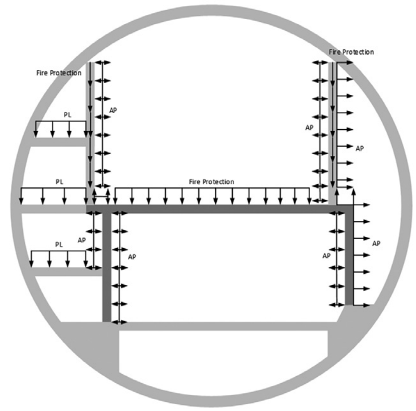

Tunnel Load Rating Examples: A Supplement to the Reference Guide for Load Rating of Tunnel Structures

Publication No. FHWA-HIF-20-058
Office of Infrastructure, Office of Bridges and Structures

December 2020

U.S. Department of Transportation
Federal Highway Administration

FOREWORD

The National Tunnel Inspection Standards (NTIS, 23 CFR 650.513(g)) require 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(b)(1))." However, the AASHTO Manual for Bridge Evaluation (MBE) lacks specific information on the load rating of tunnels. In addition, Section 5.4 of FHWA's Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual, published in 2015 only provides limited information. As a result, FHWA published the Reference Guide for Load Rating of Tunnel Structures in 2019.

This report is intended to be used as a supplement to that Reference Guide. It presents two practical examples of load rating for actual, in-service tunnel structures. The examples are based on the information contained within the Reference Guide and the computations are presented in similar detail as the load rating examples included in the Reference Guide.

Subject matter experts from several State DOTs and FHWA provided technical review of this report. Their advice, counsel and contributions during the preparation of this report are greatly appreciated.

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Office of Infrastructure
Federal Highway Administration

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16. Abstract
This supplement to the FHWA Reference Guide for Load Rating of Tunnel Structures (Guide) presents two practical examples of real, in-service tunnel structures. The examples are based on the information contained within the Guide, and computations are presented in similar details as the load rating examples included in the Guide. The ratings are provided for LRFR only. The examples are rated for the HL-93 and EV vehicles. Additionally, a nominal permit load is applied and rated. For each example, one cross-section with one set of parameters (e.g. dead loads, soil parameters, etc.) is rated. Repetitive hand calculations are shown once in detail (equations) with the subsequent results summarized in tables. Elements from the original drawings and inspection reports (where applicable) are used in the examples where legible and useful. Supplemental sketches are developed as necessary in a similar style as the Guide. Where actual dimensions or properties are unclear or unavailable, reasonable approximations are made. The results of the two examples may be used to validate the load ratings in the records of the tunnels.

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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

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
BDS	AASHTO LRFD Bridge Design Specifications, 8th Edition (2017a)
CFR	Code of Federal Regulations
FHWA	Federal Highway Administration
Guide	Reference Guide for Load Rating of Tunnel Structures (FHWA 2019)
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)
NTI	National Tunnel Inventory
NTIS	National Tunnel Inspection Standards
SNTI	Specifications for the National Tunnel Inventory (FHWA 2015b)
SSI	Soil-Structure Interaction
TOMIE	Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual (FHWA 2015a)

CHAPTER 1 - INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this non-binding Supplement to the voluntary FHWA Reference Guide for Load Rating of Tunnel Structures (Guide), created by the Federal Highway Administration (FHWA), is to present two practical examples of real, in-service tunnel structures. The examples are based on the information contained within the Guide, and computations are presented in similar details as the load rating examples included in the Guide. The ratings are provided for LRFR only. The examples are rated for the HL-93 and EV vehicles. Additionally, a nominal permit load is applied and rated for Example 1. For each example, one cross-section with one set of parameters (e.g. dead loads, soil parameters, etc.) is rated. Repetitive hand calculations are shown once in detail (equations) with the subsequent results summarized in tables. Elements from the original drawings and inspection reports (where applicable) are used in the examples where legible and useful. Supplemental sketches are developed as necessary in a similar style as the Guide. Where actual dimensions or properties are unclear or unavailable, reasonable approximations are made. The results of the two examples may be used to validate the load ratings in the records of the tunnels. Unless otherwise specified, all references refer to the voluntary Guide (FHWA 2019), the non-binding MBE 3rd Edition (AASHTO 2018) (see Section 1.2 for the components and editions incorporated by reference in the Federal regulations), or the TOMIE Manual (FHWA 2015a) [23 CFR 650.517(c)(1)].

The following is an overview of the Supplement's organizational structure:

- Chapter 1 comprises introductory information on the two load rating examples, applicable standards and specifications, an overview of the guide, and descriptions of each example tunnel along with assumptions and approximations.

- Chapter 2 covers the load rating of Example 1 – Circular Tunnel Liner with Invert Structures.

- Chapter 3 contains the load rating of Example 2 – Box Tunnel Roof Girder.

1.2 APPLICABLE DOCUMENTS

National Tunnel Inspection Standards (NTIS) [23 CFR 650.513(g)]. The NTIS requires "tunnel owners to establish a program for the inspection of highway tunnels" (Federal Register, July 14, 2015, 80 FR 41350) 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. Federal regulation at 23 CFR 650.513(g) provides the regulations regarding the load rating of tunnels. The NTIS incorporates, by reference, the Specifications for the National Tunnel Inventory (SNTI) [23 CFR 650.517((c)(2)] and TOMIE Manual [23 CFR 650.517(c)(1)]. Refer to the paragraph on the MBE below for components and editions incorporated by reference into the Federal regulations.

Specifications for the National Tunnel Inventory (SNTI) (FHWA 2015b) [23 CFR 650.517 (c)(2)]. 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 regarding how to log information for each item's code, and how the Engineer decides 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) [23 CFR 650.517(c)(1)]. 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 is considered a primer. 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). This voluntary edition of 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 describes non-binding criteria for load rating and posting of bridges and informed several of the load rating concepts presented in the Guide. The most current edition at the time of publication of the Guide is the 3rd Edition. Subsequent references to the MBE in this document are relevant to the voluntary 3rd Edition of the MBE. The portions and editions of the MBE incorporated into the Federal regulations by reference in the NTIS, which are binding, are Sections 6 and 8 of the 2nd Edition, 2011 [23 CFR 650.517(b)(1)], the 2011 Interim Revisions to Section 6 [23 CFR 650.517(b)(2)], the 2013 Interim Revisions to Section 6 [23 CFR 650.517(b)(3)], the 2014 Interim Revisions to Section 6 [23 CFR 650.517(b)(4)], and the 2015 Interim Revisions to Section 6 [23 CFR 650.517(b)(5)].

AASHTO LRFD Bridge Design Specifications, 8th Edition (BDS) (AASHTO 2017a) [23 CFR 625.4(d)(1)(v)]. Concepts such as standard loads, load factors, material resistance, and resistance factors that are used in the 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). The non-binding 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 voluntary and non-binding memorandum provides explanations of the ratings for the emergency vehicles enacted by the Fixing America's Surface Transportation Act (FAST Act) [23 CFR 650.313(c)].

1.3 REFERENCE GUIDE FOR LOAD RATING OF TUNNEL STRUCTURES

The **FHWA Reference Guide for Load Rating of Tunnel Structures (Guide) (FHWA 2019)** is voluntary, non-binding reference guide created by the FHWA to illuminate the technical aspects for the load rating of tunnel structures. It provides practical and representative step-by-step examples of load rating that meets the FHWA's general load rating guidance. It also provides technical details and breadth appropriate to explain the load rating specifications in the non-binding AASHTO MBE (AASHTO 2018) and the FHWA TOMIE Manual (FHWA 2015a) [23 CFR 650.517(c)(1)]. A set of examples illustrate the specifications, procedures, and methods for load rating.

1.4 GENERAL DISCUSSION

The two examples in this Supplement illustrate how to load rate 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.

1.4.1 Example Tunnels

Example 1 – The first example is located on the west coast of the United States. The tunnel is a single, large diameter circular bored tunnel with a concrete liner. Internally, there are two levels of traffic, the lower supported by an invert slab and the upper by a rigid frame. The tunnel liner is comprised of precast reinforced concrete segments. The actual tunnel is located a distance below the surface that is more than the diameter of the tunnel, which puts it outside the live load zone of influence limits for culverts per the MBE (maximum of 8 feet or culvert width). Per discussions with the FHWA, it will be assumed the tunnel is closer to the surface for the purposes of this example. The invert slab, rigid frame structure, and tunnel liner will be rated for this example. The invert slab will be analyzed by hand as a one-way prestressed slab on simple supports and rated for shear and moment. The rigid frame will be analyzed with a simple 2D analysis and rated for shear, moment, and thrust at controlling locations. The tunnel liner will be modeled in Plaxis to determine forces in the liner due to earth loads and surface live loads. Based on geotechnical information, a representative soil profile will be selected. The street surface above the tunnel will be located a distance less than the tunnel diameter in order to be within the zone of influence. Traffic above will be assumed to travel normal to the tunnel. The ring segment reinforcement will be checked for bending, shear and thrust. The joints between sections will be checked against crushing, shear, and the formation of a mechanism (bending overcoming thrust pre-compression and opening joint).

Example 2 – The second example is located on the east coast of the United States. The tunnel is one of several adjacent boxes with steel roof girders connected by shear-only steel bracket connections to support walls. The tunnel supports a transverse roadway directly above. The simply supported composite steel tunnel roof girder will be rated for this example. Forces will be determined using hand calculations. The tunnel roof girder will be rated for moment and shear. The capacity of the shear connection will be investigated as well.

1.4.2 Assumptions

The following general assumptions were used in the development of the examples and final document:

- The document is entirely separate and acts as a supplement to the Guide with a separate report number supplied by the FHWA.
- The examples are based on the information contained within the Guide.
- The examples are in a similar format as the ones in the Guide.
- The ratings are provided for LRFR only. The Guide adequately demonstrates the difference between LRFR and LFR with parallel calculations in the examples; the purpose of the examples is to apply the methodology to actual structures.
- The examples are rated for the HL-93 and EV vehicles. Additionally, a nominal permit load is applied and rated.
- For each example, one cross-section with one set of parameters (e.g. dead loads, soil parameters, etc.) is rated.
- Repetitive hand calculations are shown once in detail (equations) with the subsequent results summarized in tables.
- Elements from the original drawings and inspection reports (where applicable) are used in the examples where legible and useful. Supplemental sketches are developed as necessary in a similar style as the Guide.

1.4.3 Approximations

- Where actual dimensions or properties are unclear or unavailable, reasonable approximations are made.
- For these examples, the area of steel reinforcement bars for concrete elements are directly calculated based on their nominal diameters to the third decimal point. However, and alternatively acceptable and common practice, is to use values from design tables provided by associations such as the Concrete Reinforcing Steel Institute (CRSI), often given to the second decimal place.

CHAPTER 2 - EXAMPLE 1 – CIRCULAR TUNNEL LINER, RIGID FRAME, & SLAB

Example 1 presents a bored tunnel with an internal, reinforced concrete frame and a prestressed bottom slab. The internal roadway is a bi-level, reinforced concrete frame. The bottom roadway surface consists of a prestressed concrete invert slab. The tunnel is also subjected to surface loads influencing the tunnel liner. The tunnel is subjected to vertical dead loads, earth loads, and live loads as well as lateral earth, building surcharge and live load surcharge. The tunnel is rated with the LRFR method for the design vehicles (HL-93 Inventory and Operating) and emergency vehicles (EV-2 and EV-3) at the legal load limit.

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

2.1 STRUCTURE DATA

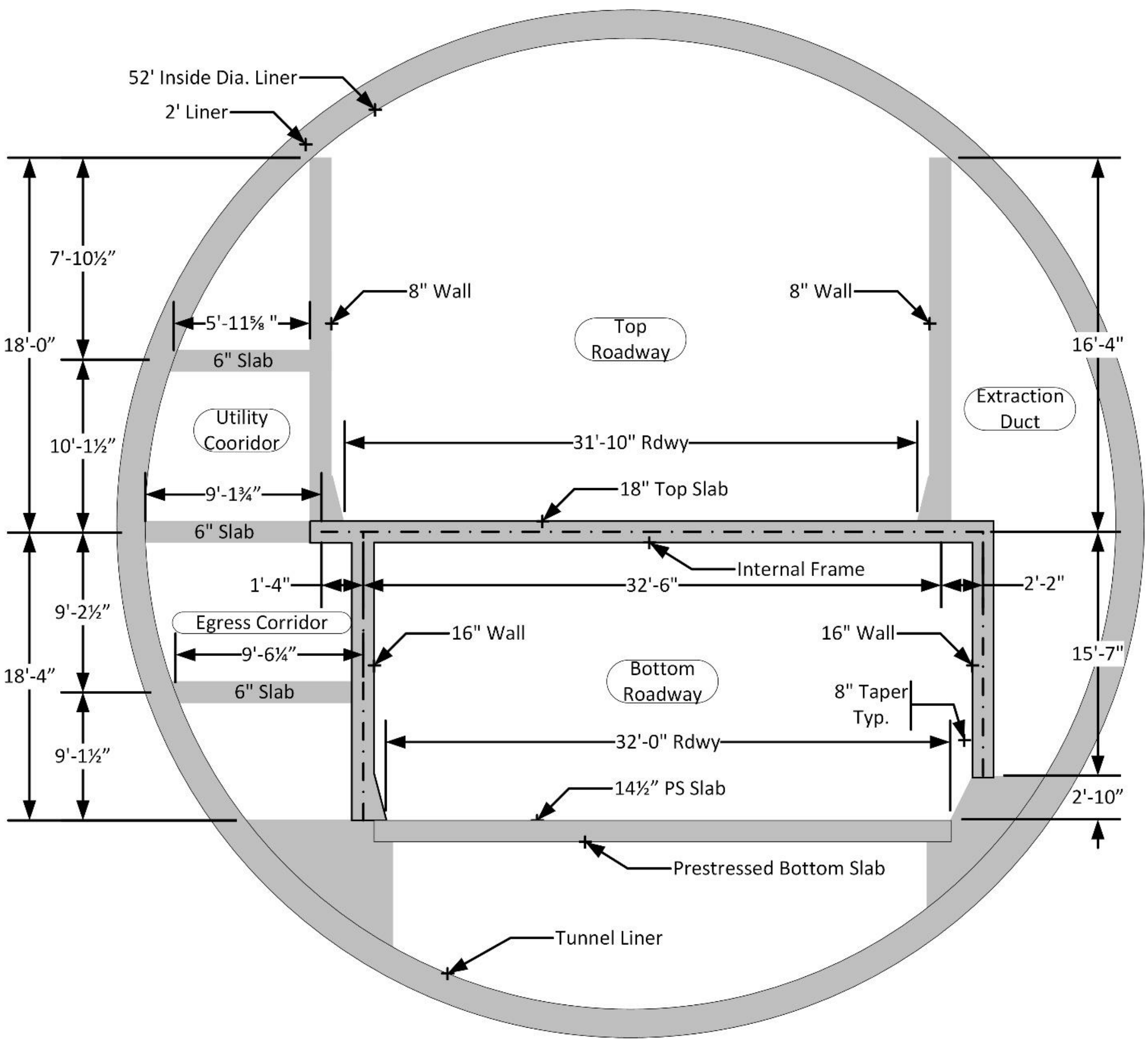
2.1.1 Materials

Materials are known, otherwise MBE Articles 6A5.2.1 and 6A5.2.2 may be used. 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 wet tunnel construction conditions and a 5% soil volume loss.

Concrete:	f'_c	=	4.0 ksi (Internal Frame)
	f'_c	=	7.0 ksi (Prestressed Bottom Slab)
	f'_c	=	7.0 ksi (Tunnel Liner)
Reinforcing Steel:	f_y	=	60.0 ksi
Soil:	γ_{soil}	=	0.117 kcf
	ϕ_{soil}	=	32°

2.1.2 Dimensions

See Figure 1 for the tunnel cross-section geometry and dimensions. The elements rated in this example are outlined in the figure.



Source: FWHA

Figure 1. Illustration. Cross-Section Showing Tunnel Geometry.

Top slab thickness:	t_{ts}	=	18 inches
Bottom slab thickness:	t_{bs}	=	14.5 inches
Wall thickness:	t_{wall}	=	16 inches
Top slab roadway width:	w_{top}	=	31.83 feet
Bottom slab roadway width:	w_{bot}	=	32.00 feet
Fill depth:	h_{fill}	=	10.00 feet

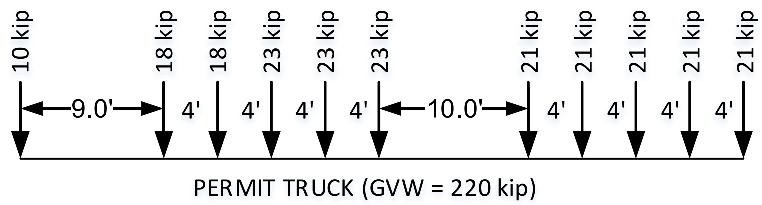
2.1.3 Example Notes

- This cross-section has been selected as representative to demonstrate rating of tunnel internal frames, external loading on liners and rating of PS sections.
- The live load carrying members included the driving surfaces, first level walls, and the tunnel liner. The non-live load carrying members need to be included in the structural model to account for the load distribution these members may contribute to the overall structural behavior.
- If the roadway has an overlay (in addition to the initial 2-inch design overlay included in the prestressed bottom slab ratings), 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 reinforced concrete tunnel.
- Mechanical equipment, signage, and architectural systems should be included in the dead load analysis. This example includes fire protection that would be similarly classified.
- The focus of this example is LRFR.
- This example focuses on the load rating of an interior section of a reinforced concrete 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 recommendations. 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 affected 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, are a vertically restrained bottom slab and a laterally restrained exterior wall for the upper slab. 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 and other sectional or material change locations.

2.1.4 Rating Approach/Assumptions

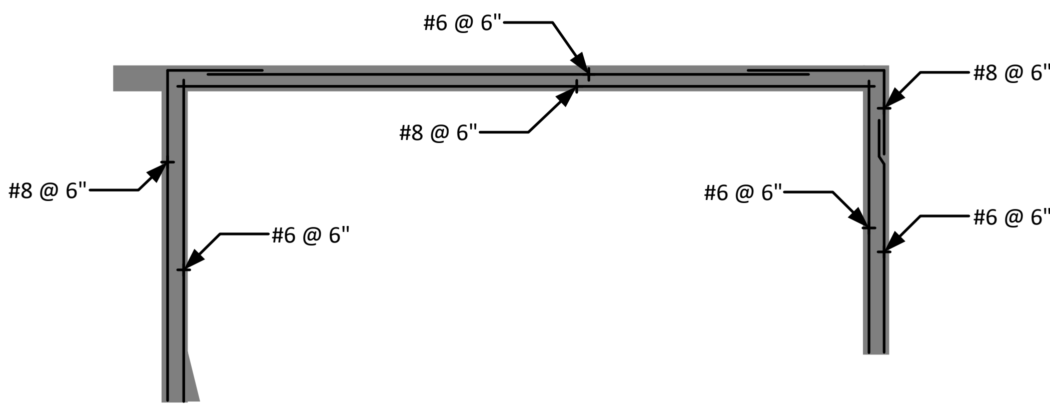
- LRFR evaluation is performed for a 1-foot wide strip (perpendicular to direction of traffic inside the tunnel and parallel to the direction of traffic above the box tunnel).
- The pavement on the ground above has approximately the same unit weight as the soil and is therefore included with the soil vertical loads (that is, gravel pavement surface).
- Compacted gravel fill acts along the sides of the tunnel with soil parameters of $\phi = 32°$ and $\gamma = 117$ pcf. Additionally, the tunnel is partially submerged in ground water.

