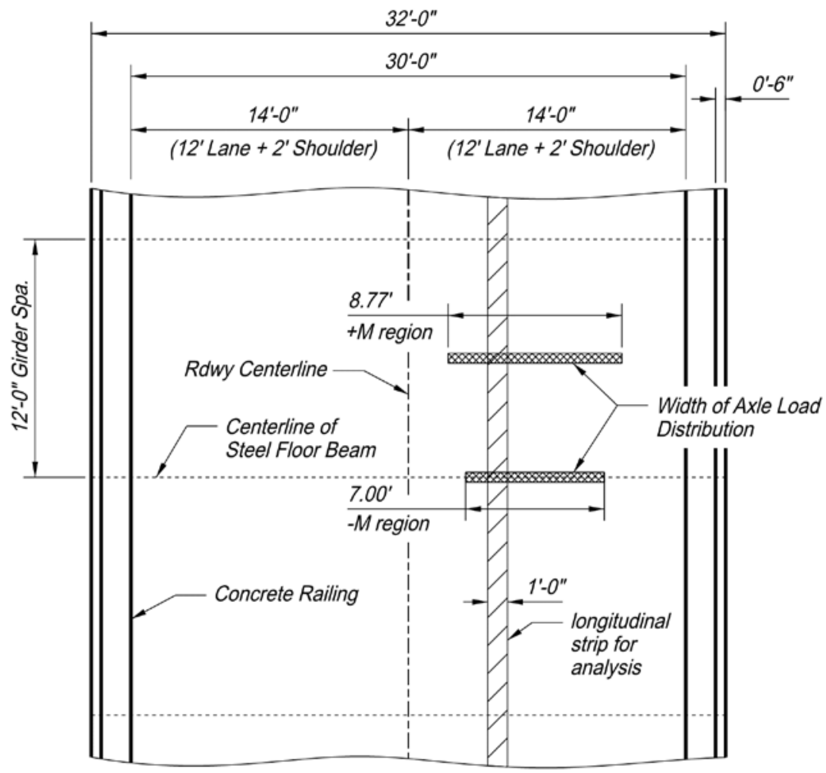
$$w_{pos} = 26.0 + 6.6(12.00) = 105.2 \text{ inches} = 8.77 \text{ ft} \leq S = 12.00 \text{ ft}$$

Width of equivalent interior strip in negative moment regions:

$$w_{neg} = 48.0 + 3.0(12.00) = 84.0 \text{ inches} = 7.00 \text{ ft} \leq S = 12.00 \text{ ft}$$

For this example, the HL-93 Design Truck/Tandem (MBE Article 6A.2.3) and the AASHTO legal trucks will be evaluated. The lane portion of the HL-93 design truck will not be applied in accordance with LRFD BDS Article 3.6.1.3.3. Axle configurations are shown below.

Axle loads for each vehicle can be distributed transversely over the equivalent interior strip width. The axle load(s) are then divided by the distribution width to give an equivalent load to be analyzed in the 1-foot-wide longitudinal section used for structural analysis. The following diagram (Figure 69) depicts how the axle loads are distributed for analysis on a 1-foot-wide longitudinal strip.



Source: FHWA

Figure 69. Illustration. Distribution of live load axles to equivalent slab analysis width.

The equations from LRFD BDS Article 4.6.2.1.3, as used for this example, are an approximate method considered acceptable for this type of structure. However, a more refined structural model (MBE Article 6A.3.3) and analysis, such as a full 3D finite element analysis considering the distribution of wheel loads using two-way slab action, may be used to determine model live loads if a more refined result is required.

9.3.4.1.2 HL-93 Live Load

Determine the equivalent HL-93 deign vehicle concentrated axle loads for application to the line girder analysis.

HL-93 truck equivalent load on 1-foot-wide analysis strip:

$$HL93_{trk\_axle} = \begin{bmatrix} 8 & 32 & 32 \end{bmatrix} \text{ kip}$$

$$P_{HL93trk\_pos} = \frac{HL93_{trk\_axle}}{w_{pos}} = \frac{\begin{bmatrix} 8 & 32 & 32 \end{bmatrix}}{8.77} = \begin{bmatrix} 0.913 & 3.650 & 3.650 \end{bmatrix} \text{ kip}$$

$$P_{HL93trk\_neg} = \frac{HL93_{trk\_axle}}{w_{neg}} = \frac{\begin{bmatrix} 8 & 32 & 32 \end{bmatrix}}{7.00} = \begin{bmatrix} 1.143 & 4.571 & 4.571 \end{bmatrix} \text{ kip}$$

HL-93 tandem equivalent load on 1-foot-wide analysis strip:

$$HL93_{tdm\_axle} = \begin{bmatrix} 25 & 25 \end{bmatrix} \text{ kip}$$

$$P_{HL93tdm\_pos} = \frac{HL93_{tdm\_axle}}{w_{pos}} = \frac{\begin{bmatrix} 25 & 25 \end{bmatrix}}{8.77} = \begin{bmatrix} 2.852 & 2.852 \end{bmatrix} \text{ kip}$$

$$P_{HL93tdm\_neg} = \frac{HL93_{tdm\_axle}}{w_{neg}} = \frac{\begin{bmatrix} 25 & 25 \end{bmatrix}}{7.00} = \begin{bmatrix} 3.571 & 3.571 \end{bmatrix} \text{ kip}$$

9.3.4.1.3 Legal Live Load

Determine the equivalent AASHTO legal load concentrated axle loads for application to the line girder analysis.

Type 3 equivalent load on 1-foot-wide analysis strip:

$$Type3_{axle} = \begin{bmatrix} 16 & 17 & 17 \end{bmatrix} \text{ kips}$$

$$P_{Type3\_pos} = \frac{Type3_{axle}}{w_{pos}} = \frac{\begin{bmatrix} 16 & 17 & 17 \end{bmatrix}}{8.77} = \begin{bmatrix} 1.825 & 1.939 & 1.939 \end{bmatrix} \text{ kip}$$

$$P_{Type3\_neg} = \frac{Type3_{axle}}{w_{neg}} = \frac{\begin{bmatrix} 16 & 17 & 17 \end{bmatrix}}{7.00} = \begin{bmatrix} 2.286 & 2.429 & 2.429 \end{bmatrix} \text{ kip}$$

Type 3S2 equivalent load on 1-ft wide analysis strip:

$$Type3S2_{axle} = \begin{bmatrix} 10 & 15.5 & 15.5 & 15.5 & 15.5 \end{bmatrix} \text{ kip}$$

$$P_{Type3S2\_pos} = \frac{Type3S2_{axle}}{w_{pos}} = \frac{\begin{bmatrix} 10 & 15.5 & 15.5 & 15.5 & 15.5 \end{bmatrix}}{8.77}$$
$$= \begin{bmatrix} 1.141 & 1.768 & 1.768 & 1.768 & 1.768 \end{bmatrix} \text{ kip}$$

$$P_{Type3S2\_neg} = \frac{Type3S2_{axle}}{w_{neg}} = \frac{\begin{bmatrix} 10 & 15.5 & 15.5 & 15.5 & 15.5 \end{bmatrix}}{7.00}$$
$$= \begin{bmatrix} 1.429 & 2.214 & 2.214 & 2.214 & 2.214 \end{bmatrix} \text{ kip}$$

Type 3-3 equivalent load on 1-foot-wide analysis strip:

$$Type3\_3_{axle} = \begin{bmatrix} 12 & 12 & 12 & 16 & 14 & 14 \end{bmatrix} \text{ kip}$$

$$P_{Type3\_3\_pos} = \frac{Type3\_3_{axle}}{w_{pos}} = \frac{\begin{bmatrix} 12 & 12 & 12 & 16 & 14 & 14 \end{bmatrix}}{8.77}$$
$$= \begin{bmatrix} 1.369 & 1.369 & 1.369 & 1.825 & 1.597 & 1.597 \end{bmatrix} \text{ kip}$$

$$P_{Type3\_3\_neg} = \frac{Type3\_3_{axle}}{w_{neg}} = \frac{\begin{bmatrix}12 & 12 & 12 & 16 & 14 & 14\end{bmatrix}}{7.00}$$

$$= \begin{bmatrix}1.714 & 1.714 & 1.714 & 2.286 & 2.000 & 2.000\end{bmatrix} \text{ kip}$$

9.3.4.1.4 Impact Load

The live load impact factor is in accordance with MBE Article 6A.4.4.3 and LRFD BDS Table 3.6.2.1-1:

$$IM = 33\%$$

9.3.4.2 Steel Invert Girder

9.3.4.2.1 Live Load Distribution

For transverse invert girders that directly support a concrete slab, live load distribution is in accordance with LRFD BDS Article 4.6.2.2.2f. For a concrete deck, the deck span length/invert girder spacing is outside of the range of applicability shown in LRFD BDS Table 4.6.2.2.2f-1; therefore, the lever rule is applied in accordance with LRFD BDS Article 4.6.2.2.2f. LRFD BDS Article 4.6.2.2.2f also indicates that all of the live load is applied; therefore, the lane portion of the HL-93 will be incorporated for the invert girder rating, even though it was not included for the slab rating.

9.3.4.2.2 Wheel Loads

The following are resulting live load reactions of the invert girder due to the applied wheel loads for each rating vehicle. These values were determined using static force equilibrium of the invert girder pinned at the support walls. MBE Appendix D6B can also be used (except for the tandem axles) to determine the wheel reactions at the invert girder. This cross section has a span of 30 feet from inside wall to inside wall with the exterior wheel placed 1 foot inside of curb (See Figure 65).

Vehicular portion of HL-93 design vehicle wheel load reactions (tandem controls over design truck) noting the pair of axles are 4 feet apart and the girders are spaced at 12 feet.:

$$Axle_{HL93} = 25 + 25\left(\frac{12-4}{12}\right) = 41.67 \text{ kip}$$

$$whl_{HL93\_tdm} = \frac{Axle_{HL93}}{2} = \frac{41.67}{2} = 20.84 \text{ kip wheel load}$$

Lane portion of HL-93 design vehicle wheel load reactions (for simplicity, lane is broken out into equivalent wheels – 1 wheel equals ½ lane):

$$Lane_{HL93} = 0.64klf\,(12\,ft\ spacing) = 7.68 \text{ kip}$$

$$whl_{HL93\_lane} = \frac{Lane_{HL93}}{2} = \frac{7.68}{2} = 3.84 \text{ kip wheel load}$$

Legal load vehicle wheel load reactions noting that controlling axles for reaction on girders are spaced 4 feet apart for all legal loads:

$$Axle_{Type3} = 17 + 17\left(\frac{12-4}{12}\right) = 28.32 \text{ kip}$$

$$whl_{Type3} = \frac{Axle_{Type3}}{2} = \frac{28.32}{2} = 14.16 \text{ kip wheel load}$$

$$Axle_{Type3S2} = 15.5 + 15.5\left(\frac{12-4}{12}\right) = 25.83 \text{ kip}$$

$$whl_{Type3S2} = \frac{Axle_{Type3S2}}{2} = \frac{25.83}{2} = 12.92 \text{ kip wheel load}$$

$$Axle_{Type3\_3} = 14 + 14\left(\frac{12-4}{12}\right) = 23.33 \text{ kip}$$

$$whl_{Type3\_3} = \frac{Axle_{Type3\_3}}{2} = \frac{23.33}{2} = 11.66 \text{ kip wheel load}$$

9.3.4.2.3 Impact Load

The live load impact factor is in accordance with MBE Article 6A.4.4.3 and LRFD BDS Table 3.6.2.1-1:

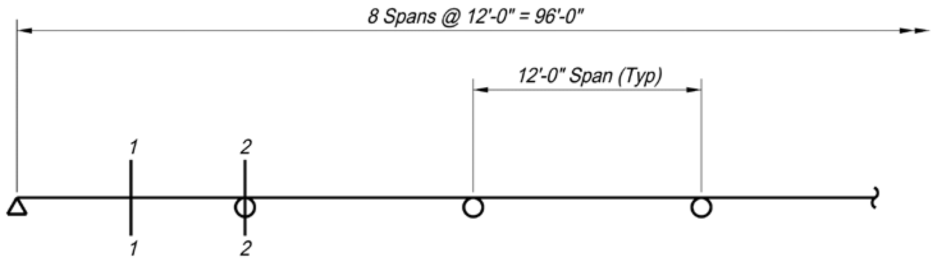
$$IM = 33\%$$

9.3.5 Structural Analysis

9.3.5.1 Concrete Slab

A continuous beam analysis procedure is applied for the 1-foot-wide longitudinal analysis strip. The analysis strip is assumed to be an 8-span continuous beam over 9 transverse steel invert girders (Figure 70). Controlling positive moments are found in Spans 1 and 8 (end spans), and controlling negative moments are found over the 2nd and 8th invert girders. Structural analysis of the longitudinal strip assumes a pin at the first invert girder and roller support at invert girders 2 thru 9.

The critical failure mode for the concrete slab is due to flexure. LRFD BDS Article 4.6.2.1.6 indicates that the design section for negative moment can be taken at 1/4 of the flange width from the centerline of the support; however, this is typically for a slab in a situation with cross-section types (a) or (c) with the main invert girders supporting the deck framing parallel to the roadway centerline (direction of traffic). Since the main invert girders are perpendicular to centerline (direction of traffic), this example will conservatively use the negative moment directly over the support (invert girder) centerline. The positive moment is checked at the maximum live load moment location at 40 percent of the end span.



Source: FHWA

Figure 70. Illustration. Rating locations of structural slab.

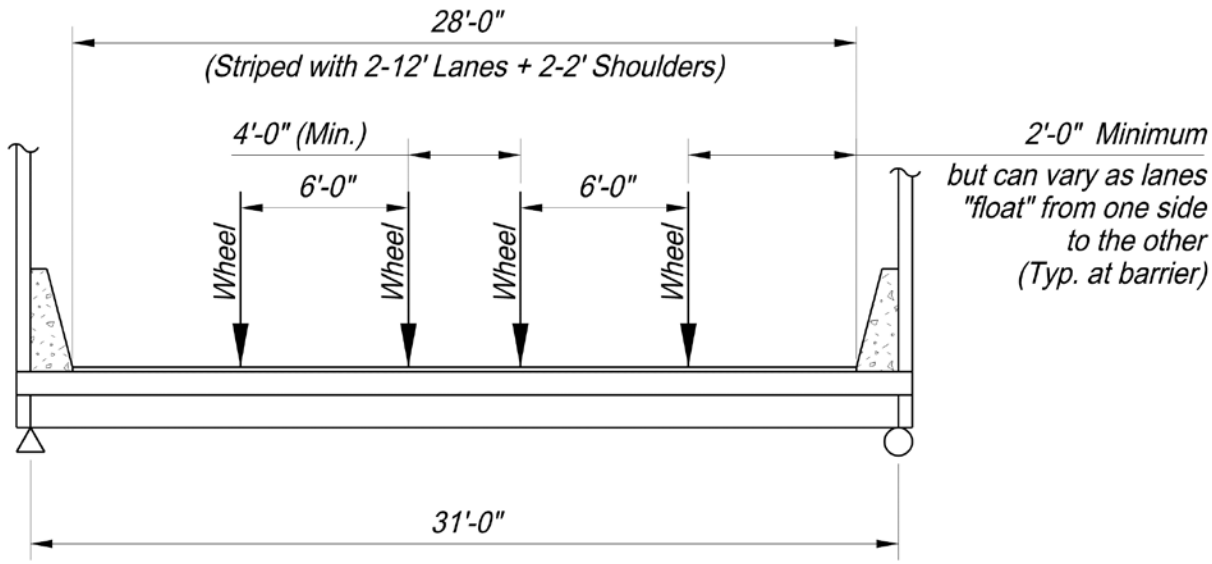
Table 24 presents the analysis results for unfactored moments for the HL-93 and legal live loads. The unfactored live load force effects incorporate the multiple presence factors (MPF = 1.20) in accordance with LRFD BDS Article 3.6.1.1.2 and the dynamic load allowance (IM = 33 percent).

Table 24. Analysis results for slab moment (kip-inch) at critical sections.

Section	**1**	**2**
Description	Bottom Slab at Maximum Positive Moment	Top Slab Over Support (Invert Girder)
DC	33.62	-45.53
DW	2.69	-3.65
LL+IM (HL-93)	176.64	-164.19
LL+IM (Type 3)	120.14	-105.89
LL+IM (Type 3S2)	113.38	-111.66
LL+IM (Type 3-3)	98.92	-85.72

9.3.5.2 Steel Invert Girder

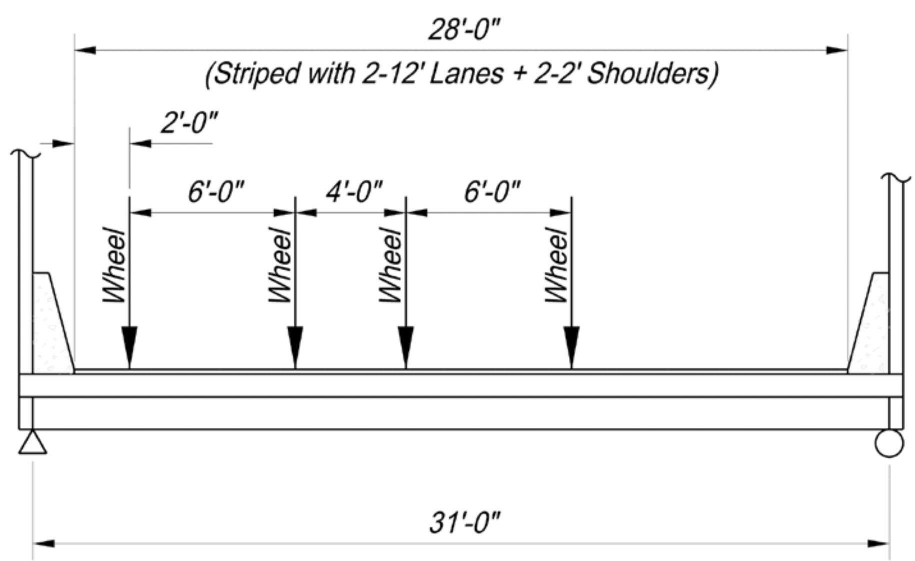
Using the wheel loads and a standard spacing of 6.0 feet between wheels, the invert girder maximum moment, and shear values due to live load can then be determined by moving the wheels within the lane as required in LRFD BDS Article 3.6.1.3 and MBE Article 6A.2.3.2. The wheels are allowed to move within standard 12 foot lanes, and the lanes are allowed to "float" between the barriers. The MBE also allows for an alternate analysis, with the bridge Owner's permission, by limiting the truck wheels to only move within the currently configured striped lanes as delineated in the field.



Source: FHWA

Figure 71. Illustration. Wheel load locations (2 lanes loaded) to produce maximum moment at invert girder mid-span.

Note: For this example, the lanes were allowed to "float" and cross over the striped centerline that is assumed to be centered in the cross-section (14 feet from each barrier face). If a load rating were desired that more closely matched the current normal operating conditions, MBE Article 6A.2.3.2 allows the alternate analysis as described above.



Source: FHWA

Figure 72. Illustration. Wheel load locations (2 lanes loaded) to produce maximum shear at invert girder left support.

The analysis results for unfactored moment, shear, and flange stress for the HL-93 and legal live loads at the controlling mid-span location are presented in Table 25. The unfactored live load force effects incorporate the multiple presence factors (MPF = 1. 0 for controlling 2 lane condition) in accordance with LRFD BDS Article 3.6.1.1.2 and the dynamic load allowance (IM = 33 percent).

Table 25. Analysis results for steel invert girder at critical location.

Load	Moment (kip-inch)	Shear (kip)	Flange Stress (ksi)
DC1	2295.44	24.86	10.78
DC2	0.00	24.00	0.00
DW	399.84	3.92	1.88
HL-93 Lane	967.68	9.66	4.54
HL-93 Tandem	6983.06	69.72	32.78
Type 3	4747.54	47.40	22.29
Type 3S2	4328.59	43.21	20.32
Type 3-3	3909.64	39.04	18.36

9.3.6 Resistance Calculations

9.3.6.1 Concrete Slab

The depth, analysis section width, and reinforcement for each of the critical sections are tabulated below in Table 26.

Table 26. Critical section data.

Section	Depth (inch)	Width (inch)	Bar No.	Bar Spa. (inch)	Cover (inch)
1-Bottom	10.00	12.00	6	8.00	1.00
2-Top	10.00	12.00	6	8.00	2.50

Concrete Properties:

f'_c = 4.0 ksi
α_1 = 0.85 — LRFD BDS 5.6.2.2
β_1 = 0.85 if $f'_c \leq 4.0$ ksi — LRFD BDS 5.6.2.2
λ = 1.0 (normal weight concrete) — LRFD BDS 5.4.2.8
γ_3 = 0.67 — LRFD BDS 5.6.3.3
γ_1 = 1.6 — LRFD BDS 5.6.3.3

Modulus of rupture:

$f_r = 0.24\lambda\sqrt{f'_c} = 0.24(1.0)\sqrt{4.0} = 0.480$ ksi — LRFD BDS 5.4.2.6

Compression reinforcement in flexural capacity calculations is conservatively ignored. Calculated results are based on a per foot analysis width.

9.3.6.1.1 Section 1 – Bottom of Slab at Mid-span

Rectangular section height:	h	=	10.00 inches
Rectangular section width:	b	=	12.00 inches
Clear distance to rebar from tension face:	clr	=	1.00 inches
Area of single rebar:	A_{s_bar}	=	0.44 inch^2
Diameter of rebar:	dia_{bar}	=	0.750 inches
Spacing of rebar:	s	=	8 inches

Determine equivalent area of reinforcing bar:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.44(12.00)}{8} = 0.66 \text{ inch}^2$$

Determine distance from the extreme compression fiber to the centroid of the reinforcement:

$$d_s = h - \frac{dia_{bar}}{2} - clr = 10.00 - \frac{0.750}{2} - 1.00 = 8.625 \text{ inches}$$

Determine distance from the extreme compression fiber to the neutral axis for non-prestressed tension reinforcement only:

$$c = \frac{A_s f_y}{\alpha_1 \beta_1 f'_c b} = \frac{0.66(60)}{0.85(0.85)(4.0)(12.00)} = 1.14 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-

Determine the equivalent stress block:

$$a = \beta_1 c = 0.85(1.14) = 0.97 \text{ inches}$$ LRFD BDS Eq. 5.6.2.2

Determine resistance factor:

$$\varepsilon_{cl} = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.0021 \le 0.002 \;\therefore\; \varepsilon_{cl} = 0.002$$ LRFD BDS 5.6.2.1

$$\varepsilon_{tl} = 0.005$$ LRFD BDS 5.6.2.1

The net tensile strain is determined using similar triangles based on a strain limit of 0.003 in the extreme compression fiber, see LRFD BDS Article C5.6.2.1.

$$\varepsilon_t = \frac{0.003}{c}(d_s - c) = \frac{0.003}{1.14}\left(8.625 - 1.14\right) = 0.020$$

Since $\varepsilon_t \ge \varepsilon_{tl}$, the section is tension controlled and 0.90 is used for the resistance factor in accordance with LRFD BDS Article 5.5.4.2. If $\varepsilon_t \le \varepsilon_{cl}$, it would be compression controlled and the resistance factor would be 0.75 in accordance with LRFD BDS Article 5.5.4.2 and MBE Article 6A.5.5. If ε_t was between ε_{cl} and ε_{tl}, then LRFD BDS Eq. 5.5.4.2-2 would be used to interpolate a resistance factor.

$\phi_f = 0.90$ LRFD BDS 5.5.4.2

Calculate nominal moment resistance:

$$M_{n\_bott1} = A_s f_y \left(d_s - \frac{a}{2} \right) = 0.66(60)\left(8.625 - \frac{0.97}{2} \right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 322.3 \text{ kip-inch}$$

Note that the nominal moment resistance based on top reinforcement is not critical at mid-span and need not be checked.

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(10.00)^2}{6} = 200.00 \text{ inch}^3$$ LRFD BDS 5.6.3.3

The cracking moment equation simplifies to the following (only monolithic sections and no prestress forces):

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(200.0) = 102.9 \text{ kip-inch}$$ LRFD BDS Eq. 5.6.3.3-1

$$\phi_f M_{n\_bott1} = 0.90(322.3) = 290.1 \text{ kip-inch}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

9.3.6.1.2 Section 2 – Top of Slab over Support

Rectangular section height:	h	=	10.00 inches
Rectangular section width:	b	=	12.00 inches
Clear distance to rebar from tension face:	clr	=	2.50 inches
Area of single rebar:	A_{s_bar}	=	0.44 inch^2
Diameter of rebar:	dia_{bar}	=	0.750 inches
Spacing of rebar:	s	=	8 inches

Determine equivalent area of reinforcing bar:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.44(12.00)}{8} = 0.66 \text{ inch}^2$$

Determine distance from the extreme compression fiber to the centroid of the reinforcement:

$$d_s = h - \frac{dia_{bar}}{2} - clr = 10.00 - \frac{0.750}{2} - 2.50 = 7.125 \text{ inches}$$

Determine distance from the extreme compression fiber to the neutral axis for non-prestressed tension reinforcement only:

$$c = \frac{A_s f_y}{\alpha_1 \beta_1 f'_c b} = \frac{0.66(60)}{0.85(0.85)(4.0)(12.00)} = 1.14 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4

Determine the equivalent stress block:

$$a = \beta_1 c = 0.85(1.14) = 0.97 \text{ inches} \qquad \text{LRFD BDS Eq. 5.6.2.2}$$

Determine resistance factor, where ε_{cl} (0.002) and ε_{tl} (0.005) have been determine previously in Section 9.3.6.1.1:

$$\varepsilon_t = \frac{0.003}{c}(d_s - c) = \frac{0.003}{2.08}\left(13.062 - 2.08\right) = 0.016 \geq \varepsilon_{tl} \;\therefore$$

$$\phi_f = 0.90 \qquad \text{LRFD BDS 5.5.4.2}$$

Calculate nominal moment resistance:

$$M_{n\_top2} = A_s f_y \left(d_s - \frac{a}{2}\right) = 0.66(60)\left(7.125 - \frac{0.97}{2}\right) \qquad \text{LRFD BDS Eq. 5.6.3.2.2-1}$$

$$= 262.9 \text{ kip-inch}$$

Note that the nominal moment resistance based on top reinforcement is not critical at mid-span and need not be checked.

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(10.00)^2}{6} = 200.00 \text{ inch}^3 \qquad \text{LRFD BDS 5.6.3.3}$$

The cracking moment equation simplifies to the following (only monolithic sections and no prestress forces):

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(200.0) = 102.9 \text{ kip-inch} \qquad \text{LRFD BDS Eq. 5.6.3.3-1}$$

$$\phi_f M_{n\_top2} = 0.90(262.9) = 236.6 \text{ kip-inch}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

9.3.6.2 Steel Invert Girder

W27x84 Properties:

E = 29,000 ksi
t_f = 0.640 inches
b_f = 10.0 inches
d = 26.7 inches
t_w = 0.460 inches
S_x = 213 inch^3

Resistance factors:

$$\phi_f = 1.00$$ LRFD BDS 6.5.4.2

$$\phi_v = 1.00$$ LRFD BDS 6.5.4.2

In accordance with MBE Article 6A.6.9.3, the non-composite compression flange (top flange) is considered adequately braced. In accordance with MBE Article 6A.6.9.5, the cross-section provisions of LRFD BDS Article 6.10.2 need not be considered for existing structures. In accordance with MBE Article 6A.6.9.1, flange lateral stresses, f_l, need not be considered.

9.3.6.2.1 Flexural Capacity (Strength-I Limit)

Determine if LRFD BDS Appendix A6 is applicable:

Clear distance between flanges:

$$D = d - 2t_f = 26.7 - 2(0.640) = 25.42 \text{ inches}$$

Depth of the web in compression in the elastic range (non-composite section):

$$D_c = \frac{D}{2} = \frac{25.42}{2} = 12.71 \text{ inches}$$

Compute moments of inertia for the tension (bottom) and compression (top) flanges about the vertical axis (beam is doubly-symmetric):

$$I_{yc} = \frac{t_f b_f^3}{12} = 53.33 \text{ inch}^4$$

$$I_{yt} = I_{yc} = 53.33 \text{ inch}^4$$

Check the non-compact slenderness limit in accordance with LRFD BDS Equation A6.1-1:

$$\frac{2D_c}{t_w} = \frac{2(12.71)}{0.460} = 55.26 < 5.7\sqrt{\frac{E}{F_y}} = 5.7\sqrt{\frac{29{,}000}{50}} = 137.27 \therefore \text{ OK}$$

Check the flange ratio in accordance with LRFD BDS Equation A6.1-2:

$$\frac{I_{yc}}{I_{yt}} = \frac{53.33}{53.33} = 1.0 > 0.3 \therefore \text{ OK}$$

Since the yield strength of the steel is less than 70 ksi, the non-composite slenderness limit is met (LRFD BDS Equation A6.1-1), and the flange ratio is met (LRFD BDS Equation A6.1-2), Appendix A6 will be used.

Calculate the yield moment:

$$M_y = S_x F_y = 213(50) = 10,650 \text{ kip-inch}$$

Calculate the plastic moment in accordance with LRFD BDS Appendix D6.1:

$$D_{cp} = \frac{D}{2} = 12.71 \text{ inches}$$

$$\frac{2D_{cp}}{t_w} = \frac{2(12.71)}{0.460} = 55.26$$

$$\overline{Y} = \frac{D}{2} = 12.71 \text{ inches}$$

$$P_w = F_y D t_w = 50(25.42)(0.460) = 584.66 \text{ kip}$$

$$d_w = 0 \text{ inches}$$

$$P_t = P_c = F_y b_f t_f = 50(10.0)(0.640) = 320 \text{ kip}$$

$$d_t = d_c = \overline{Y} + \frac{t_f}{2} = 12.71 + \frac{0.640}{2} = 13.03 \text{ inches}$$

$$\begin{aligned} M_p &= \frac{P_w}{2D}\left[\overline{Y}^2 + (D-\overline{Y})^2\right] + P_t d_t + P_c d_c \\ &= \frac{584.66}{2(25.42)}\left[12.71^2 + (25.42-12.71)^2\right] + 320(13.03) + 320(13.03) \\ &= 12,055 \text{ kip-inch} \end{aligned}$$

The section is homogeneous, therefore in accordance with LRFD BDS Article 6.10.1.10.1:

$$R_h = 1.0$$

Check the compactness of the web in accordance with LRFD BDS Appendix A6.2.1:

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29,000}{50}} = 137.27$$ LRFD BDS Eq. A6.2.1-3

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} = \frac{\sqrt{\frac{29,000}{50}}}{\left(0.54\frac{12,055}{1.0(10,650)} - 0.09\right)^2}$$ LRFD BDS Eq. A6.2.1-2

$$= 88.65 \le \lambda_{rw}\frac{D_{cp}}{D_c} = 137.27\frac{12.71}{12.71} = 137.27$$

$$\frac{2D_{cp}}{t_w} = 55.26 < 88.65 \therefore \text{ compact web section} \qquad \text{LRFD BDS Eq. A6.2.1-1}$$

Since it qualifies as a compact web section in accordance with LRFD BDS Appendix A6.2.1, the web plastification factor can be taken as:

$$R_p = \frac{M_p}{M_y} = \frac{12,055}{10,650} = 1.13 \qquad \text{LRFD BDS Eq. A6.2.1-4 \& 5}$$

Compute the factored moment resistance for the compression flange:

$$M_{r\_c} = \phi_f R_p M_y = 1.0(1.13)(10,650) = 12,055 \text{ kip-inch} \qquad \text{LRFD BDS Eq. A6.1.3-1}$$

9.3.6.2.2 Permanent Deformations (Service-II Limit)

In accordance with LRFD BDS Article 6.10.4.2.2, stresses in both flanges shall not exceed the following:

$$f_f \le 0.80 R_h F_{yf} = 0.80(1.0)(50) = 40.0 \text{ ksi} \qquad \text{LRFD BDS Eq. 6.10.4.2.2-3}$$

Additionally, in accordance with LRFD BDS Article 6.10.4.2.2, compression flange stress shall not exceed F_{crw} in accordance with Article 6.10.1.9:

$$f_c \le F_{crw} \qquad \text{LRFD BDS Eq. 6.10.4.2.2-4}$$

$$k = \frac{9}{\left(\frac{D_c}{D}\right)^2} = \frac{9}{\left(\frac{12.71}{25.42}\right)^2} = 36.0 \qquad \text{LRFD BDS Eq. 6.10.1.9.1-2}$$

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} = \frac{0.9(29,000)(36.0)}{\left(\frac{25.42}{0.460}\right)^2} = 307.69 \text{ ksi} \qquad \text{LRFD BDS Eq. 6.10.1.9.1-1}$$

$$\le \frac{F_{yw}}{0.7} = \frac{50}{0.7} = 71.4 \text{ ksi} \le R_h F_{yc} = 1.0(50) = 50 \text{ ksi}$$

Therefore, the resisting stress for Service-II is:

$$F_{r\_svc2} = \min\left(0.80 R_h F_{yf},\ F_{crw}\right) = \min\left(40.0,\ 50.0\right) = 40.0 \text{ ksi}$$

9.3.6.2.3 Shear Capacity

The diaphragms are spaced greater than 1.5D at the bearings (end panel) and no additional transverse stiffeners are provided for the wide flange beam; therefore, in accordance with LRFD BDS Article 6.10.9.1, the web is unstiffened and the resistance will be calculated in accordance with LRFD BDS Article 6.10.9.2.

$k = 5.0$ (unstiffened) LRFD BDS 6.10.9.2

$$V_p = 0.58F_y D t_w = 0.58(50)(25.42)(0.460) = 339.1 \text{ kip}$$ LRFD BDS Eq. 6.10.9.2-2

$$\frac{D}{t_w} = \frac{25.42}{0.460} = 55.26$$

LRFD BDS 6.10.9.3.2

$$< 1.12\sqrt{\frac{Ek}{F_y}} = 1.12\sqrt{\frac{29,000(5.0)}{50}} = 60.31$$

Therefore:

$C = 1.0$ LRFD BDS Eq. 6.10.9.3.2-4

$$V_n = V_{cr} = CV_p = 1.0(339.1) = 339.1 \text{ kip}$$

Therefore, the factored shear resistance is:

$$V_r = \phi_v V_n = 1.0(339.1) = 339.1 \text{ kip}$$

9.3.7 LRFR Results

9.3.7.1 Concrete Slab

The structural condition of the slab is satisfactory and the system factor falls under the category for "All Other Girder Bridges and Slab Bridges." Therefore:

Condition factor:	ϕ_c	=	1.00	MBE Table 6A.4.2.3-1
System factor:	ϕ_s	=	1.00	MBE Table 6A.4.2.4-1

The equation for calculating the rating factor is based on MBE Equation 6A.4.2.1-1, which has been simplified for the load types being applied. The component for permanent loads other than dead loads, *P*, has been removed.

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

For the Strength Limit State:

$$C = \phi_c \phi_s \phi R_n$$ MBE Eq. 6A.4.2.1-2

Table 27 shows the results of the overall capacity based on MBE Equation 6A.4.2.1-2

Table 27. Moment capacity, $\phi_c\phi_s\phi_f M_n$.