- LRFR live load ratings are evaluated for HL-93 design loading (Truck or Tandem), the EV-2 and EV-3 emergency vehicles (FHWA Memo), and a permit truck shown in Figure 2 (for the internal frame and prestressed bottom slab only).



Source: FHWA

Figure 2. Illustration. Permit Truck.



Source: FHWA

Figure 3. Illustration. Reinforcing Steel Details of Internal Frame.

2.2 INTERNAL FRAME

2.2.1 Load Calculations

2.2.1.1 Dead Load Component, DC

Dead load components include the self-weight of the concrete internal frame and barriers on the frame. 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 Article C3.5.1

The self-weight of the frame is applied to the model by including self-weight on the geometry detailed in Figure 1. A 233 plf barrier load is applied on top of the frame at each side of the top roadway.

A 10 psf fire protection load was applied to the interior surface of the upper roadway as indicated in the Original Construction Plans (OCPs), this includes the upper walls and the underside of the upper roadway. In addition to the fire protection load, 10 psf utility loads were applied to the upper roadway surface along with the horizontal walkway surface slabs to the left side of the internal frame supporting the upper roadway. Additional utility loads of 220 plf and 200 plf were applied to the center of the middle and lower of these walking surfaces. Details of these loads can be seen in Figure 5.

2.2.1.2 Wearing Surface, DW

This internal frame has no wearing surfaces applied and therefore were not included in the load calculations.

2.2.1.3 Air Pressure, AP

Air pressures were applied to the Egress Areas, roadways and Extraction Duct as shown in Figure 1. ±21 psf was applied to the Egress Area, ±10 psf was applied to the roadways and -68 psf was applied to the Extraction Duct, according to the OCPs.

2.2.1.4 Temperature, TU

A temperature range of ±20 degrees was applied to the structure as a whole. This load was modelled by applying the Temperature tool within the structural analysis program.

2.2.1.5 Creep and Shrinkage, CR & SH

Creep and shrinkage is a displacement based force and needs to be modelled in the program as a temperature variation. LRFD BDS Article 5.4.2.3.1 specifies the creep and shrinkage displacement is calculated as *"c"* times *"L"*. The temperature range can then be backed out of the temperature displacement equation. This temperature is then input into the structural analysis program to determine the creep and shrinkage forces. Per Article 5.4.2.3.1, in the absence of more accurate data, the shrinkage coefficient may be assumed to be 0.0005 after one year of drying. The coefficient of thermal expansion, α, is specified in LRFD BDS Article 5.4.2.2

$$cL = \delta = \alpha \Delta TL$$

$$0.0005 = 6.0e^{-6} \Delta T$$

$$\Delta T = 83.4°F$$

2.2.1.6 Pedestrian Live Load, PL

A 100 psf pedestrian live load is applied to both the Egress Path Corridor and Utility Corridor floor slabs. A 60 psf pedestrian live load is applied to the Ceiling Slab.

2.2.1.7 Live Load Application, LL

The internal frame is a reinforced concrete frame and behaves similarly to the concrete deck of a slab bridge. The distribution of axle loads based on an equivalent strip width, *E*, to cast-in-place slabs acting perpendicular to the direction of traffic is provided in LRFD BDS Table 4.6.2.1.3-1, where *S* is the span length. Per the LRFD BDS, separate distribution factors are computed for the positive and negative moments in the roadway slab using a span, S, of 36 feet. The live load applied to the walls is dependent on the forces imparted to them due to the fixed end moments from the roadway slab. As such, this example determines the live load forces in the walls based on the equivalent strip width, E, computed for negative moments in the slab, E_{Slab-}.

$$E_{pos} = 26.0 + 6.6S$$

$$E_{neg} = 48.0 + 3.0S$$

$$E_{Slab+} = 26.0 + 6.6(36) = 263.60 \text{ inch} = 21.97 \text{ feet}$$ LRFD BDS Table 4.6.2.1.3-1

$$E_{Slab-} = 48.0 + 3.0(36) = 156.00 \text{ inch} = 13.00 \text{ feet}$$

$$E_{Wall1+} = E_{Wall1-} = E_{Wall2+} = E_{Wall2-} = E_{Slab-} = 13.00 \text{ feet}$$

Since the design strip is transverse to the direction of traffic, the wheel loads are included in the live load evaluation; however, the lane load is not included per LRFD BDS Article 3.6.1.3.3. The maximum number of trucks to be placed on the structure is obtained by taking the roadway width and dividing by a 12-foot lane, per LRFD BDS Article 3.6.1.1.1, resulting in two possible trucks to be placed parallel. This results in axle loads that are placed transversely such to produce maximum forces at the critical locations, identified at mid-span and wall-slab connection. Maximum connection forces were obtained by placing the first axle load 2 feet from the edge of the driving surface per LRFD BDS Article 3.6.1.3.1. The second truck is placed such that there is 4 feet between wheel placements. Maximum positive moment is obtained when the centerline of the span is midway between the center of gravity of loads and the nearest concentrated load. Live load placement is shown in Figure 4.

Wheel loads for unit, or 1-foot, analysis widths are determined by multiplying the equivalent unit width wheel load, P, by the appropriate impact factor (IM) and multiple presence factor (MPF). The equivalent wheel load is determined by dividing the full wheel load by the equivalent strip width, E, over which the axle load is distributed. However, if the axle spacing is less than the equivalent strip width, E, then multiple axles can load the same unit-width analysis strip. In these cases, the applied load is not simply the full wheel load divided by E; the overlap of axle loads should be considered. For positive bending, the E_{Slab+} width of 21.97 feet exceeds the axle spacing of the HL-93 Truck (14-foot spacing), HL-93 Tandem (4-foot spacing), both EV vehicles (15-foot for EV-2 and 15- and 4-foot for EV-3), and the permit truck (4- to 10-foot spacings). For negative bending and the walls, the E_{Slab-} width of 13.00 feet exceeds the axle spacing of the HL-93 Tandem, EV-2 vehicle (rear axle) and the permit truck. The unit width wheel load, P, is computed below using equivalent axle loads where required. For a detailed discussion of modified equivalent widths and figures showing the HL-93 and EV vehicle axle configurations, refer to Section 9.2.4.1 of the *Reference Guide for Load Rating of Tunnel Structures* (FHWA, 2019). Refer to Figure 2 for the permit truck axle configuration.

First, compute P for various axle configurations using the roadway slab positive bending equivalent slab width, E_{Slab+}, to determine the controlling load for each rating vehicle:

HL-93 Design Truck:

$$P_{HL93-Truck1} = \frac{\left(\dfrac{32 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}\right)} = \frac{16}{21.97} = 0.73 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Truck2} = \frac{\left(\dfrac{32 \text{ kip} + 32 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}/2 + 14 + E_{Slab+}/2\right)} = \frac{32}{35.97} = 0.89 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Truck3} = \frac{\left(\dfrac{32 \text{ kip} + 32 \text{ kip} + 8 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}/2 + 14 + 14 + E_{Slab+}/2\right)} = \frac{36}{49.97} = 0.72 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Truck+} = \max\left(P_{HL93-Truck1}, P_{HL93-Truck2}, P_{HL93-Truck3}\right) = 0.89 \frac{\text{k/wheel}}{\text{ft}}$$

HL-93 Design Tandem:

$$P_{HL93-Tandem1} = \frac{\left(\dfrac{25 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}\right)} = \frac{12.5}{21.97} = 0.57 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Tandem2} = \frac{\left(\dfrac{25 \text{ kip} + 25 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}/2 + 4 + E_{Slab+}/2\right)} = \frac{25}{25.97} = 0.96 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Tandem+} = \max\left(P_{HL93-Tandem1}, P_{HL93-Tandem2}\right) = 0.96 \frac{\text{k/wheel}}{\text{ft}}$$

EV-2:

$$P_{EV2-1} = \frac{\left(\dfrac{33.5 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}\right)} = \frac{16.75}{21.97} = 0.76 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{EV2-2} = \frac{\left(\dfrac{33.5 \text{ kip} + 24 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}/2 + 15 + E_{Slab+}/2\right)} = \frac{28.75}{36.97} = 0.78 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{EV2+} = \max\left(P_{EV2-1}, P_{EV2-2}\right) = 0.78 \frac{\text{k/wheel}}{\text{ft}}$$

EV-3:

$$P_{EV3-1} = \frac{\left(\dfrac{31 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}\right)} = \frac{15.5}{21.97} = 0.71 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{EV3-2} = \frac{\left(\dfrac{31 \text{ kip} + 31 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}/2 + 4 + E_{Slab+}/2\right)} = \frac{31}{25.97} = 1.19 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{EV3-3} = \frac{\left(\dfrac{31 \text{ kip} + 31 \text{ kip} + 24 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}/2 + 4 + 15 + E_{Slab+}/2\right)} = \frac{43}{40.97} = 1.05 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{EV3+} = \max\left(P_{EV3-1}, P_{EV3-2}, P_{EV3-3}\right) = 1.19 \frac{\text{k/wheel}}{\text{ft}}$$

Permit Truck:

As stated, all the permit truck axle spacings are less than E_{Slab+}; therefore, all 11 axle load possibilities should be considered. The calculations follow the format as above and *P* varies from 0.48 to 1.64 over the 11 permutations. The calculation for the controlling 10-axle configuration is shown below.

$$P_{Permit+} = \frac{\left(\dfrac{21+21+21+21+21+23+23+23+18+18}{2 \text{ wheels}}\right)}{\left(E_{Slab+}/2 + 4 + 4 + 4 + 4 + 10 + 4 + 4 + 4 + 4 + E_{Slab+}/2\right)} = \frac{105}{63.97} = 1.64 \frac{\text{k/wheel}}{\text{ft}}$$

Next, compute *P* for various axle configurations using the roadway slab negative bending equivalent slab width, E_{Slab-}, which also applies to the walls, to determine the controlling load for each rating vehicle:

HL-93 Design Truck:

E_{Slab-};is less than the HL-93 Truck axle spacings; therefore, only one configuration is computed.

$$P_{HL93-Truck-} = \frac{\left(\dfrac{32 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}\right)} = \frac{32}{13.00} = 1.23 \frac{\text{k/wheel}}{\text{ft}}$$

HL-93 Design Tandem:

$$P_{HL93-Tandem1} = \frac{\left(\dfrac{25 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}\right)} = \frac{25}{17.00} = 0.96 \frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Tandem2}=\frac{\left(\dfrac{25\text{ kip}+25\text{ kip}}{2\text{ wheels}}\right)}{\left(E_{Slab+}/2+4+E_{Slab+}/2\right)}=\frac{50}{17.00}=1.47\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Tandem-}=\max\left(P_{HL93-Tandem1},P_{HL93-Tandem2}\right)=1.47\frac{\text{k/wheel}}{\text{ft}}$$

EV-2:

E_{Slab-};is less than the EV-2 axle spacing; therefore, only one configuration is computed.

$$P_{EV2-}=\frac{\left(\dfrac{33.5\text{ kip}}{2\text{ wheels}}\right)}{\left(E_{Slab+}\right)}=\frac{33.5}{13.00}=1.29\frac{\text{k/wheel}}{\text{ft}}$$

EV-3:

E_{Slab-};is less than the front EV-3 axle spacing; therefore, only two configurations are computed.

$$P_{EV3-1}=\frac{\left(\dfrac{31\text{ kip}}{2\text{ wheels}}\right)}{\left(E_{Slab+}\right)}=\frac{31}{13.00}=1.19\frac{\text{k/wheel}}{\text{ft}}$$

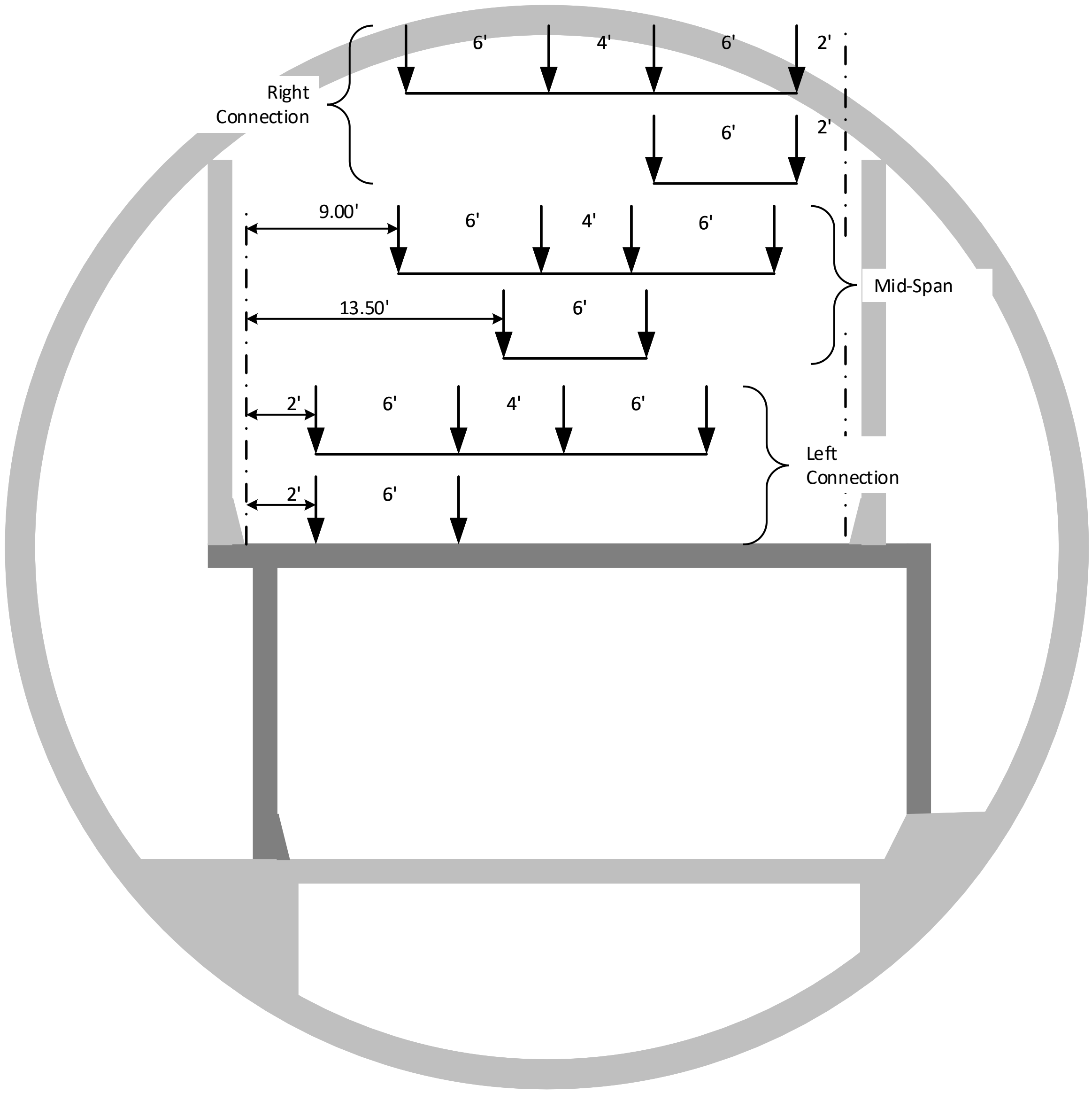
$$P_{EV3-2}=\frac{\left(\dfrac{31\text{ kip}+31\text{ kip}}{2\text{ wheels}}\right)}{\left(E_{Slab+}/2+4+E_{Slab+}/2\right)}=\frac{62}{17.00}=1.82\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{EV3-}=\max\left(P_{EV3-1},P_{EV3-2}\right)=1.82\frac{\text{k/wheel}}{\text{ft}}$$

Permit Truck:

Similar to the positive bending calculations, all the permit truck axle spacings are less than E_{Slab-}. P varies from 0.81 to 1.91 over the 11 permutations. The calculation for the controlling 10-axle configuration is shown below.

$$P_{Permit-}=\frac{\left(\dfrac{21+21+21+21+21+23+23+23+18+18}{2\text{ wheels}}\right)}{\left(E_{Slab+}/2+4+4+4+4+10+4+4+4+4+E_{Slab+}/2\right)}=\frac{105}{55.00}=1.91\frac{\text{k/wheel}}{\text{ft}}$$



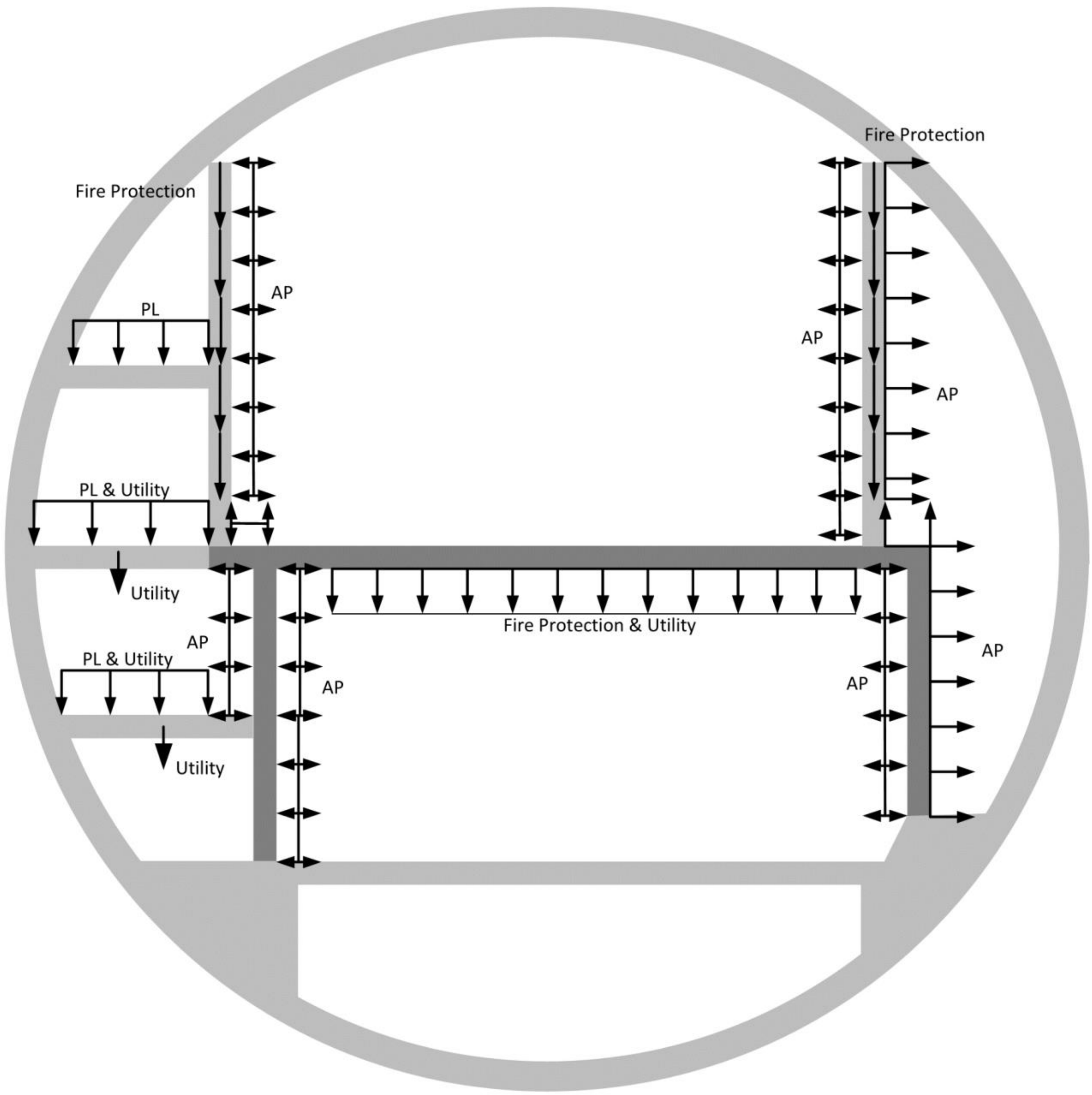
Source: FHWA

Figure 4. Illustration. Live Load Placement for Internal Frame.

Single trucks are amplified by a multiple presence factor, *MPF*, of 1.2 while the two truck effects utilize a multiple presence factor of 1.0 per LRFD BDS Table 3.6.1.1.2-1. For this example, MBE Article 6A.4.4.3 is used to determine the dynamic impact factor, *IM*, of 1.33 for all vehicles. Additionally, from the equivalent wheel load calculations, it can be seen that the HL-93 tandem will control over the HL-93 truck, and the EV-3 vehicle controls over the EV-2.

2.2.2 Structural Analysis

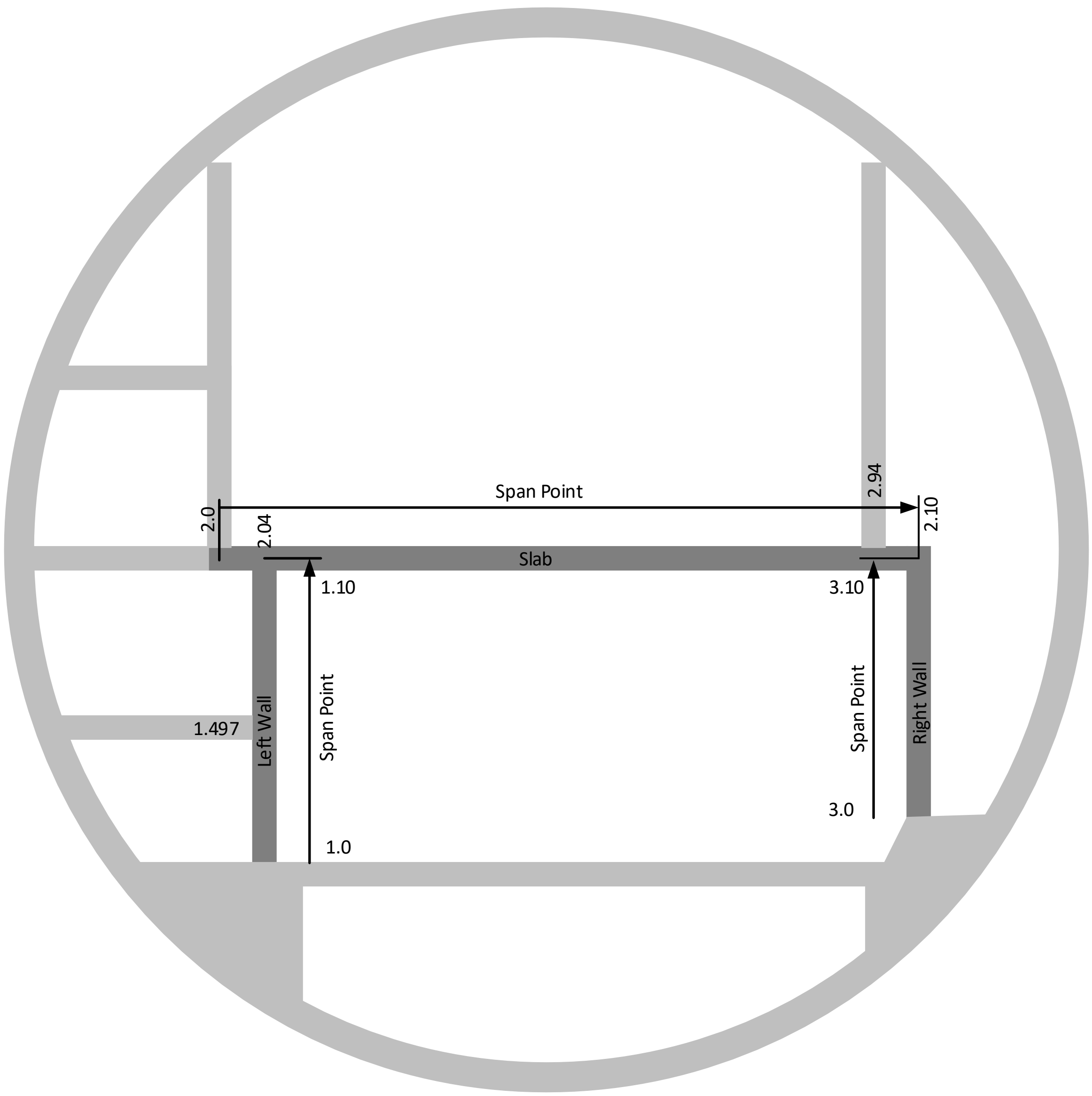
A frame analysis procedure is applied for the internal frame model with beam-column elements based on gross section properties. All the load effects and member resistances are calculated using this 1-foot wide strip representation. Structural analysis of the internal frame is based on a frame model with moment resisting connections between the slab and the wall joints. The frame is assumed to be pinned where the concrete frame connects to the tunnel liner, except at the top walls where they are assumed to be free to vertically translate with respect to the liner. A P-Delta analysis is also included to account for secondary effects. 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 [23 CFR 625.4(d)(1)(v)]. The applied loads are shown in Figure 5, and Table 1 through Table 18 provide the resulting force effects.



Source: FHWA

Figure 5. Illustration. Internal Frame Tunnel Applied Loads.

The *LRFD Road Tunnel Design and Construction Guide Specifications*, 1st Edition (AASHTO, 2017b), Table 3.4-1 identifies the applicable minimum and maximum load factors for each load specified above. These minimum and maximum load factors are matched with the appropriate load direction and summed to identify the critical design forces, including moments, shears, and axial forces. The live load factors for the HL-93 are identified in Table 3.4-1, and the live load factor for the EV vehicles is 1.3 as identified in the FHWA Emergency Vehicle Memorandum (FHWA, 2016) and the live load factor for the permit vehicle was identified as 1.3 from MBE 3rd Edition (AASHTO, 2018) Table 6A.4.5.4.2a-1, corresponding to unlimited crossings, mixed with traffic, two or more lanes, ADTT greater than 5000, and a permit weight factor ratio (GVW/AL) greater than 3.



Source: FHWA

Figure 6. Illustration. Member Identifiers and Directions.

Figure 6 shows the span point identification for the tunnel frame. The whole number represents the member identification while the 10ths number indicates the percentage along the span length.

The forces were factored to produce controlling Strength I (or Strength II for permit loads) as well as Service I load combinations. These resulting unfactored forces are given in Table 1 through Table 18 and the member identifiers and directions are shown in Figure 6.

Table 1. Left Wall Axial Forces (kip).

Sp Pt	DC	AP+	AP-	TU+	TU-	CR&SH
1.0	-12.41	-0.01	0.00	-0.13	0.14	0.56
1.1	-12.04	-0.01	0.00	-0.13	0.14	0.56
1.2	-11.68	-0.01	0.00	-0.13	0.14	0.56
1.3	-11.31	-0.01	0.00	-0.13	0.14	0.56
1.4	-10.94	-0.01	0.00	-0.13	0.14	0.56
1.497	-9.94	-0.01	0.00	-0.11	0.11	0.44
1.6	-9.59	-0.01	0.00	-0.11	0.11	0.44
1.7	-9.24	-0.01	0.00	-0.11	0.11	0.44
1.8	-8.89	-0.01	0.00	-0.11	0.11	0.44
1.9	-8.55	-0.01	0.00	-0.11	0.11	0.44
1.10	-8.19	-0.01	0.00	-0.11	0.11	0.44

Table 2. Left Wall Shear (kip).

Sp Pt	DC	AP+	AP-	TU+	TU-	CR&SH
1.0	0.44	0.04	0.01	0.22	0.22	0.92
1.1	0.44	0.02	0.01	0.22	0.22	0.92
1.2	0.44	0.01	0.01	0.22	0.22	0.92
1.3	0.44	0.01	0.02	0.22	0.22	0.92
1.4	0.44	0.01	0.04	0.22	0.22	0.92
1.497	1.99	0.10	0.10	0.11	0.11	0.44
1.6	1.99	0.06	0.06	0.11	0.11	0.44
1.7	1.99	0.03	0.03	0.11	0.11	0.44
1.8	1.99	0.01	0.01	0.11	0.11	0.44
1.9	1.99	0.05	0.05	0.11	0.11	0.44
1.10	1.99	0.08	0.08	0.11	0.11	0.44

Table 3. Left Wall Moment (kip-ft).

Sp Pt	DC	AP+	AP-	TU+	TU-	CR&SH
1.0	0.00	0.00	0.00	0.00	0.00	0.00
1.1	0.80	-0.06	0.02	-0.40	0.40	-1.68
1.2	1.60	-0.08	0.03	-0.81	0.81	-3.36
1.3	2.41	-0.07	0.05	-1.21	1.21	-5.04
1.4	3.21	-0.07	0.07	-1.61	1.61	-6.72
1.497	2.75	-0.09	0.09	-1.71	1.71	-7.13
1.6	-0.71	-0.05	0.05	-1.53	1.53	-6.37
1.7	-4.17	-0.13	0.13	-1.34	1.34	-5.61
1.8	-7.63	-0.14	0.14	-1.16	1.16	-4.85
1.9	-11.09	-0.09	0.09	-0.98	0.98	-4.08
1.10	-14.65	-0.04	0.10	-0.79	0.79	-3.30

Table 4. Right Wall Axial Forces (kip).

Sp Pt	DC	AP+	AP-	TU+	TU-	CR&SH
3.0	-9.07	0.00	0.02	-0.07	0.07	0.30
3.1	-8.76	0.00	0.02	-0.07	0.07	0.30
3.2	-8.45	0.00	0.02	-0.07	0.07	0.30
3.3	-8.13	0.00	0.02	-0.07	0.07	0.30
3.4	-7.82	0.00	0.02	-0.07	0.07	0.30
3.5	-7.51	0.00	0.02	-0.07	0.07	0.30
3.6	-7.20	0.00	0.02	-0.07	0.07	0.30
3.7	-6.89	0.00	0.02	-0.07	0.07	0.30
3.8	-6.58	0.00	0.02	-0.07	0.07	0.30
3.9	-6.26	0.00	0.02	-0.07	0.07	0.30
3.10	-5.95	0.00	0.02	-0.07	0.07	0.30

Table 5. Right Wall Shear (kip).

Sp Pt	DC	AP+	AP-	TU+	TU-	CR&SH
3.0	1.05	0.00	0.06	0.21	0.21	0.89
3.1	1.05	0.00	0.05	0.21	0.21	0.89
3.2	1.05	0.00	0.03	0.21	0.21	0.89
3.3	1.05	0.00	0.01	0.21	0.21	0.89
3.4	1.05	0.00	0.00	0.21	0.21	0.89
3.5	1.05	0.02	0.00	0.21	0.21	0.89
3.6	1.05	0.03	0.00	0.21	0.21	0.89
3.7	1.05	0.05	0.00	0.21	0.21	0.89
3.8	1.05	0.07	0.00	0.21	0.21	0.89
3.9	1.05	0.08	0.00	0.21	0.21	0.89
3.10	1.05	0.10	0.00	0.21	0.21	0.89

Table 6. Right Wall Moment (kip-ft).

Sp Pt	DC	AP+	AP-	TU+	TU-	CR&SH
3.0	0.00	0.00	0.00	0.00	0.00	0.00
3.1	1.64	0.00	0.09	-0.33	0.33	-1.38
3.2	3.28	0.00	0.15	-0.66	0.66	-2.77
3.3	4.92	0.00	0.18	-1.00	1.00	-4.15
3.4	6.56	-0.01	0.19	-1.33	1.33	-5.53
3.5	8.20	-0.01	0.18	-1.66	1.66	-6.92
3.6	9.84	-0.01	0.14	-1.99	1.99	-8.30
3.7	11.48	-0.01	0.07	-2.32	2.32	-9.68
3.8	13.12	-0.02	0.01	-2.65	2.65	-11.07
3.9	14.76	-0.14	0.01	-2.99	2.99	-12.45
3.10	16.40	-0.28	0.01	-3.32	3.32	-13.83

Table 7. Slab Axial Forces (kip).

Sp Pt	DC	AP+	AP-	TU+	TU-	CR&SH
2.0	1.09	-0.10	0.15	-0.18	0.18	0.74
2.04	-1.00	0.00	0.20	-0.28	0.28	1.17
2.2	-0.97	0.00	0.20	-0.28	0.28	1.17
2.3	-0.95	0.00	0.20	-0.28	0.28	1.17
2.4	-0.93	0.00	0.20	-0.28	0.28	1.17
2.5	-0.91	0.00	0.20	-0.28	0.28	1.17
2.6	-0.90	0.00	0.20	-0.28	0.28	1.17
2.7	-0.88	0.00	0.20	-0.28	0.28	1.17
2.8	-0.86	0.00	0.20	-0.28	0.28	1.17
2.94	-1.05	0.00	0.10	-0.21	0.21	0.89
2.10	-1.05	0.00	0.10	-0.21	0.21	0.89

Table 8. Slab Shear (kip).

Sp Pt	DC	AP+	AP-	TU+	TU-	CR&SH
2.0	2.99	0.00	0.00	0.18	0.18	0.74
2.04	4.52	0.00	0.00	0.08	0.08	0.33
2.2	3.08	0.00	0.00	0.08	0.08	0.33
2.3	2.20	0.00	0.00	0.08	0.08	0.33
2.4	1.32	0.00	0.00	0.08	0.08	0.33
2.5	0.44	0.00	0.00	0.08	0.08	0.33
2.6	0.45	0.00	0.00	0.08	0.08	0.33
2.7	1.33	0.00	0.00	0.08	0.08	0.33
2.8	2.21	0.00	0.00	0.08	0.08	0.33
2.94	5.42	0.01	0.00	0.07	0.07	0.30
2.10	5.95	0.00	0.02	0.07	0.07	0.30

Table 9. Slab Moment (kip-ft).

Sp Pt	DC	AP+	AP-	TU+	TU-	CR&SH
2.0	-3.77	-0.09	0.02	-0.01	0.01	0.03
2.04	-23.88	-0.05	0.05	-0.49	0.49	-2.06
2.2	-1.78	-0.04	0.04	-0.06	0.06	-0.14
2.3	7.83	-0.03	0.03	-0.25	0.25	1.03
2.4	14.24	-0.02	0.02	-0.53	0.53	2.21
2.5	17.28	-0.02	0.02	-0.81	0.81	3.38
2.6	17.18	-0.02	0.01	-1.09	1.09	4.56
2.7	13.92	-0.03	0.00	-1.37	1.37	5.73
2.8	7.53	-0.04	0.00	-1.66	1.66	6.91
2.94	-4.08	-0.01	0.27	-3.16	3.16	13.18
2.10	-16.40	-0.01	0.28	-3.32	3.32	13.83

Unfactored live load envelopes, including PL, HL-93, EVs, and Permit are presented in Table 10 through Table 18.

Table 10. Left Wall Axial Live Load (kip).

Sp Pt	PL	HL-93 Negative	HL-93 Positive	EV Negative	EV Positive	Permit Negative	Permit Positive
1.0	-1.46	-6.08	0.00	-7.52	0.00	-7.90	0.00
1.1	-1.46	-6.08	0.00	-7.52	0.00	-7.90	0.00
1.2	-1.46	-6.08	0.00	-7.52	0.00	-7.90	0.00
1.3	-1.46	-6.08	0.00	-7.52	0.00	-7.90	0.00
1.4	-1.46	-6.08	0.00	-7.52	0.00	-7.90	0.00
1.497	-0.83	-6.10	0.00	-7.55	0.00	-7.93	0.00
1.6	-0.83	-6.10	0.00	-7.55	0.00	-7.93	0.00
1.7	-0.83	-6.10	0.00	-7.55	0.00	-7.93	0.00
1.8	-0.83	-6.10	0.00	-7.55	0.00	-7.93	0.00
1.9	-0.83	-6.10	0.00	-7.55	0.00	-7.93	0.00
1.10	-0.83	-6.10	0.00	-7.55	0.00	-7.93	0.00

Table 11. Left Wall Shear Live Load (kip).

Sp Pt	PL	HL-93 Max	HL-93 Min	EV Max	EV Min	Permit Max	Permit Min
1.0	0.05	0.53	0.14	0.65	0.15	0.69	0.22
1.1	0.05	0.53	0.14	0.65	0.15	0.69	0.22
1.2	0.05	0.53	0.14	0.65	0.15	0.69	0.22
1.3	0.05	0.53	0.14	0.65	0.15	0.69	0.22
1.4	0.05	0.53	0.14	0.65	0.15	0.69	0.22
1.497	0.25	3.06	0.81	3.79	0.85	3.98	1.25
1.6	0.25	3.06	0.81	3.79	0.85	3.98	1.25
1.7	0.25	3.06	0.81	3.79	0.85	3.98	1.25
1.8	0.25	3.06	0.81	3.79	0.85	3.98	1.25
1.9	0.25	3.06	0.81	3.79	0.85	3.98	1.25
1.10	0.25	3.06	0.81	3.79	0.85	3.98	1.25

Table 12. Left Wall Moment Live Load (kip-ft).