Section	**Section 1**	**Section 2**
Description	Bottom of Slab at Mid-span	Top of Slab Over Support (Invert Girder)
Moment (kip-inch)	290.10	236.64

Load factors are taken from MBE Table 6A.4.2.2-1 (for reinforced concrete, only Strength-I applies). The factor for legal loads is dependent on the ADTT level in MBE Table 6A4.4.2.3a-1, which was updated in the 2013 interim.

DC	γ_{DC}	=	1.25	MBE Table B6A-1
DW	γ_{DW}	=	1.50	MBE Table B6A-1
HL-93 LL (Inventory)	$\gamma_{LL,inv}$	=	1.75	MBE Table B6A-1
HL-93 LL (Operating)	$\gamma_{LL,opr}$	=	1.35	MBE Table B6A-1
Routine Commercial Traffic	$\gamma_{LL,legal}$	=	1.45	MBE Table 6A.4.4.2.3a-1

Table 28 shows the results for rating factors based on MBE Equation 6A.4.2.1-1 as discussed above:

Table 28. LRFR rating factors for concrete slab moment at critical sections.

Rating Vehicle	**Rating Level**	**Section 1**	**Section 2**
HL-93	Inventory	0.79	0.61
HL-93	Operating	1.02	0.79
Type 3	Legal	1.40	1.13
Type 3S2	Legal	1.48	1.08
Type 3-3	Legal	1.70	1.40

Since the HL-93 design truck rating factor is less than 1.0 for Inventory and Operating levels, a second level evaluation is warranted at the Legal level in accordance with MBE Article 6A.4.3 and Article 6A.4.4.1. The AASHTO legal trucks are shown for this example, but the notional rating load (NRL) should also be evaluated at the legal level in accordance with MBE Article 6A.4.4.2.1-b. If the NRL produces a legal rating less than 1.0, then the individual SU4 - SU7 trucks should also be evaluated. If all legal ratings are greater than 1.0, then no posting would be required.

9.3.7.2 Steel Invert Girder

The structural condition of the steel invert girder is satisfactory and the system factor falls under the category for "All Other Girder Bridges and Slab Bridges." Therefore:

Condition factor:	ϕ_c	=	1.00	MBE Table 6A.4.2.3-1
System factor:	ϕ_s	=	1.00	MBE Table 6A.4.2.4-1

The equation for calculating the rating factor is based on MBE Equation 6A.4.2.1-1, which has been simplified for the load types being applied. The component for permanent loads other than dead loads, *P*, has been removed.

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

For the Strength Limit State:

$$C = \phi_c \phi_s \phi R_n$$ MBE Eq. 6A.4.2.1-2

For the Service Limit State:

$$C = f_R$$ MBE Eq. 6A.4.2.1-4

The following shows the results of the overall capacity based on MBE Equation 6A.4.2.1-2:

Moment capacity (section is non-composite and symmetric):

$$C_m = \phi_c \phi_s M_{r\_c} = 12{,}055 \text{ kip-inch}$$

Shear capacity:

$$C_v = \phi_c \phi_s V_r = 339.1 \text{ kip}$$

Service-II stress capacity:

$$C_f = \phi_c \phi_s F_{r\_svc2} = 40 \text{ ksi}$$

Load factors are taken from MBE Table 6A.4.2.2-1 (for steel, Strength-I and Service-II applies). The factor for legal loads for Strength-I is dependent on the ADTT level in MBE Table 6A4.4.2.3a-1, which was updated in the 2013 interim.

Strength-I:

DC	γ_{DC}	=	1.25	MBE Table B6A-1
DW	γ_{DW}	=	1.50	MBE Table B6A-1
HL-93 LL (Inventory)	$\gamma_{LL,inv}$	=	1.75	MBE Table B6A-1
HL-93 LL (Operating)	$\gamma_{LL,opr}$	=	1.35	MBE Table B6A-1
Routine Commercial Traffic	$\gamma_{LL,legal}$	=	1.45	MBE Table 6A.4.4.2.3a-1

Service-II:

DC	γ_{DC}	=	1.00	MBE Table B6A-1
DW	γ_{DW}	=	1.00	MBE Table B6A-1
HL-93 LL (Inventory)	$\gamma_{LL,inv}$	=	1.30	MBE Table B6A-1
HL-93 LL (Operating)	$\gamma_{LL,opr}$	=	1.00	MBE Table B6A-1
Routine Commercial Traffic	$\gamma_{LL,legal}$	=	1.30	MBE Table B6A-1

Table 29 shows the results for rating factors based on MBE Equation 6A.4.2.1-1 as discussed above:

Table 29. LRFR rating factors for steel invert girder at critical section.

Rating Vehicle	Rating Level	Strength-I Moment	Strength-I Shear	Service-II Flange Stress
HL-93	Inventory	0.62	1.96	0.56
HL-93	Operating	0.80	2.54	0.73
Type 3	Legal	1.25	3.96	0.94
Type 3S2	Legal	1.36	4.34	1.03
Type 3-3	Legal	1.51	4.81	1.15

The legal ratings are below 1.0; posting evaluation is necessary.

9.3.8 Posting Evaluation

Since the HL-93 Inventory and Operating ratings are less than 1.0, the legal ratings need to be reviewed for possible posting requirements. The Type 3 legal rating is below 1.0, while the Type 3S2 and 3-3 units are at or above 1.0. Therefore, a posting analysis will be performed for the Type 3 truck as follows:

Calculate the gross vehicle weight of the Type 3 truck:

$$w = 16 + 17 + 17 = 50.0 \text{ kip}$$

The controlling Type 3 truck rating factor is:

$$RF_{type3} = 0.94$$

Calculate the posting load:

$$Posting_{load} = \frac{w}{0.7}(RF_{type3} - 0.3) = 45.71 \text{ kip}$$ MBE Eq. 6A.8.3-1

Rounding the results down to the nearest ton gives:

$$Posting_{Ton} = \frac{45.71}{2 \text{ kip/ton}} = 22.9 \text{ tons} \rightarrow 22 \text{ tons}$$

Therefore, this structure should be load posted for the Type 3 truck at 22 tons.

Note that AASHTO SHVs and State legal loads (if applicable) are not considered in this example, simply because the example is only to demonstrate the load rating procedure, and it is not a complete load rating package.

9.3.9 LFR Results

9.3.9.1 Concrete Slab

LFR rating methodology for the concrete slab is presented below based on the following:

- Dead load analysis and unfactored loads would be determined using the same process as for LRFR.
- DW loads are included in the overall dead loads.
- The design truck for live load is designated as HS-20 truck for LFR instead of the HL-93 used for LRFR, but the truck axle configurations are the same.
- AASHTO Standard Specifications Article 3.24.3.2 – Case B for Main Reinforcement Parallel to Traffic will be used for live load distribution. Unlike LRFR, the lane load is not explicitly excluded from application to deck slab analysis. Article 3.24.3.2 specifies moments in continuous spans shall be determined by suitable analysis using the truck or appropriate lane loading. However, for this example it has been verified that the lane loading does not control for the loading of the concrete slab when modeled as a continuous span as required.
- The multiple presence factor for LRFR is replaced by Standard Specifications Article 3.12. For one or two lanes, 100 percent of live load effect is used.
- The dynamic load (Impact) is also different in LFR and uses Standard Specifications Equation 3-1 in Article 3.8.2.1.

9.3.9.1.1 Loads

Dead loads from LRFR DC and DW cases are combined into dead load for LFR:

$$DL = q_{slab\_DC1} + q_{rail\_DC2} + q_{panel\_DC2} + q_{DW} = 0.125 + 0.031 + 0.094 + 0.020 = 0.270 \text{ klf}$$

Live load for LFR is the HS-20 truck, which is the same as the truck portion from LRFR:

$$HS20 = HL93_{trk\_axle} = \begin{bmatrix} 8.00 & 32.00 & 32.00 \end{bmatrix} \text{ kip}$$

Dynamic load (Impact) for LFR is based on the Standard Specifications Equation 3-1 in Article 3.8.2.1:

$$L = S = S_{FB\_spa} = 12.00 \text{ ft}$$

$$I = \min\left(\frac{50}{L+125}, 30\%\right) = \min\left(\frac{50}{12+125}, 30\%\right) = 30\% \qquad \text{Standard Specifications Eq. 3-1}$$

Live Load distribution in accordance with Standard Specifications Article 3.24.3.2:

Wheel distribution width:

$$E_{wheel} = \min\left(4 + 0.06S,\ 7\right) = 4.72 \text{ ft}$$

Lane distribution width:

$$E_{lane} = 2E_{wheel} = 2(4.72) = 9.44 \text{ ft}$$

The lane distribution width is used for both the positive and negative moment regions for LFR, whereas LRFR used two different distribution widths.

HS-20 Live Load:

HS-20 truck equivalent load on 1-foot-wide analysis strip:

$$HS20_{trk\_axle} = \begin{bmatrix} 8 & 32 & 32 \end{bmatrix} \text{ kip}$$

$$P_{HS20} = \frac{HS20_{trk\_axle}}{E_{lane}} = \frac{\begin{bmatrix} 8 & 32 & 32 \end{bmatrix}}{9.44} = \begin{bmatrix} 0.847 & 3.390 & 3.390 \end{bmatrix} \text{ kip}$$

AASHTO Legal Trucks:

Type 3 equivalent load on 1-foot-wide analysis strip:

$$Type3_{axle} = \begin{bmatrix} 16 & 17 & 17 \end{bmatrix} \text{ kip}$$

$$P_{Type3} = \frac{Type3_{axle}}{E_{lane}} = \frac{\begin{bmatrix} 16 & 17 & 17 \end{bmatrix}}{9.44} = \begin{bmatrix} 1.695 & 1.801 & 1.801 \end{bmatrix} \text{ kip}$$

Type 3S2 equivalent load on 1-foot-wide analysis strip:

$$Type3S2_{axle} = \begin{bmatrix} 10 & 15.5 & 15.5 & 15.5 & 15.5 \end{bmatrix} \text{ kip}$$

$$P_{Type3S2} = \frac{Type3S2_{axle}}{E_{lane}} = \frac{\begin{bmatrix} 10 & 15.5 & 15.5 & 15.5 & 15.5 \end{bmatrix}}{9.44}$$
$$= \begin{bmatrix} 1.059 & 1.642 & 1.642 & 1.642 & 1.642 \end{bmatrix} \text{ kip}$$

Type 3-3 equivalent load on 1-foot-wide analysis strip:

$$Type3\_3_{axle} = \begin{bmatrix} 12 & 12 & 12 & 16 & 14 & 14 \end{bmatrix} \text{ kip}$$

$$P_{Type3\_3} = \frac{Type3\_3_{axle}}{E_{lane}} = \frac{\begin{bmatrix} 12 & 12 & 12 & 16 & 14 & 14 \end{bmatrix}}{9.44}$$
$$= \begin{bmatrix} 1.271 & 1.271 & 1.271 & 1.695 & 1.483 & 1.483 \end{bmatrix} \text{ kip}$$

Table 30 presents the analysis results for unfactored moments for the HS-20 and legal live loads. The unfactored live load force effects incorporate the multiple presence factors (MPF = 1.00) and the dynamic load allowance (I = 30 percent).

Table 30. Analysis results for slab moment (kip-inch) at critical sections.

Section	**1**	**2**
Description	Bottom Slab at Mid-span	Top Slab Over Support (Invert Girder)
DC	33.62	-45.53
DW	2.69	-3.65
LL+I (HS-20)	133.47	-99.17
LL+I (Type 3)	90.90	-63.96
LL+I (Type 3S2)	85.80	-67.44
LL+I (Type 3-3)	74.86	-51.78

9.3.9.1.2 Section 1 – Bottom of Slab at Mid-span Flexural Resistance

See Section 9.3.1 for critical section data including depth, width, material properties, and reinforcement details.

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.44(12.00)}{8} = 0.66 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 10.00 - \frac{0.750}{2} - 1.00 = 8.625 \text{ inches}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.66(60)}{0.85(4.0)(12.00)} = 0.97 \text{ inches}$$ Standard Spec. Eq. 8-17

$$\phi_f = 0.90$$ Standard Spec. 8.16.1.2.2

$$\phi M_{n\_bott1} = \phi_f A_s f_y \left(d - \frac{a}{2} \right)$$

$$= 0.90(0.66)(60)\left(8.625 - \frac{0.97}{2} \right) = 290.1 \text{ kip-inch}$$ Standard Spec. Eq. 8-16

Note, the nominal moment resistance on the top reinforcement is not critical at mid-span and need not be checked.

Check maximum reinforcement requirement:

$$\beta_1 = 0.85$$ Standard Spec. 8.16.2.7

$$\rho_b = \frac{0.85\beta_1 f'_c}{f_y}\left(\frac{87{,}000}{87{,}000+f_y}\right)$$

Standard Spec. Eq. 8-18

$$= \frac{0.85(0.85)(4{,}000)}{60{,}000}\left(\frac{87{,}000}{87{,}000+60{,}000}\right) = 0.0285$$

$$\rho = \frac{A_s}{bd_s} = \frac{0.66}{12.00(8.625)} = 0.0064$$

Standard Spec. 8.1.2

$$\rho = 0.0064 < 75\%\rho_b = 0.0214 \therefore \text{OK}$$

Standard Spec. 8.16.3.1.1

Maximum reinforcement requirements at this section are satisfied.

Check minimum reinforcement requirement:

Gross moment of inertia:

$$I_g = \frac{bh^3}{12} = \frac{12.00(10.00)^3}{12} = 1000 \text{ inch}^4$$

Standard Spec. 8.1.2

Distance from centroidal axis of gross section to extreme fiber in tension:

$$y_t = \frac{h}{2} = \frac{10.00}{2} = 5.00 \text{ inches}$$

Standard Spec. 8.1.2

$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{4{,}000} = 474 \text{ psi}$$

Standard Spec. 8.15.2.1.1

Cracking moment:

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{474(1000)}{5.00} = 94.9 \text{ kip-inch}$$

Standard Spec. Eq. 8-2

$$\phi M_{n\_bott1} = 290.1 \text{ kip-inch} > 1.2M_{cr} = 1.2(94.9) = 113.9 \text{ kip-inch} \therefore \text{ OK}$$

Minimum reinforcement requirements at this section are satisfied.

9.3.9.1.3 Section 2 – Top of Slab over Support Flexural Resistance

See Section 9.3.1 for critical section data including depth, width, material properties, and reinforcement details.

$$A_s = \frac{A_{s\_bar}b}{s} = \frac{0.44(12.00)}{8} = 0.66 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 10.00 - \frac{0.750}{2} - 2.500 = 7.125 \text{ inches}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.66(60)}{0.85(4.0)(12.00)} = 0.97 \text{ inches}$$

Standard Spec. Eq. 8-17

$\phi_f = 0.90$ Standard Spec. 8.16.1.2.2

$$\phi M_{n\_top2} = \phi_f A_s f_y \left(d - \frac{a}{2} \right)$$

Standard Spec. Eq. 8-16

$$= 0.90(0.66)(60)\left(7.125 - \frac{0.97}{2} \right) = 236.6 \text{ kip-inch}$$

Note, the nominal moment resistance on the top reinforcement is not critical at mid-span and need not be checked.

Check maximum reinforcement requirement:

$\beta_1 = 0.85$ Standard Spec. 8.16.2.7

$$\rho_b = \frac{0.85\beta_1 f'_c}{f_y}\left(\frac{87,000}{87,000 + f_y} \right)$$

Standard Spec. Eq. 8-18

$$= \frac{0.85(0.85)(4,000)}{60,000}\left(\frac{87,000}{87,000 + 60,000} \right) = 0.0285$$

$$\rho = \frac{A_s}{bd_s} = \frac{0.66}{12.00(7.125)} = 0.0077$$

Standard Spec. 8.1.2

$\rho = 0.0077 < 75\%\rho_b = 0.0214 \therefore \text{OK}$ Standard Spec. 8.16.3.1.1

Maximum reinforcement requirements at this section are satisfied.

Check minimum reinforcement requirement:

Gross moment of inertia:

$$I_g = \frac{bh^3}{12} = \frac{12.00(10.00)^3}{12} = 1000 \text{ inch}^4$$

Standard Spec. 8.1.2

Distance from centroidal axis of gross section to extreme fiber in tension:

$$y_t = \frac{h}{2} = \frac{10.00}{2} = 5.00 \text{ inches}$$

Standard Spec. 8.1.2

$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{4,000} = 474 \text{ psi}$$

Standard Spec. 8.15.2.1.1

Cracking moment:

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{474(1000)}{5.00} = 94.9 \text{ kip-inch}$$

Standard Spec. Eq. 8-2

$$\phi M_{n\_top2} = 236.6 \text{ kip-inch} > 1.2M_{cr} = 1.2(94.9) = 113.9 \text{ kip-inch} \;\therefore\; \text{OK}$$

Minimum reinforcement requirements at this section are satisfied.

9.3.9.1.4 Rating Factors

The equation for calculating the rating factor is taken from Article 6B.4.1 of the BME:

$$RF = \frac{C - A_1 D}{A_2 L(1+I)} \qquad \text{MBE Eq. 6B.4.1-1}$$

The load factors are taken from MBE Article 6B.4.3:

Dead Load	A_1	=	1.3
Live load at inventory	$A_{2,inv}$	=	2.17
Live load at operating	$A_{2,opr}$	=	1.3

The capacity at these sections are taken as:

$$C_{bott1} = \phi M_{n\_bott1} = 290.10 \text{ kip-inch}$$

$$C_{top2} = \phi M_{n\_top2} = 236.64 \text{ kip-inch}$$

The unfactored loads from Table 30:

$$D_1 = DC + DW = 36.31 \text{ kip-inch}$$

$$D_2 = DC + DW = -49.18 \text{ kip-inch}$$

Utilizing the live loads from above for each vehicle at each section. MBE Equation 6B.4.1-1 can be applied to determine rating factors shown below in Table 31.

Table 31. Rating factors for slab sections.

Rating Vehicle	Rating Level	Section 1	Section 2
HS-20	Inventory	0.84	0.80
HS-20	Operating	1.40	1.34
Type 3	Inventory	1.23	1.24
Type 3	Operating	2.06	2.08
Type 3S2	Inventory	1.30	1.18
Type 3S2	Operating	2.18	1.97
Type 3-3	Inventory	1.50	1.54
Type 3-3	Operating	2.50	2.57

9.3.9.2 Steel Invert Girder

The LFR rating methodology is similar to the concrete slab; see Section 9.3.2 for details. See Section 9.3.1 for the dimensions and section properties of the W27x84 invert girder.

9.3.9.2.1 Loads

The dead loads are calculated in the same fashion as was performed in Section 9.3.2 and 9.3.3 of the calculations for LRFR. The live loads are calculated in the same fashion as was performed in Section 9.3.4 of the calculations for LRFR. Standard Specifications Article 3.23.3 dictates the live load distribution for the invert girders. The applicable equation from Standard Specifications Table 3.23.3.1 is outside the range of applicability (S is greater than the denominator in the distribution equation); therefore the lever rule is applied for LFR, just as in LRFR.

The following are the resulting reactions to the support invert girder, expressed as wheel loads for each rating vehicle. These values were determined by simple static analysis using the lever rule. MBE Appendix D6B can also be used to determine the wheel reactions at the invert girder.

Dynamic load (Impact) for LFR is based on the Standard Specifications Equation 3-1 in Article 3.8.2.1. Article 3.8.2.2(b) specifies the length of member (floor beam/invert girder) for use in loaded length, *L*:

$$L = S_{FB\_span} = 31.00 \text{ ft}$$

$$I = \min\left(\frac{50}{L+125}, 30\%\right) = \min\left(\frac{50}{31+125}, 30\%\right) = 30\% \qquad \text{Standard Specifications Eq. 3-1}$$

HS-20 truck wheel load:

$$whl_{HS20} = \frac{Axle_{HS20}}{2} = \frac{32.00}{2} = 16.00 \text{ kip wheel load}$$

Design lane load for simplicity is broken out into equivalent wheels; 1 wheel = 1/2 lane). The maximum lane load for shear (V) includes a 26 kip concentrated load while for moment (M) an 18 kip concentrated load is included along with the 640 plf uniform loading:

$$whl_{laneV} = \frac{Lane_V}{2} = \frac{33.68}{2} = 16.84 \text{ kip wheel load}$$

$$whl_{laneM} = \frac{Lane_M}{2} = \frac{25.68}{2} = 12.84 \text{ kip wheel load}$$

Single wheel load reactions:

$$whl_{Type3} = \frac{Axle_{Type3}}{2} = \frac{28.33}{2} = 14.16 \text{ kip wheel load}$$

$$whl_{Type3S2} = \frac{Axle_{Type3S2}}{2} = \frac{26.68}{2} = 13.34 \text{ kip wheel load}$$

$$whl_{Type3\_3} = \frac{Axle_{Type3\_3}}{2} = \frac{23.33}{2} = 11.66 \text{ kip wheel load}$$

The analysis results for forces acting on the steel invert girder are presented in Table 32:

Table 32. Maximum moment and shear (I=30 percent).

Load	Moment (kip-inch)	Shear (kip)	Flange Stress (ksi)
DC1	2295.44	24.86	10.78
DC2	0.00	24.00	0.00
DW	399.84	3.92	1.88
Lane Load	4206.38	49.43	19.75
HS-20 Truck	5241.60	46.97	24.61
Type 3	4640.45	41.58	21.79
Type 3S2	4233.12	37.93	19.87
Type 3-3	3821.45	34.24	17.94

9.3.9.2.2 Flexural Capacity

Standard Specifications Article 10.48 lists some required and preferred limits for a fabricated beam. Since this invert girder is a rolled section, the required/preferred limits do not apply.

Check Standard Specifications Article 10.48.1.1 items (a) through (d) to determine if section is compact:

The clear distance between flanges:

$$D = d - 2t_f = 26.70 - 2(0.64) = 25.42 \text{ inches}$$

$$\frac{b_f}{t_f} = \frac{10.00}{0.64} = 15.63 \le \frac{4,110}{\sqrt{F_y}} = \frac{4,100}{\sqrt{50,000}} = 18.38$$ Standard Specifications Eq. 10-93

$$\frac{D}{t_w} = \frac{25.42}{0.46} = 55.26 \le \frac{19,230}{\sqrt{F_y}} = \frac{19,230}{\sqrt{50,000}} = 86.00$$ Standard Specifications Eq. 10-94

Both *b*/*t* and D/t_w do not exceed 75 percent of the above limits, therefore Standard Specifications Equation 10-95 does not apply. The invert girder is in positive bending only and the top flange is embedded in the concrete deck, therefore the compression flange lateral bracing limit of Equation 10-96 does not apply. Finally, the invert girder is not subjected to axial load, therefore Equation 10-97 does not apply. Therefore, the section is compact in accordance with Article 10.48.1 and the flexural strength is in accordance with Equation 10-92:

$$M_u = F_y Z = (50)(244) = 12,200 \text{ kip-inch}$$ Standard Specifications Eq. 10-92

9.3.9.2.3 Overload (Serviceability)

In accordance with Standard Specifications Article 10.57.1, the maximum overload stress shall not exceed $0.8F_y$:

$$0.8F_y = 0.8(50) = 40.0 \text{ ksi}$$

Article 10.57 requires the consideration of web-bending buckling for overload in accordance with Equation 10-173. The maximum overload compressive stress in the web shall not exceed:

$$k = 9\left(\frac{D}{D_c}\right)^2 = 9\left(\frac{25.42}{(25.42/2)}\right)^2 = 36.0 \qquad \text{Standard Specifications Article 10.6.1.1}$$

In accordance with Article 10.6.1.1 for members without longitudinal stiffeners:

$$\alpha = 1.3$$

$$f_b \le \frac{26,200,000\alpha k}{\left(\frac{D}{t_w}\right)^2} = \frac{26,200,000(1.3)(36.0)}{\left(\frac{25.42}{0.46}\right)^2} \qquad \text{Standard Specifications Eq. 10-173}$$

$$= 401.4 \text{ ksi} \le F_y = 50 \text{ ksi}$$

Since the invert girder is non-composite, the value of D_c is constant. Also, since the limiting stress in the stress in the web is greater than the limiting stress in the flanges, the flange stress will control the ratings. The capacity for overload stress (in the controlling, compression flange) is 40.0 ksi.

9.3.9.2.4 Shear Capacity

The shear capacity is determined by Standard Specifications Article 10.48.8. The wide-flange beam is not stiffened, therefore use Equation 10-113 and 10-115:

$$V_p = 0.58F_yDt_w = 0.58(50)(25.42)(0.46) = 339.1 \text{ kip} \qquad \text{Standard Specifications Eq. 10-115}$$

For unstiffened webs the shear buckling coefficient, k, is equal to 5.0 in accordance with Article 10.48.8.1. Check the limits of Article 10.48.8.1 to determine the value of constant C:

$$\frac{D}{t_w} = 55.26 \le \frac{6000\sqrt{k}}{\sqrt{F_y}} = \frac{6000\sqrt{5.0}}{\sqrt{50,000}} = 60.00 \therefore C = 1.0$$

Therefore, the shear capacity of the invert girder is:

$$V_u = CV_p = 1.0(339.1) = 339.1 \text{ kip} \qquad \text{Standard Specifications Eq. 10-113}$$

9.3.9.2.5 Rating Factors

The equation for calculating the rating factor is taken from Article 6B.4.1 of the BME:

$$RF = \frac{C - A_1 D}{A_2 L(1+I)}$$ MBE Eq. 6B.4.1-1

The load factors are taken from MBE Article 6B.4.3:

Dead Load	A_1	=	1.3
Live load at inventory	$A_{2,inv}$	=	2.17
Live load at operating	$A_{2,opr}$	=	1.3

The overload factors are taken from the Standard Specifications Article 10.57:

Dead Load	A_1	=	1.0
Live load at HS design load	$A_{2,inv}$	=	5/3 = 1.67
Live load at legal loads	$A_{2,opr}$	=	1.0

The capacity at these sections are taken as:

$$C_M = M_u = 12,200 \text{ kip-inch}$$

$$C_{OL} = 40.0 \text{ ksi}$$

$$C_v = V_u = 339.1 \text{ kip}$$

The dead and live loads for rating are summarized in Table 33:

Table 33. Maximum moment, shear and flange stress (I = 30 percent).

Load	**Moment** (kip-inch)	**Shear** (kip)	**Flange Stress** (ksi)
D = DC+DW	2695	52.8	12.7
HS-20 (Max of Truck/Lane)	5242	49.4	24.6
Type 3	4640	41.6	21.8
Type 3S2	4233	37.9	19.9
Type 3-3	3821	34.2	17.9

MBE Equation 6B.4.1-1 can be applied to determine rating factors shown in Table 34.

Table 34. LFR Inventory and Operating rating factors.

Rating Vehicle	Rating Level	Moment	Shear	Flange Stress
HS-20	Inventory	0.76	2.52	0.67
HS-20	Operating	1.28	4.21	1.11
Type 3	Inventory	0.86	3.00	0.75
Type 3	Operating	1.44	5.00	1.26
Type 3S2	Inventory	0.95	3.28	0.83
Type 3S2	Operating	1.58	5.46	1.38
Type 3-3	Inventory	1.05	3.64	0.91
Type 3-3	Operating	1.75	6.08	1.52

Since all Operating rating factors for the considered legal loads are greater than 1.0, no load posting is required for this tunnel to restrict the AASHTO legal loads Type 3, 3S2 and 3-3.

9.4 EXAMPLE 3 – REINFORCED CONCRETE BOX TUNNEL

Example 3 presents a single barrel box tunnel supported by walls and a bottom slab. The roadway is supported by the bottom slab, or a slab-on-grade (see Section 2.4.1.3). The tunnel is subjected to vertical dead loads, earth loads and live loads as well as lateral earth and live load surcharge. The box tunnel is rated with the LRFR method for the design vehicles (HL-93 Inventory and Operating) and required legal and emergency vehicles (AASHTO Type 3, AASHTO Type 3S2, AASHTO Type 3-3, NRL, EV2 and EV3) at the legal load limit. For illustrative purposes, Live Load effects and load rating are only presented for EV3 for Example 3.

This example will perform the following steps to rate this reinforced concrete box tunnel:

1. Structure data
2. Example notes
3. Rating approach/assumptions
4. Load application
5. Structural analysis
6. Resistance calculations
7. LRFR rating calculations
8. LFR rating calculations

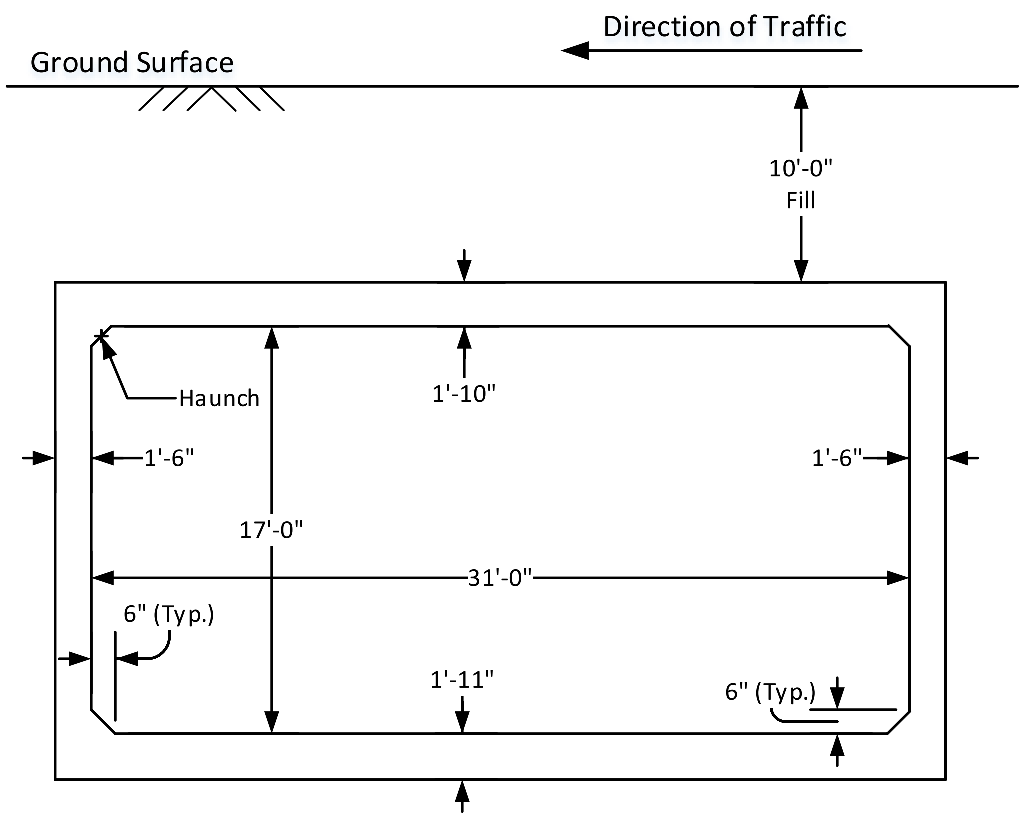
9.4.1 Structure Data

9.4.1.1 Materials

Materials are known, otherwise use MBE Article 6A5.2.1 and 6A5.2.2. Soil parameters were randomly selected for the example. Full soil descriptions and evaluation should be performed for actual tunnel ratings so accurate soil parameters can be obtained. This example also assumed dry soil. Water table elevation, water pressure and buoyancy shall be considered for actual tunnel ratings as these forces may induce additional moments into the bottom slab and reduce lateral moments into the exterior walls.

Concrete:	f'_c	=	4.0 ksi
Reinforcing Steel:	f_y	=	60.0 ksi
Soil:	γ_{soil}	=	0.117 kcf
	ϕ_{soil}	=	32°

9.4.1.2 Dimensions



Source: FWHA

Figure 73. Illustration. Cross-section showing invert slab and integral structure.

Top slab thickness:	t_{ts}	=	22 inches
Bottom slab thickness:	t_{bs}	=	23 inches
Wall thickness:	t_{wall}	=	18 inches

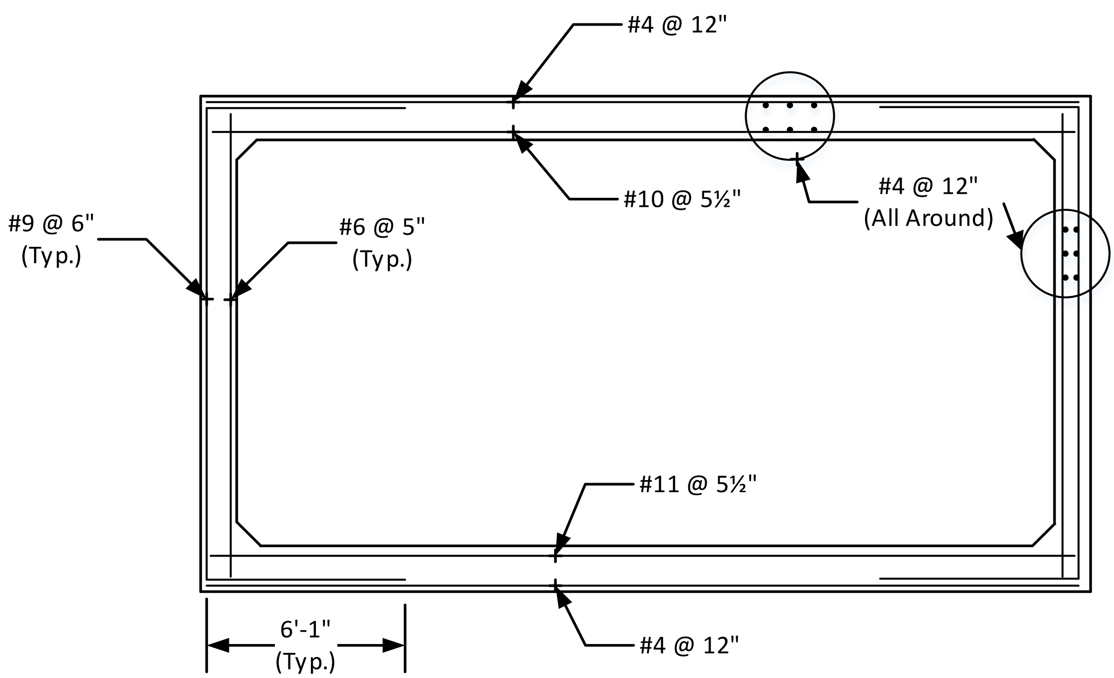
Haunch size:	t_{haunch}	=	6 inches
Width of tunnel opening:	w_{tunnel}	=	31.00 ft
Height of tunnel opening:	h_{tunnel}	=	17.00 ft
Fill depth:	h_{fill}	=	10.00 ft

9.4.2 Example Notes

- This cross-section is fictitious and only for the purpose of demonstrating the principles of rating box tunnels.
- Section for this box tunnel is a cast-in-place reinforced concrete, box tunnel that behaves as a box frame.
- Internal live loading on the box tunnel is only applicable to the bottom slab (traffic is driving directly on the bottom slab). Application of this live load will put the slab into opposite flexure as the dead load and soil load reaction forces. Therefore, worst case scenario does not include internal tunnel live load acting down on the bottom slab.
- Live loading over the box tunnel will increase force affects and is therefore included in the rating of the box culvert.
- If roadway has additional barrier, overlay, or sidewalk loads, they should be included in the dead load analysis. This example assumes that these items are not included in the geometry of this box tunnel.
- While not assumed here, any other mechanical (including dynamic equipment loads) and architectural systems supported by the tunnel should be included in the dead load.
- Focus of this example is LRFR.
- This example focuses on the load rating of an interior section of a reinforced concrete box tunnel. A section taken near the ends of the tunnel would need to account for increases in load effects due to reduced live load distribution near the edge.
- The rating Engineer should evaluate the existing record drawings to determine if the rebar lap lengths and development lengths meet current design code requirements. This example is based on the assumption that rebar lap lengths and development lengths meet current design code requirements. In instances where development and lap length deficiencies are present, a reduced rebar area proportional to the percent developed should be used within the portion of the bar effected by the deficiency.
- The rating Engineer should review the record drawings and inspection reports carefully to properly identify the support condition (pinned, expansion, fixed). Typical conditions, and assumed for this example, is a vertically restrained bottom slab and a laterally restrained exterior wall. This is achieved by providing a laterally restrained roller at the top of the wall.
- Rating is performed only at maximum shear and moment locations. Typical ratings include evaluation at all tenth points.