Sp Pt	PL	HL-93 Negative	HL-93 Positive	EV Negative	EV Positive	Permit Negative	Permit Positive
1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.1	0.07	0.00	0.97	0.00	1.20	0.00	1.26
1.2	0.14	0.00	1.94	0.00	2.40	0.00	2.52
1.3	0.20	0.00	2.91	0.00	3.61	0.00	3.78
1.4	0.27	0.00	3.88	0.00	4.81	0.00	5.04
1.497	-0.92	0.00	5.09	0.00	6.30	0.00	6.61
1.6	-0.48	-0.41	0.13	-0.51	0.16	-0.54	0.16
1.7	-0.04	-5.55	0.00	-6.87	0.00	-7.21	0.00
1.8	0.40	-10.87	0.00	-13.46	0.00	-14.13	0.00
1.9	0.84	-16.19	0.00	-20.05	0.00	-21.04	0.00
1.10	1.29	-21.67	0.00	-26.83	0.00	-28.16	0.00

Table 13. Right Wall Axial Live Load (kip).

Sp Pt	PL	HL-93 Negative	HL-93 Positive	EV Negative	EV Positive	Permit Negative	Permit Positive
3.0	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00
3.1	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00
3.2	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00
3.3	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00
3.4	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00
3.5	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00
3.6	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00
3.7	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00
3.8	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00
3.9	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00
3.10	0.02	-4.82	0.00	-5.97	0.00	-6.26	0.00

Table 14. Right Wall Shear Live Load (kip).

Sp Pt	PL	HL-93 Max	HL-93 Min	EV Max	EV Min	Permit Max	Permit Min
3.0	0.01	1.22	0.15	1.51	0.16	1.59	0.24
3.1	0.01	1.22	0.15	1.51	0.16	1.59	0.24
3.2	0.01	1.22	0.15	1.51	0.16	1.59	0.24
3.3	0.01	1.22	0.15	1.51	0.16	1.59	0.24
3.4	0.01	1.22	0.15	1.51	0.16	1.59	0.24
3.5	0.01	1.22	0.15	1.51	0.16	1.59	0.24
3.6	0.01	1.22	0.15	1.51	0.16	1.59	0.24
3.7	0.01	1.22	0.15	1.51	0.16	1.59	0.24
3.8	0.01	1.22	0.15	1.51	0.16	1.59	0.24
3.9	0.01	1.22	0.15	1.51	0.16	1.59	0.24
3.10	0.01	1.22	0.15	1.51	0.16	1.59	0.24

Table 15. Right Wall Moment Live Load (kip-ft).

Sp Pt	PL	HL-93 Negative	HL-93 Positive	EV Negative	EV Positive	Permit Negative	Permit Positive
3.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.1	-0.01	0.00	1.90	0.00	2.36	0.00	2.47
3.2	-0.03	0.00	3.81	0.00	4.71	0.00	4.94
3.3	-0.04	0.00	5.71	0.00	7.07	0.00	7.42
3.4	-0.05	0.00	7.61	0.00	9.42	0.00	9.89
3.5	-0.06	0.00	9.51	0.00	11.78	0.00	12.36
3.6	-0.08	0.00	11.42	0.00	14.14	0.00	14.83
3.7	-0.09	0.00	13.32	0.00	16.49	0.00	17.31
3.8	-0.10	0.00	15.22	0.00	18.85	0.00	19.78
3.9	-0.11	0.00	17.13	0.00	21.20	0.00	22.25
3.10	-0.13	0.00	19.03	0.00	23.56	0.00	24.72

Table 16. Slab Axial Live Load (kip).

Sp Pt	PL	HL-93 Negative	HL-93 Positive	EV Negative	EV Positive	Permit Negative	Permit Positive
2.0	-0.25	0.00	2.18	0.00	2.70	0.00	2.43
2.04	0.01	-1.11	0.00	-1.37	0.00	-1.44	0.00
2.2	0.01	-1.11	0.00	-1.37	0.00	-1.44	0.00
2.3	0.01	-1.11	0.00	-1.37	0.00	-1.44	0.00
2.4	0.01	-1.11	0.00	-1.37	0.00	-1.44	0.00
2.5	0.01	-1.06	0.00	-1.32	0.00	-1.38	0.00
2.6	0.01	-1.03	0.00	-1.27	0.00	-1.33	0.00
2.7	0.01	-1.01	0.00	-1.25	0.00	-1.32	0.00
2.8	0.01	-0.99	0.00	-1.22	0.00	-1.28	0.00
2.94	0.01	-1.22	0.00	-1.51	0.00	-1.59	0.00
2.10	0.01	-1.22	0.00	-1.51	0.00	-1.59	0.00

Table 17. Slab Shear Live Load (kip).

Sp Pt	PL	HL-93 Max	HL-93 Min	EV Max	EV Min	Permit Max	Permit Min
2.0	0.81	0.11	0.03	0.14	0.03	0.14	0.05
2.04	0.02	5.99	0.82	7.42	0.86	7.78	1.27
2.2	0.02	4.35	0.82	5.38	0.86	5.65	1.27
2.3	0.02	3.10	0.24	3.83	0.25	4.02	0.37
2.4	0.02	2.94	0.34	3.64	0.36	3.82	0.53
2.5	0.02	1.02	0.51	1.26	0.53	1.32	0.79
2.6	0.02	1.98	0.82	2.45	0.86	2.57	1.27
2.7	0.02	2.49	0.42	3.09	0.44	3.24	0.65
2.8	0.02	3.47	0.34	4.30	0.36	4.51	0.53
2.94	0.02	4.82	0.34	5.97	0.35	6.26	0.52
2.10	0.02	4.82	0.34	5.97	0.35	6.26	0.52

Table 18. Slab Moment Live Load (kip-ft).

Sp Pt	PL	HL-93 Negative	HL-93 Positive	EV Negative	EV Positive	Permit Negative	Permit Positive
2.0	-0.78	-3.95	0.00	-3.47	0.00	-5.14	0.00
2.04	-0.46	-27.29	0.00	-23.95	0.00	-35.46	0.00
2.2	-0.35	-4.77	2.76	-4.19	5.23	-6.20	4.72
2.3	-0.29	0.00	8.51	0.00	16.14	0.00	14.55
2.4	-0.23	0.00	13.18	0.00	24.98	0.00	22.51
2.5	-0.17	0.00	15.76	0.00	29.87	0.00	26.92
2.6	-0.10	0.00	15.35	0.00	29.11	0.00	26.23
2.7	-0.04	0.00	12.06	0.00	22.87	0.00	20.61
2.8	0.02	-1.08	6.65	-0.95	12.61	-1.41	11.37
2.94	0.09	-9.89	0.00	-8.68	0.00	-12.85	0.00
2.10	0.13	-19.02	0.00	-16.69	0.00	-24.72	0.00

2.2.3 Resistance Calculations

The depth, analysis section width, and reinforcement for each of the critical sections are tabulated below in Table 19.

Table 19. Critical Section Data.

Section	**Depth** (in)	**Width** (in)	**Bar Number**	**Bar Spacing** (in)	**Cover** (in)
1 - Left Wall Inside	16.00	12.00	6	6.00	2.00
2 – Left Wall Outside	16.00	12.00	8	6.00	2.00
3 – Right Wall Outside	16.00	12.00	6	6.00	2.00
4 – Right Wall Inside	16.00	12.00	6	6.00	2.00
5 – Slab Mid-Span	18.00	12.00	8	6.00	2.00
6 – Slab Left Connection	18.00	12.00	8	6.00	2.50
7 – Slab Right Connection	18.00	12.00	8	6.00	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 (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.

2.2.3.1 Section 1 – Left Wall Inside Steel

Rectangular section height:	h	=	16.00 inch	
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	
Area of single rebar:	A_{s_bar}	=	0.442 inch^2	
Diameter of rebar:	dia_{bar}	=	0.75 inch	
Spacing of rebar:	s	=	6.0 inch	
Flexural resistance factor:	ϕ_f	=	TBD	LRFD BDS 5.5.4.2

Moment Resistance:

Determine equivalent area of reinforcing bar:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.44(12.00)}{6.0} = 0.884 \text{ inch}^2$$

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

$$d_s = h - \frac{dia_{bar}}{2} - clr = 16.00 - \frac{0.75}{2} - 2.00 = 13.63 \text{ inches}$$

Determine distance from the equivalent stress block for tension controlled, non-prestressed tension reinforcement only:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.884(60)}{0.85(4.0)(12.00)} = 1.30 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

$$\beta_1 = 0.85 \Rightarrow c = \frac{a}{\beta_1} = \frac{1.30}{0.85} = 1.53 \text{ inches}$$ LRFD BDS 5.6.2.2

$$\varepsilon_s = \varepsilon_c\left(\frac{d_s}{c} - 1\right) = 0.003\left(\frac{13.63}{1.53} - 1\right) = 0.024 > 0.005$$

$$\therefore \phi_f = 0.90$$ LRFD BDS 5.5.4.2, 5.6.2.1

Calculate factored moment resistance:

$$\phi M_n = \phi_f A_s f_y\left(d_s - \frac{a}{2}\right) =$$

$$0.9(0.884)(60)\left(13.63 - \frac{1.30}{2}\right)\left(\frac{1 \text{ ft}}{12 \text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 51.59 \text{ kip-ft}$$

Axial Resistance:

Walls need to be checked for axial thrust and flexural interaction. Check flexure/axial interaction using the largest factored HL-93 Inventory axial load in the wall to check applicability:

$$0.1\phi_c f_c^{'} A_g = 0.1(0.7)(4.0)(16)(12) = 53.76 \text{ kip}$$

$$\text{N}_u = 28.28 \text{ kip (Strength-I)}$$ LRFD BDS 5.6.4.5

$$0.1\phi_c f_c^{'} A_g \geq N_u \therefore \text{Neglect Axial Thrust}$$

Minimum Steel:

Determine minimum reinforcement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(16.00)^2}{6} = 512 \text{ inch}^3$$ LRFD BDS 5.6.3.3

The cracking moment equation simplifies to the following (only monolithic sections and no prestress forces), where f_r is the modulus of rupture of concrete per LRFD BDS Article 5.4.2.6:

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(512)\left(\frac{1\text{ ft}}{12\text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.3-1

$$= 21.95\text{ kip-ft}$$

Since $\phi_f M_n > M_{cr}$, minimum reinforcement at this section is 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.

$$d_v = \max\begin{Bmatrix} 0.72h \\ 0.9d_s \\ d_s - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(16) = 11.52 \\ 0.9(13.63) = 12.27 \\ 13.63 - 1.30/2 = 12.98 \end{Bmatrix} = 12.98\text{ inches}$$ LRFD BDS 5.7.2.8

For this example, conservatively combine the largest factored HL-93 Inventory forces along the wall (points 1.00 to 1.10 in Figure 6) to compute the minimum shear resistance. Concurrent results along the length of the wall could be used to obtain more refined results when rating factors are below 1.00:

$$M_{u,max,LTwall} = 60.55\text{ kip-ft}$$
$$V_{u,max,LTwall} = 9.00\text{ kip}$$
$$\mathrm{N}_{u,max,LTwall} = 28.28\text{ kip}$$

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 not less than 16 inches and there is no shear reinforcement. The maximum aggregate size, a_g, is set to 0.75 inches.

$$M_u \geq V_u d_v \Rightarrow$$
$$60.55(12) = 726.00\text{ kip-in} \geq 9.00(12.98) = 116.82\text{ kip-inch}$$ LRFD BDS 5.7.3.4.2

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u|\right)}{E_s A_s}$$ LRFD BDS Eq. 5.7.3.4.2-4

$$= \frac{\left(\frac{|60.55(12)|}{12.98} + 0.5(28.28) + 9.00\right)}{0.884(29000)} = 0.0031$$

$$0 \leq \varepsilon_s \leq 0.006$$

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500(0.0031) = 39.85^o$$ LRFD BDS Eq. 5.7.3.4.2-3

$$s_x = d_v \leq h - 2cl - d_b = 12.98\,\text{in} \leq 16 - 2(2) - 0.75 = 11.25\,\text{inch}$$

Where *cl* is the clear cover and the smaller #6 bar diameter is assumed on each face;

$$s_{xe} = 12\text{ in} \leq s_x \frac{1.38}{a_g + 0.63} \leq 80\text{ inch}$$

LRFD BDS Eq. 5.7.3.4.2-7

$$s_{xe} = 12\text{ in} \leq 11.25\frac{1.38}{0.75 + 0.63} \leq 80\text{ in} \Rightarrow 12.00\,\text{inch}$$

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \frac{51}{(39 + s_{xe})}$$

LRFD BDS Eq. 5.7.3.4.2-2

$$= \frac{4.8}{[1 + 750(0.0031)]} \frac{51}{[39 + 12.00]} = 1.44$$

$$\phi V_n = \phi_v \left[0.0316\beta\sqrt{f_c^{'}} \leq 0.25 f_c^{'} \right] b_v d_v$$

$$= 0.90\left[0.0316(1.44)\sqrt{4.0} \leq 0.25(4.0) \right] 12(12.98)$$

LRFD BDS 5.7.3.3

$$= 12.76\text{ kip}$$

The following LRFD BDS checks have not been performed for this example but should be checked for actual rating calculations: regions requiring transverse reinforcement (5.7.2.3) and longitudinal reinforcement checks (5.7.3.5) [23 CFR 625.4(d)(1)(v)].

2.2.3.2 Section 2 – Left Wall Outside Steel

Rectangular section height:	*h*	=	16.00 inch	
Rectangular section width:	*b*	=	12.00 inch	
Clear distance to rebar from tension face:	*clr*	=	2.00 inch	
Area of single rebar:	A_{s_bar}	=	0.785 inch^2	
Diameter of rebar:	dia_{bar}	=	1.00 inch	
Spacing of rebar:	*s*	=	6.0 inch	
Flexural resistance factor:	ϕ_f	=	TBD	LRFD BDS 5.5.4.2

Moment Resistance:

Determine equivalent area of reinforcing bar:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.785(12.00)}{6.0} = 1.571\,\text{inch}^2$$

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

$$d_s = h - \frac{dia_{bar}}{2} - clr = 16.00 - \frac{1.00}{2} - 2.00 = 13.50\text{ inches}$$

Determine distance from the equivalent stress block for tension controlled, non-prestressed tension reinforcement only:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.571(60)}{0.85(4.0)(12.00)} = 2.31 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

$$\beta_1 = 0.85 \Rightarrow c = \frac{a}{\beta_1} = \frac{2.31}{0.85} = 2.72 \text{ inches}$$ LRFD BDS 5.6.2.2

$$\varepsilon_s = \varepsilon_c \left(\frac{d_s}{c} - 1 \right) = 0.003 \left(\frac{13.50}{2.72} - 1 \right) = 0.012 > 0.005$$
$$\therefore \phi_f = 0.90$$ LRFD BDS 5.5.4.2, 5.6.2.1

Calculate factored moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$
$$0.9(1.571)(60)\left(13.50 - \frac{2.31}{2}\right)\left(\frac{1 \text{ ft}}{12 \text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1
$$= 87.26 \text{ kip-ft}$$

Axial Resistance:

It was shown in Section 1 above that axial thrust can be neglected for the left wall.

Minimum Steel:

Determine minimum reinforcement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(16.00)^2}{6} = 512 \text{ inch}^3$$ LRFD BDS 5.6.3.3

The cracking moment equation simplifies to the following (only monolithic sections and no prestress forces), where f_r is the modulus of rupture of concrete per LRFD BDS Article 5.4.2.6:

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(512)\left(\frac{1 \text{ ft}}{12 \text{ inch}}\right)$$
$$= 21.95 \text{ kip-ft}$$ LRFD BDS Eq. 5.6.3.3-1

Since $\phi_f M_n > M_{cr}$, minimum reinforcement at this section is satisfied.

Shear Resistance:

Shear resistance is calculated for the wall with Section 1 above.

2.2.3.3 Section 3 – Right Wall Inside Steel

Rectangular section height:	*h*	=	16.00 inch

Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	
Area of single rebar:	A_{s_bar}	=	0.442 inch^2	
Diameter of rebar:	dia_{bar}	=	0.75 inch	
Spacing of rebar:	s	=	6.0 inch	
Flexural resistance factor:	ϕ_f	=	TBD	LRFD BDS 5.5.4.2

Moment Resistance:

Determine equivalent area of reinforcing bar:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.44(12.00)}{6.0} = 0.884 \text{ inch}^2$$

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

$$d_s = h - \frac{dia_{bar}}{2} - clr = 16.00 - \frac{0.75}{2} - 2.00 = 13.63 \text{ inches}$$

Determine distance from the equivalent stress block for tension controlled, non-prestressed tension reinforcement only:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.884(60)}{0.85(4.0)(12.00)} = 1.30 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

$$\beta_1 = 0.85 \Rightarrow c = \frac{a}{\beta_1} = \frac{1.30}{0.85} = 1.53 \text{ inches}$$ LRFD BDS 5.6.2.2

$$\varepsilon_s = \varepsilon_c \left(\frac{d_s}{c} - 1 \right) = 0.003 \left(\frac{13.63}{1.53} - 1 \right) = 0.024 > 0.005$$ LRFD BDS 5.5.4.2, 5.6.2.1

$$\therefore \phi_f = 0.90$$

Calculate factored moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(0.884)(60)\left(13.63 - \frac{1.30}{2}\right)\left(\frac{1 \text{ ft}}{12 \text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 51.59 \text{ kip-ft}$$

Axial Resistance:

Walls need to be checked for axial thrust and flexural interaction. Check flexure/axial interaction using the largest factored HL-93 Inventory axial load in the wall to check applicability:

$0.1\phi_c f_c^{'} A_g = 0.1(0.7)(4.0)(16)(12) = 53.76 \text{ kip}$

$\text{N}_u = 19.54 \text{ kip}$ LRFD BDS 5.6.4.5

$0.1\phi_c f_c^{'} A_g \geq N_u \therefore$ Neglect Axial Thrust

Minimum Steel:

Determine minimum reinforcement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(16.00)^2}{6} = 512 \text{ inch}^3$$ LRFD BDS 5.6.3.3

The cracking moment equation simplifies to the following (only monolithic sections and no prestress forces), where f_r is the modulus of rupture of concrete per LRFD BDS Article 5.4.2.6:

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(512)\left(\frac{1 \text{ ft}}{12 \text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.3-1

$$= 21.95 \text{ kip-ft}$$

Since $\phi_f M_n > M_{cr}$, minimum reinforcement at this section is 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.

$$d_v = \max\begin{Bmatrix} 0.72h \\ 0.9d \\ d - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(16) = 11.52 \\ 0.9(13.63) = 12.27 \\ 13.63 - 1.30/2 = 12.98 \end{Bmatrix} = 12.98 \text{ inch}$$ LRFD BDS 5.7.2.8

Determining the nominal shear resistance. For this example, conservatively combine the largest factored HL-93 Inventory forces along the wall (points 3.00 to 3.10 in Figure 6) to compute the minimum shear resistance. Concurrent results along the length of the wall could be used to obtain more refined results when rating factors are below 1.00):

$M_{u,max,RTwall} = 42.99 \text{ kip-ft}$

$V_{u,max,RTwall} = 4.78 \text{ kip}$

$\text{N}_{u,max,RTwall} = 19.54 \text{ kip}$

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 not less than 16 inches and there is no shear reinforcement.

$$M_u \geq V_u d_v \Rightarrow 42.99(12) = 515.88 \text{ kip-inch}$$
$$\geq 4.78(12.98) = 62.04 \text{ kip-inch}$$

LRFD BDS5.7.3.4.2

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u|\right)}{E_s A_s}$$

LRFD BDS Eq. 5.7.3.4.2-4

$$= \frac{\left(\frac{|515.88|}{12.98} + 0.5(19.54) + 4.78\right)}{0.884(29000)} = 0.0021$$

$$0 \leq \varepsilon_s \leq 0.006$$

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500(0.0021) = 36.35^o$$

LRFD BDS Eq. 5.7.3.4.2-3

$$s_x = d_v \leq h - 2cl - d_b = 12.98 \text{ in} \leq 16 - 2(2) - 0.75 = 11.25 \text{ inch}$$

Where *cl* is the clear cover;

$$s_{xe} = 12 \text{ in} \leq s_x \frac{1.38}{a_g + 0.63} \leq 80 \text{ inch}$$

LRFD BDS Eq. 5.7.3.4.2-7

$$s_{xe} = 12 \text{ in} \leq 11.25 \frac{1.38}{0.75 + 0.63} \leq 80 \text{ in} \Rightarrow 12.00 \text{ inch}$$

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \frac{51}{(39 + s_{xe})}$$

LRFD BDS Eq. 5.7.3.4.2-2

$$= \frac{4.8}{[1 + 750(0.0021)]} \frac{51}{[39 + 12.00]} = 1.86$$

$$\phi V_n = \phi_v \left[0.0316\beta\sqrt{f_c'} \leq 0.25 f_c'\right] b_v d_v$$
$$= 0.90\left[0.0316(1.86)\sqrt{4.0} \leq 0.25(4.0)\right] 12(12.98)$$
$$= 16.48 \text{ kip}$$

LRFD BDS 5.7.3.3

The following LRFD BDS checks have not been performed for this example but should be checked for actual rating calculations: regions requiring transverse reinforcement (5.7.2.3) and longitudinal reinforcement checks (5.7.3.5) [23 CFR 625.4(d)(1)(v)].