9.4.3 Rating Approach/Assumptions:

- LRFR evaluation is performed for a 1-foot-wide strip (parallel to direction of traffic above the box tunnel).
- Pavement has approximately the same unit weight as the soil and is therefore included with the soil vertical loads (that is, gravel pavement surface).
- Compacted gravel fill acts along the side of the tunnels with soil parameters of ϕ = 32° and γ = 117 pcf. Additionally, this box tunnel is in a dry soil condition.
- LRFR live load ratings are evaluated for HL-93 design loading (Truck or Tandem), AASHTO Legal Loads and the EV2 and EV3 emergency vehicles (FHWA Memo).
- Corner haunch are included for dead load but not for stiffness in the structural analysis.
- Vertical reactions on the bottom slab are uniformly distributed across the out-to-out box tunnel width.
- Horizontal side sway of the box tunnel is restrained and simulated by pinning the top of the box tunnel.
- There are no signs of distress or deterioration; therefore, the box tunnel is considered to be in satisfactory physical condition.
- There are no post/pre-tensioning forces; therefore, MBE Article 6A.2.2.2 does not apply.



Source: FHWA

Figure 74. Illustration. Reinforced concrete box tunnel reinforcing steel details.

9.4.4 Load Calculations

9.4.4.1 Dead Load Component, DC

Dead load components include the self-weight of the concrete box tunnel as well as barriers, sidewalks, railings or fixtures to the box tunnel. These loads are described in MBE Article 6A.5.12.10.1 and LRFD BDS Article 3.5.1.

Normal weight of reinforced concrete:

$$w_c = 0.145 \text{ kcf} + 0.005 \text{ kcf} = 0.150 \text{ kcf}$$ LRFD BDS Table 3.5.1-1 and Table C3.5.1

The following weights are calculated for the analysis strip width:

$$w_{strip} = 1.00 \text{ ft}$$

Top slab weight:

$$w_{top\_DC} = t_{ts} w_c w_{strip} = \left(22/12\right)(0.150)(1.00) = 0.275 \text{ klf}$$

Bottom slab weight:

$$w_{bottom\_DC} = t_{bs} w_c w_{strip} = \left(23/12\right)(0.150)(1.00) = 0.288 \text{ klf}$$

Wall weight:

$$w_{wall\_DC} = t_{wall} w_c w_{strip} = \left(18/12\right)(0.150)(1.00) = 0.225 \text{ klf}$$

Haunch weight:

$$P_{haunch\_DC} = 1/2\, t_{haunch}^2 w_c w_{strip} = \left(1/2\right)(0.5^2)\left(0.150\right)(1.00) = 0.019 \text{ kip}$$

Wall and haunch weight (one side only):

$$P_{wall\_DC} = w_{wall\_DC}(h_{wall} + t_{ts}/2 + t_{bs}/2) + 2P_{haunch}$$

$$= \left(0.225\right)\left(17 + 22/2\left(12\right) + 23/2\left(12\right)\right) + 2(0.019) = 4.285 \text{ kip}$$

The haunch weight is assumed to be part of wall load in the structural model.

9.4.4.2 Wearing Surface, DW

Wearing surface was assumed to have the same unit weight as the soil. Wearing surface loads will have to be calculated in the scenario where the unit weight is different. These calculations are outlined in LRFD BDS Article 3.5.1.

9.4.4.3 Vertical Earth Loads, EV

Vertical earth loads include the weight of the soil acting on the top of the box tunnel. These calculations are outlined in MBE Article 6A.5.12.10.2a and LRFD BDS Article 3.5.1 and Article 12.11.2.2. This rating example assumes an embankment installation resulting in the following vertical earth load calculations. In the instances where trench installations are appropriate, refer to LRFD BDS Article 12.11.2.2.1 for detailed calculation procedures.

Vertical Earth Loads:

$$B_c = 31.00\,\text{span} + 2\left(18/12\right)\text{walls} = 34.00\text{ft}$$

$$F_e = 1 + 0.20\,{}^{h_{fill}}\!/_{B_c} \le 1.15 \rightarrow 1 + 0.20\left(10/34\right) = 1.059 \le 1.15$$

$$W_E = F_e \gamma_{soil} B_c h_{fill} = 1.058(0.120)(34.00)(10.00) = 43.21\,\text{kip}$$

$$w_{soil,EV} = {}^{W_E}\!/_{B_c} = 43.21/34.00 = 1.27\,\text{klf}$$

9.4.4.4 Horizontal Earth Loads, EH

Horizontal earth loads include the lateral soil pressure acting on the walls of the box tunnel. Rigid frame behavior of the box tunnel requires an at-rest lateral earth pressure condition to be considered; therefore, at-rest lateral earth pressure coefficients can be calculated in accordance with LRFD BDS Article 3.11.5.2 or Article 3.11.5.5. Subsequently, LRFD BDS Table 3.11.5.5-1 can be used for conservative and approximate soil pressures in the case better information is not available and the backfill is free draining. These calculations are outlined in MBE Article 6A.5.12.10.2b/c and LRFD BDS Article 3.11.5. Lateral earth loads are calculated at the top slab and bottom slab working points (intersection between centerline slab and centerline wall) as follows:

Horizontal earth loads at top working point of the box tunnel:

$$k_o = 1 - \sin\phi_f^{'} = 1 - \sin(32°) = 0.470$$

$$w_{EH,\max} = k_o \gamma_{fill} h_{fill} w_{strip} = 0.470(0.117)(10.917)(1.00) = 0.60\,\text{klf}$$

Horizontal earth loads at bottom working point of the box tunnel:

$$w_{EH,\max} = k_o \gamma_{fill} h_{fill} w_{strip} = 0.470(0.117)(27.79)(1.00) = 1.638\,\text{klf}$$

As outlined later in this problem, a minimum and maximum lateral earth pressure is required to capture the force envelopes. BDS Article 3.11.7 specifies maximum lateral earth pressure is obtained by evaluating the maximum lateral earth pressure while the minimum lateral earth pressure is obtained by taking half the maximum lateral earth pressure. Therefore, the minimum lateral earth pressures are:

Horizontal earth loads at top working point of the box tunnel:

$$w_{EH,\min} = \frac{k_o \gamma_{fill} h_{fill} w_{strip}}{2} = \frac{0.470(0.117)(10.917)(1.00)}{2} = 0.30 \text{ klf}$$

Horizontal earth loads at bottom working point of the box tunnel:

$$w_{EH,\min} = \frac{k_o \gamma_{fill} h_{fill} w_{strip}}{2} = \frac{0.470(0.117)(27.79)(1.00)}{2} = 0.820 \text{ klf}$$

9.4.4.5 Live Load Application, LL

The box tunnel spans in the direction parallel to the direction of traffic. Since this structure is under more than 2 feet of fill, MBE Article 6A.5.12.10.3a and LRFD BDS Article C4.6.2.10.1 indicate that live load may be distributed to the top slab using the equations specified in LRFD BDS Article 3.6.1.2.5 and 3.6.1.2.6 for distribution of wheel load through earth fills of 2 feet or more when main reinforcing is parallel to the direction of traffic.

The equivalent wheel loading is determined as:

- *LLDF* from BDS Table 3.6.1.2.6a-1 is 1.15,
- S_w is a wheel spacing of 6 feet (BDS Article 3.6.1.2.2),
- w_t is a tire patch width of 20 inches (BDS Article 3.6.1.2.5),
- l_t is a tire patch length of 10 inches (BDS Article 3.6.1.2.5),
- D_i is the inside clear span of culvert of 31 feet (372 inches):

$$H_{\text{int}} = \frac{S_w - \frac{w_t}{12} - \frac{0.06D_i}{12}}{LLDF} = \frac{6 - \frac{20}{12} - \frac{0.06(372)}{12}}{1.15} = 2.15 \text{ ft}$$

- Since $H \geq H_{int}$:

$$w_w = \frac{w_t}{12} + S_w + LLDF\left(h_{fill}\right) + \frac{0.06D_i}{12} = \frac{20}{12} + 6 + 1.15(10) + \frac{0.06(372)}{12} = 21.03 \text{ ft}$$

- The live load distribution length needs to be considered for all axle spacing. This rating example evaluates the Inventory and Operating capacity of HL-93 and Legal Load capacities of AASHTO Type 3, AASHTO Type 3S2, AASHTO Type 3-3, Notional Rating Load (NRL), Emergency Vehicle EV2 and Emergency Vehicle EV3. Re-arranging BDS Equation 3.6.1.2.6b-4 gives us the maximum axle spacing in which the axle loads will not interact with the given fill depth:

$$s = h_{fill}(LLDF) + {l_t}/{12} = 10(1.15) + \left({10}/{12}\right) = 12.33 \text{ ft}$$

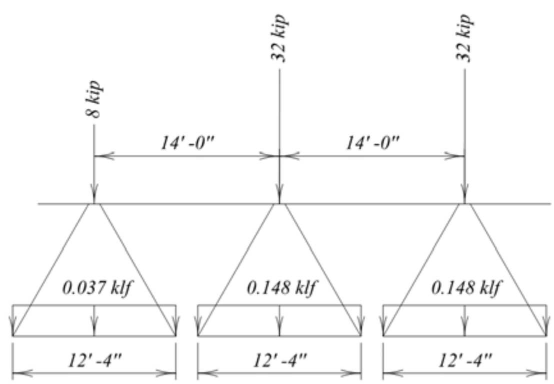
Therefore, BDS Equation 3.6.1.2.6b-5 shall be applied for a longitudinal axle spacing less than 12.33 feet while equation 3.6.1.2.6b-6 shall be applied for a longitudinal axle spacing greater than or equal to 12.33 ft. This results in the distribution lengths shown in Figure 75 through Figure 82.

- Distributing the axle load(s) over the applicable transverse and longitudinal distribution lengths in accordance with BDS Equation 3.6.1.2.6b-7 will provide the distributed loads to be applied to the box tunnel. Dynamic allowance is calculated in accordance with BDS Equation 3.6.2.2-1 and the multiple presence factor of 1.2 is used for the HL-93 Truck and HL-93 Tandem in accordance with BDS Article 3.6.1.1.2. A multiple presence factor of 1.0 is used for all legal loads including EV's in accordance with MBE Article 6A.5.12.10.3. Therefore, the dynamic allowance is:

  $$IM = 1 + 33\left(1.0 - 0.125D_E\right) \geq 1.00 \Rightarrow 1 + 0.33\left[1.0 - 0.125\left(10\right)\right] = 1.00$$

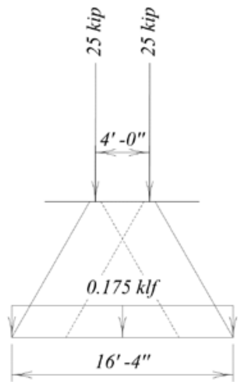
  Where D_E is the fill depth of 10ft.

Applying these multiple presence and dynamic allowance factors combined with the applicable axle loads and distribution widths results in the distributed loads shown in Figure 75 through Figure 82.



Source: FHWA

Figure 75. Illustration. Rating live loads and soil distribution – HL-93 Truck.



Source: FHWA

Figure 76. Illustration. Rating live loads and soil distribution – HL-93 Tandem.

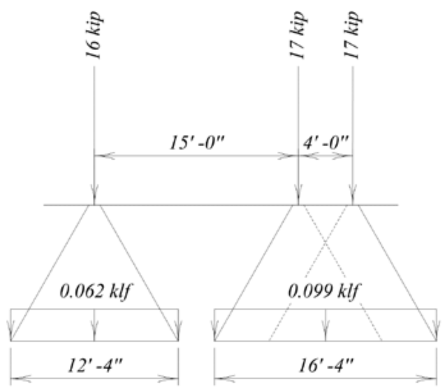


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Figure 77. Illustration. Rating live loads and soil distribution – AASHTO Type 3.

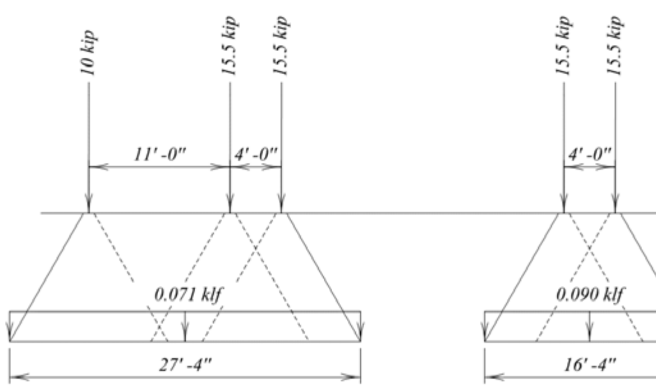

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Figure 78. Illustration. Rating live loads and soil distribution – AASHTO Type 3S2.

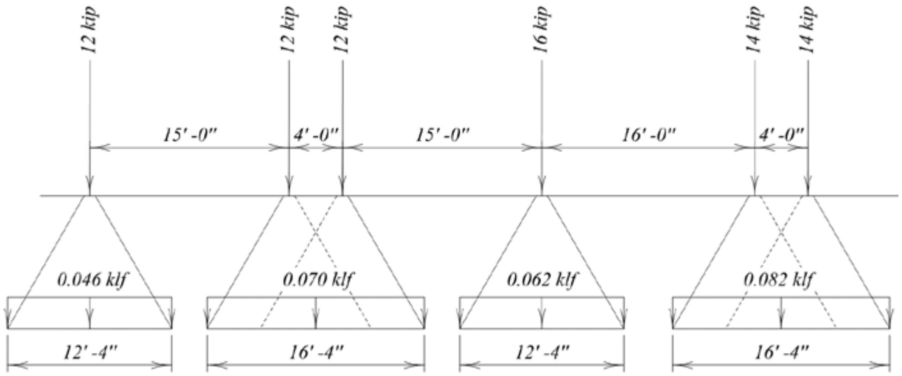

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Figure 79. Illustration. Rating live loads and soil distribution – AASHTO Type 3-3.

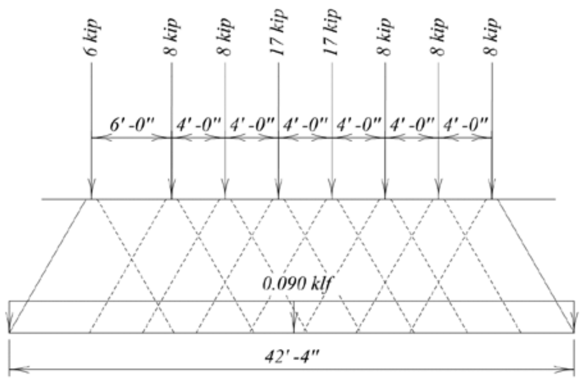

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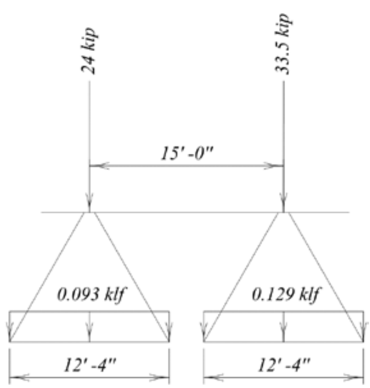
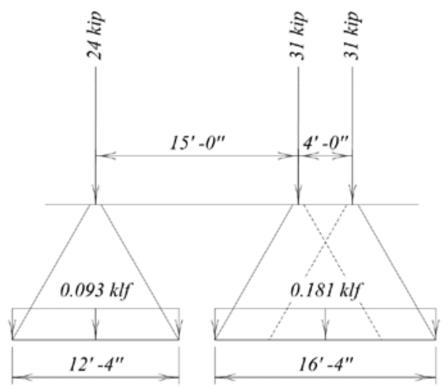
Figure 80. Illustration. Rating live loads and soil distribution – AASHTO NRL.



Source: FHWA

Figure 81. Illustration. Rating live loads and soil distribution – AASHTO EV2.



Source: FHWA

Figure 82. Illustration. Rating live loads and soil distribution – AASHTO EV3.

The equations outlined here are an approximate method considered acceptable for this type of structure. However, a more refined structural model (MBE Article 6A.3.3) and analysis, such as a full 3D analysis considering the distribution of wheel loads through soil boundary conditions, may be used to determine model live loads if desired.

9.4.4.6 Live Load Surcharge, LS

Live load surcharge, LS, consists of a lateral soil pressure acting on the walls of the box tunnel. This surcharge accounts for increased lateral pressure due to induced live load surcharge. The depth of equivalent surcharge fill acting on the box tunnel is variable dependent on the fill depth of the box tunnel as specified in BDS Table 3.11.6.4-1 and is a constant earth pressure. Subsequently, the equivalent live load surcharge height is 2.0 feet for the specified 10 foot fill depth. Similar to the horizontal earth pressure, at-rest conditions are used to calculate the equivalent lateral earth pressure due to live load surcharge. These calculations are outlined in MBE Article 6A.5.12.10.3c and LRFD BDS Article 3.11.6.4.

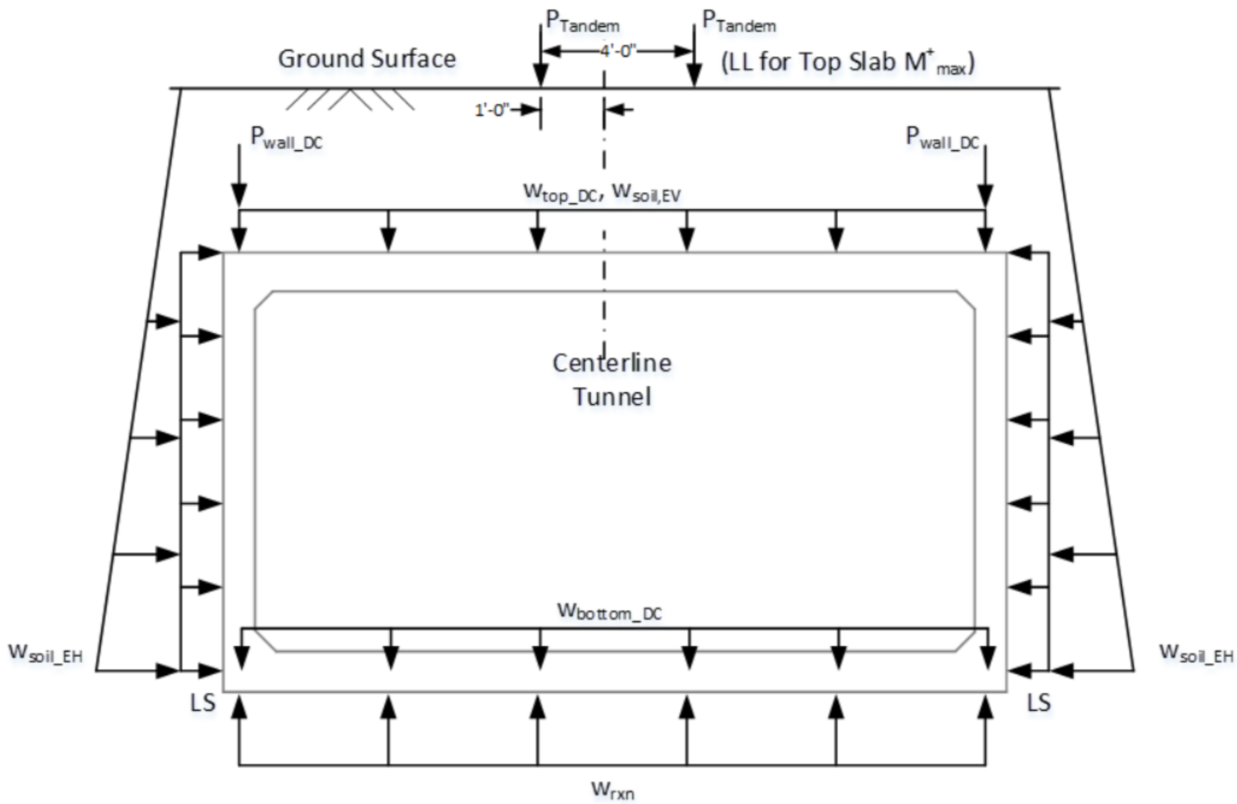
Live load surcharge:

$$w_{LS} = k_o \gamma_{fill} h_{eq} w_{strip} = 0.470(0.117)(2.00)(1.00) = 0.110\text{ klf}$$

9.4.5 Structural Analysis

A frame analysis procedure is applied for the box tunnel model with beam-column elements based on gross section properties. All the load effects and member resistances are calculated using this 1-foot-wide strip representation. Structural analysis of the box tunnel is based on a frame model with moment resisting connections between the slab and the wall joints. The frame is assumed to have a uniform vertical reaction across the bottom slab and pinned at the top of the walls to prevent side sway. A P-Delta analysis is not necessary to account for secondary effects due to the braced conditions. However, in the event of large box tunnel openings, a P-Delta analysis may be evaluated in the event secondary effects may begin to develop. Although not shown in this example, consideration for slenderness in the walls may be required in accordance with LRFD BDS Article 5.6.4.3. Additionally, corner haunches may be considered in the frame stiffness. However, it is common to neglect the corner haunches in the structural analysis.

Haunch stiffness is neglected in the rating example. The applied loads are shown in Figure 83, and Table 35 through Table 37 provide the resulting force effects.



Source: FHWA

Figure 83. Illustration. Box tunnel applied loads.

BDS Article C12.11.2.1 identifies three loading scenarios necessary to envelope the force effects on a buried structure. These load combinations will encapsulate the maximum positive and negative moment and shear effects on the wall as well as the slabs. Additionally, these will capture the maximum axial forces in the walls. Standard practice does not typically include evaluation of axial effects into the slabs as these are largely flexural members and it has been identified that the axial stresses are negligible. However, under conditions of excessively large lateral forces, axial effects into the slabs shall be considered. The three critical scenarios are:

1. Maximum Vertical, Maximum Horizontal,
2. Maximum Vertical, Minimum Horizontal,
3. Minimum Vertical, Maximum Horizontal.

Scenario one will capture the maximum negative moment in the slab in conjunction with the maximum negative moment in the wall resulting in controlling negative joint moments. Scenario two will capture the controlling positive moment in the slabs, while scenario three captures the controlling positive moment in the walls. Each scenario is matched with the appropriate live load envelope to produce the worst case. Typically, scenario 1 is used to calculate the maximum negative moment at the slab to wall joints. Scenario 2 is used to calculate the maximum positive moment in slab while scenario 3 is used to calculate the maximum positive moment in the exterior walls. Alternative variations may be considered if these do not envelope all controlling moments, shears and axial forces acting on the box tunnel frame.

The maximum and minimum effects are captured by utilization of the load factors presented in BDS Table 3.4.1-2. The one exception is the horizontal earth pressure. Maximum and minimum force effects due to horizontal earth pressure are obtained by combining the maximum load factor for an at-rest condition with the full calculated horizontal earth pressure and half the calculated horizontal earth pressure, respectively. These loads are presented in Table 35 through Table 37. A positive (+) moment indicates positive bending (indicative of the moment at mid span) while negative (-) moment indicates negative bending (indicative of the moment over a continuous support). Shears positive and negative or correspondingly to their respective moments.

Table 35. Exterior wall permanent load envelopes.

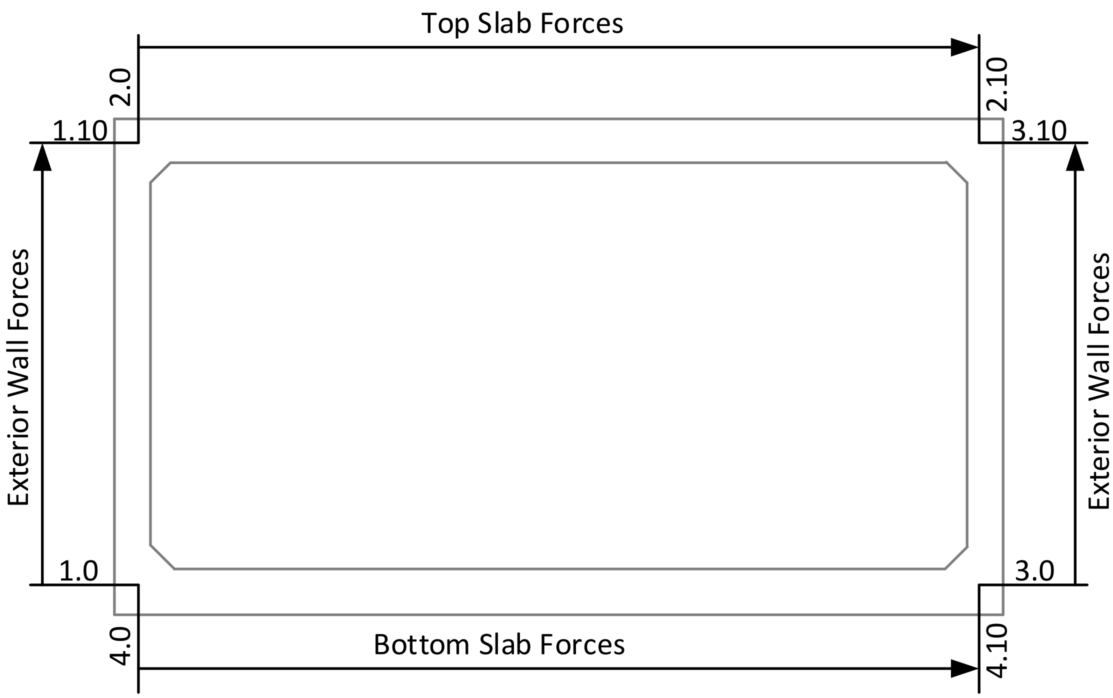
Sp Pt	**Moment** (k-ft) **DC**	**Moment** (k-ft) **EV**	**Moment** (k-ft) **EH**	**Shear** (kip) **DC**	**Shear** (kip) **EV**	**Shear** (kip) **EH**
1.0	-24.27	-49.61	-19.36	0.80	-0.30	12.37
1.1	-22.76	-50.18	1.18	0.80	-0.30	9.38
1.2	-21.26	-50.76	16.24	0.80	-0.30	6.58
1.3	-19.75	-51.33	26.20	0.80	-0.30	3.98
1.4	-18.24	-51.91	31.44	0.80	-0.30	1.57
1.5	-16.73	-52.48	32.32	0.80	-0.30	-0.64
1.6	-15.22	-53.06	29.21	0.80	-0.30	-2.66
1.7	-13.71	-53.63	22.48	0.80	-0.30	-4.47
1.8	-12.21	-54.20	12.50	0.80	-0.30	-6.10
1.9	-10.70	-54.78	-0.35	0.80	-0.30	-7.52
1.10	-9.19	-55.35	-15.72	0.80	-0.30	-8.76

Table 36. Top slab permanent load envelopes.

Sp Pt	Moment (k-ft) DC	Moment (k-ft) EV	Moment (k-ft) EH	Shear (kip) DC	Shear (kip) EV	Shear (kip) EH
2.0	-9.19	-55.35	-16.03	4.54	20.68	0.00
2.1	3.92	5.14	-16.03	3.58	16.55	0.00
2.2	14.09	52.19	-16.03	2.68	12.41	0.00
2.3	21.35	85.80	-16.03	1.79	8.27	0.00
2.4	25.71	105.96	-16.03	0.89	4.14	0.00
2.5	27.16	112.69	-16.03	0.00	0.00	0.00
2.6	25.71	105.96	-16.03	-0.89	-4.14	0.00
2.7	21.35	85.80	-16.03	-1.79	-8.27	0.00
2.8	14.09	52.19	-16.03	-2.68	-12.41	0.00
2.9	3.92	5.14	-16.03	-3.58	-16.55	0.00
2.10	-9.19	-55.35	-16.03	-4.54	-20.68	0.00

Table 37. Bottom slab permanent load envelopes.

Sp Pt	Moment (k-ft) DC	Moment (k-ft) EV	Moment (k-ft) EH	Shear (kip) DC	Shear (kip) EV	Shear (kip) EH
4.0	-24.27	-49.61	-19.36	8.79	20.68	0.00
4.1	1.44	10.89	-19.36	7.03	16.55	0.00
4.2	21.44	57.94	-19.36	5.27	12.41	0.00
4.3	35.72	91.54	-19.36	3.52	8.27	0.00
4.4	44.30	111.71	-19.36	1.76	4.14	0.00
4.5	47.15	118.43	-19.36	0.00	0.00	0.00
4.6	44.30	111.71	-19.36	-1.76	-4.14	0.00
4.7	35.72	91.54	-19.36	-3.52	-8.27	0.00
4.8	21.44	57.94	-19.36	-5.27	-12.41	0.00
4.9	1.44	10.89	-19.36	-7.03	-16.55	0.00
4.10	-24.27	-49.61	-19.36	-8.79	-20.68	0.00



Source: FHWA

Figure 84. Illustration. Member directions.

Live load placements also need to be considered to capture the maximum and minimum force envelopes. Live loads need to be applied to the top slab to capture the maximum positive and negative moment, maximum shear, and maximum wall axial forces. Therefore, placement of the live load needs to be determined by either developing influence lines of the box tunnel frame or executing a moving load analysis. Minimum vertical load will control the positive moment in the walls which will occur without live load application. Therefore, the wall positive moment will not control rating factors of a box tunnel.

Live load surcharge is a transient load and therefore will be included when evaluating the maximum horizontal load used in scenario one for controlling negative slab moments and wall moments. Minimum horizontal load, used in scenario two when calculating positive slab moments, will occur with the absence of a live load surcharge.

A full live load analysis is conducted for each of the selected live loads and resultant moment and shear diagrams are compared for each member. Evaluation and comparison found EV3 produces the maximum service moment as well as the maximum service shear forces. However, factored moments and shears also need to be considered. A multiple presence of 1.2 is applied to the HL-93 truck as well as the HL-93 Tandem while a multiple presence factor of 1.0 is applied to all legal loads. Additionally, live load factors of 1.75 and 1.35 are applied to the HL-93 Truck and HL-93 Tandem for inventory and operating forces, respectively. A legal load factor of 2.0 is applied to all legal loads including EV2 and EV3 in accordance with MBE 6A.5.12.10.3 and FHWA Memorandum Load Rating for the FAST Act's Emergency Vehicles.

Standard procedure included rating the structure for HL-93 live loads for inventory and operating levels. If the rating factor is less than one, further evaluation needs to be performed for Legal Load ratings. For example purposes, the legal live loads will be considered along with HL-93 live loads. Through this process, the EV3 produces the lowest rating value. Therefore, the resulting moments (kip-ft) and shears (kip) for the EV3 are shown in Table 38 through Table 40

displayed without the live load factor. Additionally, live load surcharge is presented.

Table 38. Exterior wall transient load envelopes.

Sp Pt	LS M	LS V	EV3 M+	EV3 M-	EV3 V+	EV3 V-
1.0	-1.8	1.1	0.00	-6.80	0.04	-0.31
1.1	0.0	0.8	0.00	-7.71	0.04	-0.31
1.2	1.4	0.6	0.00	-8.63	0.04	-0.31
1.3	2.4	0.4	0.00	-9.63	0.04	-0.31
1.4	3.0	0.2	0.00	-10.62	0.04	-0.31
1.5	3.2	0.0	0.00	-11.65	0.04	-0.31
1.6	3.0	-0.2	0.00	-12.73	0.04	-0.31
1.7	2.4	-0.4	0.00	-13.85	0.04	-0.31
1.8	1.5	-0.6	0.00	-14.79	0.04	-0.31
1.9	0.1	-0.8	0.00	-15.22	0.04	-0.31
1.10	-1.7	-1.0	0.00	-15.65	0.04	-0.31

Table 39. Top slab transient load envelopes.

Sp Pt	LS M	LS V	EV3 M+	EV3 M-	EV3 V+	EV3 V-
2.0	-1.7	0.0	0.00	-12.04	4.95	0.00
2.1	-1.7	0.0	3.96	-3.55	4.14	-0.05
2.2	-1.7	0.0	12.38	0.00	3.38	-0.22
2.3	-1.7	0.0	19.55	0.00	2.67	-0.50
2.4	-1.7	0.0	23.57	0.00	2.02	-0.90
2.5	-1.7	0.0	24.60	0.00	1.42	-1.42
2.6	-1.7	0.0	23.57	0.00	0.90	-2.02
2.7	-1.7	0.0	19.55	0.00	0.50	-2.67
2.8	-1.7	0.0	12.38	0.00	0.22	-3.38
2.9	-1.7	0.0	3.96	-3.55	0.05	-4.14
2.10	-1.7	0.0	0.00	-12.04	0.00	-4.95

Table 40. Bottom slab transient load envelopes.

Sp Pt	LS M	LS V	EV3 M+	EV3 M-	EV3 V+	EV3 V-
4.0	-1.8	0.0	0.00	-5.23	2.53	0.00
4.1	-1.8	0.0	2.66	-0.03	2.03	0.00
4.2	-1.8	0.0	8.50	0.00	1.53	0.00
4.3	-1.8	0.0	12.50	0.00	1.02	0.00
4.4	-1.8	0.0	14.87	0.00	0.52	0.00
4.5	-1.8	0.0	15.61	0.00	0.04	-0.04
4.6	-1.8	0.0	14.87	0.00	0.00	-0.52
4.7	-1.8	0.0	12.50	0.00	0.00	-1.02
4.8	-1.8	0.0	8.50	0.00	0.00	-1.53
4.9	-1.8	0.0	2.66	-0.03	0.00	-2.03
4.10	-1.8	0.0	0.00	-5.23	0.00	-2.53

Several loading combinations need to be considered to capture the controlling force envelopes. Strength-I will generally control the maximum and minimum effects on the slabs as well as the walls. Strength-II shall be considered for maximum and minimum effects on the slabs and walls when performing permit load rating. Service I will need to be considered in the case where crack control is a concern in accordance with BDS Article 5.6.7. However, standard rating procedures does not check the crack control and this design consideration need not be evaluated in accordance with MBE 6A.5.12.5.

Maximum and minimum load factors used to capture the extreme moment, shear, and axial effects are presented in MBE Table 6A.5.12.5-1 and are therefore not presented in full within this rating example. Minimum and maximum load factors are utilized to obtain force envelopes for DC, EV, EH, LS and LL. These load factors are used in the rating equation presented later in this example.

9.4.6 Resistance Calculations

The depth, analysis section width, and reinforcement for each of the critical sections are tabulated below in Table 41.

Table 41. Critical section data.

Section	Depth (in)	Width (in)	Bar Number	Bar Spacing (in)	Cover (in)
1 - Top Slab Center	22.00	12.000	10	5.50	2.00
2 – Bottom Slab Center	23.00	12.000	11	5.50	2.50
3 – Top Slab End	22.00	12.000	9	6.00	2.00
4 – Bottom Slab End	23.00	12.000	9	6.00	2.00
5 – Exterior Wall End	18.00	12.000	9	6.00	2.00
6 – Exterior Wall Middle	18.00	12.000	6	5.00	2.00

Concrete Properties:

f'_c = 4.0 ksi	
α_1 = 0.85	LRFD BDS 5.6.2.2
β_1 = 0.85 if $f'_c \leq 4.0$ ksi	LRFD BDS 5.6.2.2
λ = 1.0 (normal weight concrete)	LRFD BDS 5.4.2.8
γ_3 = 0.67 (AASHTO M31 Grade 60	LRFD BDS 5.6.3.3
γ_1 = 1.6	LRFD BDS 5.6.3.3

Modulus of rupture:

$$f_r = 0.24\lambda\sqrt{f'_c} = 0.24(1.0)\sqrt{4.0} = 0.480 \text{ ksi}$$ LRFD BDS 5.4.2.6

Compression reinforcement in flexural capacity calculations is conservatively ignored.
Calculated results are based on a per foot analysis width.

9.4.6.1 Section 1 – Top Slab Center

Rectangular section height:	h	=	22.00 inch	
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	1.267 inch^2	
Diameter of rebar:	dia_{bar}	=	1.270 inch	
Spacing of rebar:	s	=	5.5 inch	
Flexural resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5

Moment Resistance:

Determine equivalent area of reinforcing bar:

$$A_s = \frac{A_{s\_bar}b}{s} = \frac{1.267(12.00)}{5.5} = 2.76 \text{ inch}^2$$

Determine distance of the extreme compression fiber to the centroid of the reinforcement:

$$d_s = h - \frac{dia_{bar}}{2} - clr = 22.00 - \frac{1.270}{2} - 2.00 = 19.37 \text{ inches}$$

Determine distance from the equivalent stress block for tension controlled, non-prestressed tension reinforcement only:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.76(60)}{0.85(4.0)(12.00)} = 4.06 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(2.76)(60)\left(19.37 - \frac{4.06}{2}\right)\left(\frac{\text{ft}}{12\text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 215.57 \text{ kip-ft}$$

Minimum Steel Requirements:

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(22.00)^2}{6} = 968.00 \text{ inch}^3$$ LRFD BDS 5.6.3.3

The cracking moment equation simplifies to the following (only monolithic sections and no prestress forces):

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(968)\left(\frac{\text{ft}}{12\text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.3-1

$$= 41.51 \text{ kip-ft}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Shear resistance is critical at the end of the span and therefore shear resistance does not need to be checked at mid-span.

9.4.6.2 Section 2 – Bottom Slab Center

Rectangular section height:	h	=	23.00 in – 1 in = 22.00 inch	
Note: 1 inch is subtracted from thickness since it was cast against ground.				
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.50 inch	
Area of single rebar:	A_{s_bar}	=	1.561 inch^2	
Diameter of rebar:	dia_{bar}	=	1.41 inch	
Spacing of rebar:	s	=	5.5 inch	
Flexural resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5

Moment Resistance:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{1.561(12.00)}{5.5} = 3.406 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 22.00 - \frac{1.41}{2} - 2.50 = 18.80 \text{ inch}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.406(60)}{0.85(4.0)(12.00)} = 5.01 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(3.406)(60)\left(18.80 - \frac{5.01}{2}\right)\left(\frac{1\text{ft}}{12\text{in}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 249.75 \text{ kip-ft}$$

Minimum Steel Requirements:

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(22.00)^2}{6} = 968.00 \text{ inch}^3$$ LRFD BDS 5.6.3.3

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(968)\left(\frac{\text{ft}}{12 \text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.3-1

$$= 41.51 \text{ kip-ft}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Shear resistance is critical at the end of the span and therefore shear resistance does not need to be checked at mid-span.

9.4.6.3 Section 3 – Top Slab End

Rectangular section height:	h	=	22.00 inch	
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	0.999 inch^2	
Diameter of rebar:	dia_{bar}	=	1.128 inch	
Spacing of rebar:	s	=	6 inch	
Resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5
	ϕ_v	=	0.85	LRFD BDS 12.5.5

Moment Resistance:

Note: The flexural reinforcement being evaluated is the horizontal leg of the exterior wall bar as depicted in Figure 74. Load rater should verify the development length of rebar. If not fully developed, reduction in reinforcing steel area should be used. The section at rebar cut-off location needs to be considered in the load rating.

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.999(12.00)}{6} = 1.999 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 22.00 - \frac{1.128}{2} - 2.00 = 19.44 \text{ inch}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.999(60)}{0.85(4.0)(12.00)} = 2.939 \text{ inch}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(1.999)(60)\left(19.44 - \frac{2.94}{2}\right)\left(\frac{1\text{ft}}{12\text{in}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 161.59 \text{ kip-ft}$$

Minimum Steel Requirements:

Since $\varphi_f M_n > M_{cr} = 41.51$ k-ft, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Critical section for shear is d_v from the face of the support in accordance with LRFD BDS 5.7.3.2.

$$d_v = \max \begin{Bmatrix} 0.72h \\ 0.9d \\ d - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(22) = 15.84 \\ 0.9(19.44) = 17.50 \\ 19.44 - 2.94/2 = 17.97 \end{Bmatrix} = 17.97 \text{ inch}$$ LRFD BDS 5.7.2.8

Determining the nominal shear resistance requires the use of the applied shears and moments at the section of interest. Separate shears and moments will be required for every live load factor being evaluated. However, in this example, it was identified that the EV3 will control live load ratings and therefore only this live load is considered for the shear resistance. The concurrent, critical factored shears and moments at this location are:

$$M_u @ d_v = 55.02 \text{ k-ft}$$

$$V_u @ d_v = 36.80 \text{ kip}$$

Using these numbers, the nominal shear resistance when subjected to the EV3, in accordance with LRFD BDS 5.12.7.3 is:

$$\phi V_n = \left[0.0948\sqrt{f_c'} \le \left(0.0676\sqrt{f_c'} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right) \le 0.126\sqrt{f_c'}\right] bd_e$$

$$= \left[0.0948\sqrt{4.0} \le 0.0676\sqrt{4.0} + 4.6\frac{1.999(36.80)(19.44)}{12(19.44)(55.02\times 12)} \le 0.126\sqrt{4.0}\right] 12(19.44)$$

$$= \left[0.19 \le 0.18 \le 0.25\right] 12(19.44) = 41.22 \text{ kip}$$

Note that the value of $V_u d_e/M_u$ was checked and found to be less than 1.0, where d_e is the distance from the maximum compressive fiber to the centroid of tension steel.

9.4.6.4 Section 4 – Bottom Slab End

Rectangular section height:	h	=	23.00 inches – 1 inch = 22.00 inches	
Note: 1 inch is subtracted from thickness since it was cast against ground.				
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	0.999 inch^2	
Diameter of rebar:	dia_{bar}	=	1.128 inch	
Spacing of rebar:	s	=	6 inch	
Resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5
	ϕ_v	=	0.85	LRFD BDS 12.5.5

Moment Resistance:

Note: The flexural reinforcement being evaluated is the horizontal leg of the exterior wall bar as depicted in Figure 74.

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.999(12.00)}{6} = 1.999 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 22.00 - \frac{1.128}{2} - 2.00 = 19.44 \text{ inch}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.999(60)}{0.85(4.0)(12.00)} = 2.939 \text{ inch}$$ LRFD BDS Eq. 5.6.3.1.1-4 & 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2}\right) =$$

$$0.9(1.999)(60)\left(19.44 - \frac{2.94}{2}\right)\left(\frac{1\text{ft}}{12\text{in}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 161.59 \text{ kip-ft}$$

Minimum Steel Requirements:

Since $\varphi_f M_n > M_{cr} = 41.51$ k-ft, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Critical section for shear is d_v from the face of the support in accordance with LRFD BDS 5.7.3.2.

$$d_v = \max\begin{Bmatrix} 0.72h \\ 0.9d \\ d - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(22) = 15.84 \\ 0.9(19.44) = 17.50 \\ 19.44 - 2.94/2 = 17.97 \end{Bmatrix} = 17.97 \text{ inches}$$ LRFD BDS 5.7.2.8

Determining the nominal shear resistance requires the use of the applied shears and moments at the section of interest. Separate shears and moments will be required for every live load factor being evaluated. However, in this example, it was identified that the EV3 will control live load ratings and therefore only this live load is considered for the shear resistance. The concurrent, critical factored shears and moments at this location are:

$$M_u @ d_v = 51.27 \text{ k-ft}$$
$$V_u @ d_v = 37.01 \text{ kip}$$

Using these numbers, the nominal shear resistance, when subjected to the EV3, in accordance with LRFD BDS 5.12.7.3 is:

$$\phi V_n = \phi_v \left[0.0948\sqrt{f_c'} \le \left(0.0676\sqrt{f_c'} + 4.6 \frac{A_s}{bd_e} \frac{V_u d_e}{M_u} \right) \le 0.126\sqrt{f_c'} \right] bd_e$$
$$= 0.85\left[0.0948\sqrt{4.0} \le 0.0676\sqrt{4.0} + 4.6 \frac{1.999(37.01)(19.44)}{12(19.44)(51.27 \times 12)} \le 0.126\sqrt{4.0} \right] 12(19.44)$$
$$= [0.19 \le 0.18 \le 0.25] 12(19.44) = 37.59 \text{ kip}$$

Note that the value of $V_u d_e / M_u$ was checked and found to be less than 1.0, where d_e is the distance from the maximum compressive fiber to the centroid of tension steel.

9.4.6.5 Section 5 – Exterior Wall End

Rectangular section height:	h	=	18.00 inch	
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	0.999 inch^2	
Diameter of rebar:	dia_{bar}	=	1.128 inch	

Diameter of aggregate	d_{ag}	=	0.75 inch	
Spacing of rebar:	s	=	6 inch	
Resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5
	ϕ_v	=	0.85	LRFD BDS 12.5.5
	ϕ_c	=	0.70	LRFD BDS 5.5.4.2

Moment Resistance:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.999(12.00)}{6} = 1.999 \text{ in}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 18.00 - \frac{1.128}{2} - 2.00 = 15.44 \text{ in}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.999(60)}{0.85(4.0)(12.00)} = 2.939 \text{ in}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(1.999)(60)\left(15.44 - \frac{2.94}{2}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 125.61 \text{ kip-ft}$$

Walls need to be checked for axial thrust and flexural interaction. This check is typically only completed for walls; however, should also be checked in the top and bottom slabs if significant axial thrust is experienced in these members. Check flexure/axial interaction:

$$0.1\phi_c f_c' A_g = 0.1(0.7)(4.0)(18)(12) = 60.48 \text{ kip}$$

$$\text{N}_u = 40.29 \text{ kip}$$ LRFD BDS 5.6.4.5

$$0.1\phi_c f_c' A_g \geq N_u \therefore \text{Neglect Axial Thrust}$$

Minimum Steel Requirements

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(18.00)^2}{6} = 648 \text{ in}^3$$ LRFD BDS 5.6.3.3

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(648)\left(\frac{\text{ft}}{12 \text{ in}}\right)$$ LRFD BDS Eq. 5.6.3.3-1

$$= 27.78 \text{ kip-ft}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

Shear Resistance

Critical section for shear is d_v from the face of the support in accordance with LRFD BDS 5.7.3.2.

$$d_v = \max\begin{Bmatrix} 0.72h \\ 0.9d \\ d - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(18) = 12.96 \\ 0.9(15.44) = 13.90 \\ 15.44 - 2.94/2 = 13.97 \end{Bmatrix} = 13.97 \text{ in} \qquad \text{LRFD BDS 5.7.2.8}$$

Determining the nominal shear resistance requires the use of the applied shears and moments at the section of interest. Separate shears and moments will be required for every live load factor being evaluated. However, in this example, it was identified that the EV3 will control live load ratings and therefore only this live load is considered for the shear resistance. Additionally, consideration needs to be given to whether the maximum shear force occurs at the top or the bottom of the exterior wall and subsequent shear calculations need to correspond to the critical location. In this example, the critical shear occurs d_v from the top of bottom slab. The concurrent, critical factored shears and moments at this location are:

$$M_u @ d_v = 112.85 \text{ k-ft}$$
$$V_u @ d_v = 14.36 \text{ kip}$$

The shear resistance of the walls needs to be evaluated using LRFD BDS 5.7.3.3. Shear factors β and θ need to be calculated since the member thickness is greater than 16 in. and there is no shear reinforcement. The shear factors in accordance with LRFD BDS 5.7.3.4.2, using EV3 shears and moments shown above, are:

$$M_u \geq V_u d_v \Rightarrow 112.86(12) = 1354.32 \text{ kip-in} \geq 14.36(13.97) = 200.61 \text{ kip-in}$$

$$\varepsilon_s = \frac{\left(\dfrac{|M_u|}{d_v} + 0.5N_u + |V_u|\right)}{E_s A_s} = \frac{\left(\dfrac{|112.85(12)|}{13.97} + 0.5(-40.29) + 14.36\right)}{1.999(29000)} = 0.0016$$

$$0 \leq \varepsilon_s \leq 0.006$$

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500(0.0016) = 34.6^o$$

$$s_x = d_v \leq h - 2CL - d_b = 13.97 \text{ in} \leq 18 - 2(2) - 0.75 = 13.25 \text{ inch}$$

Where CL is the clear cover;

$$s_{xe} = 12 \text{ in} \le s_x \frac{1.38}{a_g + 0.63} \le 80 \text{ inch}$$

$$s_{xe} = 12 \text{ in} \le 13.25 \frac{1.38}{0.75 + 0.63} \le 80 \text{ in} \Rightarrow 13.25 \text{ inch}$$

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \frac{51}{(39 + s_{xe})} = \frac{4.8}{[1 + 750(0.0016)]} \frac{51}{[39 + 13.25]} = 2.13$$

$$\phi V_n = \phi_v \left[0.0316\beta\sqrt{f_c^{'}} \le 0.25 f_c^{'} \right] b_v d_v$$

$$= 0.85 \left[0.0316(2.13)\sqrt{4.0} \le 0.25(4.0) \right] 12(13.97)$$

$$= 19.45 \text{ kip}$$

9.4.6.6 Section 6 –Exterior Wall Mid-span

Rectangular section height:	*h*	=	18.00 inch	
Rectangular section width:	*b*	=	12.00 inch	
Clear distance to rebar from tension face:	*clr*	=	2.00 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	0.442 inch^2	
Diameter of rebar:	dia_{bar}	=	0.75 inch	
Spacing of rebar:	*s*	=	5 inch	
Resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5
	ϕ_v	=	0.85	LRFD BDS 12.5.5
	ϕ_c	=	0.70	LRFD BDS 5.5.4.2

Moment Resistance:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.442(12.00)}{5.5} = 1.060 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 18.00 - \frac{0.75}{2} - 2.00 = 15.63 \text{ inch}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.06(60)}{0.85(4.0)(12.00)} = 1.56 \text{ inch}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(1.06)(60)\left(15.63 - \frac{1.56}{2}\right)\left(\frac{1\text{ft}}{12\text{in}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 70.83 \text{ kip-ft}$$

Walls need to be checked for axial thrust and flexural interaction. This check is typically only checked for walls; however, should also be checked in the top and bottom slabs if significant axial thrust is experienced in these members. Check flexure/axial interaction:

$0.1\phi_c f_c^{'} A_g = 0.1(0.7)(4.0)(18)(12) = 60.48 \text{ kip}$

$\mathrm{N}_u = 40.29 \text{ kip}$ LRFD BDS 5.6.4.5

$0.1\phi_c f_c^{'} A_g \geq N_u \therefore$ Neglect Axial Thrust

Minimum Steel Requirements:

Since $\varphi_f M_n > M_{cr} = 27.78$ k-ft, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Shear was checked with end of wall moment and found to satisfy required conditions. Shear is less critical at mid-span and is therefore sufficient at this location.

9.4.7 LRFR Rating Calculations

The structural condition of the box tunnel is satisfactory and the system factor falls under the category for "All Other Girder Bridges and Slab Bridges." Therefore:

Condition factor:	ϕ_c	=	1.00	MBE Table 6A.4.2.3-1
System factor:	ϕ_s	=	1.00	MBE Table 6A.4.2.4-1

Resistance factors are based on LRFD BDS Table 12.5.5-1:

Flexure resistance factor:	ϕ_f	=	0.90	LRFD BDS 12.5.5
Shear resistance factor:	ϕ_v	=	0.85	LRFD BDS 12.5.5
Thrust resistance factor:	ϕ_a	=	0.70	LRFD BDS 5.5.4.2

The equation for calculating the rating factor is based on MBE Equation 6A.4.2.1-1, which has been simplified for the load types being applied. The component for permanent loads other than dead loads, *P*, has been removed.

$$RF = \frac{C \pm \gamma_{DC} DC \pm \gamma_{DW} DW \pm \gamma_{EV} EV \pm \gamma_{EH} EH \pm \gamma_{ES} ES}{\gamma_{LL}(LL + IM) \pm \gamma_{LS} LS}$$ MBE Eq. 6A.5.12.4-1

For the Strength Limit State:

$$C = \phi_c \phi_s \phi R_n$$ MBE Eq. 6A.5.12.4-2

Table 42 though Table 50 shows the results of the overall capacity based on MBE Equation 6A.5.12.4-2 as well as the rating load factors based on MBE Equation 6A.5.12.4-1. It is also noted that negative moments and shears are designated with a negative sign as well as corresponding negative moment capacities. It is imperative to coordinate positive and negative moments and shears when calculating the rating factor. Ratings are performed at the critical section defined in LRFD BDS 12.11.5.2 for flexure and 5.7.3.2 for shear. Consequently, the critical moment section (x_{cr} for moment) is at the end of the haunched corner and the critical shear (x_{cr} for shear) is at a distance d_v from the inside face of the wall or slab. A live load distribution factor of 2.0 is used in accordance with MBE 6A.5.12.10.3. Additionally, the minimum lateral earth pressure load factor was taken as 1.35/2 per LRFD BDS 3.11.7.

Table 42. Exterior wall rating load factors.

Load	DC	EV	EH	LS	LL
M-	1.25	1.30	1.35	2.00	2.00
M+	0.90	0.90	1.35	2.00	0.00
V	1.25	1.30	1.35	2.00	2.00

Table 43. Exterior wall moments (k-ft) and rating factor (RF).

x_{cr} (in)	DC	EV	EH	LS	LL+	LL-	C	RF
209.00	-10.36	-54.91	-3.84	-0.30	0.00	-15.32	-125.61	1.16
113.25	-16.73	-52.48	32.32	3.16	0.00	-11.65	70.83	14.16

Table 44. Exterior wall shears (kip) and rating factor (RF).

x_{cr} (in)	DC	EV	EH	LS	LL+	LL-	C	RF
24.97	0.80	-0.30	9.09	0.82	0.04	-0.31	19.45	6.45

Table 45. Top slab rating load factors.

Load	DC	EV	*EH	LS	LL
M-	1.25	1.30	1.35	2.00	2.00
M+	1.25	1.30	0.68	0.00	2.00
V	1.25	1.30	1.35	2.00	2.00

Table 46. Top slab moments (k-ft) and rating factor (RF).

x_{cr} (in)	DC	EV	EH	LS	LL+	LL-	C	RF
29.50	0.73	-9.59	-16.03	-1.65	3.00	-5.62	-161.59	8.83
195	27.16	112.69	-16.03	-1.65	24.60	0.00	215.57	0.93

Table 47. Top slab shears (kip) and rating factor (RF).

x_{cr} (in)	DC	EV	EH	LS	LL+	LL-	C	RF
26.97	3.88	17.82	0.00	0.00	4.39	-0.03	41.22	1.50

Table 48. Bottom slab rating load factors.

Load	DC	EV	EH	LS	LL
M-	1.25	1.30	1.35	2.00	2.00
M+	1.25	1.30	0.68	0.00	2.00
V	1.25	1.30	1.35	2.00	2.00

Table 49. Bottom slab moments (k-ft) and rating factor (RF).

x_{cr} (in)	DC	EV	EH	LS	LL+	LL-	C	RF
29.50	-4.82	-3.85	-19.36	-1.82	2.01	-1.30	-161.59	19.96
195	47.15	118.43	-19.36	-1.82	15.61	0.00	249.74	1.60

Table 50. Bottom slab shears (kip) and rating factor (RF).

x_{cr} (in)	DC	EV	EH	LS	LL+	LL-	C	RF
26.97	7.57	17.82	0.00	0.00	2.18	0.00	41.22	1.96

It was observed that the rating factor for the EV3 for all but the top slab positive moment is greater than 1.0 indicating that these locations are sufficiently rated. However, the top slab positive moment has a rating factor of 0.93 showing this location is insufficient to pass an EV3 Emergency Vehicle. The equivalent loading can be determined by multiplying the rating factor by the vehicle total weight as shown below:

$$\text{Safe Posting Load} = \frac{W}{0.7}\left[\left(RF\right)-0.3\right] = \frac{43}{0.7}\left[0.93-0.3\right] = 38 \text{ tons}$$

9.4.8 LFR Rating Calculations

Rating of cut and cover tunnels is similar in process and procedures as rating of box culverts as defined in the MBE. However, the MBE only provides guidance and examples for rating of boxes using LRFR. Though rating of boxes using LFR is not directly specified in the rating specifications, interpretation can be used to rate box tunnels using LFR methodologies.

There are several alterations and interpretations that are required to rate a box tunnel using LFR methodology. This includes, but may not be limited to, the inclusion of live load surcharge on the denominator of the rating equation, utilization of LFR load factors and resistance factors, definition of dead loads and wearing surfaces, earth load calculations, and capacity evaluations. These factors are performed slightly different with LRFR and LFR procedures.

The example detailed in this document was evaluated and loads were calculated using LRFR procedures and guidelines. For rating procedure example, the following shows the rating procedures using LFR methodology. However, the member capacity and forces are replicated from previous section rather than recalculated using LFR procedures.

The critical section shown in previous section was top slab, mid span moment. For simplicity, the following calculations are just showing rating for this critical section. Typical rating procedures would require both moment and shear evaluation at all critical locations and possibly all 10th span points.

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

Where

$C = 215.57$ kip-ft

$A_1 = 1.3$

$A_2 = 1.3$ (Operating Level)

$D = 27.16 + 112.69 - 16.03 = 123.82$ kip-ft

$L = 24.60$ kip-ft

$I = 10\%$ (Standard Specification Section 3.8.2.3)

$$RF = \frac{215.57 - 1.3(123.82)}{1.3(24.6)(1+0.10)} = 1.55$$

It is observed that LRFR analysis results in a load rating factor of 0.93 while LFR analysis results in a load rating factor of 1.55. The main reason for the difference is the live load factor of 1.3 (LFR) vs 2.0 (LRFR). If 2.0 would be used in LFR as well, it would yield a lower rating factor. The increased rating factor is also due to the example assumption that the capacities between LRFR and LFR would be essentially the same. It should be noted this phenomenon is isolated and does not represent general relationship between LRFR and LFR.

9.5 EXAMPLE 4 – REINFORCED CONCRETE BOX WITH CIRCULAR TOP

9.5.1 Problem Statement

Example 4 considers a reinforced concrete box with a cast-in-place reinforced concrete circular arch top, as illustrated in Figure 85. The tunnel opening has a width of 31 feet and a height of 17 feet at the edges and 23 feet 8 ½ inches at the center. The tunnel roadway is supported by a slab-on-grade (see Section 2.4.1.3). Cut-and-cover construction is assumed, with the box structure completed prior to backfilling. For this type of situation, a flat roof would be more common; however, the circular top is assumed to provide geometric complexity to the example. Backfill is assumed to be placed in 4 foot lifts. The at-grade roadway is 5 feet above the top of the arch, supported on fill.

Structure backfill is assumed to be compacted silty sand (SM). For the sake of simplicity in this example, the native soils outside the cut-and-cover areas are assumed to be the same as the backfill.

Structural components subject to load rating analysis include the Arch, Left Wall, Right Wall, and Bottom Slab. For this example, structure load effects are assumed to come from: self-weight of the structure or Dead Load Component (DC); Vertical Earth Loads (EV); Horizontal Earth Loads (EH); and Live Loads (LL).

Two-Dimensional (2D) Finite Element (FE) analysis is used to determine structural load effects in the form of moment and shear forces along each structural component. The commercially-available FE software Plaxis 2D (2018) is used for this example. A 2D model is generated which includes the structural elements surrounded by a continuum representing the soil, allowing the model to account for the effects of soil-structure-interaction (SSI).

The resulting shear and moment diagrams are used in conjunction with the reinforcement layout, shown in Figure 86, to determine the capacity and load effects on each member of the tunnel to evaluate the rating factor.

This example will perform the following steps to rate this reinforced concrete box tunnel:

1. Structure data
2. FE model description
3. FE analysis procedure
4. FE analysis results
5. Resistance calculations
6. LRFR rating calculations
7. LFR rating calculations

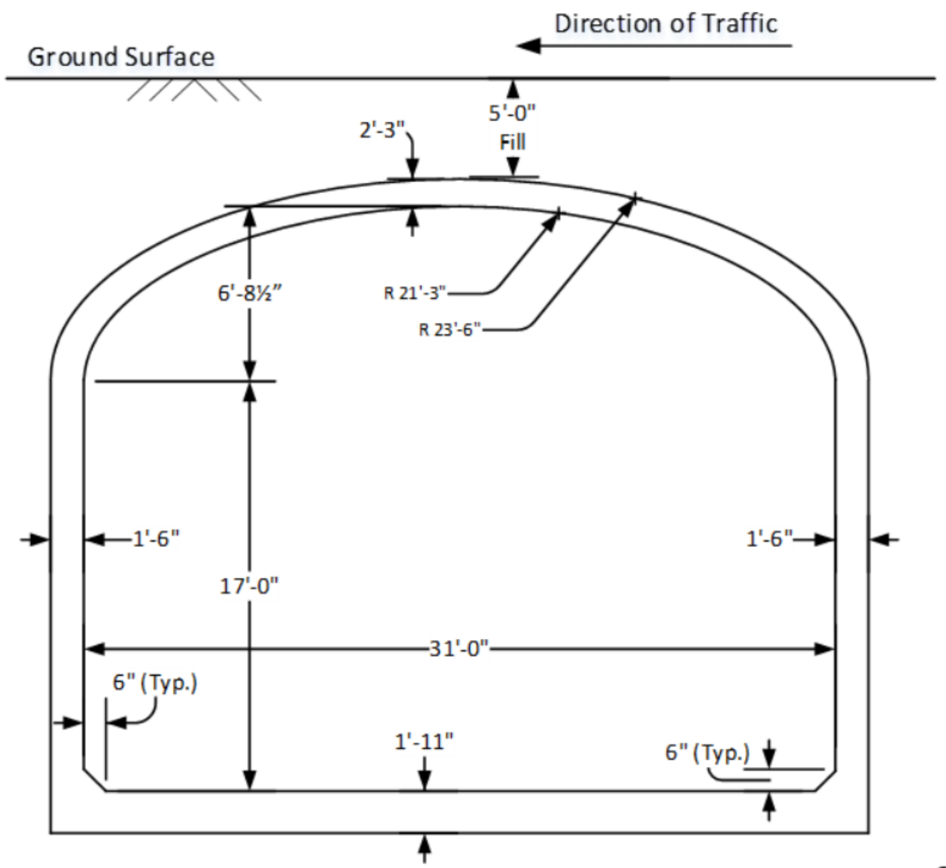
9.5.1.1 Structure Data

Concrete Properties:

	f'_c	=	4.0 ksi	
	E_c	=	3,644 ksi	(LRFD BDS 5.4.2.4-1)
	w_c	=	0.145 kcf	

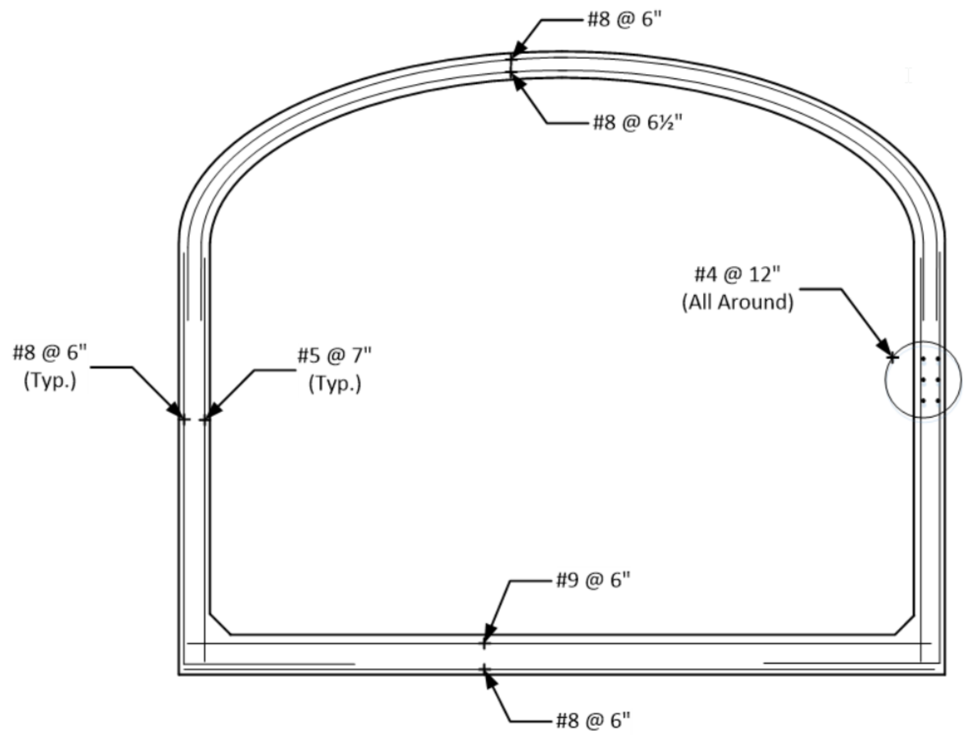
Dimensions (Figure 85):

Arch thickness:	t_{ts}	=	27 inches
Bottom Slab thickness:	t_{bs}	=	23 inches
Wall thickness:	t_{wall}	=	18 inches
Width of tunnel opening:	w_{tunnel}	=	31 feet
Height of tunnel opening:	h_{tunnel}	=	varies
Fill Depth:	h_{fill}	=	varies



Source: FHWA

Figure 85. Illustration. Tunnel dimensions for Example 4.



Source: FHWA

Figure 86. Illustration. Reinforcing layout for Example 4.

9.5.1.2 Example Notes

- This cross section is fictitious and is only for the purpose of demonstrating the principles of using FE models to perform analysis to aid in rating tunnels.
- Section for this box tunnel is a cast-in-place reinforced concrete, box tunnel that behaves as a box frame.
- Live loading inside the tunnel is only applicable to the Bottom Slab. Application of live load will put the slab into opposite flexure as the dead load and soil load reaction forces. Therefore, the worst case scenario does not include live load acting down on the Bottom Slab. As a result, live loads are not included on the Bottom Slab in the analysis.
- Live loading on the ground surface over the tunnel will increase force effects and is therefore included in the load rating analysis.
- In general, if the roadway has additional barrier, overlay, sidewalk, or other loads, they should be included in the dead load analysis for tunnel load rating. This example assumes that these items are not included in the geometry.
- While not assumed here, any other mechanical (including dynamic equipment loads) and architectural systems supported by the tunnel should be included in the dead load.
- This example focuses on an interior section of the tunnel. A section taken near the ends of the tunnel may need to account for increases in load effects due to reduced live load distribution near the edge.

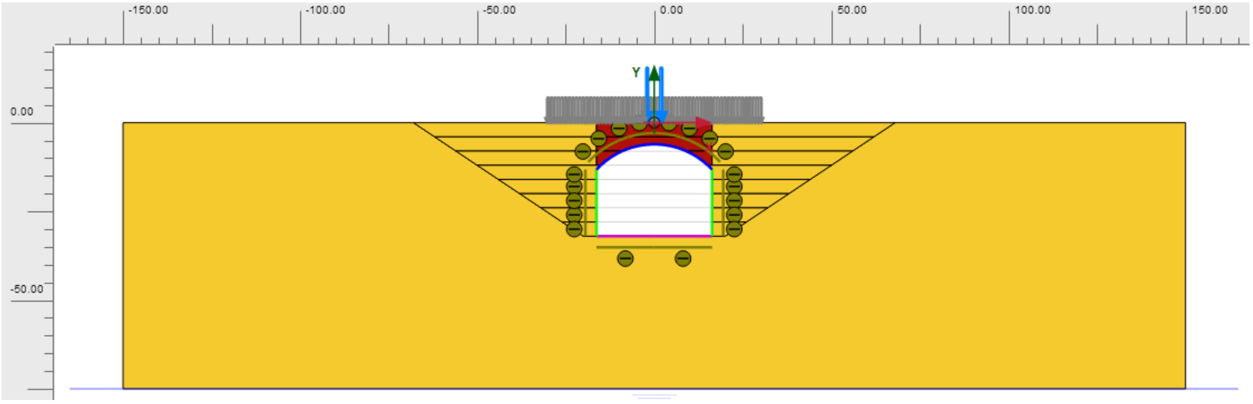
- The focus of this example is LRFR.
- The rating Engineer should evaluate the existing record drawings to determine if the rebar lap lengths and development lengths meet current design code requirements. This example is based on the assumption that rebar lap lengths and development lengths meet current design code requirements. In instances where development and lap length deficiencies are present, a reduced rebar area should be used within the portion of the bar affected by the deficiency by the ratio of provided development length to required development length.
- The rating Engineer should review the record drawings and inspection reports carefully to properly identify the support condition (pinned, expansion, fixed). Typical conditions, and assumed for this example, are a vertically restrained bottom slab and a laterally restrained exterior wall with fixity at the member ends, typically at the top slab and wall intersection working point.
- Rating is performed only at maximum shear and moment locations. Typical ratings include evaluation at all tenth points.

9.5.1.3 Rating Approach/Assumptions

- The LRFR evaluation is performed for a 1-foot-wide strip of tunnel and surrounding soil (parallel to the direction of traffic above the tunnel).
- Pavement has approximately the same unit weight as the soil and is therefore included with the soil vertical loads, consequently the vertical earth pressure load factors are applied for the wearing surface.
- Groundwater is assumed to be significantly deeper than the Bottom Slab, and does not impact the load rating analysis of this tunnel. As a result, groundwater is not included in the FE model.
- In this example, the only live loads considered are the HL-93 Truck and Tandem, for the sake of simplicity. All relevant live loads must be considered in an actual load rating analysis (see Example 3).
- There are no signs of distress or deterioration; therefore, the box tunnel is considered to be in satisfactory physical condition.
- There are no post/pre-tensioning forces; therefore, MBE Article 6A.2.2.2 does not apply.
- Structural load effects are calculated by performing 2D FE analysis including the soil and structure and applicable external loads.