2.2.3.4 Section 4 – Right Wall Outside Steel

Rectangular section height:	*h*	=	16.00 inch
Rectangular section width:	*b*	=	12.00 inch
Clear distance to rebar from tension face:	*clr*	=	2.00 inch
Area of single rebar:	A_{s_bar}	=	0.442 inch^2
Diameter of rebar:	dia_{bar}	=	0.75 inch

Spacing of rebar:	s	=	6.0 inch	
Flexural resistance factor:	ϕ_f	=	TBD	LRFD BDS 5.5.4.2

Moment Resistance:

The reinforcement and factored moment resistance are the same as calculated for Section 3 above.

Axial Resistance:

It was shown in Section 3 above that axial thrust can be neglected for the left wall.

Minimum Steel:

It was shown in Section 3 above that the minimum steel reinforcement is met.

Shear Resistance:

Shear resistance is calculated for the wall with Section 3 above.

2.2.3.5 Section 5 – Slab

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	
Area of single rebar:	A_{s_bar}	=	0.785 inch^2	
Diameter of rebar:	dia_{bar}	=	1.00 inch	
Spacing of rebar:	s	=	6.0 inch	
Flexural resistance factor:	ϕ_f	=	TBD	LRFD BDS 5.5.4.2

Moment Resistance:

Determine equivalent area of reinforcing bar:

$$A_s = \frac{A_{s\_bar} b}{s} = \frac{0.785(12.00)}{6.0} = 1.571 \text{ inch}^2$$

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

$$d_s = h - \frac{dia_{bar}}{2} - clr = 18.00 - \frac{1.00}{2} - 2.00 = 15.50 \text{ inches}$$

Determine distance from the equivalent stress block for tension controlled, non-prestressed tension reinforcement only:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.571(60)}{0.85(4.0)(12.00)} = 2.31 \text{ inches} \qquad \text{LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2}$$

$$\beta_1 = 0.85 \Rightarrow c = \frac{a}{\beta_1} = \frac{2.31}{0.85} = 2.72 \text{ inches} \qquad \text{LRFD BDS 5.6.2.2}$$

$$\varepsilon_s = \varepsilon_c \left(\frac{d_s}{c} - 1 \right) = 0.003 \left(\frac{15.50}{2.72} - 1 \right) = 0.014 > 0.005$$ LRFD BDS 5.5.4.2, 5.6.2.1

$$\therefore \phi_f = 0.90$$

Calculate factored moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$

$$0.9(1.571)(60)\left(15.50 - \frac{2.31}{2} \right)\left(\frac{1\,\text{ft}}{12\,\text{inch}} \right)$$ LRFD BDS Eq. 5.6.3.2.2-1

$$= 101.40 \text{ kip-ft}$$

Axial Resistance:

Slab need to be checked for axial thrust and flexural interaction using the largest factored HL-93 Inventory axial load in the wall to check applicability:

$$0.1\phi_c f_c^{'} A_g = 0.1(0.7)(4.0)(18)(12) = 60.48 \text{ kip}$$

$$\text{N}_u = 2.76 \text{ kip}$$ LRFD BDS 5.6.4.5

$$0.1\phi_c f_c^{'} A_g \geq N_u \therefore \text{Neglect Axial Thrust}$$

Minimum Steel:

Determine minimum reinforcement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(18.00)^2}{6} = 648\,\text{inch}^3$$ LRFD BDS 5.6.3.3

The cracking moment equation simplifies to the following (only monolithic sections and no prestress forces), where f_r is the modulus of rupture of concrete per LRFD BDS Article 5.4.2.6:

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.67(1.6)(0.480)(648)\left(\frac{1\,\text{ft}}{12\,\text{inch}} \right)$$ LRFD BDS Eq. 5.6.3.3-1

$$= 27.79 \text{ kip-ft}$$

Since $\phi_f M_n > M_{cr}$, minimum reinforcement at this section is satisfied.

Shear Resistance

The shear depth, d_v, is calculated in accordance with LRFD BDS 5.7.3.2.

$$d_v = \max\begin{Bmatrix} 0.72h \\ 0.9d \\ d - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(18) = 12.96 \\ 0.9(15.50) = 13.95 \\ 15.50 - 2.31/2 = 14.35 \end{Bmatrix} = 14.35 \text{ inches}$$ LRFD BDS 5.7.2.8

Determining the nominal shear. For this example, conservatively combine the largest factored HL-93 Inventory forces along the slab (points 2.00 to 2.10 in Figure 6) to compute the minimum shear resistance. Concurrent results along the length of the slab could be used to obtain more refined results when rating factors are below 1.00:

$$M_{u,max,slab} = 81.27 \text{ kip-ft}$$
$$V_{u,max,slab} = 16.61 \text{ kip}$$
$$\mathrm{N}_{u,max,slab} = 2.76 \text{ kip}$$

The shear resistance of the slab needs to be evaluated using LRFD BDS 5.7.3.3. Shear factors β and θ need to be calculated since there is no shear reinforcement.

$$M_u \geq V_u d_v \Rightarrow 81.27(12) = 975.24 \text{ kip-inch}$$
$$\geq 16.61(14.35) = 268.35 \text{ kip-inch}$$ LRFD BDS5.7.3.4.2

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u|\right)}{E_s A_s}$$ LRFD BDS Eq. 5.7.3.4.2-4
$$= \frac{\left(\frac{|975.24|}{14.35} + 0.5(2.76) + |16.61|\right)}{1.571(29000)} = 0.0019$$

$$0 \leq \varepsilon_s \leq 0.006$$

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500(0.0019) = 35.65^o$$ LRFD BDS Eq. 5.7.3.4.2-3

$$s_x = d_v \geq h - \left(cl_{top} + cl_{bot}\right) - \frac{\left(d_{b,top} + d_{b,bot}\right)}{2} = 18 - \left(2.50 + 2.00\right) - \frac{\left(0.75 + 1.00\right)}{2} = 12.625 \text{ inch}$$

Where *cl* is the clear cover;

$$s_{xe} = 12 \text{ in} \leq s_x \frac{1.38}{a_g + 0.63} \leq 80 \text{ inch}$$ LRFD BDS Eq. 5.7.3.4.2-7
$$s_{xe} = 12 \text{ in} \leq 12.625 \frac{1.38}{0.75 + 0.63} \leq 80 \text{ in} \Rightarrow 12.625 \text{ inch}$$

$$\beta = \frac{4.8}{\left(1+750\varepsilon_s\right)}\frac{51}{\left(39+s_{xe}\right)}$$

$$= \frac{4.8}{\left[1+750(0.0019)\right]}\frac{51}{\left[39+12.625\right]} = 1.96$$ LRFD BDS Eq. 5.7.3.4.2-2

$$\phi V_n = \phi_v\left[0.0316\beta\sqrt{f_c^{'}} \le 0.25 f_c^{'}\right]b_v d_v$$

$$= 0.90\left[0.0316(1.95)\sqrt{4.0} \le 0.25(4.0)\right]12(14.35)$$ LRFD BDS 5.7.3.3

$$= 19.10 \text{ kip}$$

The following LRFD BDS checks have not been performed for this example but should be checked for actual rating calculations: regions requiring transverse reinforcement (5.7.2.3) and longitudinal reinforcement checks (5.7.3.5) [23 CFR 625.4(d)(1)(v)].

2.2.3.6 Section 6 –Slab Left Connection

Rectangular section height:	h	=	18.00 inch	
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}	=	0.785 inch^2	
Diameter of rebar:	dia_{bar}	=	1.00 inch	
Spacing of rebar:	s	=	6.0 inch	
Flexural resistance factor:	ϕ_f	=	TBD	LRFD BDS 5.5.4.2

Moment Resistance:

Determine equivalent area of reinforcing bar:

$$A_s = \frac{A_{s\_bar}b}{s} = \frac{0.785(12.00)}{6.0} = 1.571 \text{ inch}^2$$

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

$$d_s = h - \frac{dia_{bar}}{2} - clr = 18.00 - \frac{1.00}{2} - 2.50 = 15.00 \text{ inches}$$

Determine distance from the equivalent stress block for tension controlled, non-prestressed tension reinforcement only:

$$a = \frac{A_s f_y}{0.85 f'_c\, b} = \frac{1.571(60)}{0.85(4.0)(12.00)} = 2.31 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

$$\beta_1 = 0.85 \Rightarrow c = \frac{a}{\beta_1} = \frac{2.31}{0.85} = 2.72 \text{ inches}$$ LRFD BDS 5.6.2.2

$$\varepsilon_s = \varepsilon_c\left(\frac{d_s}{c} - 1\right) = 0.003\left(\frac{15.00}{2.72} - 1\right) = 0.014 > 0.005$$

$$\therefore \phi_f = 0.90$$

LRFD BDS 5.5.4.2, 5.6.2.1

Calculate factored moment resistance:

$$\phi M_n = \phi_f A_s f_y\left(d_s - \frac{a}{2}\right) =$$

$$0.9(1.571)(60)\left(15.00 - \frac{2.31}{2}\right)\left(\frac{1\text{ ft}}{12\text{ inch}}\right)$$

LRFD BDS Eq. 5.6.3.2.2-1

$$= 97.86\text{ kip-ft}$$

Axial Resistance:

It was shown in Section 5 above that axial thrust can be neglected for the left wall.

Minimum Steel:

Determine minimum reinforcement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(18.00)^2}{6} = 648\text{ inch}^3$$

LRFD BDS 5.6.3.3

The cracking moment equation simplifies to the following (only monolithic sections and no prestress forces), where f_r is the modulus of rupture of concrete per LRFD BDS Article 5.4.2.6:

$$M_{cr} = \gamma_3\gamma_1 f_r S_c = 0.67(1.6)(0.480)(648)\left(\frac{1\text{ ft}}{12\text{ inch}}\right)$$

LRFD BDS Eq. 5.6.3.3-1

$$= 27.79\text{ kip-ft}$$

Since $\phi_f M_n > M_{cr}$, minimum reinforcement at this section is satisfied.

Shear Resistance:

Shear resistance is calculated for the slab with Section 5 above.

2.2.3.7 Section 7 – Slab Right Connection

Rectangular section height:	h	=	18.00 inch	
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.50 inch	LRFD BDS 5.10.1
Area of single rebar:	A_{s_bar}	=	0.785 inch^2	
Diameter of rebar:	dia_{bar}	=	1.0 inch	
Spacing of rebar:	s	=	6.0 inch	
Flexural resistance factor:	ϕ_f	=	TBD	LRFD BDS 5.5.4.2

The slab right connection details are the same as the left connection details (Section 6); therefore, the factored moment and shear resistances are the same.

2.2.4 LRFR Rating Calculations

The structural condition of the internal frame 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 Section 5 and specified in the calculations of Section 2.2.3 of this document.

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.

$$RF = \frac{C \pm \gamma_{DC} DC \pm \gamma_{DW} DW \pm \gamma_{EV} EV \pm \gamma_{EH} EH \pm \gamma_{ES} ES}{\gamma_{LL}(LL + IM)}$$ 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 20 through Table 24 shows the results of the overall capacity based on MBE Equation 6A.4.2.1-2 as well as the rating load factors based on MBE Equation 6A.4.2.1-1. It is also noted that negative moments 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. For this example, shear forces were taken as positive as a simplification because controlling component shear loads acted on the rating sections in the same direction. 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 wall to slab interface and the critical shear (x_{cr} for shear) is at a distance d_v from the inside face of the wall or slab. A summary of these locations are:

Moment @ 1:	Left Wall at centerline of intersection with horizontal Egress Corridor Slab
Moment @ 2:	Left Wall at underside of Roadway Slab
Moment @ 3&4:	Right Wall at underside of Roadway Slab
Moment @ 5:	Roadway Slab maximum positive moment based on review of tenth point total factored forces
Moment @ 6:	Roadway Slab at inside face of Left Wall
Moment @ 7:	Roadway Slab at inside face of Right Wall
Shear @ 1&2:	Left Wall at d_v below Roadway Slab based on review of tenth point total factored forces
Shear @ 3&4:	Right Wall at d_v below Roadway Slab based on review of tenth point total factored forces

Shear @ 5-7: Roadway Slab at d_v from inside face of Left Wall based on review of tenth point total factored forces

Maximum and minimum load factors are selectively used to obtain the maximum absolute value of the force effects. The selected load factors are shown in Table 20. Note that a minimum load factor of 0.00 for CR and SH is defined in case actual forces are less than computed; therefore, any beneficial effects of CR and SH will not be counted. Additionally, the design truck controls the HL-93 ratings and the EV-3 controls the EV ratings. The moments, shears and respective capacities are in kip-ft and kip in Table 21 to Table 24.

Table 20. Load Factors.

Max/Min	DC	AP	TU	CR&SH	PL	LL Inventory	LL Operating	EV Legal	Permit
Max	1.25	1	0.5	1.25	0	1.75	1.35	1.3	1.3
Min	0.9	1	0.5	0	1.75	1.75	1.35	1.3	1.3

Table 21. HL-93 Inventory Rating Factors.

Force	Section	x_{cr} (in)	DC	AP+	AP-	TU+	TU-	CR&SH	PL	HL-93	C	RF
Moment	1	109.50	2.74	-0.09	0.09	-1.71	1.71	-7.13	-0.92	5.07	51.59	5.32
Moment	2	210.96	-13.21	-0.06	0.09	-0.87	0.87	-3.62	1.10	-19.45	-87.26	1.93
Moment	3 & 4	178.00	15.61	-0.21	0.01	-3.16	3.16	-13.17	-0.12	18.11	51.59	0.96
Moment	5	259.20	17.17	-0.02	0.01	-1.09	1.09	4.56	-0.10	15.35	101.4	2.74
Moment	6	24.00	-21.37	-0.05	0.05	-0.44	0.44	-1.84	-0.44	-24.73	-97.86	1.56
Moment	7	424.00	-12.61	-0.01	0.28	-3.27	3.27	13.63	0.12	-16.22	-97.86	2.83
Shear	1 & 2	198.02	1.99	0.05	0.05	0.11	0.11	0.44	0.25	3.06	12.76	1.66
Shear	3 & 4	16.02	1.05	0.00	0.05	0.21	0.21	0.89	0.01	0.15	16.84	50.62
Shear	5 - 7	38.35	4.06	0.00	0.00	0.08	0.08	0.33	0.02	5.47	19.1	1.41

Table 22. HL-93 Operating Rating Factors.