9.5.2 FE Model Description

A 2D plane strain model was developed to assess the load demands on the tunnel due to dead loads, earth pressures, and live loads. The model is illustrated in Figure 87, representing a 1-foot--wide strip (into the page) of the tunnel and surrounding soil. Structural elements are used to represent the tunnel and continuum elements are used to represent the soil. An interface is included around the outside of the tunnel to simulate the interaction between the soil and structure. Model length units are feet, and force units are kip.



Source: FHWA

Figure 87. Illustration. 2D plane strain FE model of tunnel cross section. Length units are in feet.

9.5.2.1 Structural Elements

The tunnel liner is modeled with linear-elastic plate elements, which are essentially beam elements (Plaxis, 2018), that are used to model slender structures in the ground with a significant flexural rigidity, *EI,* and normal stiffness, *EA*. Generically, beam or frame elements are used for modeling the liner; plate element is the specific nomenclature used by the software in this example. The assumption of linear-elastic structural behavior is expected to be sufficient for load rating purposes, where loading is restricted to typical service-type conditions. However, after the FE analysis is completed, the resulting structural demands should be checked against capacities to verify this assumption.

Different properties are assigned to the Arch, Left and Right Walls, and Bottom Slab (indicated by different colors in Figure 87), based on the thicknesses listed in Section 9.5.1.1. The properties are listed in Table 51. In the 1-foot-wide plane strain model, structure properties are defined per foot of model width. Plate elements have zero thickness, while the actual tunnel liner has non-zero thickness. The plate elements are modeled along the centerlines of the actual tunnel liner elements. Weights of the structural elements are based on the actual dimensions and a unit weight of 0.15 kcf for reinforced concrete.

The tunnel is centered horizontally at an X coordinate of zero feet. The Bottom Slab is at a Y coordinate of -32 feet (32 feet below grade).

Table 51. Structure properties.

Tunnel Element	Structure Properties			Plane Strain Model Inputs		
	E (ksf)	A (ft^2/ft)	I (ft^4/ft)	EA (kip/ft)	EI (kip-ft^2/ft)	Weight, w (kip/ft/ft)
Arch	525,000	2.25	0.949	1,181,250	498,225	0.338
Left and Right Walls	525,000	1.5	0.281	787,500	147,525	0.225
Bottom Slab	525,000	1.917	0.587	1,006,425	308,175	0.288

Notes:

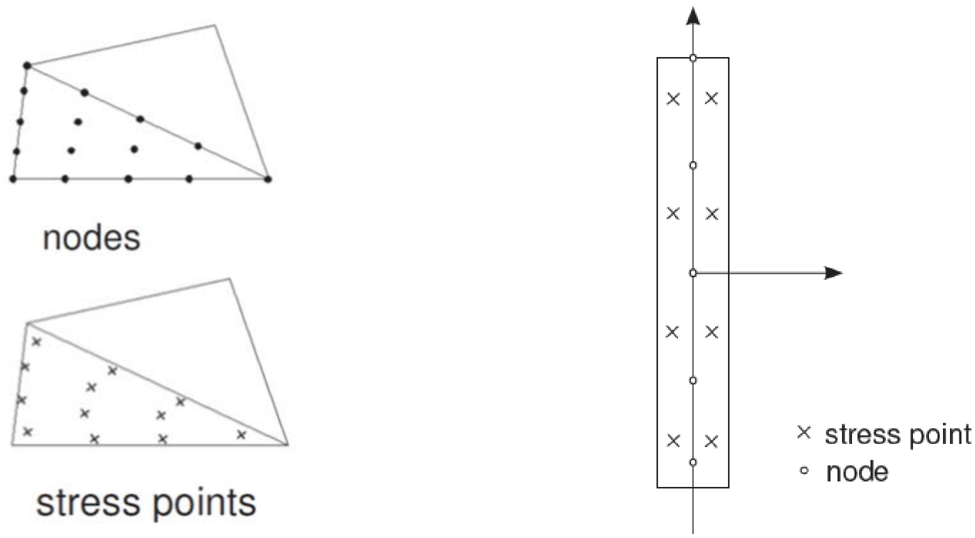
1. E = Young's modulus
2. A = Area
3. I = Moment of inertia

9.5.2.2 Soil Domain

The model soil domain surrounding the tunnel (Figure 87) has overall dimensions of 300 feet in width (X coordinates of -150 feet to +150 feet), and 75 feet in depth (Y coordinates of +0 to -75 feet). In SSI modeling, overall dimensions should be sufficiently large such that they do not significantly influence the model response and the results of interest. The soil dimensions were determined for this example by first starting with an initial assumption with relatively small dimensions and running the model to determine the structure forces (moment and shear). Then, the dimensions were increased incrementally and the model was re-run until the forces became insensitive to further increases.

9.5.2.3 FE Mesh

15-node 2D triangular, continuum elements (Figure 88-A) were used to discretize the soil domain. The plate elements have five nodes per adjacent triangular continuum element (Figure 88-B). Plaxis (2018) generates the mesh automatically based on the settings and adjustments selected by the Engineer. Similar to selecting the domain size, the fineness of the mesh should be selected such that it does not significantly influence the model response and the results of interest. An appropriate mesh was selected for this example by comparing the results for different mesh sizes.



(A) 15-node triangle (B) 5-node plate

Source: FHWA

Figure 88. Illustrations. Node distribution for (Plaxis 2018): a) 15-node triangular continuum element and b) 5-node plate element.

9.5.2.4 Soil Properties

The soil domain in the FE model is controlled by constitutive models (stress-strain relationships) and soil properties which are assigned by the Engineer. Actual stress-strain behavior of soil tends to involve significant variability and uncertainty. In order to address this variability in SSI analyses and ensure a reliable load rating, a range of soil properties may need to be considered. Accordingly, sets of Upper and Lower Estimate (UE and LE) soil parameters (listed in Table 52) were used in this example. These parameters represent a reasonable range of unit weight, strength, and stiffness for medium dense to dense silty sand (compacted fill or native material). Section 9.5.2.7 explains how the UE and LE parameters were implemented in the FE analyses in order to address variability and uncertainty.

The Mohr-Coulomb (Plaxis, 2018) linear elastic-perfectly plastic model is used to approximate nonlinear soil behavior in this example. Five parameters are typically required to define the Mohr-Coulomb model behavior for each soil type, including: unit weight, friction angle, ϕ, cohesion intercept, c, Young's modulus, E, and Poisson's ratio, ν. An advanced feature was also used to allow for increasing soil stiffness with depth based on a sixth input parameter, E_{incr}.

Table 52. Soil parameters.

Soil Type	Unit wt. (pcf)	ϕ (deg)	c (ksf)	E (ksf) [1]	E_{inc} (ksf/ft) [2]	Poisson's Ratio
Silty Sand (SM) *Lower Estimate*	110	30	0.1	500	20	0.3
Silty Sand (SM) *Upper Estimate*	130	38	0.1	750	30	0.3

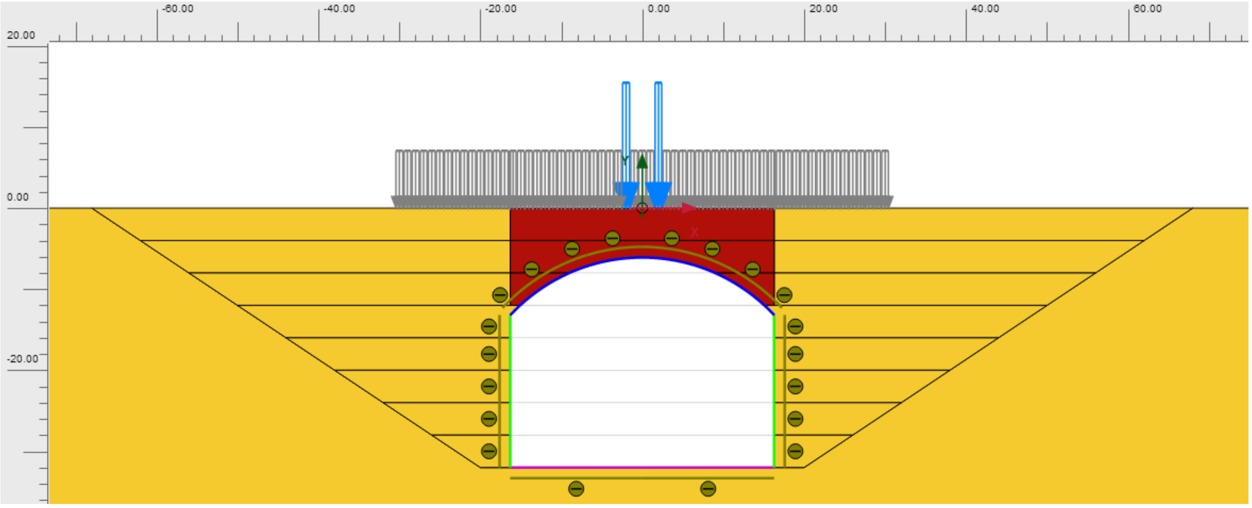
Notes:
1. E = Young's modulus at Y coordinate of zero
2. E_{inc} = Rate of change of stiffness with depth below Y coordinate of zero

9.5.2.5 Soil-Structure Interface

A soil-structure interface is included around the outside perimeter of the tunnel with strength equal to two-thirds (0.67) that of the adjacent soil (that is, $2/3\phi$). The interface provides a realistic limit to the amount of stress that can be transferred between the structure and the soil. Small circles with a minus sign inside them indicate the interface around the perimeter of the tunnel in Figure 87 and in later figures.

9.5.2.6 Applied Loads

Permanent loads including the dead weight of the tunnel (DC), vertical earth pressures (EV), and horizontal earth pressures (EH) are generated automatically in the model based on the assigned structure and soil properties. Live loads are applied as downward distributed loads on the surface of the model, as illustrated in Figure 89.



Source: FHWA

Figure 89. Illustration. Close-up view of tunnel in FE model.

Surface traffic is assumed to be moving in the direction from right to left across the tunnel, as illustrated in Figure 85. Based on BDS Article 3.6.1.2.5, a tire contact length (l_t) of 10 inches is assumed along the direction of traffic. In order to consider various vehicle positions above the tunnel and find the most critical location, a distributed load with a length of approximately 10 inches was placed (but not always activated) at every foot on-center from X = -30 feet to X = +30 feet (grey arrows in Figure 89). Vehicle loads are simulated by activating the distributed loads at wheel locations above the tunnel for various vehicle positions. For instance, the two activated loads (blue arrows) in Figure 89 represent HL-93 Tandem loading, which consists of a pair of wheel loads spaced at 4 feet laterally.

The two live loads included in this example for demonstration purposes are the HL-93 Truck and HL-93 Tandem loads. The HL-93 Truck consists of an 8-kip axle load, and two 32-kip loads, each spaced at 14 feet. The HL-93 Tandem consists of two 25-kip loads spaced at 4 feet, as mentioned above.

9.5.2.6.1 Distribution of Wheel Loads through Fill

Wheel loads will distribute outward laterally as they pass downward through the fill until they reach the tunnel Arch. This leads to lower magnitude pressures on the Arch than at the ground surface that are spread out over a larger area. Distribution parallel to the culvert span (to the left and right in Figure 89) will occur automatically in the 2D model. However, the 2D plane strain model cannot automatically account for the distribution transverse to the culvert span (in and out of the page in Figure 89).

The guidelines in AASHTO BDS 3.6.1.2.6 were used to account for the distribution transverse to the culvert span as follows:

- For traffic running parallel to the culvert span, and wheel load distribution transverse to culvert spans,

$$H_{\text{int}-t} = \frac{s_w - \frac{w_t}{12}\frac{0.06 D_i}{12}}{LLDF} = 3.633 \text{ feet}$$

  Where:

s_w	=	6 feet
w_t	=	20 inches
D_i	=	31 feet
$LLDF$	=	1.15

- Conservatively assume H = 5 feet of fill above the arch (H > 5 feet towards the ends of the arch, but maximum moments and impacts from wheel loads will likely occur near the mid-span).
- Since $H > H_{int\text{-}t}$:

$$w_w = \frac{w_t}{12} + s_w + LLDF(H) + 0.06\frac{D_i}{12} = 13.6 \text{ feet}$$

- The length of the distributed wheel loads in the model is 0.84 feet (approx. 10 inches / 12 inches per foot).

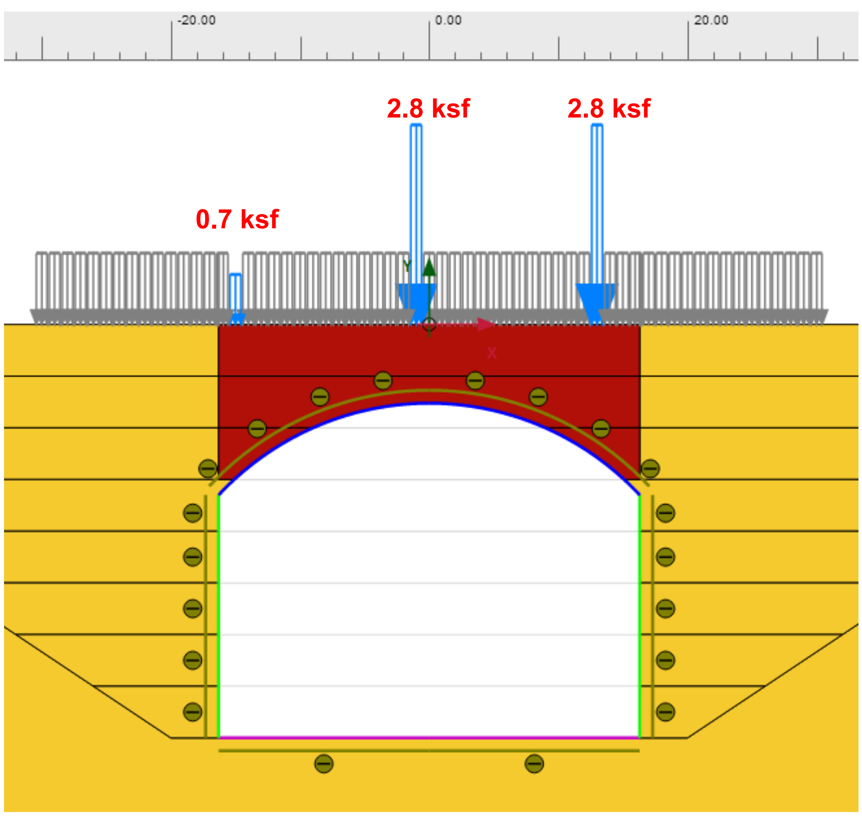
- The distributed loads to be applied in the 2D FE model are:
  - Front wheel load for Truck: 8 kip / 13.6 feet / 0.84 feet = 0.70 ksf
  - Middle and rear wheel loads for Truck: 32 kip / 13.6 feet / 0.84 feet = 2.80 ksf
  - Each wheel load for Tandem: 25 kip / 13.6 feet / 0.84 feet = 2.19 ksf

The above magnitudes are assigned to the appropriate discrete distributed load in the model to simulate each wheel. For instance, to simulate HL-93 Tandem loading at the arch mid-span as illustrated in Figure 89, the distributed loads centered at X-coordinates -2 feet and +2 feet were activated and assigned a magnitude of 2.19 ksf. All other distributed loads were left un-activated.

9.5.2.6.2 Determination of Critical Location for Live Loads

Live load placements need to be considered in order to capture the maximum and minimum force envelopes. Influence lines can sometimes be used to determine placement of live loads. However, for more complex SSI analyses, alternative methods may be required.

A relatively simple but time consuming method is to consider all relevant vehicle locations one at a time in separate FE analyses. Figure 90 presents an example for the HL-93 Truck at one specific location. The three activated loads can be moved incrementally to any other position. Structural demands can be compared for each load position in order to identify the maximum demands.



Source: FHWA

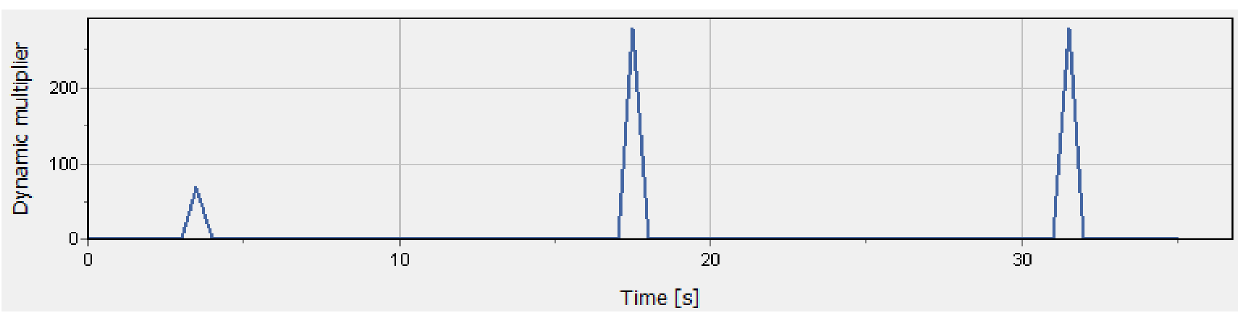
Figure 90. Illustration. Live load application for HL-93 Truck at one location.

Alternatively, a moving load analysis can be performed to identify the critical load location. Moving load analysis is more complex, but can save time if performed by an experienced modeling professional. A moving load analysis is demonstrated in this example for the HL-93 Truck using the Dynamics feature of Plaxis (2018).

For the moving load analysis demonstration, force-time functions were developed for each of the surface loads to simulate 33 different truck positions crossing the Arch span. The different truck positions were simulated at 1-second intervals occurring from 1.5 seconds to 33.5 seconds. All of the distributed loads (grey arrows in Figure 87 through Figure 90) were initially assigned a magnitude of 0.01 ksf in the downward direction.

The force-time functions consist of a multiplier at each instant in time. For each distributed load location, the multiplier is zero at instances when there is no wheel load at that location. The multiplier ramps up to the nonzero value required to generate the appropriate load when a wheel is at the location of the respective force-time history.

To illustrate, a sample force-time function is presented in Figure 91 for distributed load number 31, which is located above the center of the Arch (X = 0). In the dynamic simulation, the front wheel passes this location at T = 3.5 seconds, the middle wheel passes at T = 17.5 seconds, and the rear wheel passes at T = 31.5 seconds. The multiplier for the front wheel is set to 70, resulting in the desired distributed wheel load of 0.7 ksf (70 times 0.01 ksf). The multiplier for the middle and rear wheels is set to 280, resulting in the desired distributed wheel load of 2.8 ksf. A similar force-time function is developed for each of the 61 distributed loads that are centered at X coordinates -30 feet to +30 feet in the model.



Source: FHWA

Figure 91. Illustration. Force-time function for load number 31 located at X = 0.

The moving load analysis produces maximum and minimum force envelopes for each structural element considering all of the simulated vehicle positions. However because it is a dynamic analysis that does not precisely simulate the actual truck driving over the tunnel, it is important to verify the results by identifying the critical vehicle location and performing a corresponding static analysis. The moving load analysis is not intended to evaluate vibrations in the soil and structure (as in a typical dynamic analysis), but rather to simply allow for a large number of static load cases to be simulated in a single analysis phase. As a result, a large amount of damping may be required in order to avoid the influence of unintended dynamic effects. In this example, 100% damping was assigned to the soil layers. This led to a good match in terms of the structure forces between the moving load analysis and the static analysis for the identified critical load location. Moving load analysis for the HL-93 Truck is discussed further Section 9.5.4.2.2.

9.5.2.7 Load Factors and Combinations

Structural dead loads and earth pressures are generated automatically in the FE model. As a result, it is not possible to apply a load factor directly to the EV or EH loads. It is possible to apply load factors to live loads in the FE model. However, for consistency with the handling of permanent loads, load factors were not applied to live loads in the FE analysis. Therefore, the FE model results are for un-factored permanent and live loads. Load factors will be applied to the resulting demands later in the load rating procedure.

In typical box culvert design, different combinations of earth pressure loading on the top and sides of the box are considered in order to find the most critical force conditions. The following earth pressure load combinations are often considered:

1. Maximum Vertical and Maximum Horizontal.
2. Maximum Vertical and Minimum Horizontal.
3. Minimum Vertical and Maximum Horizontal.

Maximum horizontal earth pressure is typically taken as the at-rest earth pressure, and minimum horizontal pressure is taken as half of the maximum.

In earth pressure theory and in the FE soil-structure model, at-rest earth pressures occur when the wall does not move relative to the backfill. Earth pressures will decrease and approach an active state of equilibrium if the wall moves away from the backfill. If the wall is pushed into the backfill, earth pressures will increase, approaching the peak passive resistance. These SSI effects occur automatically in the FE model simulations. The tunnel liner elements will experience movements as each backfill lift is added, and as live loads are applied. Earth pressures will develop according to the assigned soil and interface properties and the movements of the adjacent structural elements at each location. As a result, it is not possible to simply apply half of the at-rest earth pressure as in the above load combinations. Instead, the intent of the above load combinations can be implemented by considering a range of realistic soil properties, such as the Lower Estimate (LE) and Upper Estimate (UE) parameters listed in Table 52. Two different regions are defined in the model to allow for combinations that lead to critical force conditions for load rating purposes. The zone above the arch (darker reddish color in Figure 90) generally produces larger vertical earth pressure loads (EV), while the remaining areas (lighter yellowish color) generally produces larger horizontal earth pressure loads (EH).

FE analyses were performed for the following combinations of LE and UE soil properties:

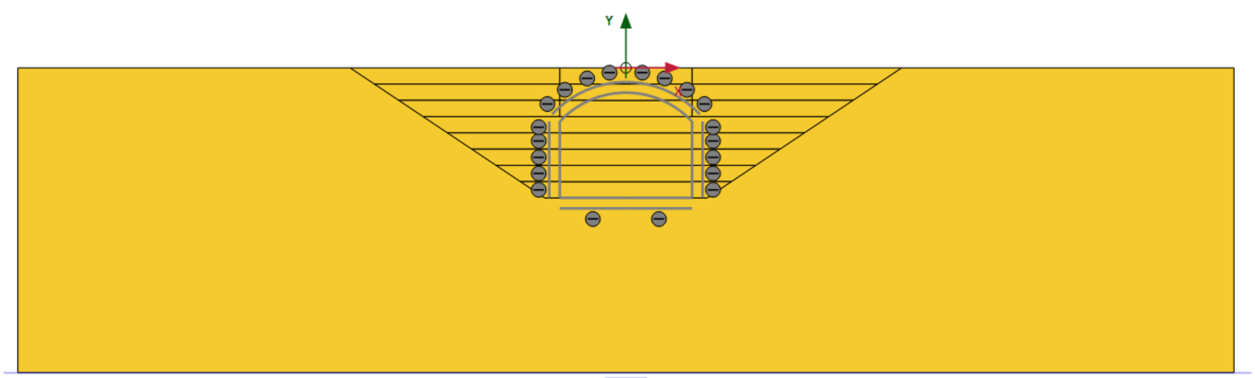
1. LE in the horizontal pressure zone and LE in the vertical pressure zone.
2. UE in the horizontal pressure zone and UE in the vertical pressure zone.
3. LE in the horizontal pressure zone and UE in the vertical pressure zone.
4. UE in the horizontal pressure zone and LE in the vertical pressure zone.

Load rating analysis will be based on the highest structure demands from these four combinations.

9.5.3 FE Analysis Procedure

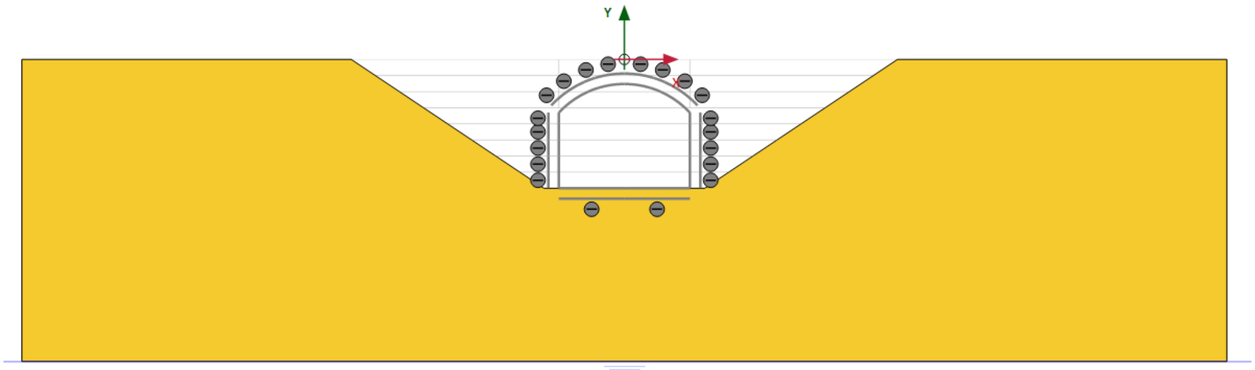
The FE analysis was performed over several phases in order to consider a realistic construction sequence and to separately evaluate demands under permanent loads only and permanent plus live loads. An initial phase was performed first with level ground conditions to generate stresses in the ground prior to the start of construction, as illustrated in Figure 92. Vertical stresses were generated based on the assigned soil unit weights, and horizontal stresses were generated based on a K_0 procedure (horizontal stress = vertical stress times K_0). The at-rest earth pressure coefficient K_0 was taken as $1 - \sin(\phi)$, which is the recommended default value in Plaxis (2018) and provided in BDS Article 3.11.5.2-1. The following staged construction phases were performed next:

- Phase 1: Excavate (Figure 93).
- Phase 2: Reset displacements to zero and activate tunnel (Figure 94).
- Phases 3-9: Add 4-foot backfill lifts, one in each phase (for example, Figure 95 shows the model after the third lift in Phase 5).
- Phase 10: Add final 4-foot backfill lift (Figure 96). All un-factored permanent loads (DC, EV, and EH) are accounted for at the end of this phase.
- Phase 11: Activate live load (Figure 97). All un-factored permanent and live loads (DC, EV, EH, and LL) are accounted for at the end of this phase.



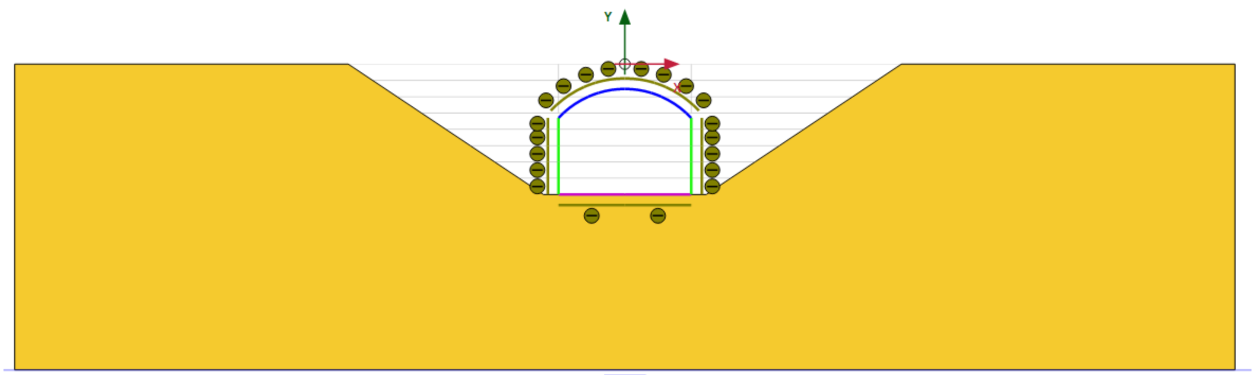
Source: FHWA

Figure 92. Illustration. Initial Phase - Generate stresses under original level ground condition.



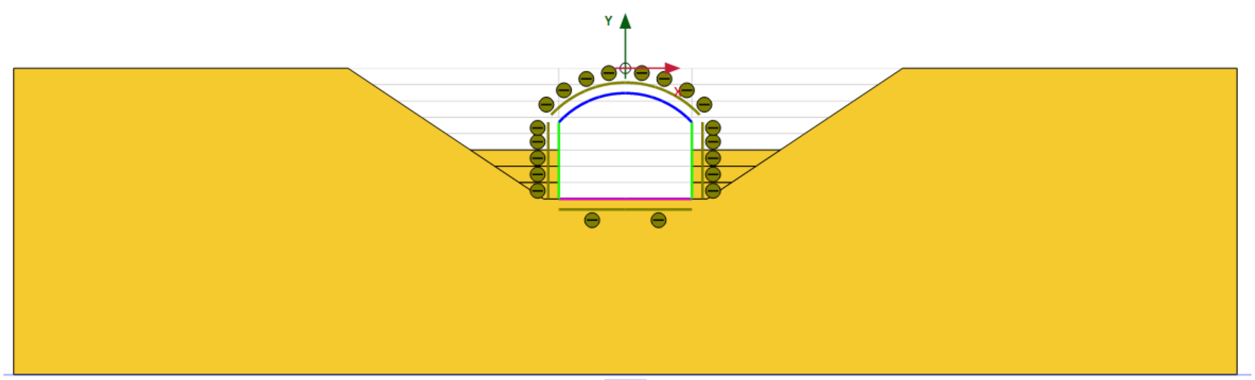
Source: FHWA

Figure 93. Illustration. Phase 1 - Excavate.



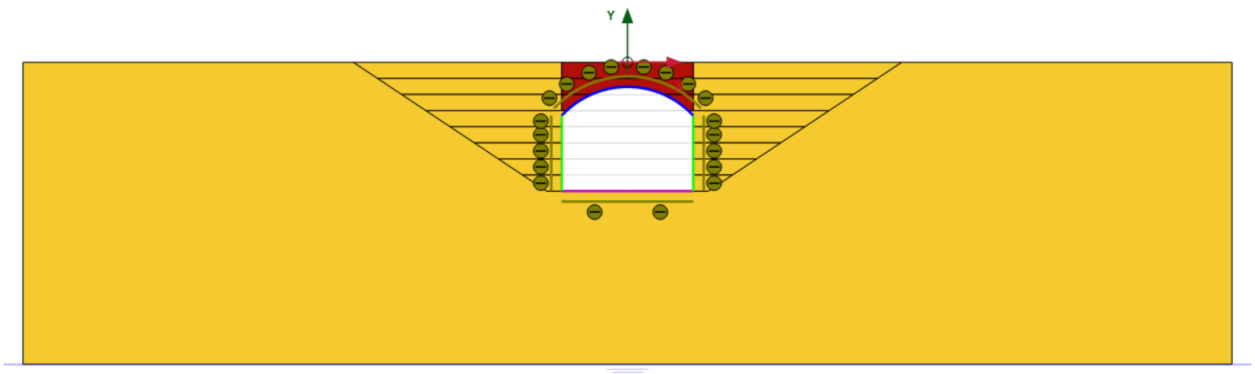
Source: FHWA

Figure 94. Illustration. Phase 2 – Activate tunnel.



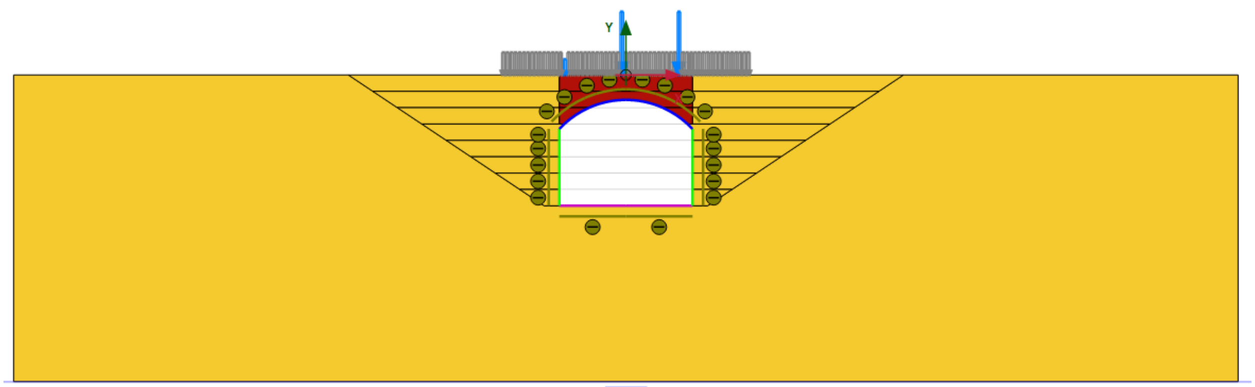
Source: FHWA

Figure 95. Illustration. Phase 5 – Add third lift (one lift added in each of Phases 3-10).



Source: FHWA

Figure 96. Illustration. Phase 10 – Add final lift.



Source: FHWA

Figure 97. Illustration. Phase 11 – Activate live load.

All of the analysis phases listed above were performed for each of the four soil property combinations described in Section 9.5.2.1.

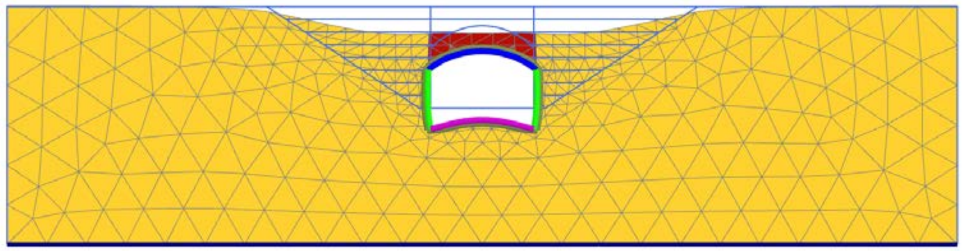
An additional dynamic analysis phase was required to perform the moving load analysis for the HL-93 Truck, which is discussed in Section 9.5.2.6.

9.5.4 FE Analysis Results

Results are presented in the following sections for use in the load rating analysis in the form of moment and shear plots along each structural element.

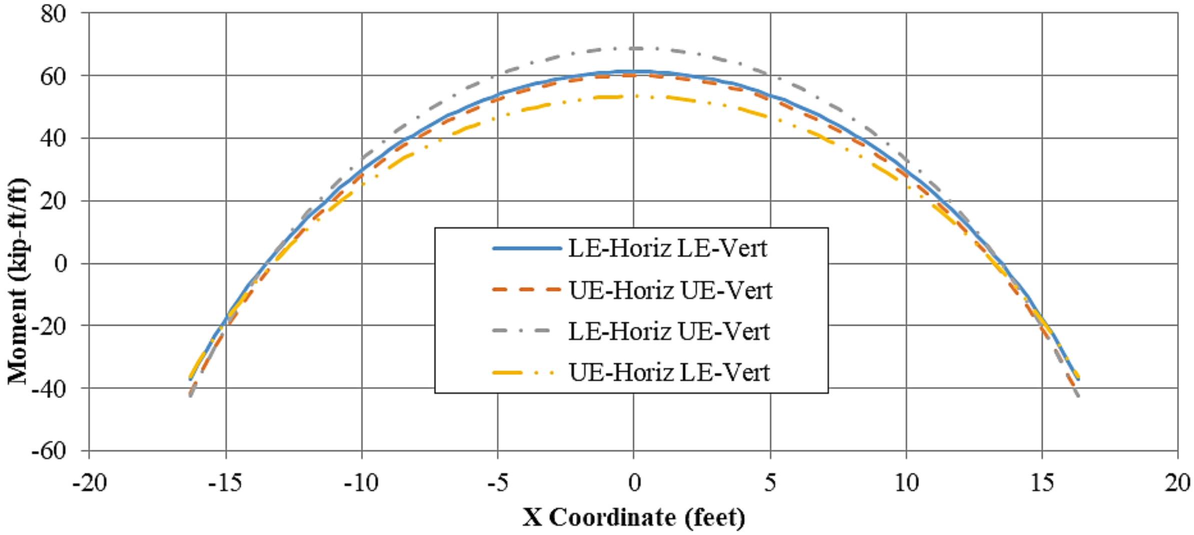
9.5.4.1 Structure Demands due to Permanent Loads

All un-factored permanent loads (DC, EV, and EH) are accounted for in the model at the end of analysis Phase 10. A representative deformed mesh after Phase 10 is presented in Figure 98. Moment and shear force diagrams due to the permanent loads are presented in Figure 99, Figure 100, Figure 101, Figure 102, Figure 103, and Figure 104 for the Arch, Bottom Slab, Left Wall, and Right Wall, respectively, considering each of the four soil property combinations listed in Section 9.5.2.