Force	Section	x_{cr} (in)	DC	AP+	AP-	TU+	TU-	CR&SH	PL	HL-93	C	RF
Moment	1	109.50	2.74	-0.09	0.09	-1.71	1.71	-7.13	-0.92	5.07	51.59	6.90
Moment	2	210.96	-13.21	-0.06	0.09	-0.87	0.87	-3.62	1.10	-19.45	-87.26	2.50
Moment	3 & 4	178.00	15.61	-0.21	0.01	-3.16	3.16	-13.17	-0.12	18.11	51.59	1.25
Moment	5	259.20	17.17	-0.02	0.01	-1.09	1.09	4.56	-0.10	15.35	101.4	3.55
Moment	6	24.00	-21.37	-0.05	0.05	-0.44	0.44	-1.84	-0.44	-24.73	-97.86	2.01
Moment	7	424.00	-12.61	-0.01	0.28	-3.27	3.27	13.63	0.12	-16.22	-97.86	3.67
Shear	1 & 2	198.02	1.99	0.05	0.05	0.11	0.11	0.44	0.25	3.06	12.76	2.15
Shear	3 & 4	16.02	1.05	0.00	0.05	0.21	0.21	0.89	0.01	0.15	16.84	65.62
Shear	5 - 7	38.35	4.06	0.00	0.00	0.08	0.08	0.33	0.02	5.47	19.1	1.83

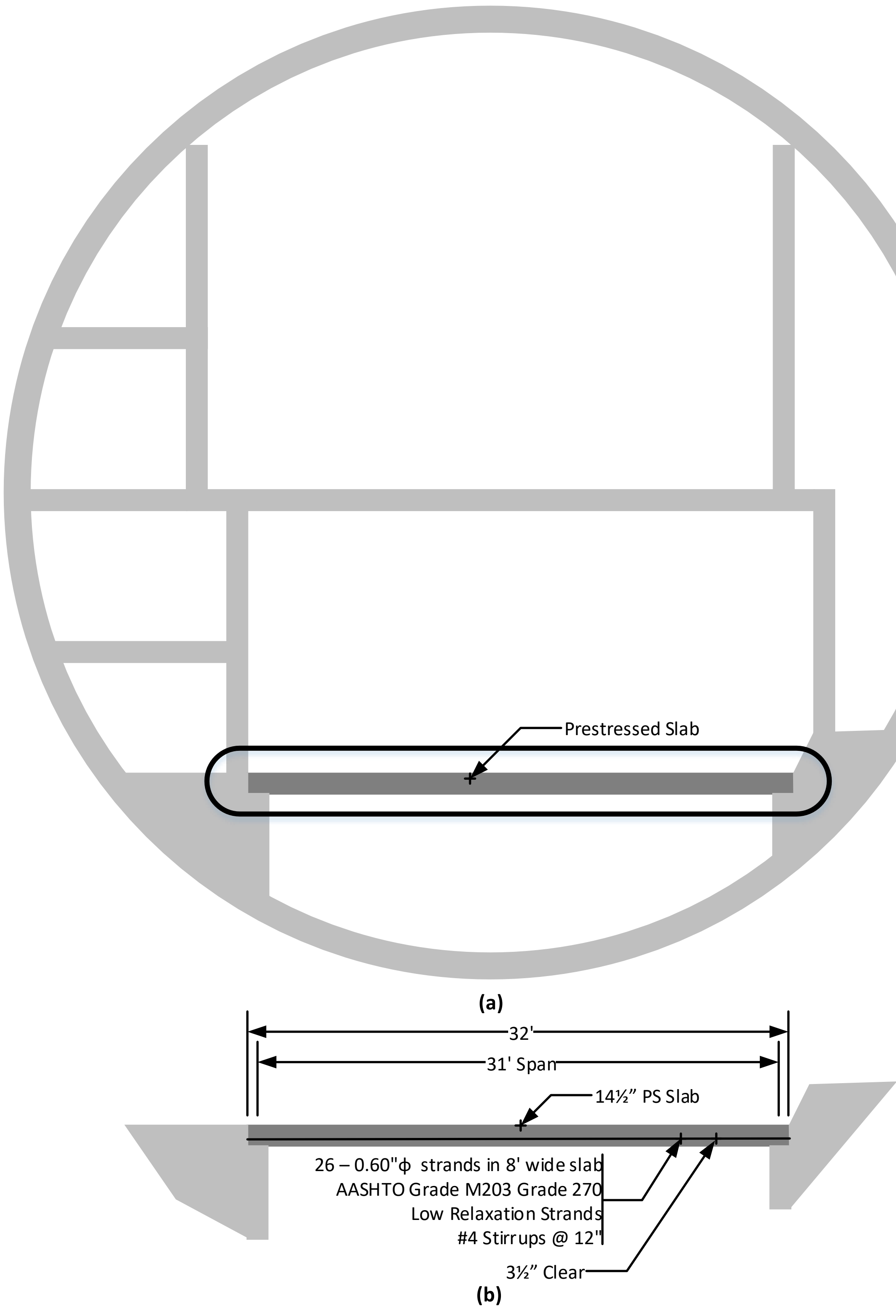
Table 23. EV-2 and EV-3 Rating Factors.

Force	Section	x_{cr} (in)	DC	AP+	AP-	TU+	TU-	CR&SH	PL	EV	C	RF
Moment	1	109.50	2.74	-0.09	0.09	-1.71	1.71	-7.13	-0.92	6.28	51.59	5.79
Moment	2	210.96	-13.21	-0.06	0.09	-0.87	0.87	-3.62	1.10	-24.08	-87.26	2.10
Moment	3 & 4	178.00	15.61	-0.21	0.01	-3.16	3.16	-13.17	-0.12	22.42	51.59	1.05
Moment	5	259.20	17.17	-0.02	0.01	-1.09	1.09	4.56	-0.10	29.11	101.4	1.95
Moment	6	24.00	-21.37	-0.05	0.05	-0.44	0.44	-1.84	-0.44	-21.70	-97.86	2.38
Moment	7	424.00	-12.61	-0.01	0.28	-3.27	3.27	13.63	0.12	-14.23	-97.86	4.35
Shear	1 & 2	198.02	1.99	0.05	0.05	0.11	0.11	0.44	0.25	3.79	12.76	1.83
Shear	3 & 4	16.02	1.05	0.00	0.05	0.21	0.21	0.89	0.01	0.16	16.84	65.13
Shear	5 - 7	38.35	4.06	0.00	0.00	0.08	0.08	0.33	0.02	6.77	19.1	1.54

Table 24. Permit Rating Factors.

Force	Section	x_{cr} (in)	DC	AP+	AP-	TU+	TU-	CR&SH	PL	Permit	C	RF
Moment	1	109.50	2.74	-0.09	0.09	-1.71	1.71	-7.13	-0.92	6.59	51.59	5.51
Moment	2	210.96	-13.21	-0.06	0.09	-0.87	0.87	-3.62	1.10	-25.27	-87.26	2.00
Moment	3 & 4	178.00	15.61	-0.21	0.01	-3.16	3.16	-13.17	-0.12	23.53	51.59	1.00
Moment	5	259.20	17.17	-0.02	0.01	-1.09	1.09	4.56	-0.10	26.23	101.4	2.16
Moment	6	24.00	-21.37	-0.05	0.05	-0.44	0.44	-1.84	-0.44	-32.13	-97.86	1.62
Moment	7	424.00	-12.61	-0.01	0.28	-3.27	3.27	13.63	0.12	-21.07	-97.86	2.94
Shear	1 & 2	198.02	1.99	0.05	0.05	0.11	0.11	0.44	0.25	3.98	12.76	1.75
Shear	3 & 4	16.02	1.05	0.00	0.05	0.21	0.21	0.89	0.01	0.24	16.84	44.69
Shear	5 thru 7	38.35	4.06	0.00	0.00	0.08	0.08	0.33	0.02	7.10	19.1	1.47

2.3 PRESTRESSED BOTTOM SLAB



Source: FHWA

Figure 7. Illustration. PS Slab Geometry and Prestressing Steel.

The prestressed bottom slab is a 14 ½-inch slab spanning 32 feet with a 12-inch bearing on each end resulting in an effective span of 31 feet. The PS slab is comprised of 7 ksi concrete, (26)-0.60 inch diameter, AASHTO Grade M203 Low Relaxation strands for each 8 foot slab with #4 shear bars at 12 inches on center in both transverse and longitudinal directions. The reinforcing steel is grade 60 steel. Adjacent precast slabs are connected via shear keys and therefore transfer forces between them.

2.3.1 Load Calculations

The slab is exposed to self-weight, air pressure, and live load. Other forces such as TU are neglected due to the simply supported boundary conditions of the PS slab. Creep and Shrinkage as well as prestressing forces are accounted for in the capacity calculations

2.3.1.1 Dead Load Component, DC

Dead load components include the self-weight of the concrete prestressed slab. Prestressed forces are accounted for in the capacity calculations.

Use a unit weight, w_c, of 0.150 kcf for reinforced concrete. The self-weight of the slab is applied to the model by including self-weight on the geometry detailed in Figure 7.

2.3.1.2 Wearing Surface, DW

The prestressed slab has a 2-inch asphalt overlay with a unit weight of 140 pcf. This produces a load pressure of 23.33 psf. However, since this is a known specified load applied during initial construction with field verification as opposed to an allowance for a future overly, the wearing surface will be included in the Dead Load Component, DC loads for this example, such that a strength load factor of 1.25 is applied per MBE 6A.2.2.3.

2.3.1.3 Air Pressure, AP

Air pressure was applied to the roadway of ±10 psf, according to the original construction plans.

2.3.1.4 Temperature, TU

No temperature loads were applied since this prestressed slab behaves as a simply supported slab.

2.3.1.5 Creep and Shrinkage, CR & SH

Creep and shrinkage is accounted for in the prestress losses within the capacity calculations.

2.3.1.6 Live Load Application, LL

The live load travels perpendicular to the span of the prestressed slab. The distribution of axle loads based on an equivalent strip width, E, to cast-in-place slabs acting perpendicular to the direction of traffic is provided in LRFD BDS Table 4.6.2.1.3-1, where S is the span length of 31 feet.

$$E_{pos} = 26.0 + 6.6S$$

$$E_{Slab+} = 26.0 + 6.6(31) = 230.60 \text{ inches} = 19.22 \text{ feet}$$

Since the design strip is transverse to the direction of traffic, the wheel loads are included in the live load evaluation; however, the lane load is not included per LRFD BDS Article 3.6.1.3.3. The maximum number of trucks to be placed on the structure is obtained by taking the roadway width and dividing by a 12-foot lane per LRFD BDS Article 3.6.1.1.1 resulting in two possible trucks to be placed parallel. This results in axle loads that are placed transversely such to produce maximum forces at the critical locations, identified at mid-span and end of span. Maximum end forces (shear) were obtained by placing the first axle load 2 feet from the barrier per LRFD BDS Article 3.6.1.3.1. The second truck is placed such that there is 4 feet between wheel placements. Maximum positive moment is obtained when the centerline of the span is midway between the center of gravity of loads and the nearest concentrated load. Live load placement is shown in Figure 8.

Wheel loads for unit, or 1-foot, analysis widths are determined by multiplying the equivalent unit width wheel load, *P*, by the appropriate impact factor (*IM*) and multiple presence factor (*MPF*). The equivalent wheel load is determined by dividing the full wheel load by the equivalent strip width, *E*, over which the axle load is distributed. Calculate the wheel loads in the same manner as the internal frame. Refer to Section 2.2.1.7 for a detailed discussion.

HL-93 Design Truck:

$$P_{HL93-Truck1} = \frac{\left(\dfrac{32 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}\right)} = \frac{16}{19.22} = 0.83\,\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Truck2} = \frac{\left(\dfrac{32 \text{ kip} + 32 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}/2 + 14 + E_{Slab+}/2\right)} = \frac{32}{33.22} = 0.96\,\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Truck3} = \frac{\left(\dfrac{32 \text{ kip} + 32 \text{ kip} + 8 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}/2 + 14 + 14 + E_{Slab+}/2\right)} = \frac{36}{47.22} = 0.76\,\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Truck+} = \max\left(P_{HL93-Truck1}, P_{HL93-Truck2}, P_{HL93-Truck3}\right) = 0.96\,\frac{\text{k/wheel}}{\text{ft}}$$

HL-93 Design Tandem:

$$P_{HL93-Tandem1} = \frac{\left(\dfrac{25 \text{ kip}}{2 \text{ wheels}}\right)}{\left(E_{Slab+}\right)} = \frac{12.5}{19.22} = 0.65\,\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Tandem2} = \frac{\left(\frac{25\text{ kip} + 25\text{ kip}}{2\text{ wheels}}\right)}{\left(E_{Slab+}/2 + 4 + E_{Slab+}/2\right)} = \frac{25}{23.22} = 1.08\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{HL93-Tandem+} = \max\left(P_{HL93-Tandem1}, P_{HL93-Tandem2}\right) = 1.08\frac{\text{k/wheel}}{\text{ft}}$$

EV-2:

$$P_{EV2-1} = \frac{\left(\frac{33.5\text{ kip}}{2\text{ wheels}}\right)}{\left(E_{Slab+}\right)} = \frac{16.75}{19.22} = 0.87\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{EV2-2} = \frac{\left(\frac{33.5\text{ kip} + 24\text{ kip}}{2\text{ wheels}}\right)}{\left(E_{Slab+}/2 + 15 + E_{Slab+}/2\right)} = \frac{28.75}{34.22} = 0.84\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{EV2+} = \max\left(P_{EV2-1}, P_{EV2-2}\right) = 0.87\frac{\text{k/wheel}}{\text{ft}}$$

EV-3:

$$P_{EV3-1} = \frac{\left(\frac{31\text{ kip}}{2\text{ wheels}}\right)}{\left(E_{Slab+}\right)} = \frac{15.5}{19.22} = 0.81\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{EV3-2} = \frac{\left(\frac{31\text{ kip} + 31\text{ kip}}{2\text{ wheels}}\right)}{\left(E_{Slab+}/2 + 4 + E_{Slab+}/2\right)} = \frac{31}{23.22} = 1.34\frac{\text{k/wheel}}{\text{ft}}$$

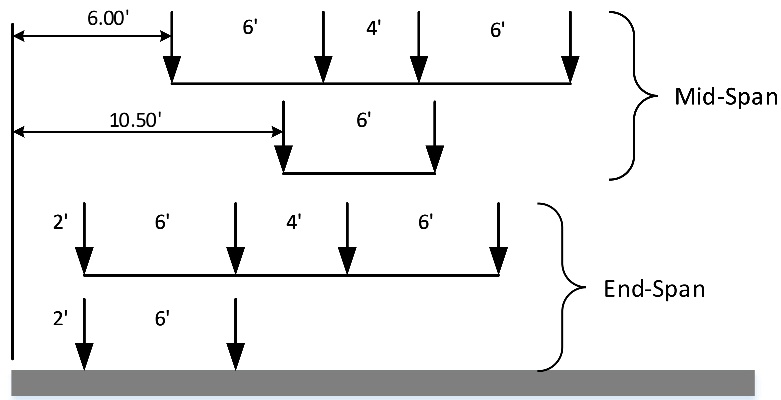
$$P_{EV3-3} = \frac{\left(\frac{31\text{ kip} + 31\text{ kip} + 24\text{ kip}}{2\text{ wheels}}\right)}{\left(E_{Slab+}/2 + 4 + 15 + E_{Slab+}/2\right)} = \frac{43}{38.22} = 1.13\frac{\text{k/wheel}}{\text{ft}}$$

$$P_{EV3+} = \max\left(P_{EV3-1}, P_{EV3-2}, P_{EV3-3}\right) = 1.34\frac{\text{k/wheel}}{\text{ft}}$$

Permit Truck:

All the permit truck axle spacings are less than E_{Slab+}; therefore, all 11 axle load possibilities should be considered. The calculations follow the format as above and P varies from 0.55 to 1.72 over the 11 permutations. The calculation for the controlling 10-axle configuration is shown below.

$$P_{Permit+} = \frac{\left(\dfrac{21+21+21+21+21+23+23+23+18+18}{2\text{ wheels}}\right)}{\left(E_{Slab+}/2+4+4+4+4+10+4+4+4+4+E_{Slab+}/2\right)} = \frac{105}{61.22} = 1.72\frac{\text{k/wheel}}{\text{ft}}$$



Source: FHWA

Figure 8. Illustration. Live Load Placement on Slab.

Single trucks are amplified by a multiple presence factor, *MPF*, of 1.2 while the two truck effects utilize a multiple presence factor of 1.0 per LRFD BDS Table 3.6.1.1.2-1. For this example, MBE Article 6A.4.4.3 is used to determine the dynamic impact factor, *IM*, of 1.33 for all vehicles.. Additionally, from the equivalent wheel load calculations, it can be seen that the HL-93 tandem will control over the HL-93 truck and the EV-3 vehicle controls over the EV-2 for this example.

2.3.2 Structural Analysis

A beam analysis is applied for the bottom slab. All the load effects and member resistances are calculated using this 1-foot wide strip representation. The slab is assumed to be simply supported and therefore can be evaluated with simple hand calculations or by a structural analysis program. For the purpose of this example, a structural analysis program was used for simplicity.

The *LRFD Road Tunnel Design and Construction Guide Specifications*, 1st Ed. (AASHTO, 2017b), Table 3.4-1 identifies the applicable minimum and maximum load factors for each load specified above. Since the slab is simply supported, only maximum load factors were necessary to produce critical design forces. The live load factors for the HL-93 is identified in Table 3.4-1, and the live load factor for the EV vehicles is 1.3 as identified in the FHWA Emergency Vehicle Memorandum and the live load factor for the permit vehicle is identified as 1.3 from MBE 3rd Edition Table 6A.4.5.4.2a-1, corresponding to unlimited crossings, mixed with traffic, two or more lanes, ADTT greater than 5000, and a permit weight factor ratio (GVW/AL) greater than 3.

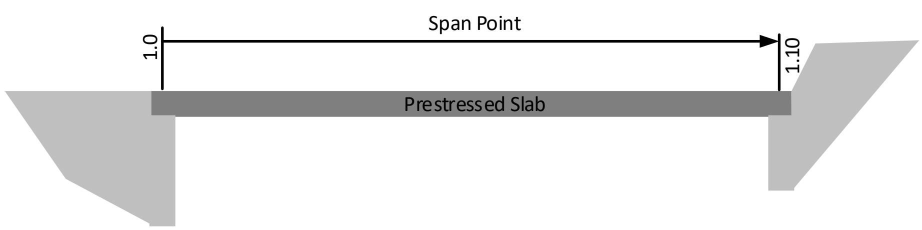
The forces were factored to produce controlling Strength I (or Strength II for permit loads) as well as Service I and Service III load combinations. These resulting unfactored forces are given in Table 25 and Table 26. Note shears are shown as absolute values.

Table 25. Slab Shear Forces (kip).

Sp Pt	DC	AP	HL-93	EV	Permit
1.0	3.45	0.16	3.89	5.22	6.20
1.1	2.76	0.13	3.15	4.22	5.02
1.2	2.07	0.10	2.46	3.29	3.91
1.3	1.38	0.06	1.95	2.61	3.10
1.4	0.69	0.03	0.28	0.37	0.44
1.5	0.00	0.00	0.28	0.37	0.44
1.6	-0.69	-0.03	-0.28	-0.37	-0.44
1.7	-1.38	-0.06	-1.95	-2.61	-3.10
1.8	-2.07	-0.10	-2.46	-3.29	-3.91
1.9	-2.76	-0.13	-3.15	-4.22	-5.02
1.10	-3.45	-0.16	-3.89	-5.22	-6.20

Table 26. Slab Moment Forces (kip-ft).

Sp Pt	DC	AP	HL-93	EV	Permit
1.0	0.00	0.00	0.00	0.00	0.00
1.1	9.62	0.45	10.49	14.06	16.70
1.2	17.10	0.80	19.25	25.81	30.65
1.3	22.44	1.05	24.56	32.93	39.12
1.4	25.64	1.20	29.30	39.29	46.67
1.5	26.71	1.25	30.16	40.44	48.04
1.6	25.64	1.20	27.29	36.59	43.46
1.7	22.44	1.05	23.70	31.78	37.75
1.8	17.10	0.80	16.09	21.57	25.62
1.9	9.62	0.45	8.04	10.79	12.81
1.10	0.00	0.00	0.00	0.00	0.00



Source: FHWA

Figure 9. Illustration. Slab Direction.

Figure 9 shows the span point identification for the prestressed slab. The whole number represents the member identification while the 10ths number indicates the percentage along the span length.

2.3.3 Resistance Calculations

2.3.3.1 Concrete Properties

f'_c = 7.0 ksi	
f'_{ci} = 5.6 ksi	LRFD BDS 5.4.2.3.2
α_1 = 0.85	LRFD BDS 5.6.2.2
β_1 = 0.70 if f'_c = 7.0 ksi	LRFD BDS 5.6.2.2
λ = 1.0 (normal weight concrete)	LRFD BDS 5.4.2.8
γ_2 = 1.1 (bonded strands)	LRFD BDS 5.6.3.3
γ_1 = 1.6	LRFD BDS 5.6.3.3
γ_3 = 1.0 (prestressed girders)	LRFD BDS 5.6.3.3

Modulus of rupture:

$$f_r = 0.24\lambda\sqrt{f'_c} = 0.24(1.0)\sqrt{7.0} = 0.635 \text{ ksi}$$ 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.

2.3.3.2 General Properties

Rectangular section height:	h	=	14.50 inch	
Rectangular section width:	b	=	12.00 inch	
Slab width:	b_{slab}	=	8.00 feet	
Distance to PS strand from tension face:	clr	=	3.50 inch	
Area of PS strand:	A_{s_PS}	=	0.217 inch^2	
Diameter of PS strand:	dia_{PS}	=	0.60 inch	
Area of single rebar:	A_{s_bar}	=	0.196 inch^2	
Diameter of rebar:	dia_{bar}	=	0.500 inch	
Spacing of rebar shear steel:	s	=	12.0 inch	
Flexural resistance factor:	ϕ_f	=	TBD	LRFD BDS 5.5.4.2
Shear resistance factor:	ϕ_v	=	0.9	LRFD BDS 5.5.4.2

Moment Resistance:

Determine equivalent area of prestressing:

$$A_s = \frac{A_{s\_PS}n}{b_{slab}} = \frac{0.217(26)}{8} = 0.705 \text{ in}^2/\text{ft}$$

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

$$d_s = h - 3.50 = 14.50 - 3.5 = 11.00 \text{ inches}$$

Eccentricity of prestressing steel:

$$e = d_s - \frac{h}{2} = 11.00 - \frac{14.50}{2} = 3.75 \text{ inches}$$

Modulus of Elasticity:

$$\gamma_c = 0.140 + 0.001 f'_c = 0.140 + 0.001(7.0) = 0.147 \text{ kcf}$$ LRFD BDS Table 3.5.1-1

$$\gamma_{ci} = 0.140 + 0.001 f'_{ci} = 0.140 + 0.001(5.6) = 0.1456 \text{ kcf}$$ LRFD BDS Table 3.5.1-1

$$\begin{aligned} E_c &= 120,000 K_1 (\gamma_c)^2 f_c'^{0.33} = 120,000(1.0)(0.147)^2 (7.0)^{0.33} \\ &= 4928 \text{ ksi} \end{aligned}$$ LRFD BDS Eq. 5.4.2.4-1

$$\begin{aligned} E_{ci} &= 120,000 K_1 (\gamma_{ci})^2 f_{ci}'^{0.33} = 120,000(1.0)(0.1456)^2 (5.6)^{0.33} \\ &= 4492 \text{ ksi} \end{aligned}$$ LRFD BDS Eq. 5.4.2.4-1

$$E_p = 28,500 \text{ ksi}$$ LRFD BDS 5.4.4.2

Section Properties:

$$A_c = bh = 12(14.50) = 174 \text{ inch}^2$$

$$y = \frac{h}{2} = \frac{14.50}{2} = 7.25 \text{ inches}$$

$$I_x = \frac{bh^3}{12} = \frac{12(14.50)^3}{12} = 3049 \text{ inch}^4$$

$$S_x = \frac{bh^2}{6} = \frac{12(14.50)^2}{6} = 420.5 \text{ inch}^3$$

2.3.3.3 Prestressed Losses

Jacking Stress: LRFD BDS Table 5.9.2.2-1

$$f_j = 0.75 f_{pu} = 0.75(270) = 202.5 \text{ ksi}$$

Elastic Shortening Losses: LRFD BDS Eq. C5.9.3.2.3a-1

$$\begin{aligned} \Delta f_{pES} &= \frac{\left[A_{ps} f_j \left(I_x + e^2 A_c \right) - e M_{SW} A_c \right]}{\left[A_{ps} \left(I_x + e^2 A_c \right) + A_c I_x E_{ci} / E_p \right]} \\ &= \frac{\left[0.705(202.5)(3049 + 3.75^2 \times 174) - 3.75(23.95 \times 12)(174) \right]}{\left[0.705(3049 + 3.75^2 \times 174) + 174(3049)(4492) / (28,500) \right]} = 6.82 \text{ ksi} \end{aligned}$$

$$f_{pi} = f_{pj} - \Delta_{pES} = 202.5 - 6.82 = 195.68 \text{ ksi}$$
$$\mathrm{P}_i = f_{pi} A_{ps} = 195.68(0.705) = 138 \text{ kip}$$

Elastic gains are neglected for this example for conservatism. Long term losses are calculated utilizing the Approximate Method as defined in LRFD BDS Article 5.9.3.3. These losses assume the specimens were cured with 70% humidity.

$$\Delta f_{pR} = 2.4 \text{ ksi}$$
$$\gamma_h = 1.7 - 0.01RH = 1.7 - 0.01(70) = 1.0$$
$$\gamma_{st} = \frac{5}{1 + f_{ci}^{'}} = \frac{5}{1 + 5.6} = 0.76$$
$$\Delta f_{p\Delta T} = \frac{10.0 f_{pj} A_{ps} \gamma_h \gamma_{st}}{A_c} + 12\gamma_h \gamma_{st} + \Delta f_{pR}$$
$$= \frac{10.0(202.5)(0.705)(1.0)(0.76)}{174} + 12(1.0)(0.76) + 2.4 = 17.76 \text{ ksi}$$

$$f_{pe} = f_{pi} - \Delta f_{p\Delta T} = 195.68 - 17.76 = 177.92 \text{ ksi}$$
$$\mathrm{P}_e = f_{pi} A_{ps} = 177.92(0.705) = 125.4 \text{ kip}$$

2.3.3.4 Concrete Stresses

Allowable concrete stresses after losses are per LRFD BDS Table 5.9.2.3.2a-1 and Table 5.9.2.3.2b-1. Note compression is represented by negative (-) and tension is represented by positive (+).

$$f_c \geq -0.45 f_c^{'} = -0.45(7.0) = -3.15 \text{ ksi} \quad (\mathrm{P_e}+\mathrm{DL})$$
$$f_c \geq -0.60 f_c^{'} = -0.60(7.0) = -4.20 \text{ ksi} \quad (\mathrm{P_e}+\mathrm{DL}+\mathrm{LL})$$
$$f_t \leq 0.19\sqrt{f_c^{'}} \leq 0.6 = 0.19\sqrt{7.0} \leq 0.6 = 0.503 \text{ ksi}$$

HL-93 stresses:

$$M_{NC} = 26.7 \text{ kip-ft}$$
$$M_C = 1.3 \text{ kip-ft}$$
$$M_{LL} = 30.2 \text{ kip-ft}$$

Top of section stresses:

$$f_{PE+DL} = \frac{-P_e}{A_c} + \frac{P_e e}{S_x} - \frac{M_{NC} + M_C}{S_x} = \frac{-125.4}{174} + \frac{125.4(3.75)}{420.5} - \frac{26.7(12) + 1.3(12)}{420.5} = -0.401 \text{ ksi}$$

$$f_{LL,HL93} = -\frac{M_{LL}}{S_x} = -\frac{(30.2)(12)}{420.5} = -0.862 \text{ ksi}$$

Bottom of section stresses:

$$f_{PE+DL} = \frac{-P_e}{A_c} - \frac{P_e e}{S_x} + \frac{M_{NC} + M_C}{S_x} = \frac{-125.4}{174} - \frac{125.4(3.75)}{420.5} + \frac{26.7(12) + 1.3(12)}{420.5} = -1.040 \text{ ksi}$$

$$f_{LL,HL93} = \frac{M_{LL}}{S_x} = \frac{(30.2)(12)}{420.5} = 0.861 \text{ ksi}$$

EV-2 and EV-3 stresses:

$$M_{LL} = 40.4 \text{ kip-ft}$$

Top of section stresses:

$$f_{LL,EV} = -\frac{M_{LL}}{S_x} = -\frac{(40.4)(12)}{420.5} = -1.153 \text{ ksi}$$

Bottom of section stresses:

$$f_{LL,EV} = \frac{M_{LL}}{S_x} = \frac{(40.4)(12)}{420.5} = 1.153 \text{ ksi}$$

Permit truck stresses:

$$M_{LL} = 48.0 \text{ kip-ft}$$

Top of section stresses:

$$f_{LL,Permit} = -\frac{M_{LL}}{S_x} = -\frac{(48.0)(12)}{420.5} = -1.370 \text{ksi}$$

Bottom of section stresses:

$$f_{LL,Permit} = \frac{M_{LL}}{S_x} = \frac{(48.0)(12)}{420.5} = 1.370 \text{ksi}$$

2.3.3.5 Ultimate Flexural Resistance

The ultimate moment strength for one foot strip of the prestressed slab is calculated below conservatively ignoring the additional capacity due to the minimal area of non-prestressed tension (or compression) reinforcement. The following calculations are outlined in LRFD BDS Eq. 5.6.3.1.1-1 and Eq. 5.6.3.1.1-4. The flexural resistance factor, which is based on a tension-controlled section, is calculated in accordance with LRFD BDS 5.6.2.1 & 5.5.4.2.

$$k = 0.28$$ LRFD BDS Table C5.6.3.1.1-1

$$c = \frac{A_{ps} f_{pu}}{0.85 f_c^{'} \beta_1 b + k A_{ps} f_{pu} / d_p} = \frac{0.705(270)}{0.85(7.0)(0.7)12 + 0.28(0.705)270/11.00} = 3.47 \text{ in}$$

$$\varepsilon_s = \varepsilon_c \left(\frac{d_s}{c} - 1 \right) = 0.003 \left(\frac{11.00}{3.47} - 1 \right) = 0.0065 > 0.005 \therefore \phi_f = 1.0$$

$$a = \beta_1 c = 0.7(3.47) = 2.43 \text{ in}$$

$$f_{ps} = f_{pu} \left(1 - \frac{kc}{d_s} \right) = 270 \left(1 - \frac{0.28 \times 3.47}{11.00} \right) = 246.15 \text{ ksi}$$

$$\phi \text{M}_n = \phi_f A_{ps} f_{ps} \left(d_s - \frac{a}{2} \right) = 1.0(0.705)(246.15) \left(11.00 - \frac{2.43}{2} \right) \left(\frac{1 \text{ ft}}{12 \text{ in}} \right) = 142 \text{ kip-ft}$$

The minimum steel per LRFD BDS 5.6.3.3 should be checked to ensure minimum strength is achieved. Since the composite and non-composite section is the same, the second term within the brackets becomes zero and can therefore be neglected.

$$f_{cps} = \frac{-P_e}{A_c} - \frac{P_e e}{S_x} = \frac{-125.4}{174} - \frac{125.4(3.75)}{420.5} = -1.84 \text{ ksi}$$

$$M_{cr} = \gamma_3 \left[\left(\gamma_1 f_r + \gamma_2 f_{cps} \right) S_x - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$$

$$= 1.0 \left[\left(1.6 \times 0.635 + 1.1 \times 1.84 \right) 420.5 \right] \left(\frac{1 \text{ ft}}{12 \text{ in}} \right) = 107 \text{ kip-ft}$$

$$\phi M_n > M_{cr} \quad \therefore \text{OK}$$

2.3.3.6 Shear Resistance

Critical section for shear is d_v from the face of the support in accordance with LRFD BDS 5.7.3.2.

$$d_v = \max\begin{Bmatrix} 0.72h \\ 0.9d \\ d - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(14.50) = 10.44 \\ 0.9(11.00) = 9.90 \\ 11.00 - 2.43/2 = 9.79 \end{Bmatrix} = 10.44 \text{ inches}$$

LRFD BDS 5.7.2.8

Applied shears and moments at the section of interest located a distance d_v from the face of the support are used to determine the nominal shear resistance. It was revealed that the shear resistance is significantly larger than any of the live load shears; therefore, for simplicity and conservatism, the maximum shear resistance will be based on the maximum shear force due to the critical live load of HL-93 Inventory. The maximum shear force and moment is:

$$M_{u-\max} = 87.4 \text{ kip-ft}$$
$$V_{u-\max} = 11.3 \text{ kip}$$

The shear resistance of the slab needs to be evaluated using LRFD BDS 5.7.3.3. Shear factors β and θ need to be calculated per LRFD BDS Article 5.7.3.4.2. #4 shear reinforcement is provided on 12-inch centers along the span and 16.5-inch centers in the direction of traffic and therefore can be included in the shear resistance.

$$M_u \geq V_u d_v \Rightarrow 87.4(12) = 1048.8 \text{ kip-inch} \geq 11.3(10.44) = 118.0 \text{ kip-inch}$$

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u| - A_{ps} f_{po}\right)}{E_{ps}A_{ps}} = \frac{\left(\frac{|1048.8|}{10.44} + 0.5(0) + 11.3 - 0.705(189)\right)}{0.705(28500)} = -0.0011 < 0.000$$

$$0 \leq \varepsilon_s \leq 0.006$$

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500(0.0000) = 29.00^o$$

$$\beta = \frac{4.8}{(1+750\varepsilon_s)} = \frac{4.8}{[1+750(0.0000)]} = 4.80$$

$$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v = 0.0316(4.80)\sqrt{7.0}\,12(10.44) = 50.28 \text{ kip}$$

$$V_s = \frac{A_v f_y d_v \cot(29°)}{s} = \frac{\left(0.196\frac{12}{16.5}\right)(60)(10.44)\cot(29°)}{12} = 10.96 \text{ kip}$$

$$\phi V_n = \phi_v[V_c + V_s] = 0.90[50.28 + 10.96] = 55.12 \text{ kip}$$

The following LRFD BDS checks have not been performed for this example but should be checked for actual rating calculations: regions requiring transverse reinforcement (5.7.2.3); detailing of transverse reinforcement (5.7.2.4); minimum shear reinforcement (5.7.2.5); and longitudinal reinforcement checks (5.7.3.5).

2.3.4 LRFR Rating Calculations

The structural condition of the prestressed 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

Resistance factors are based on LRFD BDS Section 5 and specified in the calculations of Section 2.3.3 of this document.

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.

$$RF = \frac{C \pm \gamma_{DC} DC \pm \gamma_{DW} DW \pm \gamma_{EV} EV \pm \gamma_{EH} EH \pm \gamma_{ES} ES}{\gamma_{LL}(LL + IM)}$$ 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 28 shows the results of the overall capacity based on MBE Equation 6A.4.2.1-2 as well as the load rating factors based on MBE Equation 6A.4.2.1-1. Maximum and minimum load factors are selectively used to obtain the maximum absolute value of the force effects. The load factors for strength limit states are as shown in Table 27. In Table 28, the load effects are unfactored having a unit of kip-ft or kip for moment and shear, respectively.

Table 27. Load Factors for Strength Limit States.

Max/Min	DC	AP	HL-93 Inventory Level	HL-93 Operating Level	EV Legal	Permit
Max	1.25	1	1.75	1.35	1.3	1.3
Min	0.9	N/A	N/A	N/A	N/A	N/A

Table 28. Slab Resistance Rating Factors for Strength.

Force	Rating	DC	AP	LL	C	RF
Moment	HL-93 Inventory	26.71	1.25	30.16	142	2.03
Moment	HL-93 Operating	26.71	1.25	30.16	142	2.64
Moment	EV Legal	26.71	1.25	40.44	142	2.04
Moment	Permit	26.71	1.25	48.04	142	1.72
Shear	HL-93 Inventory	3.45	0.16	3.89	55.12	7.44
Shear	HL-93 Operating	3.45	0.16	3.89	55.12	9.64
Shear	EV Legal	3.45	0.16	5.22	55.12	7.47
Shear	Permit	3.45	0.16	6.20	55.12	6.29

For prestressed concrete, rating for stresses at Service limit states is optional per MBE Table 6A.4.2.2-1 at the legal load level (EV vehicle at Service-III limit state) and permit load level (permit truck at Service-I limit state) but may be included upon the owners request. However, Service-III limit state ratings for the design load (HL-93) at the Inventory level are applicable for prestressed concrete. Per MBE Table 6A.4.2.2-1, the dead load factor (DC and AP) is 1.00 and the live load factor is 0.80. Per LRFD BDS 5.9.2.3.2, tensile stresses in prestressed slabs apply to the Service-III limit state (LRFD BDS 5.9.2.3.2b). Using the factored stresses and stress limits calculated in Section 2.3.3.4, compute the HL-93 Inventory level rating factor for tensile stress on the bottom face of the slab:

$$RF_{bot} = \frac{f_t - 1.0\left(f_{PE+DL}\right)}{0.80\left(f_{LL}\right)} = \frac{0.503 - 1.0(-1.040)}{0.80\left(0.861\right)} = 2.24$$

2.4 TUNNEL LINER

This example considers a circular bored tunnel with a precast segmental reinforced concrete lining. An earth pressure balance tunnel boring machine (EPB TBM) was used to excavate the tunnel and install the lining. The internal diameter is 52 feet and the liner segment thickness is 2 feet. A typical cross-section of the tunnel is presented in Figure 10.

For the purpose of performing a load rating of the liner, the crown of the tunnel is assumed to be only 20 feet below the at-grade roadway. In reality, such a large diameter tunnel in shallow soft ground is unlikely.

Source: FHWA

Figure 10. Illustration. Tunnel Cross-Section.

Underground conditions were modeled after the north portal area of the tunnel, where the tunnel is relatively shallow and is not fully submerged. A submerged tunnel at such shallow depths would experience instability due to buoyancy. As illustrated in Figure 11, soils were simplified into two representative layers including Recent Granular Deposits (RGD) above the tunnel and glacial Till-Like Deposits (TLD) surrounding and beneath the tunnel. The yellow color represents the RGD (upper 15 feet) and the bluish color represents the TLD (below 15 feet).

In this analysis, the only structural component subject to load rating analysis is the tunnel liner. Structure demands used in the example calculations are assumed to come from: self-weight of the structure or dead load component (DC); vertical earth loads (EV); horizontal earth loads (EH); at-grade building surcharge (BS); and live loads (LL).

Two-Dimensional (2D) Finite Element (FE) analysis is used to determine structural demands in the form of moment, shear, and axial forces along the tunnel liner. The commercially-available FE software Plaxis 2D (2019) 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 effects of soil-structure-interaction (SSI).