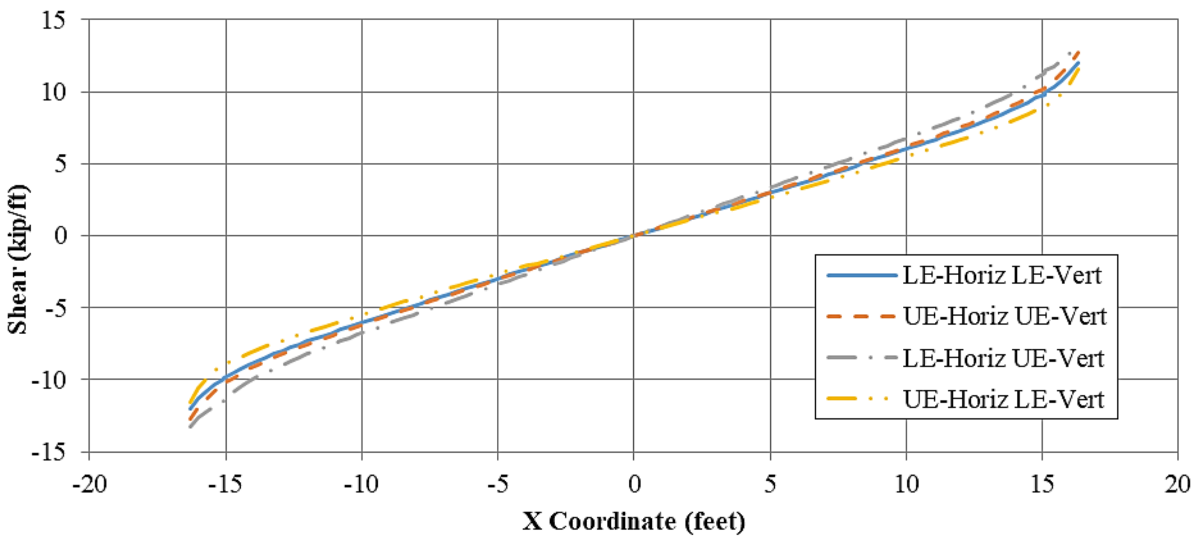


Source: FHWA

Figure 98. Illustration. Exaggerated (100 times) deformed mesh after tunnel construction and backfilling, no live loads (Phase 10).



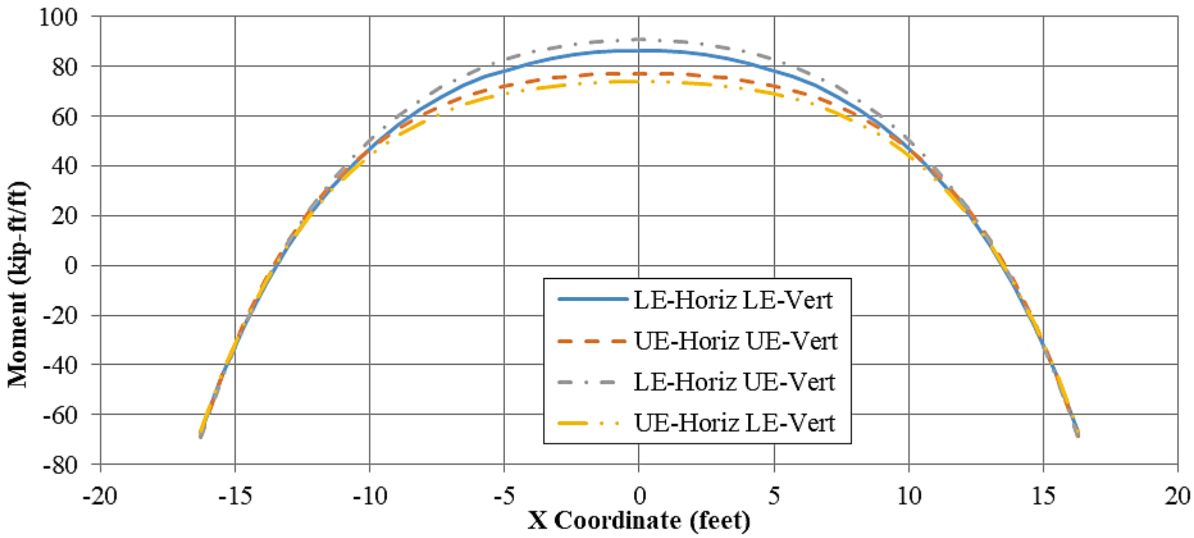
A. Analysis Results showing the Arch Moment due to permanent loads.



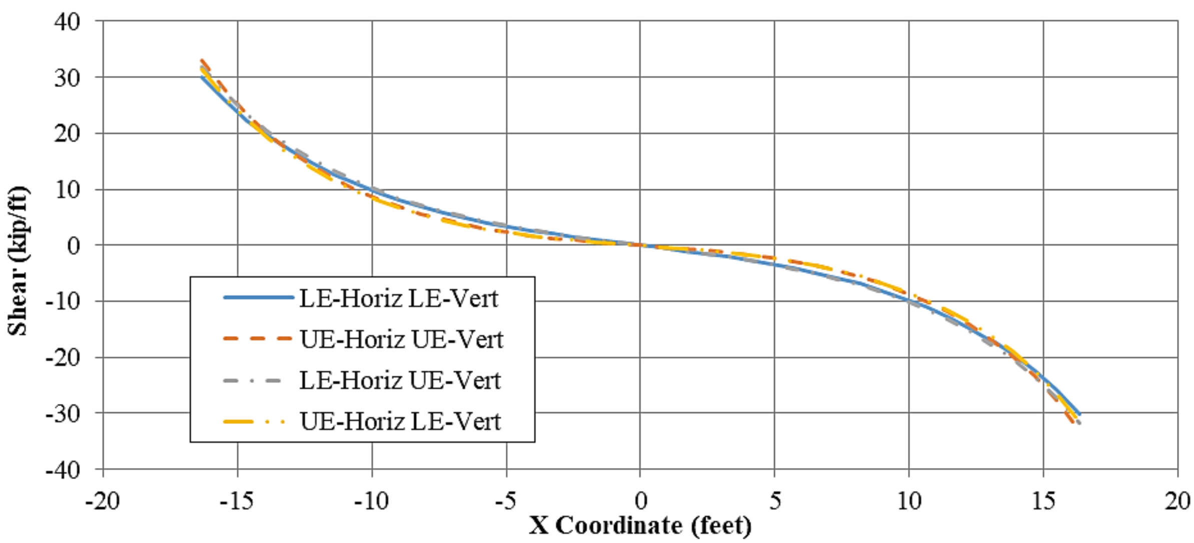
B. Analysis Results showing the Arch Shear due to permanent loads.

Source: FHWA

Figure 99. Illustrations. Analysis Results. Arch forces due to permanent loads (Phase 10).



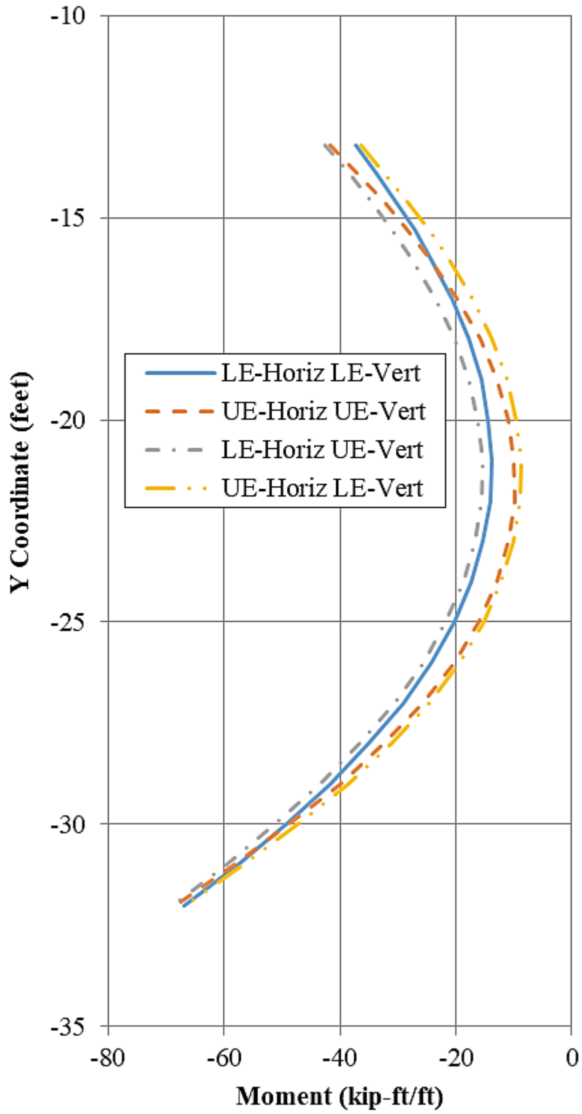
A. Analysis Results showing the Bottom Slab Moment due to permanent loads.



B. Analysis Results showing the Base Slab Shear due to permanent loads.

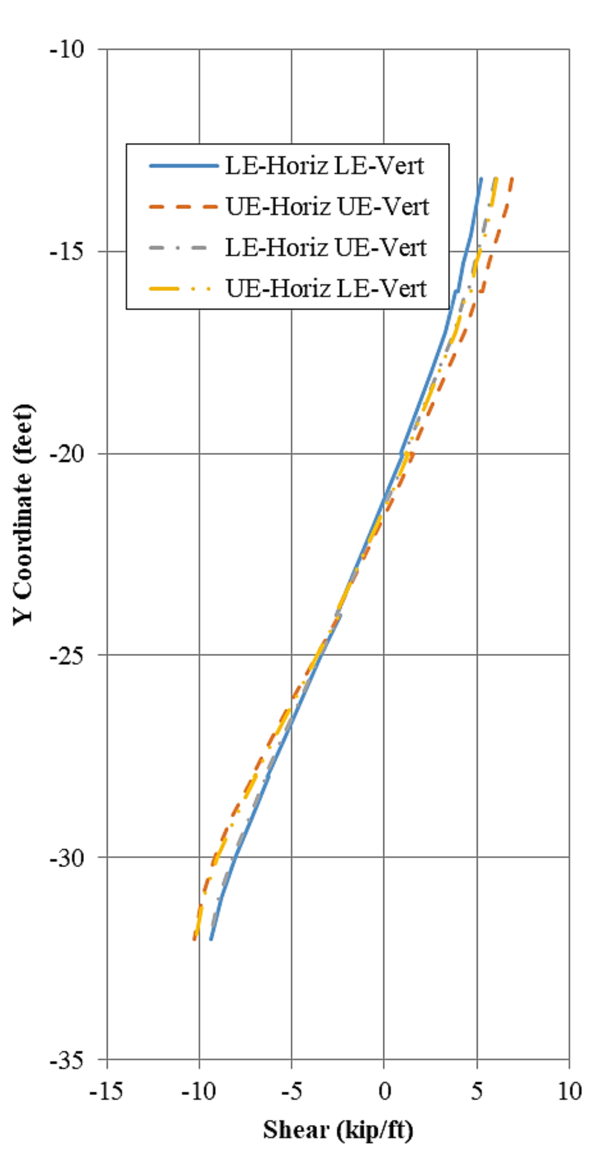
Source: FHWA

Figure 100. Illustrations. Analysis Results. Bottom slab forces due to permanent loads (Phase 10).



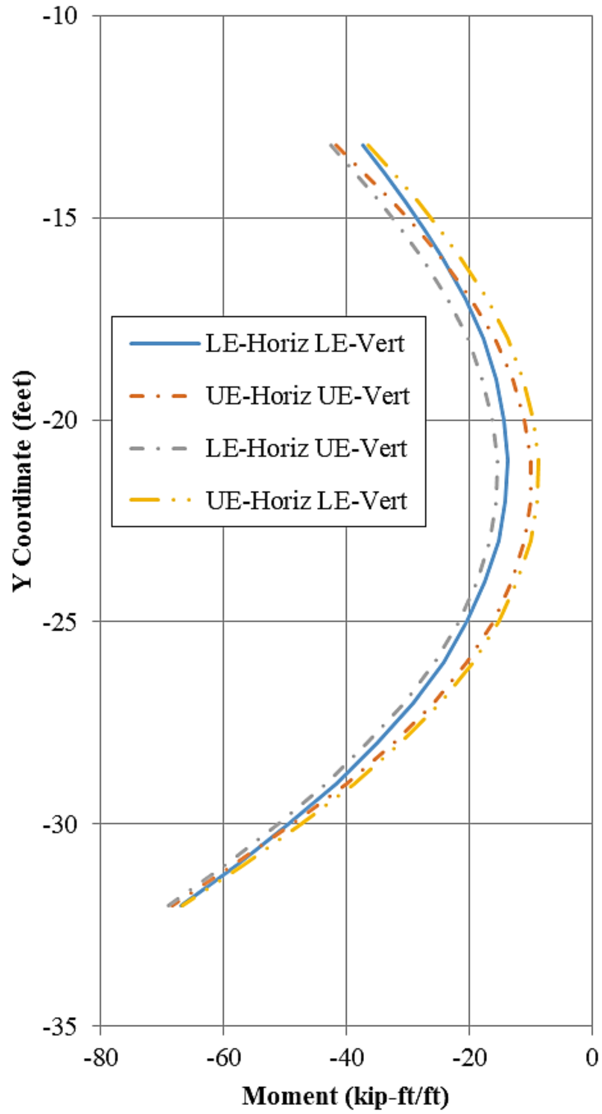
Source: FHWA

Figure 101. Illustrations. Analysis Results. Left wall moment due to permanent loads (Phase 10).



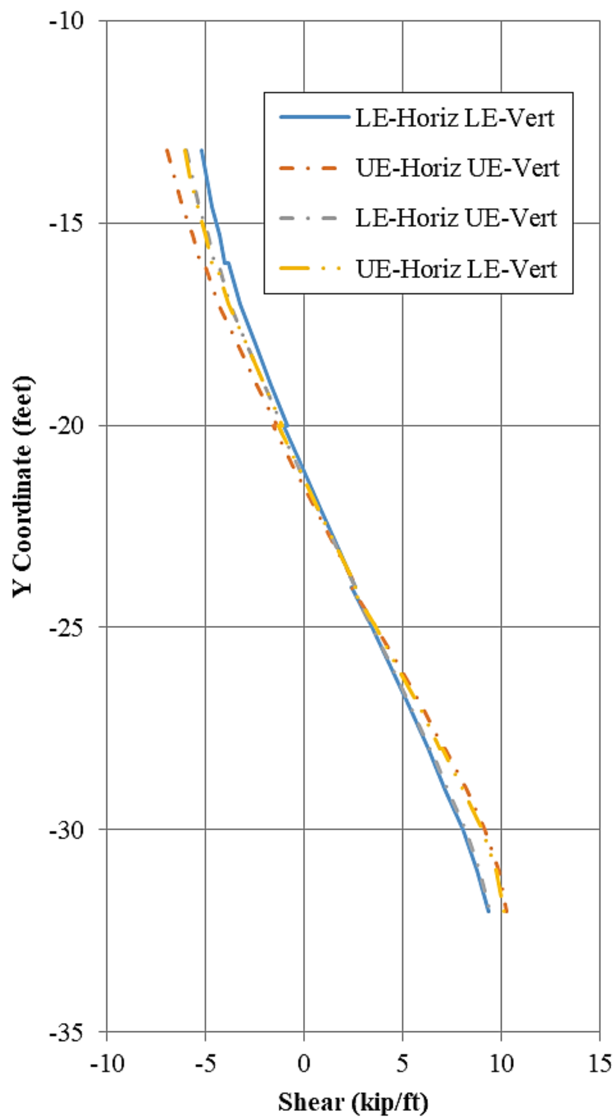
Source: FHWA

Figure 102. Illustrations. Analysis Results. Left wall shear due to permanent loads (Phase 10).



Source: FHWA

Figure 103. Illustrations. Analysis Results. Right wall moment due to permanent loads (Phase 10).



Source: FHWA

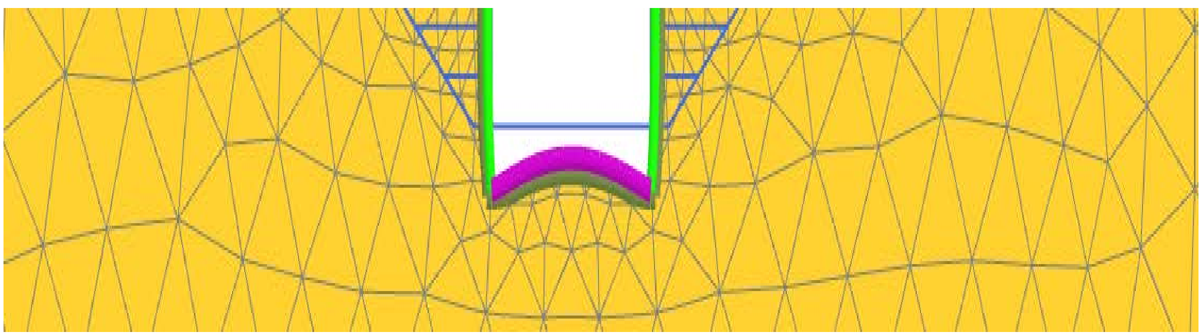
Figure 104. Illustrations. Analysis Results. Right wall shear due to permanent loads (Phase 10).

9.5.4.2 Structure Demands due to Permanent and Live Loads

9.5.4.2.1 HL-93 Tandem

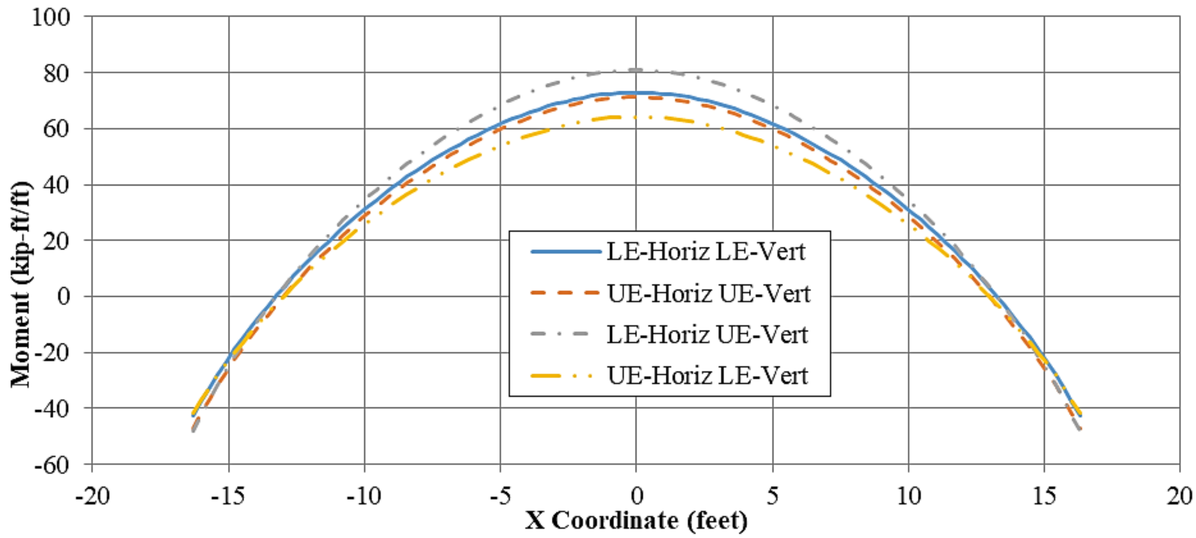
The HL-93 Tandem load consists of a pair of equal loads spaced at 4 feet apart. The critical maximum location for this fairly straightforward load configuration is assumed to be centered over mid-span of the Arch (moving load analysis is not necessary). As explained in Section 9.5.3, the live load is activated in analysis Phase 11. At the end of Phase 11, all un-factored permanent loads and the un-factored HL-93 Tandem live load are accounted for in the model.

A representative deformed mesh after Phase 11 for the HL-93 Tandem load is presented in Figure 105. Moment and shear force diagrams due to the permanent plus live loads are presented in Figure 106, Figure 107, Figure 108, Figure 109, Figure 110, and Figure 111 for the Arch, Bottom Slab, Left Wall, and Right Wall, respectively, considering each of the four soil property combinations listed in Section 9.5.2.7.

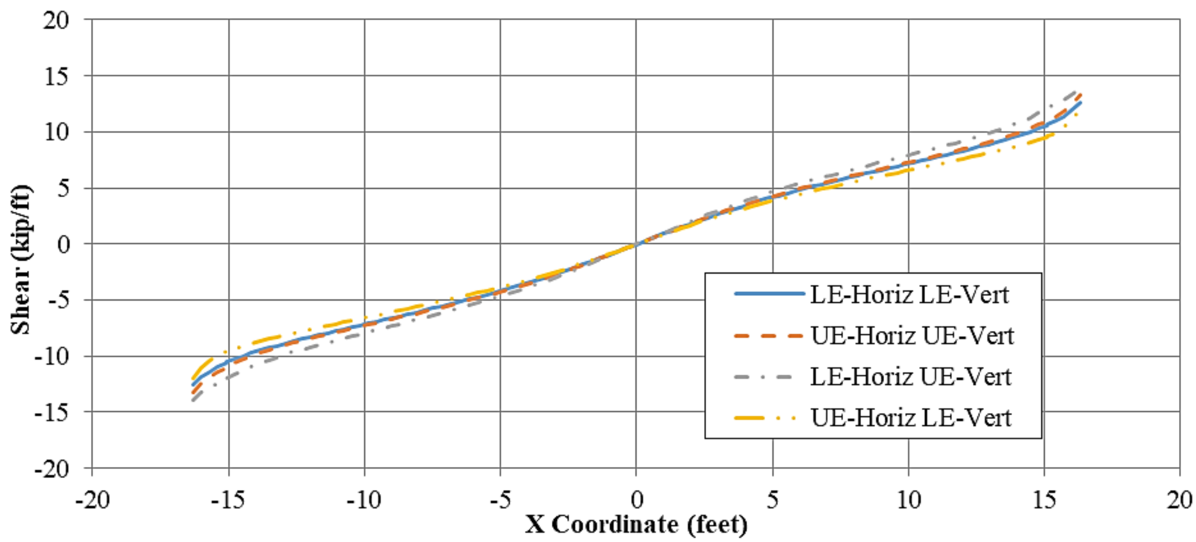


Source: FHWA

Figure 105. Illustration. Exaggerated (100 times) deformed mesh after applying HL-93 Tandem live load (Phase 11).



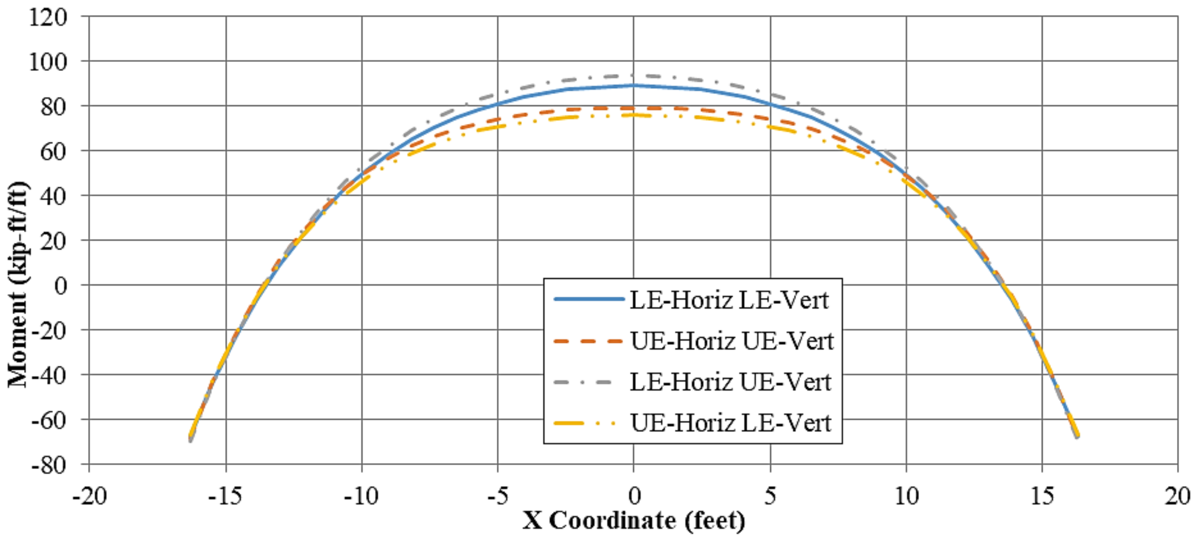
A. Analysis Results showing the Arch Moment due to permanent loads plus HL-93 Tandem live load.



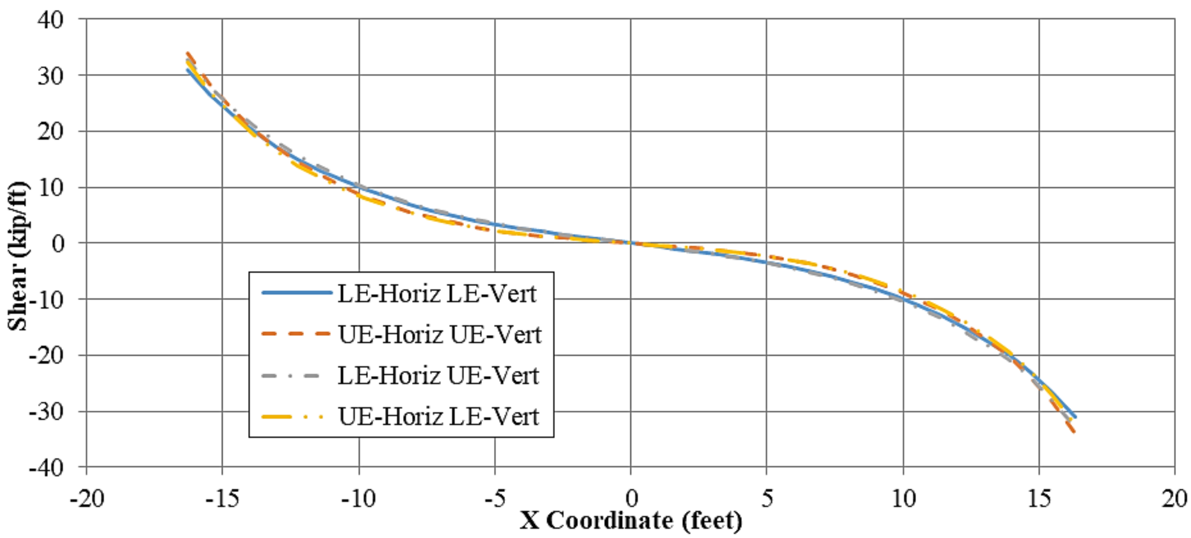
B. Analysis Results showing the Arch Shear due to permanent loads plus HL-93 Tandem live load.

Source: FHWA

Figure 106. Illustrations. Analysis Results. Arch forces due to permanent loads plus HL-93 Tandem live load (Phase 11).



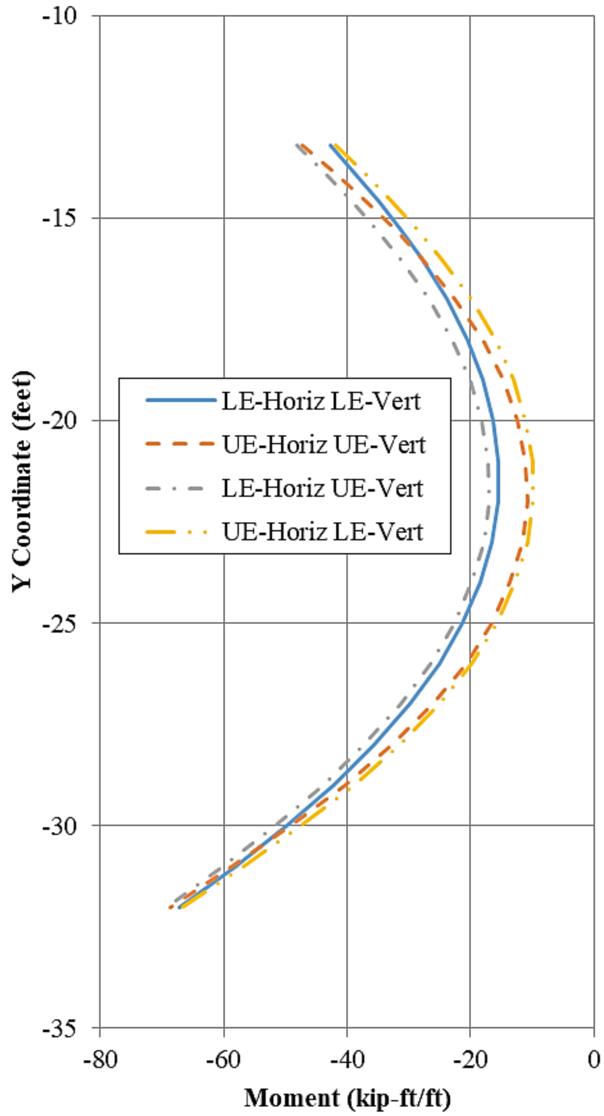
A. Analysis Results showing the Base Slab Moment due to permanent loads plus HL-93 Tandem live load.



B. Analysis Results showing the Base Slab Shear due to permanent loads plus HL-93 Tandem live load.

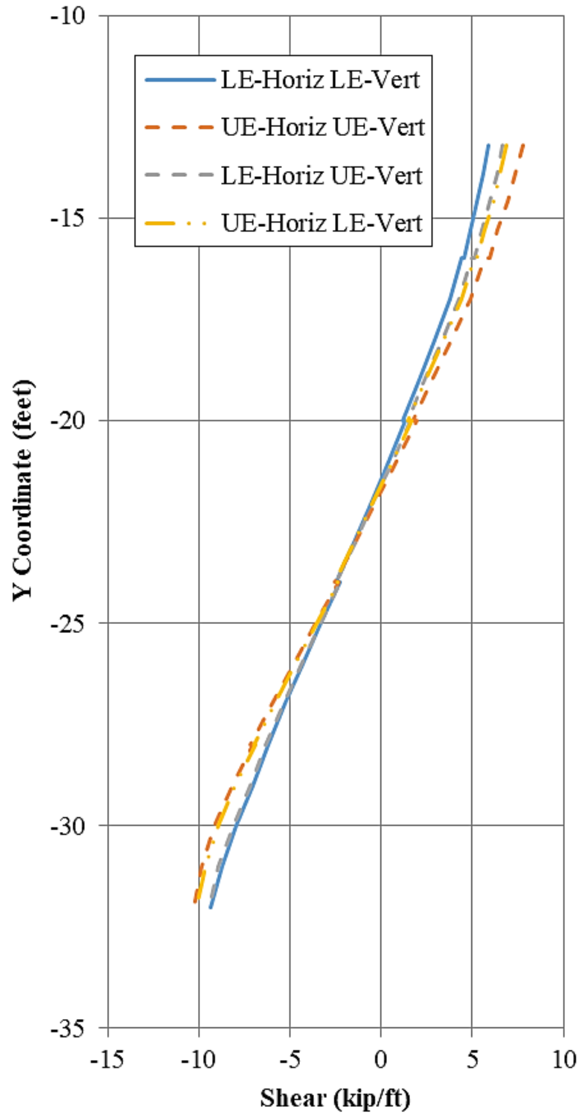
Source: FHWA

Figure 107. Illustrations. Analysis Results. Bottom slab forces due to permanent loads plus HL-93 Tandem live load (Phase 11).



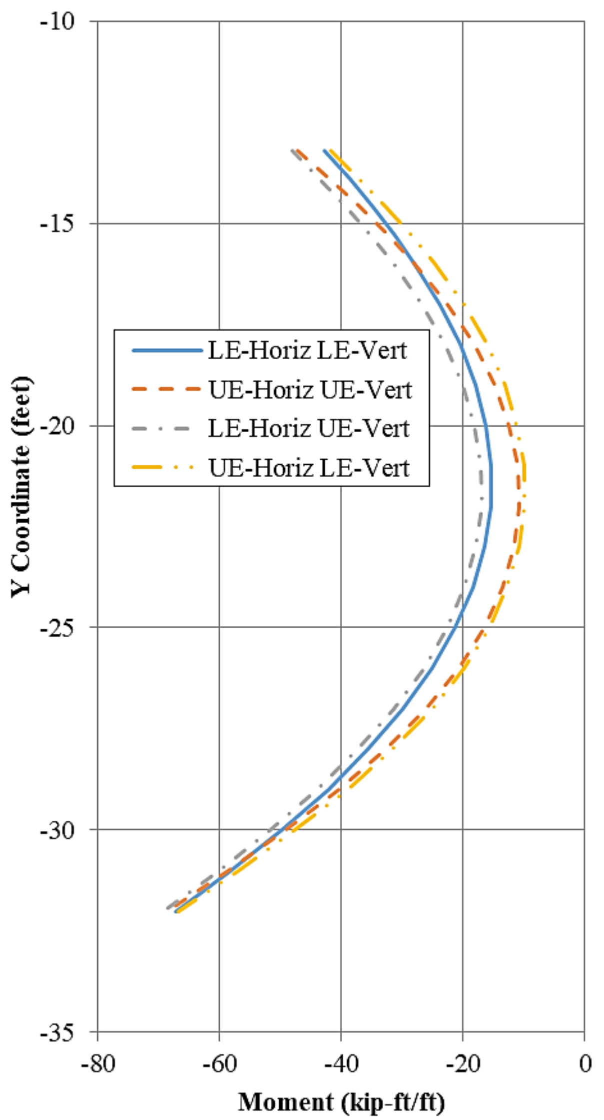
Source: FHWA

Figure 108. Illustration. Analysis Results. Left wall moment due to permanent loads plus HL-93 Tandem live load (Phase 11).



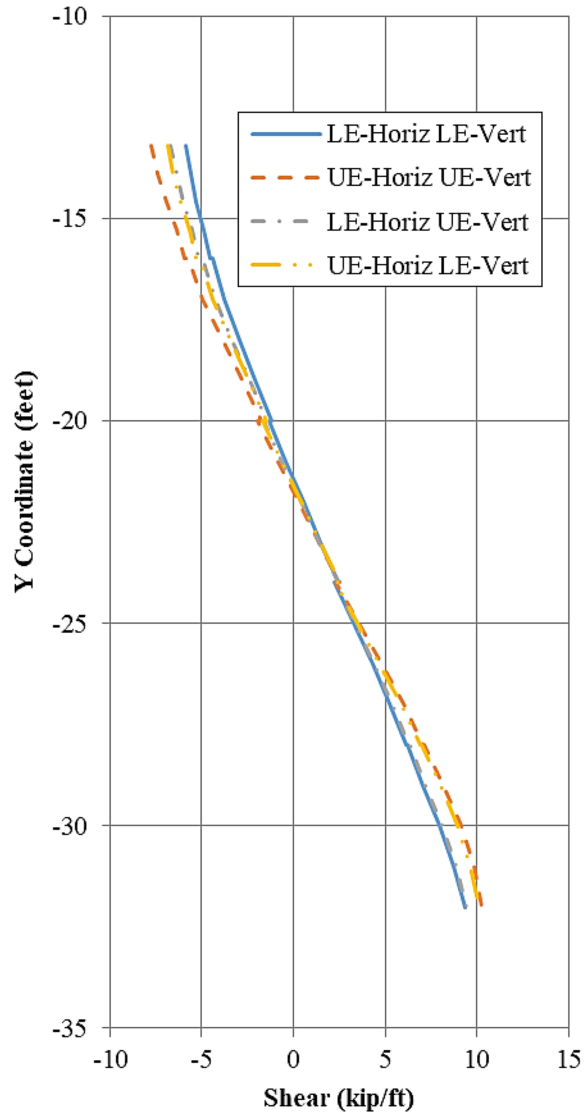
Source: FHWA

Figure 109. Illustration. Analysis Results. Left wall moment due to permanent loads plus HL-93 Tandem live load (Phase 11).



Source: FHWA

Figure 110. Illustration. Analysis Results. Right wall moment due to permanent loads plus HL-93 Tandem live load (Phase 11).



Source: FHWA

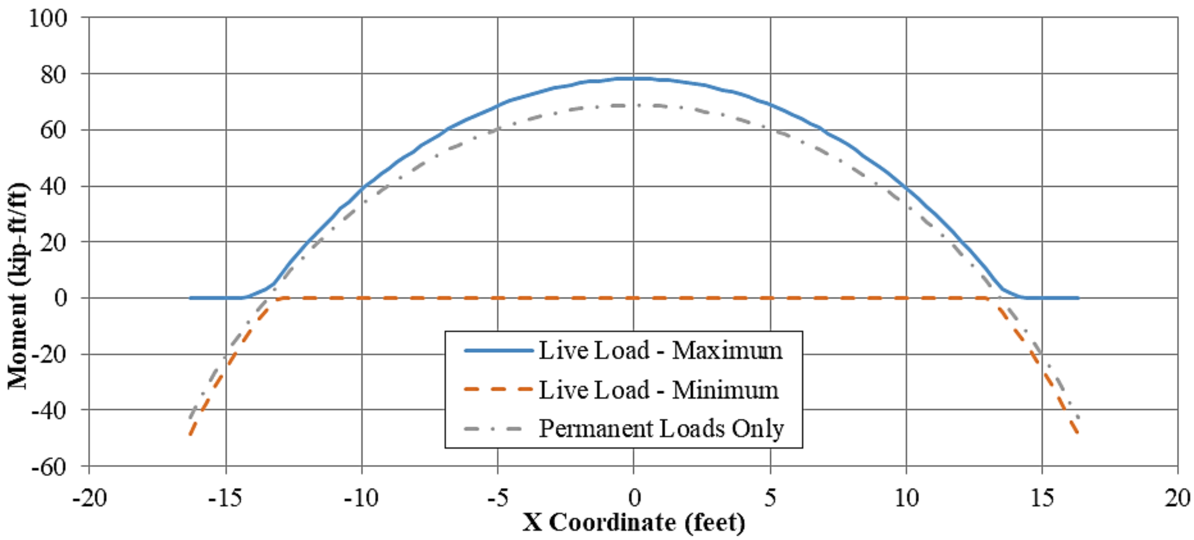
Figure 111. Illustration. Analysis Results. Right wall shear due to permanent loads plus HL-93 Tandem live load (Phase 11).

9.5.4.2.2 HL-93 Truck

The HL-93 Truck consists of an 8-kip load and two 32-kip loads each spaced at 14 feet. Determining the critical location of this vehicle load is not as straightforward as the Tandem load described in Section 9.5.4.2.1. A moving load analysis was performed for the HL-93 Truck in order to determine the critical vehicle load location in FE SSI modeling. The method was introduced in Section 9.5.2.6.2.

The moving load analysis was conducted by running a dynamic analysis in which force-time functions were assigned to all of the surface loads. Only one soil property combination was considered in the dynamic phase: LE in the horizontal pressure zone and UE in the vertical pressure zone. This combination was found to produce the maximum Arch moments in the previous analyses for the HL-93 Tandem. The critical location of the vehicle load is expected to be insensitive to minor changes in soil properties.

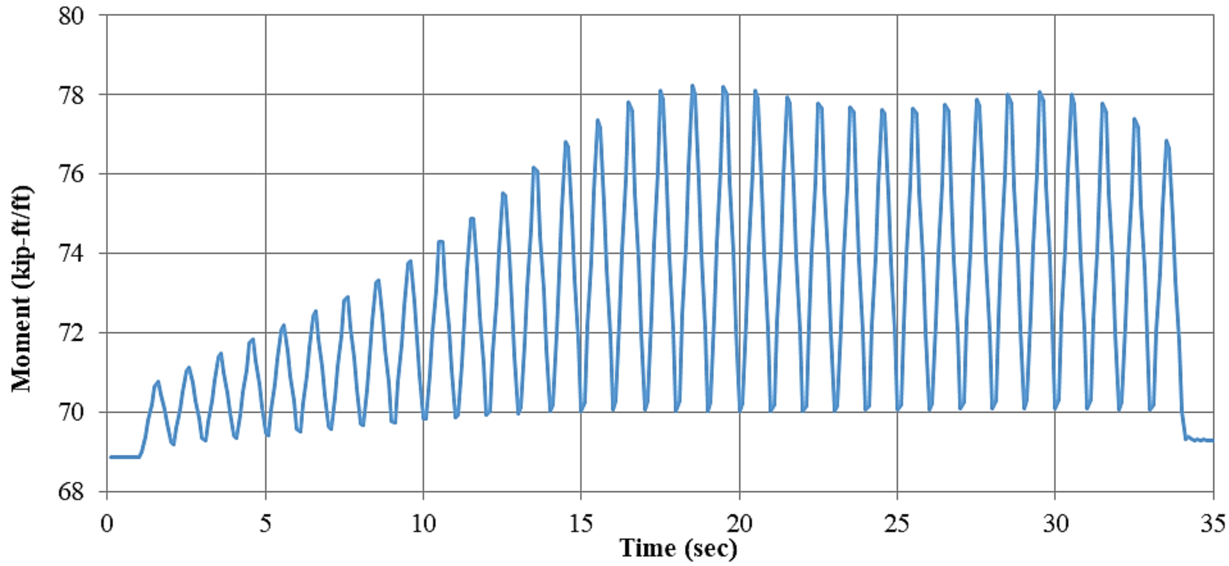
After conducting the dynamic analysis, force envelopes were inspected for each structural element. Figure 112 presents the moment envelope for the Arch. Note that the zero values on this plot are an outlier of the numerical modeling sequence and should be ignored. As expected, the maximum moment occurs near the Arch mid-span.



Source: FHWA

Figure 112. Illustration. Analysis Results. Arch moment envelopes from dynamic moving load analysis for HL-93 Truck.

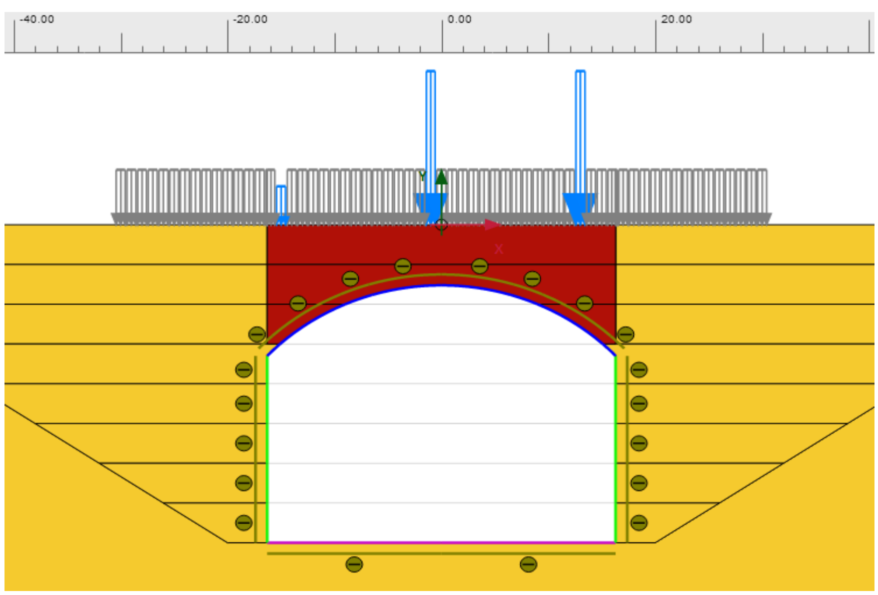
In order to determine the vehicle location which causes this maximum moment, the moment in the Arch at mid-span is plotted next as a function of time in Figure 113. The peaks of the moment-time history occur at the instants where the truck loads are fully applied, at 1-second intervals. The maximum moment of slightly over 78 kip-ft/ft occurs at 18.5 seconds.



Source: FHWA

Figure 113. Illustration. Analysis Results. Moment-time history at Arch mid-span from dynamic moving load analysis for HL-93 Truck.

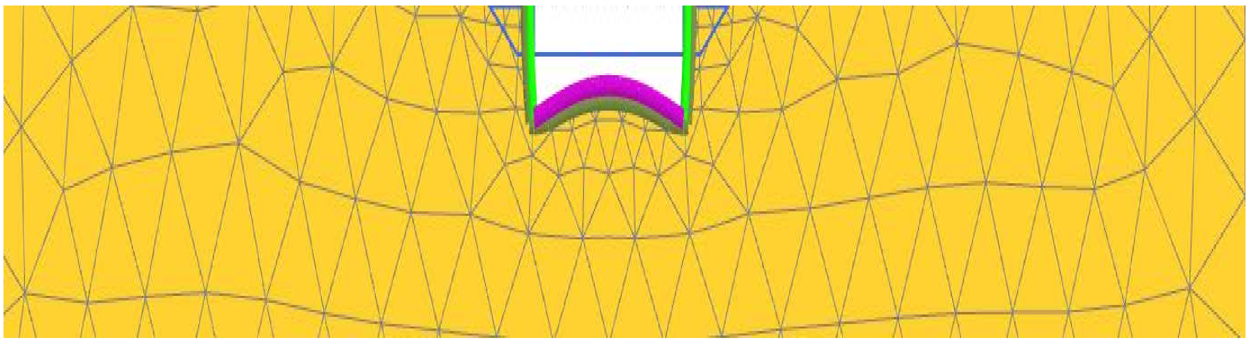
At 18.5 seconds in the analysis, the front, middle, and rear wheel loads are positioned at X = -15, -1, and +13, respectively, as illustrated in Figure 114. After this location was determined, a static analysis was conducted for the HL-93 Truck live load considering all four soil property combinations, similar to the analysis described in Section 9.5.2. Results from the moving load (dynamic) and static analyses were compared and found to be very similar, as expected (e.g., less than 1% difference in maximum moments).



Source: FHWA

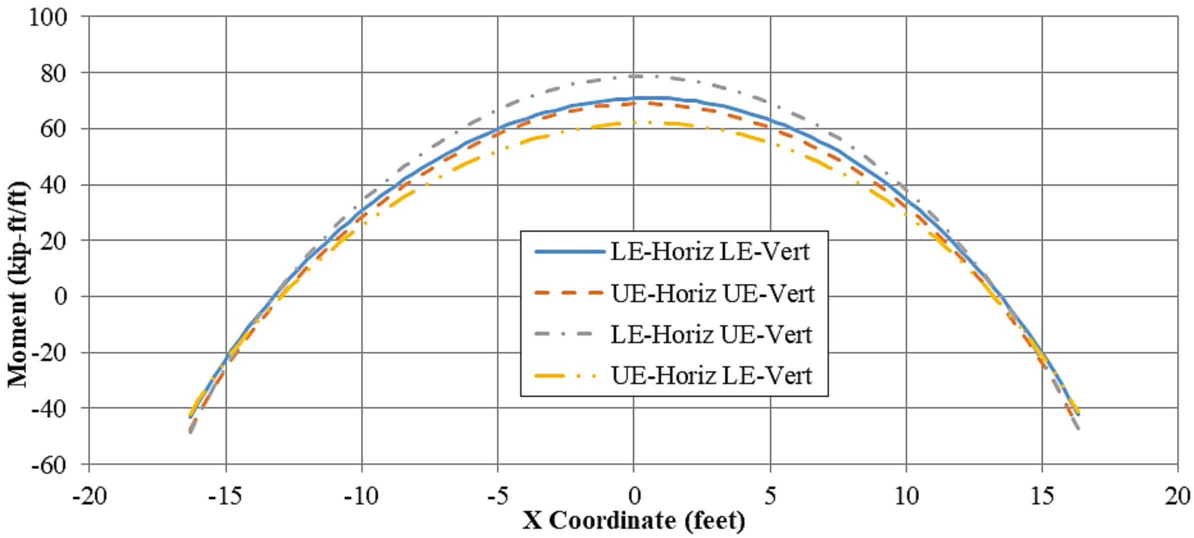
Figure 114. Illustration. Critical wheel load location for analysis of HL-93 Truck.

A representative deformed mesh after Phase 11 for the HL-93 Truck load is presented in Figure 115. Moment and shear force diagrams due to the permanent plus live loads are presented in Figure 116, Figure 117, Figure 118, Figure 119, Figure 120, and Figure 121 for the Arch, Bottom Slab, Left Wall, and Right Wall, respectively, considering each of the four soil property combinations listed in Section 9.5.2.

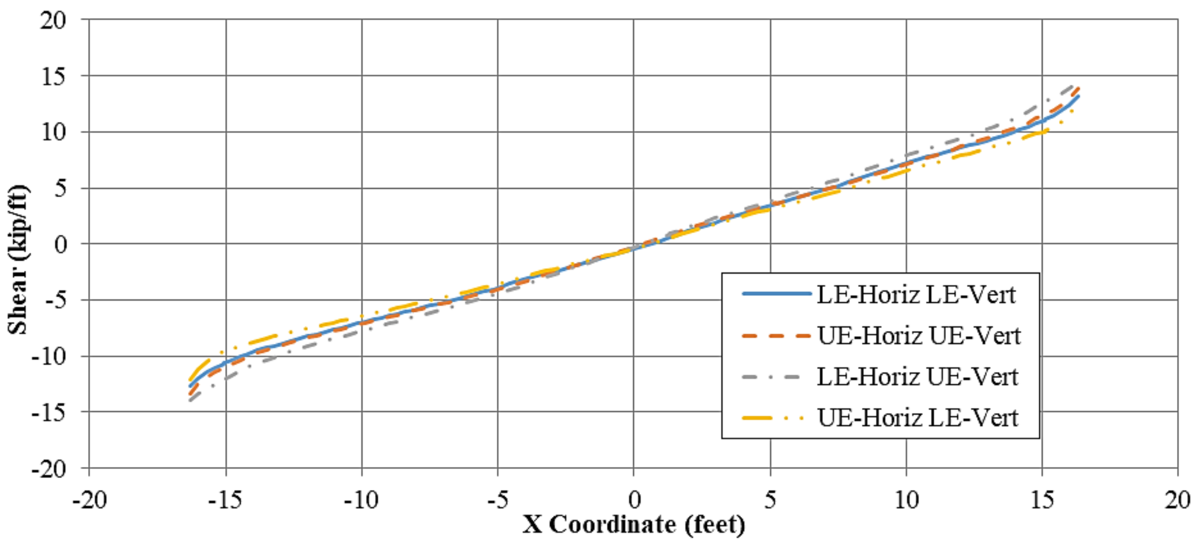


Source: FHWA

Figure 115. Illustration. Exaggerated (100 times) deformed mesh after applying HL-93 Truck live load (Phase 11).



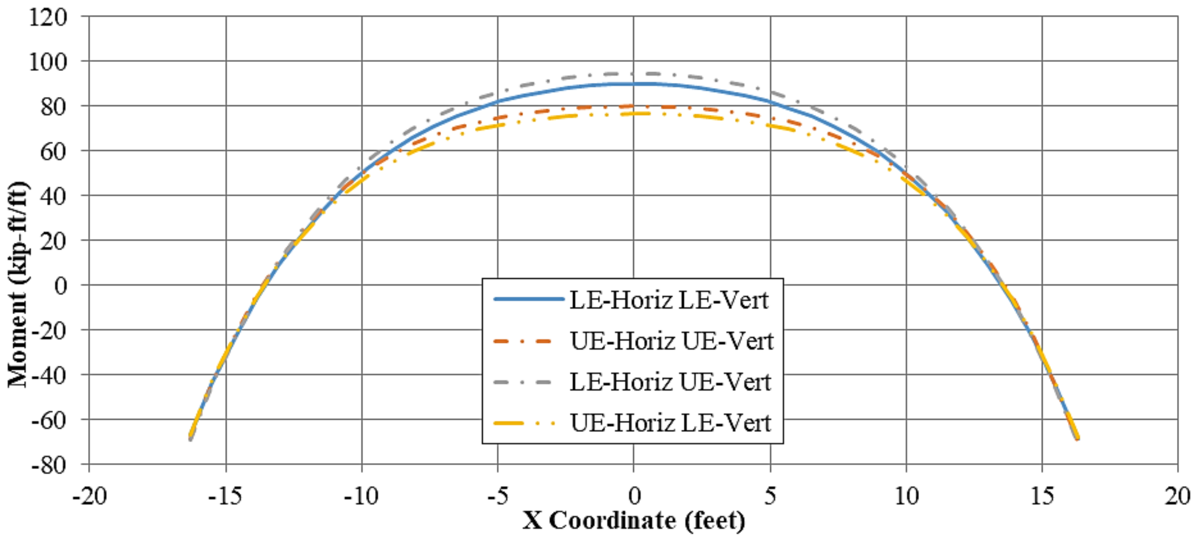
A. Analysis Results showing the Arch Moment due to permanent loads plus HL-93 Truck live load.



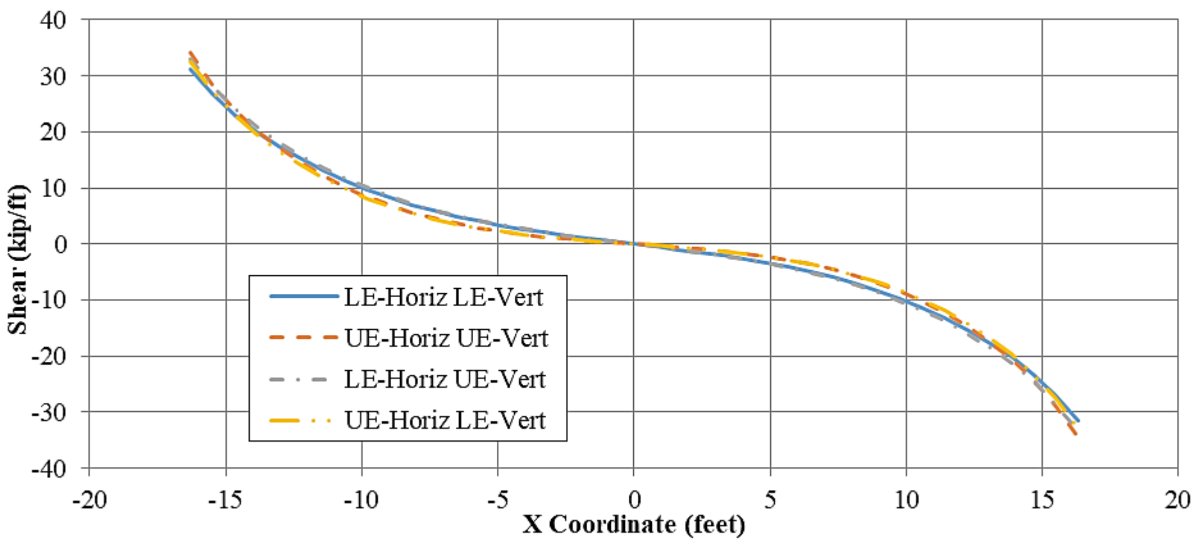
B. Analysis Results showing the Arch Shear due to permanent loads plus HL-93 Truck live load.

Source: FHWA

Figure 116. Illustrations. Analysis Results. Arch forces due to permanent loads plus HL-93 Truck live load (Phase 11).



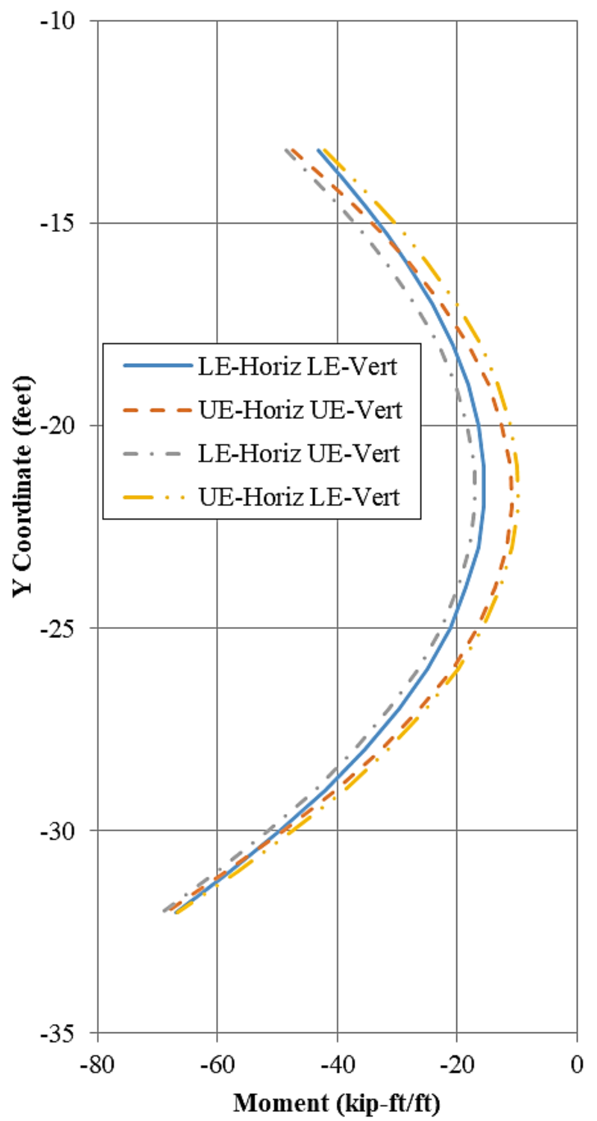
A. Analysis Results showing the Bottom Slab Moment due to permanent loads plus HL-93 Truck live load.



B. Analysis Results showing the Base Slab Shear due to permanent loads plus HL-93 Truck live load.

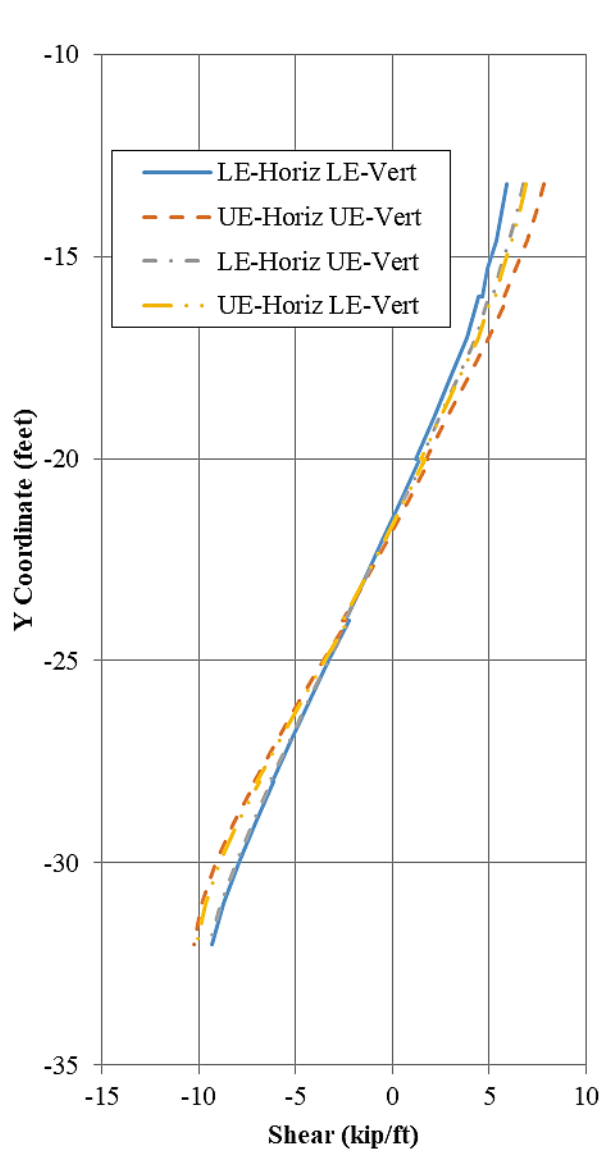
Source: FHWA

Figure 117. Illustrations. Analysis Results. Base slab forces due to permanent loads plus HL-93 Truck live load (Phase 11).



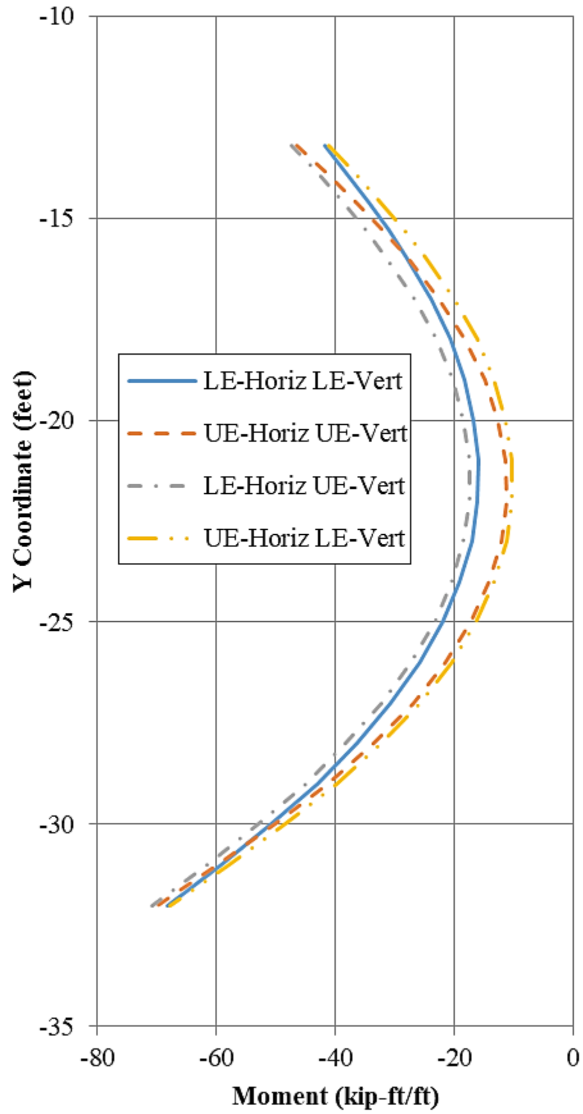
Source: FHWA

Figure 118. Illustration. Analysis Results. Left wall moment due to permanent loads plus HL-93 Truck live load (Phase 11).



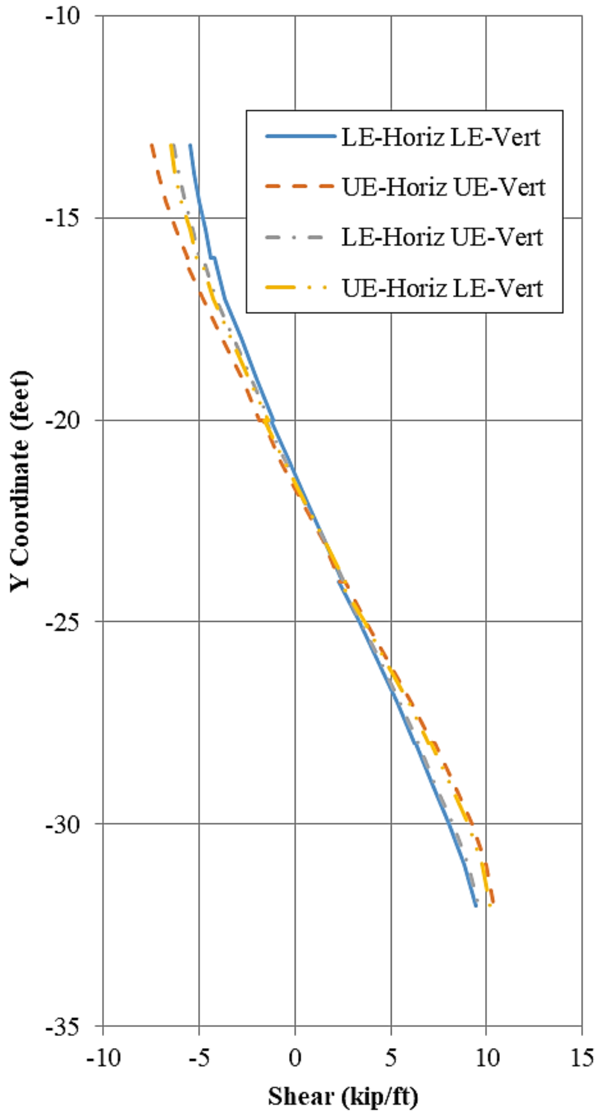
Source: FHWA

Figure 119. Illustration. Analysis Results. Left wall shear due to permanent loads plus HL-93 Truck live load (Phase 11).



Source: FHWA

Figure 120. Illustration. Analysis Results. Right wall moment due to permanent loads plus HL-93 Truck live load (Phase 11).



Source: FHWA

Figure 121. Illustration. Analysis Results. Right wall shear due to permanent loads plus HL-93 Truck live load (Phase 11).

When considering the position of vehicle loads, it is possible for the structural elements to have different locations which lead to the most critical force conditions. As a result, critical locations should be determined for each component and static live load analysis should be performed at all critical wheel load locations when performing actual load ratings.

9.5.4.2.3 Factored Load Effects

Controlling permanent and live load shear and moment diagrams are discretized to obtain controlling permanent and live loads under upper and lower extremes. These limits are then matched with the controlling force envelopes previously discussed. Due to program procedures, permanent loads cannot be separated into dead loads, earth loads and wearing surface loads and are therefore all compiled into the same shear and moment diagrams. As a result, a conservative load factor of 1.35 is applied to all permanent loads. Additionally, upper and lower estimate soil parameters were used to capture maximum and minimum effects and consequently a single load factor is applied. Therefore, a load factor of 1.35 is applied to permanent loads and a load factor of 1.75 is applied to live loads. It should also be noted that live load surcharge is encapsulated in the live load effects. Impact is accounted for in the FE model moving load analysis.

Dead load, earth loads and live loads were combined and factored in the program to produce strength and service envelopes due to the inherent nature of advanced analysis. The resulting shear and moment diagrams for service and strength are presented in Table 53 through Table 55:

Table 53. Exterior wall shears and moments.

Sp Pt	Moments (k-ft) Ser. I +	Moments (k-ft) Ser. I -	Moments (k-ft) Str I +	Moments (k-ft) Str I -	Shear (k) Str I +	Shear (k) Str I -
1.0	-36.46	-47.42	-49.23	-66.30	8.22	10.89
1.1	-26.91	-35.17	-36.33	-49.21	7.09	9.39
1.2	-21.40	-28.10	-28.89	-39.35	6.10	8.39
1.3	-13.25	-17.52	-17.89	-24.55	3.74	5.17
1.4	-9.67	-12.55	-13.05	-17.54	1.67	2.67
1.5	-8.95	-10.84	-12.09	-15.00	-0.84	-0.28
1.6	-12.21	-13.65	-16.49	-18.67	-3.46	-3.25
1.7	-18.58	-19.94	-25.08	-27.12	-6.11	-6.14
1.8	-31.17	-32.78	-42.08	-44.42	-9.43	-9.63
1.9	-43.81	-45.61	-59.14	-61.72	-11.66	-11.76
1.10	-66.62	-68.63	-89.93	-92.76	-13.71	-13.80

Table 54. Arch shears and moments.

Sp Pt	Moments (k-ft) Ser. I +	Moments (k-ft) Ser. I -	Moments (k-ft) Str I +	Moments (k-ft) Str I -	Shear (k) Str I +	Shear (k) Str I -
2.0	-48.56	-47.42	-67.96	-66.30	-19.13	-18.22
2.1	3.59	0.38	3.85	-0.55	-13.69	-12.55
2.2	35.05	29.17	47.85	39.71	-11.11	-10.21
2.3	59.86	51.93	83.19	72.21	-8.31	-7.65
2.4	74.91	65.76	105.14	92.46	-4.93	-4.57
2.5	80.80	71.23	113.88	100.62	-0.53	-0.47
2.6	74.95	65.85	105.22	92.61	4.94	4.59
2.7	61.94	53.68	86.82	75.26	8.32	7.67
2.8	38.67	32.67	54.18	45.84	11.11	10.22
2.9	3.80	0.50	4.21	-0.35	14.28	13.21
2.10	-48.12	-47.17	-67.19	-65.87	20.05	19.16

Table 55. Bottom slab shears and moments.

Sp Pt	Moments (k-ft) Ser. I +	Moments (k-ft) Ser. I -	Moments (k-ft) Str I +	Moments (k-ft) Str I -	Shear (k) Str I +	Shear (k) Str I -
4.1	-69.33	-68.63	-93.78	-92.76	44.97	46.45
4.1	10.99	11.24	15.66	15.89	24.75	23.36
4.2	55.80	51.48	76.57	70.50	13.63	11.40
4.3	79.83	70.35	109.19	96.04	6.94	4.98
4.4	91.24	78.00	124.67	106.38	2.87	1.72
4.5	94.58	80.00	129.21	109.10	0.02	0.03
4.6	91.29	78.12	124.77	106.58	-2.85	-1.70
4.7	79.86	70.51	109.25	96.31	-7.00	-4.99
4.8	55.65	51.52	76.31	70.56	-13.78	-11.54
4.9	10.58	10.93	14.95	15.36	-25.10	-23.70
4.10	-70.80	-69.93	-96.35	-95.04	-45.67	-47.21

9.5.5 Resistance Calculations

The depth, analysis section width, and reinforcement for each of the critical sections are tabulated below in Table 56.

Table 56. Critical section data.

Section	Depth (in)	Width (in)	Bar Number	Bar Spacing (in)	Cover (in)
1 – Arch Center	27.00	12.000	8	6.50	2.00
2 – Bottom Slab Center	23.00	12.000	9	6.00	2.50
3 – Arch End	27.00	12.000	8	6.00	2.00
4 – Bottom Slab End	23.00	12.000	8	6.00	2.00
5 – Exterior Wall End	18.00	12.000	8	6.00	2.00
6 – Exterior Wall Middle	18.00	12.000	5	7.00	2.00

Concrete Properties:

f'_c = 4.0 ksi
$\propto_1$ = 0.85 LRFD BDS 5.6.2.2
β_1 = 0.85 if $f'_c \leq 4.0$ ksi LRFD BDS 5.6.2.2
λ = 1.0 (normal weight concrete) LRFD BDS 5.4.2.8
γ_3 = 0.67 (AASHTO M31 Grade 60) LRFD BDS 5.6.3.3
γ_1 = 1.6 LRFD BDS 5.6.3.3

Modulus of rupture:

$$f_r = 0.24\lambda\sqrt{f'_c} = 0.24(1.0)\sqrt{4.0} = 0.480 \text{ ksi}$$ LRFD BDS 5.4.2.6

Compression reinforcement in flexural capacity calculations is conservatively ignored. Calculated results are based on a per foot analysis width.

9.5.5.1 Section 1 – Arch Center

Rectangular section height:	h	=	27.00 inch	
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	0.785 inch^2	
Diameter of rebar:	dia_{bar}	=	1.00 inch	
Spacing of rebar:	s	=	6.50 inch	
Flexural resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5

Moment Resistance:

Determine equivalent area of reinforcing bar:

$$A_s = \frac{A_{s\_bar}b}{s} = \frac{0.785(12.00)}{6.5} = 1.45 \text{ inch}^2$$

Determine distance of the extreme compression fiber to the centroid of the reinforcement:

$$d_s = h - \frac{dia_{bar}}{2} - clr = 27.00 - \frac{1.00}{2} - 2.00 = 24.50 \text{ inches}$$

Determine distance from the equivalent stress block for tension controlled, non-prestressed tension reinforcement only:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.45(60)}{0.85(4.0)(12.00)} = 2.13 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(1.45)(60)\left(24.50 - \frac{2.13}{2} \right)\left(\frac{\text{ft}}{12 \text{ inch}} \right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 152.90 \text{ kip-ft}$$

Minimum Steel Requirements:

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(27.00)^2}{6} = 1458.00 \text{ inch}^3$$ LRFD BDS 5.6.3.3

The cracking moment equation simplifies to the following (only monolithic sections and no prestress forces):

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(1458)\left(\frac{\text{ft}}{12\text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.3-1

$$= 62.52 \text{ kip-ft}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Shear resistance is critical at the end of the span and therefore shear resistance does not need to be checked at mid-span.

Axial Thrust:

Axial forces were observed to be significantly smaller in comparison to the shears and moments. By inspection, axial thrust is conservatively neglected in this problem, similar to that shown in Section 9.4.6.5.

9.5.5.2 Section 2 – Bottom Slab Center

Rectangular section height:	h	=	23.00 in – 1 in = 22.00 inch	
Note: 1 inch is subtracted from thickness since it was cast against ground.				
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.50 inch	
Area of single rebar:	A_{s_bar}	=	1.00 inch^2	
Diameter of rebar:	dia_{bar}	=	1.128 inch	
Spacing of rebar:	s	=	6.0 inch	
Flexural resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5

Moment Resistance:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{1.00(12.00)}{6.00} = 2.00 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 22.00 - \frac{1.128}{2} - 2.50 = 18.94 \text{ inch}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.00(60)}{0.85(4.0)(12.00)} = 2.94 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2}\right) =$$

$$0.9(2.00)(60)\left(18.94 - \frac{2.94}{2}\right)\left(\frac{\text{ft}}{12\text{in}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 157.23 \text{ kip-ft}$$

Minimum Steel Requirements:

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(22.00)^2}{6} = 968.00 \text{ inch}^3$$ LRFD BDS 5.6.3.3

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(968)\left(\frac{\text{ft}}{12 \text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.3-1

$$= 41.51 \text{ kip-ft}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Shear resistance is critical at the end of the span and therefore shear resistance does not need to be checked at mid-span.

9.5.5.3 Section 3 – Arch End

Rectangular section height:	h	=	27.00 inch	
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	0.785 inch^2	
Diameter of rebar:	dia_{bar}	=	1.00 inch	
Spacing of rebar:	s	=	6 inch	
Resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5
	ϕ_v	=	0.85	LRFD BDS 12.5.5

Moment Resistance

Note: The flexural reinforcement being evaluated is the horizontal leg of the exterior wall bar as depicted in Figure 85. Load rater should verify the development length of rebar. If not fully developed, reduction in reinforcing steel area should be used. The section at rebar cut-off location needs to be considered in the load rating.

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.785(12.00)}{6} = 1.57 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 27.00 - \frac{1.00}{2} - 2.00 = 24.50 \text{ inch}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.57(60)}{0.85(4.0)(12.00)} = 2.31 \text{ inch}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(1.57)(60)\left(24.50 - \frac{2.31}{2} \right)\left(\frac{\text{ft}}{12\text{in}} \right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 164.93 \text{ kip-ft}$$

Minimum Steel Requirements:

Since $\varphi_f M_n > M_{cr} = 62.52$ k-ft, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Critical section for shear is d_v from the face of the support in accordance with LRFD BDS 5.7.3.2.

$$d_v = \max \begin{Bmatrix} 0.72h \\ 0.9d \\ d - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(27) = 19.44 \\ 0.9(24.50) = 22.05 \\ 24.50 - 2.31/2 = 23.35 \end{Bmatrix} = 23.35 \text{ inch}$$ LRFD BDS 5.7.2.8

Determining the nominal shear resistance requires the use of the applied shears and moments at the section of interest. Separate shears and moments will be required for every live load factor being evaluated. However, in this example, it was identified that the HL-93 will control live load ratings and therefore only this live load is considered for the shear resistance. The concurrent, critical factored shears and moments of HL-93 at this location are:

$$M_u @ d_v = 67.96 \text{ k-ft}$$
$$V_u @ d_v = 20.05 \text{ kip}$$

Using these numbers, the nominal shear resistance when subjected to the HL-93, in accordance with LRFD BDS 5.12.7.3 is:

$$\phi V_n = \phi_v \left[0.0948\sqrt{f_c'} \le \left(0.0676\sqrt{f_c'} + 4.6 \frac{A_s}{bd_e} \frac{V_u d_e}{M_u} \right) \le 0.126\sqrt{f_c'} \right] bd_e$$

$$= 0.85\left[0.0948\sqrt{4.0} \le 0.0676\sqrt{4.0} + 4.6 \frac{1.57(20.05)(24.50)}{12(24.50)(67.96 \times 12)} \le 0.126\sqrt{4.0} \right] 12(24.50)$$

$$= 0.85\left[0.19 \le 0.15 \le 0.25 \right] 12(24.50) = 37.48 \text{ kip}$$

Note that the value of $V_u d_e / M_u$ was checked and found to be less than 1.0, where d_e is the distance from the maximum compressive fiber to the centroid of tension steel.

Axial Thrust:

Axial forces were observed to be significantly smaller in comparison to the shears and moments. By inspection, axial thrust is conservatively neglected in this problem, similar to that shown in Section 9.4.6.5.

9.5.5.4 Section 4 – Bottom Slab End

Rectangular section height:	h	=	23.00 inches – 1 inch = 22.00 inches	
Note: 1 inch is subtracted from thickness since it was cast against ground.				
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	0.785 inch^2	
Diameter of rebar:	dia_{bar}	=	1.00 inch	
Spacing of rebar:	s	=	6 inch	
Resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5
	ϕ_v	=	0.85	LRFD BDS 12.5.5

Moment Resistance:

Note: The flexural reinforcement being evaluated is the horizontal leg of the exterior wall bar as depicted in Figure 86.

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.785(12.00)}{6} = 1.57 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 22.00 - \frac{1.00}{2} - 2.00 = 19.50 \text{ inch}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.57(60)}{0.85(4.0)(12.00)} = 2.31 \text{ inch}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(1.57)(60)\left(19.50 - \frac{2.31}{2}\right)\left(\frac{\text{ft}}{12\text{in}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 129.61 \text{ kip-ft}$$

Minimum Steel Requirements:

Since $\varphi_f M_n > M_{cr} = 41.51$ k-ft, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Critical section for shear is d_v from the face of the support in accordance with LRFD BDS 5.7.3.2.

$$d_v = \max\begin{Bmatrix} 0.72h \\ 0.9d \\ d - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(22) = 15.84 \\ 0.9(19.50) = 17.55 \\ 19.50 - 2.31/2 = 18.34 \end{Bmatrix} = 18.34 \text{ inches}$$ LRFD BDS 5.7.2.8

Determining the nominal shear resistance requires the use of the applied shears and moments at the section of interest. Separate shears and moments will be required for every live load factor being evaluated. However, in this example, it was identified that the HL-93 will control live load ratings and therefore only this live load is considered for the shear resistance. The concurrent, critical factored shears and moments of EV3 at this location are:

$$M_u @ d_v = 17.87 \text{ k-ft}$$
$$V_u @ d_v = 31.75 \text{ kip}$$

Using these numbers, the nominal shear resistance, when subjected to the HL-93, in accordance with LRFD BDS 5.12.7.3 is:

$$\phi V_n = \phi_v\left[0.0948\sqrt{f_c^{'}} \le \left(0.0676\sqrt{f_c^{'}} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right) \le 0.126\sqrt{f_c^{'}}\right] bd_e$$
$$= 0.85\left[0.0948\sqrt{4.0} \le 0.0676\sqrt{4.0} + 4.6\frac{1.57(31.75)(19.50)}{12(19.50)(17.87\times 12)} \le 0.126\sqrt{4.0}\right]12(19.50)$$
$$= 0.85\left[0.19 \le 0.22 \le 0.25\right]12(19.50) = 44.61 \text{ kip}$$

Note that the value of $V_u d_e/M_u$ was checked and found to be less than 1.0, where d_e is the distance from the maximum compressive fiber to the centroid of tension steel.

9.5.5.5 Section 5 – Exterior Wall End

Rectangular section height:	h	=	18.00 inch	
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	0.785 inch^2	
Diameter of rebar:	dia_{bar}	=	1.00 inch	
Diameter of aggregate	d_{ag}	=	0.75 inch	
Spacing of rebar:	s	=	6 inch	
Resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5
	ϕ_v	=	0.85	LRFD BDS 12.5.5
	ϕ_c	=	0.70	LRFD BDS 5.5.4.2

Moment Resistance:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.785(12.00)}{6} = 1.57 \text{ in}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 18.00 - \frac{1.00}{2} - 2.00 = 15.50 \text{ in}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.57(60)}{0.85(4.0)(12.00)} = 2.31 \text{ in}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(1.57)(60)\left(15.50 - \frac{2.31}{2}\right)\left(\frac{\text{ft}}{12\text{in}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 101.35 \text{ kip-ft}$$

Walls need to be checked for axial thrust and flexural interaction. However, Example 3 showed axial thrust is insignificant and considering this structure is exposed to less fill, the axial thrust was neglected for this example.

Minimum Steel Requirements:

Check minimum reinforcement requirement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(18.00)^2}{6} = 648 \text{ in}^3$$ LRFD BDS 5.6.3.3

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(648)\left(\frac{\text{ft}}{12 \text{ in}}\right)$$ LRFD BDS Eq. 5.6.3.3-1

$$= 27.78 \text{ kip-ft}$$

Since $\varphi_f M_n > M_{cr}$, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Critical section for shear is d_v from the face of the support in accordance with LRFD BDS 5.7.3.2.

$$d_v = \max \begin{Bmatrix} 0.72h \\ 0.9d \\ d - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(18) = 12.96 \\ 0.9(15.50) = 13.95 \\ 15.50 - 2.31/2 = 14.35 \end{Bmatrix} = 14.35 \text{ in}$$ LRFD BDS 5.7.2.8

Determining the nominal shear resistance requires the use of the applied shears and moments at the section of interest. Separate shears and moments will be required for every live load factor being evaluated. However, in this example, it was identified that the HL-93 will control live load ratings and therefore only this live load is considered for the shear resistance. Additionally, consideration needs to be given to whether the maximum shear force occurs at the top or the bottom of the exterior wall and subsequent shear calculations need to correspond to the critical location. In this example, the critical shear occurs d_v from the top of bottom slab. The concurrent, critical factored shears and moments of EV3 at this location are:

$$M_u @ d_v = 92.76 \text{ k-ft}$$
$$V_u @ d_v = 13.80 \text{ kip}$$

The shear resistance of the walls needs to be evaluated using LRFD BDS 5.7.3.3. Shear factors β and θ need to be calculated since the member thickness is greater than 16 in. and there is no shear reinforcement. The shear factors in accordance with LRFD BDS 5.7.3.4.2, using HL-93 shears and moments shown above, are:

$$M_u \geq V_u d_v \Rightarrow 92.76(12) = 1113.00 \text{ kip-in} \geq 13.80(14.35) = 198.03 \text{ kip-in}$$

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u|\right)}{E_s A_s} = \frac{\left(\frac{|92.76(12)|}{14.35} + 0.5(0) + 13.80\right)}{29000(1.57)} = 0.0020$$

$$0 \leq \varepsilon_s \leq 0.006$$

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500(0.0020) = 36.02^o$$

$$s_x = d_v \leq h - 2CL - d_b = 14.35 \text{ in} \leq 18 - 2(2) - 0.625/2 - 1.00/2 = 13.19 \text{ inch}$$

Where CL is the clear cover;

$$s_{xe} = 12 \text{ in} \leq s_x \frac{1.38}{a_g + 0.63} \leq 80 \text{ inch}$$

$$s_{xe} = 12 \text{ in} \leq 13.19 \frac{1.38}{0.75 + 0.63} \leq 80 \text{ in} \Rightarrow 13.19 \text{ inch}$$

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \frac{51}{(39 + s_{xe})} = \frac{4.8}{[1 + 750(0.0020)]} \frac{51}{[39 + 13.19]} = 1.87$$

$$\phi V_n = \phi_v \left[0.0316\beta\sqrt{f_c^{'}} \leq 0.25 f_c^{'}\right] b_v d_v$$
$$= 0.85\left[0.0316(1.87)\sqrt{4.0} \leq 0.25(4.0)\right] 12(14.35)$$
$$= 17.32 \text{ kip}$$

9.5.5.6 Section 6 –Exterior Wall Mid-span

Rectangular section height:	h	=	18.00 inch	
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	0.307 inch^2	
Diameter of rebar:	dia_{bar}	=	0.625 inch	
Spacing of rebar:	s	=	7 inch	
Resistance factor:	ϕ_f	=	0.9	LRFD BDS 12.5.5
	ϕ_v	=	0.85	LRFD BDS 12.5.5
	ϕ_c	=	0.70	LRFD BDS 5.5.4.2

Moment Resistance:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.307(12.00)}{7.00} = 0.53 \text{ inch}^2$$

$$d_s = h - \frac{dia_{bar}}{2} - clr = 18.00 - \frac{0.625}{2} - 2.00 = 15.69 \text{ inch}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.53(60)}{0.85(4.0)(12.00)} = 0.77 \text{ inch}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(0.53)(60)\left(15.69 - \frac{0.77}{2}\right)\left(\frac{\text{ft}}{12\text{in}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 36.50 \text{ kip-ft}$$

Walls need to be checked for axial thrust and flexural interaction. However, Example 3 showed axial thrust is insignificant and considering this structure is exposed to less fill, the axial thrust was neglected for this example.

Minimum Steel Requirements:

Since $\varphi_f M_n > M_{cr} = 27.78$ k-ft, minimum reinforcement requirements at this section are satisfied.

Shear Resistance:

Shear was checked with end of wall moment and found to satisfy required conditions. Shear is less critical at mid-span and is therefore sufficient at this location.

9.5.6 LRFR Rating Calculations

The structural condition of the box tunnel is satisfactory and the system factor falls under the category for "All Other Girder Bridges and Slab Bridges." Therefore:

Condition factor:	ϕ_c	=	1.00	MBE Table 6A.4.2.3-1
System factor:	ϕ_s	=	1.00	MBE Table 6A.4.2.4-1

Resistance factors are based on LRFD BDS Table 12.5.5-1:

Flexure resistance factor:	ϕ_f	=	0.90	LRFD BDS 12.5.5
Shear resistance factor:	ϕ_v	=	0.85	LRFD BDS 12.5.5
Thrust resistance factor:	ϕ_c	=	0.70	LRFD BDS 5.5.4.2

The equation for calculating the rating factor is based on MBE Equation 6A.4.2.1-1, which has been simplified for the load types being applied. The component for permanent loads other than dead loads, *P*, has been removed.

$$RF = \frac{C \pm \gamma_{DC} DC \pm \gamma_{DW} DW \pm \gamma_{EV} EV \pm \gamma_{EH} EH \pm \gamma_{ES} ES}{\gamma_{LL}(LL + IM) \pm \gamma_{LS} LS} \qquad \text{MBE Eq. 6A.5.12.4-1}$$

For the Strength Limit State:

$$C = \phi_c \phi_s \phi R_n \qquad \text{MBE Eq. 6A.5.12.4-2}$$

Table 57 through Table 65 shows the results of the overall capacity based on MBE Equation 6A.5.12.4-2 as well as the rating load factors based on MBE Equation 6A.5.12.4-1. It is also noted that negative moments and shears are designated with a negative sign as well as corresponding negative moment capacities. It is imperative to coordinate positive and negative moments and shears when calculating the rating factor. Ratings are performed at the critical section defined in LRFD BDS 12.11.5.2 for flexure and 5.7.3.2 for shear subjected to the HL-93 Inventory level rating. Consequently, the critical moment section (x_{cr} for moment) is at the end of the haunched corner (or inside corner in case of zero haunch) and the critical shear (x_{cr} for shear) is at a distance d_v from the inside face of the wall or slab. The exception is at the intersection between the exterior wall and the arch. The critical sections are observed at the transition between members since there is no haunch at this location and the member continues on with minimal discontinuity. It should also be noted that the load factors for permanent loads are 1.35 as discussed in Section 9.5.4.2.3.

Table 57. Exterior wall rating load factors.

Permanent	LL
1.35	1.75

Table 58. Exterior wall moments (k-ft) and rating factor (RF).

x_{cr} (in)	Permanent	LL+	LL-	C	RF
225.60	-67.43	0.00	-0.28	-101.35	21.49
112.80	-8.97	0.00	-0.94	36.50	*N/A

*Positive moment rating factors are N/A since the extreme forces are observed without the application of live load.

Table 59. Exterior wall shears (kip) and rating factor (RF).

x_{cr} (in)	Permanent	LL+	LL-	C	RF
225.60	-10.19	0.00	-0.02	-17.32	86.65

Table 60. Arch rating load factors.

Permanent	LL
1.35	1.75

Table 61. Arch moments (k-ft) and rating factor (RF).

x_{cr} (in)	Permanent	LL+	LL-	C	RF
0.00	-41.71	-6.01	-5.71	-165.93	10.88
195.60	68.75	11.97	11.11	152.90	2.87

Table 62. Arch shears (kip) and rating factor (RF).

x_{cr} (in)	Permanent	LL+	LL-	C	RF
0.00	-12.75	-0.68	-0.58	-37.48	17.02

Table 63. Bottom slab rating load factors.

Permanent	LL
1.35	1.75

Table 64. Bottom slab moments (k-ft) and rating factor (RF).

x_{cr} (in)	Permanent	LL+	LL-	C	RF
29.60	-9.83	1.46	-1.31	-129.61	50.79
195.60	90.71	3.82	2.75	157.23	5.18

Table 65. Bottom slab shears (kip) and rating factor (RF).

x_{cr} (in)	Permanent	LL+	LL-	C	RF
27.35	21.91	0.69	0.56	44.61	12.45

9.5.7 LFR Rating Calculations

Rating of cut and cover tunnels is similar in process and procedures as rating of box culverts as defined in the MBE. However, the MBE only provides guidance and examples for rating of boxes using LRFR. Though rating of boxes using LFR is not directly specified in the rating specifications, interpretation can be used to rate box tunnels using LFR methodologies.

There are several alterations and interpretations that are required to rate a box tunnel using LFR methodology. This includes, but may not be limited to, the inclusion of live load surcharge on the denominator of the rating equation, utilization of LFR load factors and resistance factors, definition of dead loads and wearing surfaces, earth load calculations and capacity evaluations. These factors are performed slightly different with LRFR and LFR procedures.

The example detailed in this document was evaluated and loads were calculating using LRFR procedures and guidelines. For rating procedure example, the following shows the rating procedures using LFR methodology. However, the member capacity and HS-20 forces are replicated from previous section rather than recalculated using LFR procedures.

The critical and controlling section shown in previous section was the arch, mid span moment. For simplicity, the following calculations are just showing rating for this critical section. Typical rating procedures would require both moment and shear evaluation at all critical locations and possibly all 10th span points.

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

Where

$C = 152.90$ kip-ft

$A_1 = 1.3$

$A_2 = 2.17$ (Operating Level)

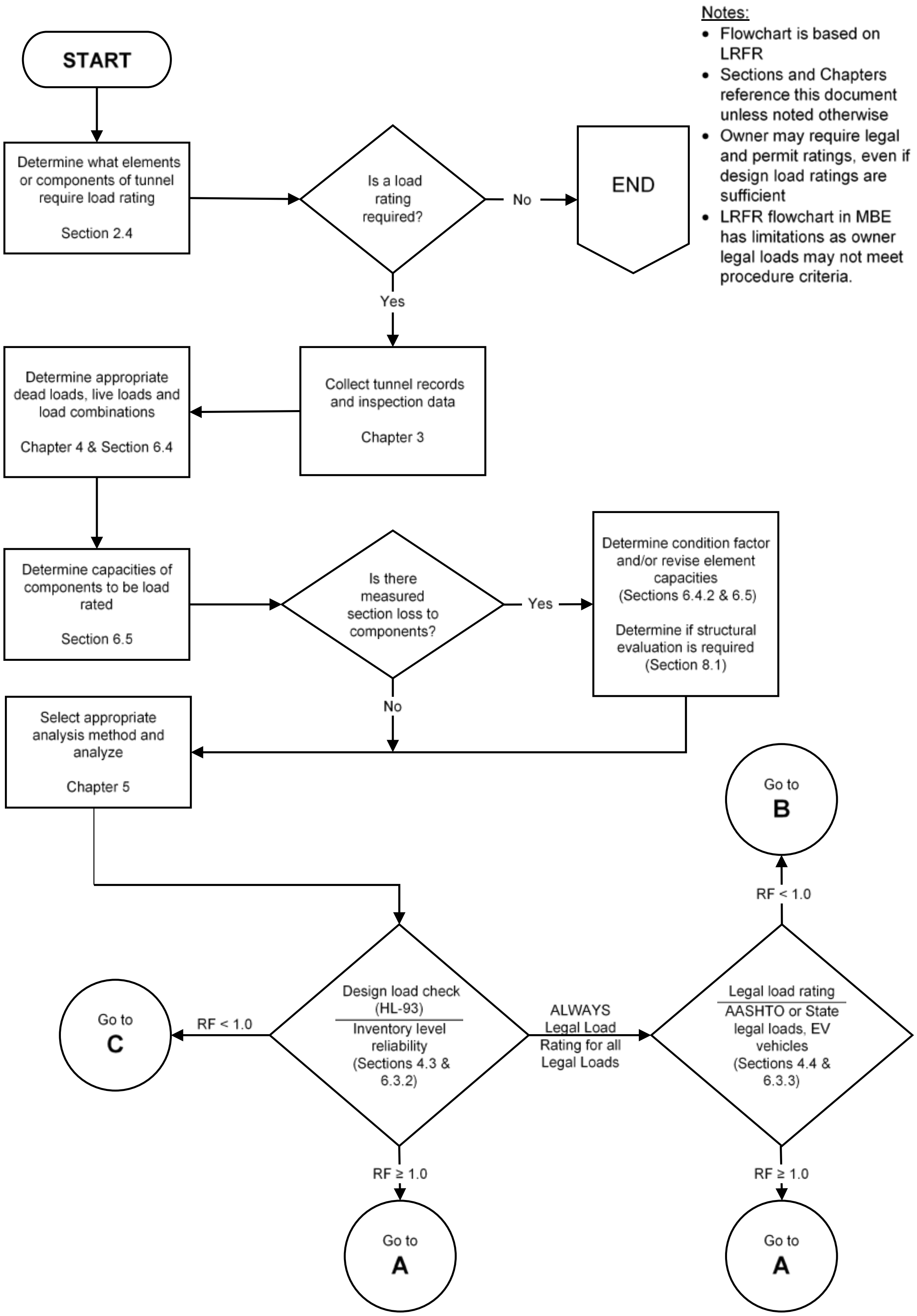
$D = 68.75$ kip-ft

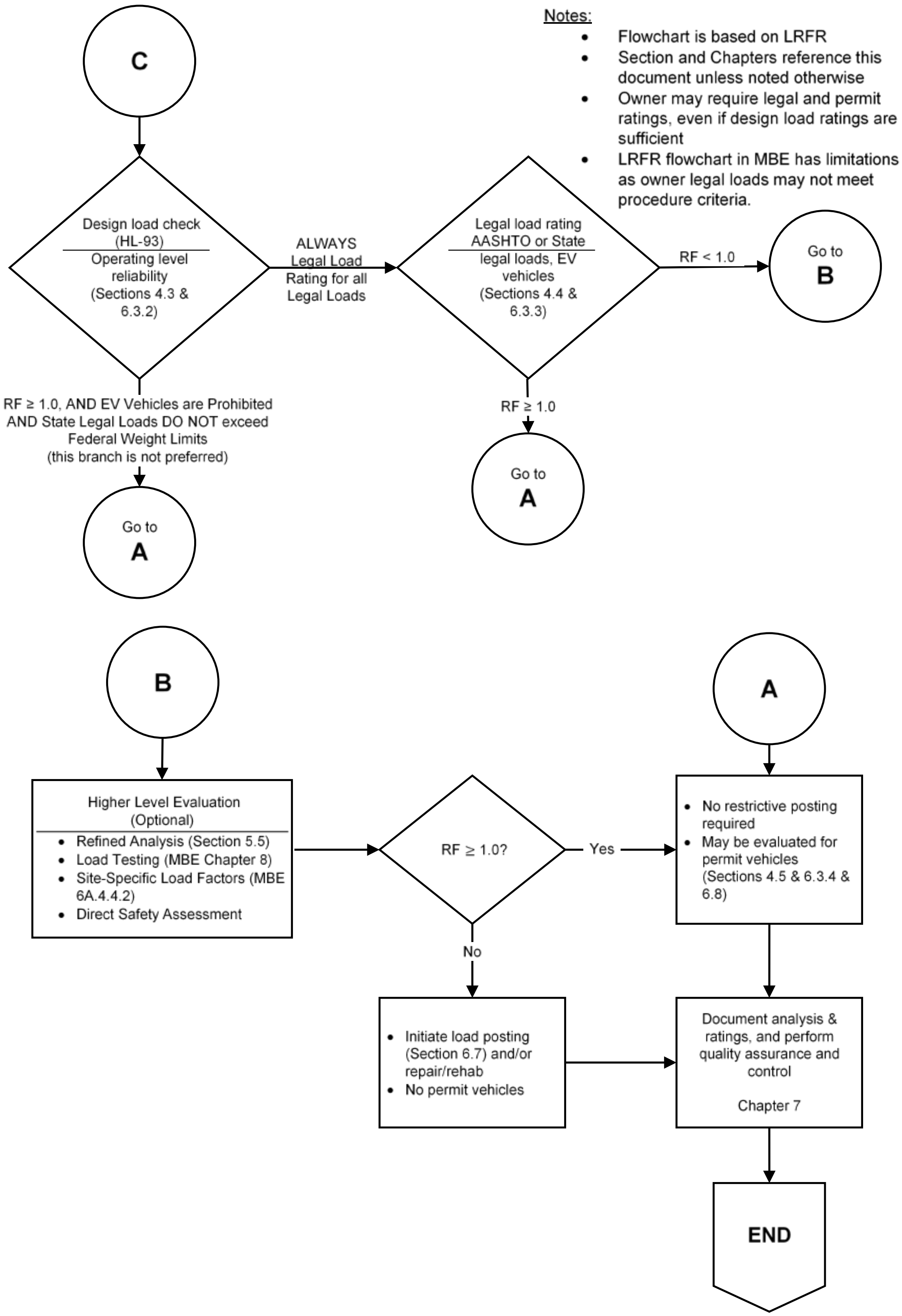
$L = 11.11$ kip-ft

$I = 10\%$ (Standard Specification Section 3.8.2.3)

$$RF = \frac{152.90 - 1.3(68.75)}{2.17(11.11)(1+0.10)} = 2.36$$

APPENDIX A – FLOWCHART




Notes:
Flowchart is based on LRFR
Section and Chapters reference this document unless noted otherwise
Owner may require legal and permit ratings, even if design load ratings are sufficient
LRFR flowchart in MBE has limitations as owner legal loads may not meet procedure criteria.
C
Design load check (HL-93)
Operating level reliability (Sections 4.3 & 6.3.2)
ALWAYS Legal Load Rating for all Legal Loads
Legal load rating AASHTO or State legal loads, EV vehicles (Sections 4.4 & 6.3.3)
RF < 1.0
Go to B
RF ≥ 1.0, AND EV Vehicles are Prohibited AND State Legal Loads DO NOT exceed Federal Weight Limits (this branch is not preferred)
Go to A
RF ≥ 1.0
Go to A
B
A
Higher Level Evaluation (Optional)
Refined Analysis (Section 5.5)
Load Testing (MBE Chapter 8)
Site-Specific Load Factors (MBE 6A.4.4.2)
Direct Safety Assessment
RF ≥ 1.0?
Yes
No restrictive posting required
May be evaluated for permit vehicles (Sections 4.5 & 6.3.4 & 6.8)
No
Initiate load posting (Section 6.7) and/or repair/rehab
No permit vehicles
Document analysis & ratings, and perform quality assurance and control
Chapter 7
END

APPENDIX B – CHECKLIST

1) Determine what elements or components of tunnel require load rating
    a. Internal elements or components
        i. Invert slab structurally supported (not at grade)?
        ii. Supporting stringers or invert girders present?
        iii. Invert slab framed into supporting elements (for example walls)?
    b. Tunnel liner and supports
        i. Roadway above tunnel within zone of influence?
        ii. Structural liner?
        iii. Roof girders present?
        iv. Internal support columns or walls?
2) Collect tunnel records and inspection data
    a. Plans (design, as-built and repair/rehab)
    b. Construction records
    c. Shop drawings
    d. Any material testing data
    e. Inspection reports
    f. Previous load ratings
    g. Traffic data
3) Determine appropriate dead loads, live loads and load combinations
    a. Dead loads
        i. Structural elements or components
        ii. Internal mechanical/electrical elements or components (including dynamic effects, if applicable)
        iii. Architectural elements or components
        iv. Pre- or post-tensioning forces
    b. Earth loads
        i. Horizontal and vertical loads
        ii. Ground water loads
        iii. Live load surcharge for adjacent traffic above
    c. Determine if any other loads affect permanent forces (such as buildings on top …)
    d. Live loads
        i. Design vehicles
        ii. Emergency (EV) vehicles
        iii. AASHTO legal loads
            1. Routine commercial traffic vehicles
            2. Specialized hauling vehicles (SHV)
                a. Notional Rating Load (NRL)
                b. Posting vehicles
        iv. State legal loads (if required)

        v. Permit vehicles (if required or permissible)
            1. Routine permit vehicles
            2. Special permit vehicles
        vi. Subject to any other live loads (for example railroad or aircraft)?
    e. Load combinations and load factors
        i. Based on elements or components to be load rated determine applicable limit states
        ii. Based on ADTT, determine live load factors as applicable
    f. Begin with reasonable and conservative assumptions for loads and refine if legal load ratings are less than 1.0 and consider site-specific load factors
4) Determine capacities of elements or components to be load rated
    a. Determine capacity of elements or components to be load rated in accordance with applicable specifications
    b. Determine if force interaction is applicable
    c. Determine if amplification or secondary force effects are applicable
    d. Start with simplified, conservative assumptions and refine if legal load ratings are less than 1.0
5) Account for deterioration
    a. Review inspection reports for deterioration in elements or components to be rated
    b. Does level of deterioration require consideration in load rating?
    c. Is condition factor sufficient to account for deterioration?
    d. Reduce section properties and recompute capacities as necessary
    e. Is deterioration significant that geometry, loads or stiffness of analysis model is affected?
    f. Refine deterioration assumptions if legal load ratings are less than 1.0
6) Select appropriate analysis method
    a. Determine what load effects are required to properly rate elements or components
    b. Determine what loads must be applied to obtain proper results
    c. Determine level of refinement necessary to get accurate results and critical locations
    d. Begin with minimum conservative analysis level that properly accounts for these requirements and refine if legal load ratings are less than 1.0
7) Analyze structure and get results
    a. Develop analysis model
    b. Document input, output and assumptions
    c. Perform sanity checks of deflections and reactions, as necessary
    d. Use hand methods to check reasonableness of results from complex models
    e. Account for secondary effects, as necessary, by approximate or refined methods
8) Load rate structure at design load inventory load level (HL-93)
    a. Strengh-I, Strengh-I, Service I, Service II, Service III and Fatigue, as applicable
9) Load rate structure at design load operating load level (HL-93)
    a. Strengh-I, Service I (segmental construction), Service II (steel structures) and Service III (segmental construction, as applicable)

10) Load rate structure for AASHTO and/or State legal loads
    a. Determine applicable legal loadings based on force effect and span length
    b. Determine generalized live load factors for legal loads based on ADTT
    c. Determine site-specific live load factors if legal load ratings are less than 1.0 and required information is available
11) Load rate structure for EV2 and EV3 vehicles at operating level
    a. Determine emergency vehicle load ratings in accordance with FHWA memo
12) Optional higher level evaluation (legal and/or EV RF < 1.0)
    a. Refine load calculations, capacities, deterioration assumptions or method of analysis
    b. Perform load testing for actual load effects
    c. Determine site-specific legal load factors, if applicable
    d. Direct safety assessment
13) Evaluate structure for permit vehicles (legal and/or EV RF ≥ 1.0)
    a. Determine applicable permit loads in accordance with Owner requirements
    b. Determine permit type (routine or special)
    c. Determine frequency of crossings
    d. Determining method of loading (mixed with traffic, escorted without other traffic, crawl)
    e. Determine load factor based on above factors, ADTT and truck weight
    f. Determine appropriate impact factor
14) Structural evaluation
    a. Has there been a change to the structure or significant deterioration that may affect original design?
    b. Should elements or components, whether subject to load rating or not, be evaluate for full design loads and load combinations in accordance with applicable design specifications?
15) Initiate load posting analysis (legal and/or EV RF < 1.0)
    a. Determine RFs for AASHTO Type 3, 3S2 and 3-3, and applicable State legal loads.
    b. If specialized hauling vehicle NRL rating is less than one, analyze structure for AASHTO SU4, SU5, SU6 and SU7. You may always elect to rate SU4 – SU7 ignoring the NRL or per Owner's direction.
    c. Determine posting levels for legal and emergency vehicles, as applicable
    d. Determine signage requirements
16) Document and QA/QC load rating
    a. Document load rating procedure
    b. Conduct QC/QA per established procedure

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