2.4.1 Materials

Concrete and section properties are listed below. In order to include the liner as a linear elastic plate element in the FE model, an effective moment of inertia was assumed (FHWA, 2009). The effective moment of inertia is significantly lower than the gross value of a 2-foot thick segment because it accounts for the joints in between the precast liner segments. It should be recognized that this effective moment of inertia is only an approximation; the actual structural behavior of a precast segmental liner with bolted connections would be more complex and is beyond the scope of this geotechnical analysis example.

2.4.1.1 Concrete Properties

Specified tunnel liner concrete properties:

Concrete strength: f'_c = 7.0 ksi (1,008 ksf)
Unit weight of concrete: w_c = 0.155 kcf
Design modulus of elasticity: E_{cd} = 5,328 ksi (767,232 ksf)

2.4.1.2 Section Properties

Liner thickness: t = 24 inches (2 feet)
Section area: A = 2 ft^2/ft (per foot of tunnel length)
Gross moment of inertia: I_g = 0.667 ft^4/ft (per foot of tunnel length)
Number of joints: n = 10 (assumed for this example)
Joint moment of inertia: I_j = 0 (no moment resistance is provided by the joints)
Effective moment of inertia: I_{eff} = 0.107 ft^4/ft (per foot of tunnel length)

$$I_{eff} = I_j + I_g\left(4/n\right)^2$$ FHWA-NHI-10-034 Equation 10-11

2.4.1.3 Example Notes

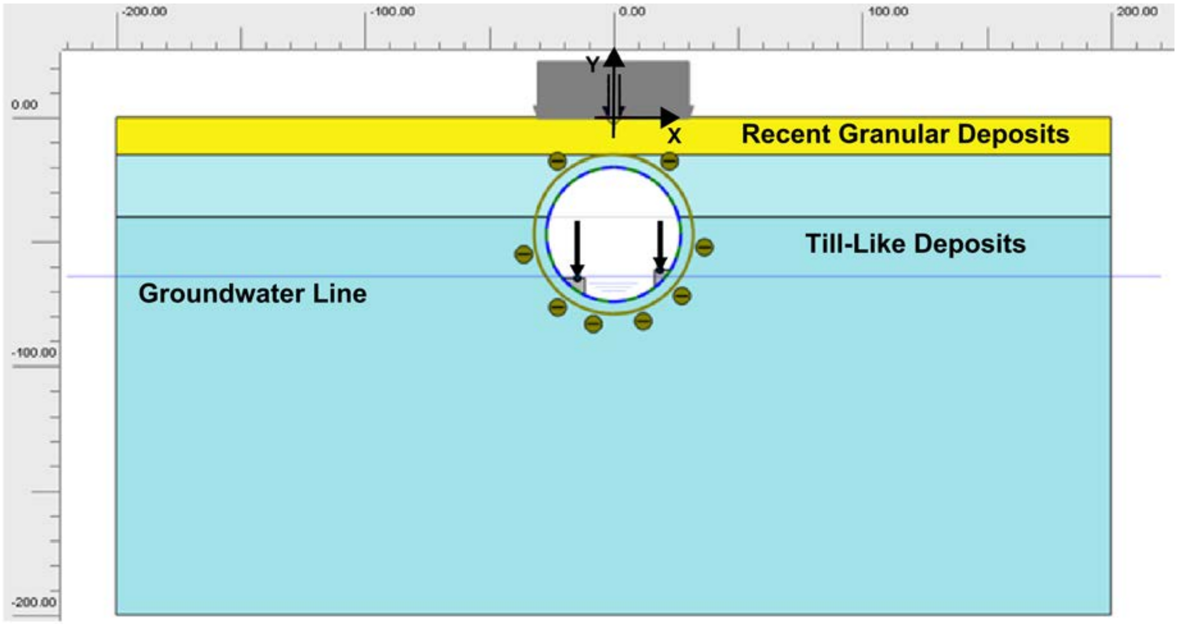
- This cross-section is fictitious and is only for the purpose of demonstrating the principles of using FE models to perform analysis to aid in rating tunnels.
- Live loading on the ground surface over the tunnel may increase force effects and is therefore included in the load rating analysis.
- Dead loads from the roadway and other structures interior to the liner are included in the analysis.
- In general, if the surface roadway passing over the tunnel has an 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 of the 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.
- The liner is modeled in this example with the approximated effective moment of inertia described above.

2.4.1.4 Rating Approach/Assumptions

- The load rating 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.
- Groundwater is assumed to be 10 feet above the tunnel invert, and is included in the FE model accordingly. The location of groundwater is shown in Figure 11.
- In this example, the HL-93 Truck, Tandem, EV-2, and EV-3 live loads are considered. All relevant live loads may be considered in an actual load rating analysis.
- There are no signs of distress or deterioration as the tunnel construction was recently completed; therefore, the tunnel is considered to be in satisfactory physical condition.
- Structural demands are calculated by performing 2D FE analysis including the soil and structure.

2.4.2 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 11, 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. Interface elements are 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 kips.



Source: FHWA

Figure 11. Illustration. 2D Plane Strain FE Model of Tunnel Cross-Section. Length Units are in Feet.

2.4.2.1 Structural Elements

The tunnel liner is modeled with linear-elastic curved plate (i.e., shell) elements (Plaxis, 2019), which are used to model slender structures in the ground with a significant flexural rigidity (*EI*) and normal stiffness (*EA*). 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.

Properties are assigned to the liner based on the structure data listed in Section 2.4.1. The assigned properties are listed in Table 29 and Table 30. 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 is 2 feet thick. The plate elements are modeled along the centerline of the actual tunnel liner elements, with a diameter of 54 feet. The liner weight is based on the actual liner thickness of 2 feet times the unit weight of 0.155 kcf for the high strength reinforced concrete. The tunnel is centered horizontally at an X coordinate of zero feet. The tunnel crown is at a Y coordinate of -20 feet (20 feet below grade), and the invert is as -74 feet (74 feet below grade).

Table 29. Structure Properties.

E (ksf)	**A** (ft^2/ft)	I_{eff} (ft^4/ft)
767,232	2	0.107

Table 30. Plane Strain Model Inputs.

EA (kip/ft)	**EI_{eff}** (kip-ft^2/ft)	**Weight, w** (kip/ft/ft)
1,534,464	82,094	0.31

Notes for Table 29 and Table 30:

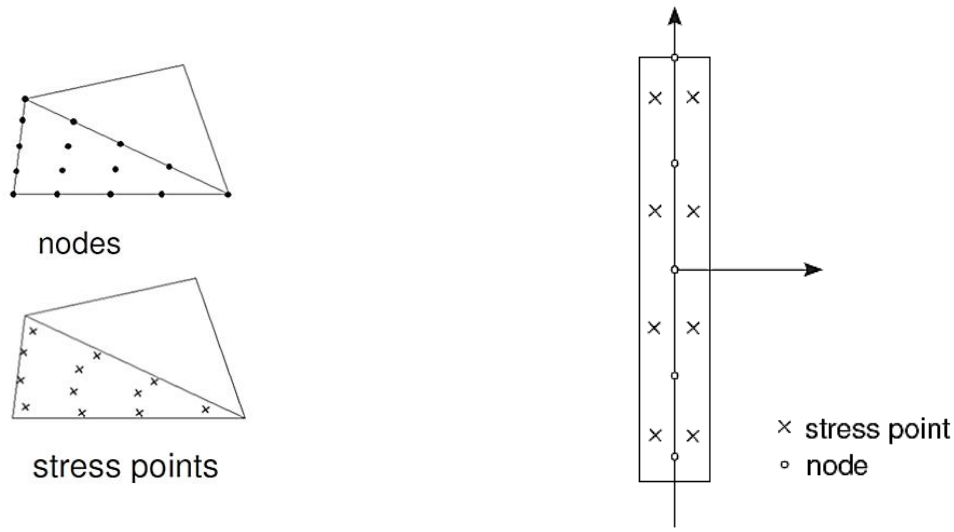
1. E = Young's modulus.
2. A = Area.
3. I_{eff} = Effective moment of inertia.
4. Precast Liner (Plate Element)

2.4.2.2 Soil Domain

The model soil domain surrounding the tunnel (Figure 11) has overall dimensions of 400 feet in width (X coordinates of -200 feet to +200 feet), and 200 feet in depth (Y coordinates of +0 to -200 feet). In soil-structure interaction (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, shear, and axial). Then, the dimensions were increased incrementally and the model was re-run until the forces became insensitive to further increases.

2.4.2.3 FE Mesh

Fifteen-node triangular continuum elements were used to discretize the soil domain and the plate elements have five nodes per adjacent triangular continuum element as shown in Figure 12. Plaxis (2019) generates the mesh automatically based on the settings and adjustments selected by the user. 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 12. Illustrations. Node distribution for (Plaxis 2019): a) 15-node triangular continuum element and b) 5-node plate element.

2.4.2.4 Soil Properties

The soil domain in the FE model is described by constitutive models (stress-strain relationships) and soil properties which are assigned by the user. 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 31) were used in this example for the Recent Granular Deposits (upper 15 feet) and Till-Like Deposits (below 15 feet).

The Mohr-Coulomb (Plaxis, 2019) linear elastic-perfectly plastic model is used to approximate nonlinear soil behavior in this example. Five parameters are typically considered to define the Mohr-Coulomb model 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}). As indicated in Table 31, the value for Young's modulus was defined at the top of the layer (E_{top}) and limited to a maximum value (E_{max}).

The at-rest earth pressure coefficient (K_0) is used to define the initial horizontal stress conditions in the model. In FE analysis of underground structures, K_0 can have a significant impact, because it significantly influences horizontal earth pressures. Construction sequencing may also affect horizontal earth pressures in FE models, which do not always exactly simulate reality. In order to account for variability, uncertainty, and potential effects of construction, a relevant range of K_0 should be considered in the load rating analysis.

The Recent Granular Deposits are modeled after loose to dense sandy soils encountered near the ground surface. These recent deposits are normally consolidated, with K_0 assigned as 1-sin(ϕ) based on the Plaxis (2019) default recommendation. It is noted that the Recent Granular Deposits are only located above the tunnel crown (Figure 11), so the K_0 value in this layer does not have as significant an impact as the Till-Like Deposits surrounding the tunnel.

Glacial till tends to be over-consolidated, strong, and stiff. In this example, the Till-Like Deposits were modeled as primarily coarse grained, with relatively high ϕ values and stiffness parameters. Lower and Upper Estimate K_0 values of 0.5 and 1.0, respectively, were considered in order to cover a typical range of over-consolidated conditions in such materials.

Table 31. Soil Parameters.

Soil Type	**Unit Wt.** (pcf)	**ϕ** (deg)	**c** (ksf)	E_{top} (ksf) [1]	E_{inc} (ksf/ft) [2]	E_{max} (ksf) [3]	**Poisson's Ratio**	K_0 [4]
Recent Granular Deposits *Lower Estimate*	105	30	0.1	500	25	875	0.3	0.50
Recent Granular Deposits *Upper Estimate*	125	36	0.1	1,000	50	1,750	0.3	0.41
Till-Like Deposits *Lower Estimate*	125	36	0.1	1,000	300	8,500	0.35	0.50
Till-Like Deposits *Upper Estimate*	145	42	0.1	2,000	600	17,000	0.35	1.00

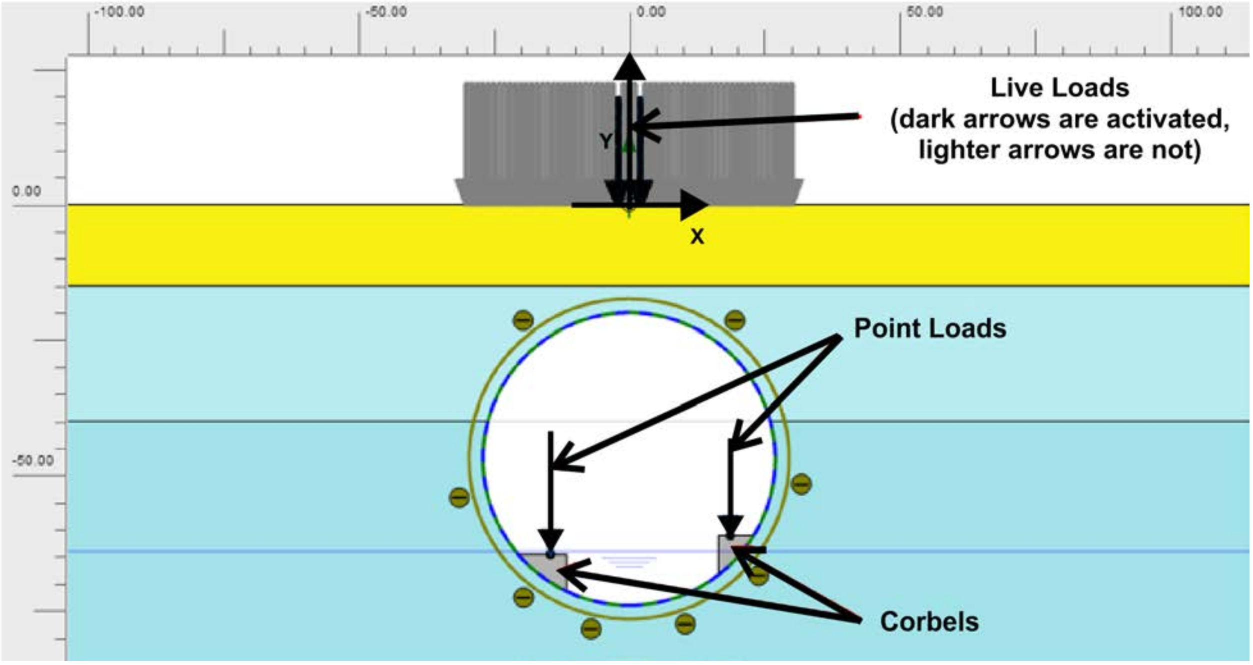
Notes:

1. E_{top} = Young's modulus at top of layer.
2. E_{inc} = Rate of change of stiffness with depth below top of layer.
3. E_{max} = Maximum value for E (no further increase with depth beyond this value).
4. K_0 = At-rest earth pressure coefficient.

2.4.2.5 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. The interface limits the amount of stress that can be transferred between the structure and the soil. Small circles with a minus sign inside them graphically indicate the interface around the perimeter of the tunnel in Figure 11 and in later figures.

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. Dead load reactions from the internal tunnel structures such as the roadway boxes (Figure 10) are applied as downward point loads acting on the corbels, as illustrated in Figure 13. Loads of 15.6 kip/ft (kips per foot model width) and 12.3 kip/ft are applied to the left and right corbels, respectively. The corbels are included as linear elastic continuum elements with concrete stiffness in order to distribute the point loads over their contact area with the tunnel liner.



Source: FHWA

Figure 13. Illustration. Close-Up View of Tunnel in FE Model.

Live loads are applied as downward distributed loads on the surface of the model (Figure 13). Surface traffic is assumed to be moving in the direction from right to left across the tunnel. Based on AASHTO 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 (lighter arrows in Figure 13). 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 (dark arrows) in Figure 13 represent HL-93 Tandem loading, which consists of a pair of wheel loads spaced at 4 feet.

The four live loads included in this example for demonstration purposes are the HL-93 Truck and HL-93 Tandem loads, EV-2, and EV-3. The HL-93 Truck consists of an 8-kip 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. EV-2 consists of a 24-kip load and a 33.5-kip load spaced at 15 feet. EV-3 includes a 24-kip wheel load followed by a 15-ft spacing and a pair of 31-kip loads spaced at 4 feet. Lane loads for the HL-93 Tandem and Truck loads were not applied, refer to Section 2.4.2.5.1 for a discussion.

2.4.2.5.1 Distribution of Wheel Loads through Fill

Axle loads will distribute outward laterally as they pass downward through the fill until they reach the tunnel. This leads to lower magnitude pressures (spread out over a larger area) on the tunnel than at the ground surface. Distribution transverse to the tunnel's longitudinal axis (to the left and right in Figure 13) will occur automatically in the 2D model. However, the 2D plane strain model cannot automatically account for the distribution along the tunnel's longitudinal direction (in and out of the page in Figure 13).

The guidelines in LRFD BDS Article 3.6.1.2.6 were used to account for the distribution along the tunnel's longitudinal direction as follows:

- For traffic running transverse to the tunnel's longitudinal axis (crossing over the tunnel), and wheel load distribution along the tunnel's longitudinal direction,

  $$H_{int\text{-}t} = \frac{s_w - \frac{w_t}{12} - \frac{0.06D_i}{12}}{LLDF} = \frac{6 - \frac{20}{12} - \frac{0.06(648)}{12}}{1.15} = 0.95 \text{ feet} \qquad \text{LRFD BDS Eq. 3.6.1.2.6b-1}$$

  Where:

s_w	= 6 feet	
w_t	= 20 inches	
D_i	= 54 feet (648 inches)	
LLDF	= 1.15	LRFD BDS Table 3.6.1.2.6a-1

- Conservatively assume H = 20 feet above the tunnel (H = 20 feet at the crown and H > 20 feet away from the crown).

- Since $H > H_{int\text{-}t}$:

  $$\begin{aligned} w_w &= \frac{w_t}{12} + s_w + LLDF\left(H\right) + 0.06\frac{D_i}{12} \\ &= \frac{20}{12} + 6 + 1.15(20) + 0.06\frac{648}{12} = 33.9 \text{ feet} \end{aligned} \qquad \text{LRFD BDS Eq. 3.6.1.2.6b-3}$$

- The length of the distributed axle 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:
  - Each axle load for HL-93 Tandem: 25 kips / 33.9 feet / 0.84 feet = 0.88 ksf
  - Front axle load for HL-93 Truck: 8 kips / 33.9 feet / 0.84 feet = 0.28 ksf
  - Middle and rear axle loads for HL-93 Truck: 32 kips / 33.9 feet / 0.84 feet = 1.12 ksf
  - Front axle load for EV-2 and EV-3: 24 kips / 33.9 feet / 0.84 feet = 0.84 ksf
  - Rear axle load for EV-2: 33.5 kips / 33.9 feet / 0.84 feet = 1.18 ksf
  - Middle and rear axle loads for EV-3: 31 kips / 33.9 feet / 0.84 feet = 1.09 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 13, the distributed loads centered at X-coordinates -2 feet and +2 feet were activated and assigned a magnitude of 0.88 ksf. All other distributed loads were left unactivated.

As mentioned previously, lane loading was not applied with the HL-93 vehicles. LRFD BDS Article 3.6.1.2.6a discusses the use of axle loads only for buried structures. However, for larger tunnel structures, lane loading should be considered unless it is negligible. In the case of this example, comparing the pressure due to the lane loading, p_{land}, at the top of the tunnel (under 20 feet of fill) with the soil pressure at that depth, p_{soil}, shows it is negligible (less than 1% of soil load):

$$p_{lane} = \frac{0.640 \text{ klf}}{\mathrm{w}_{land} + LLDF\left(H\right)} = \frac{0.640}{10 + 1.15(20)} = 0.02 \text{ ksf}$$

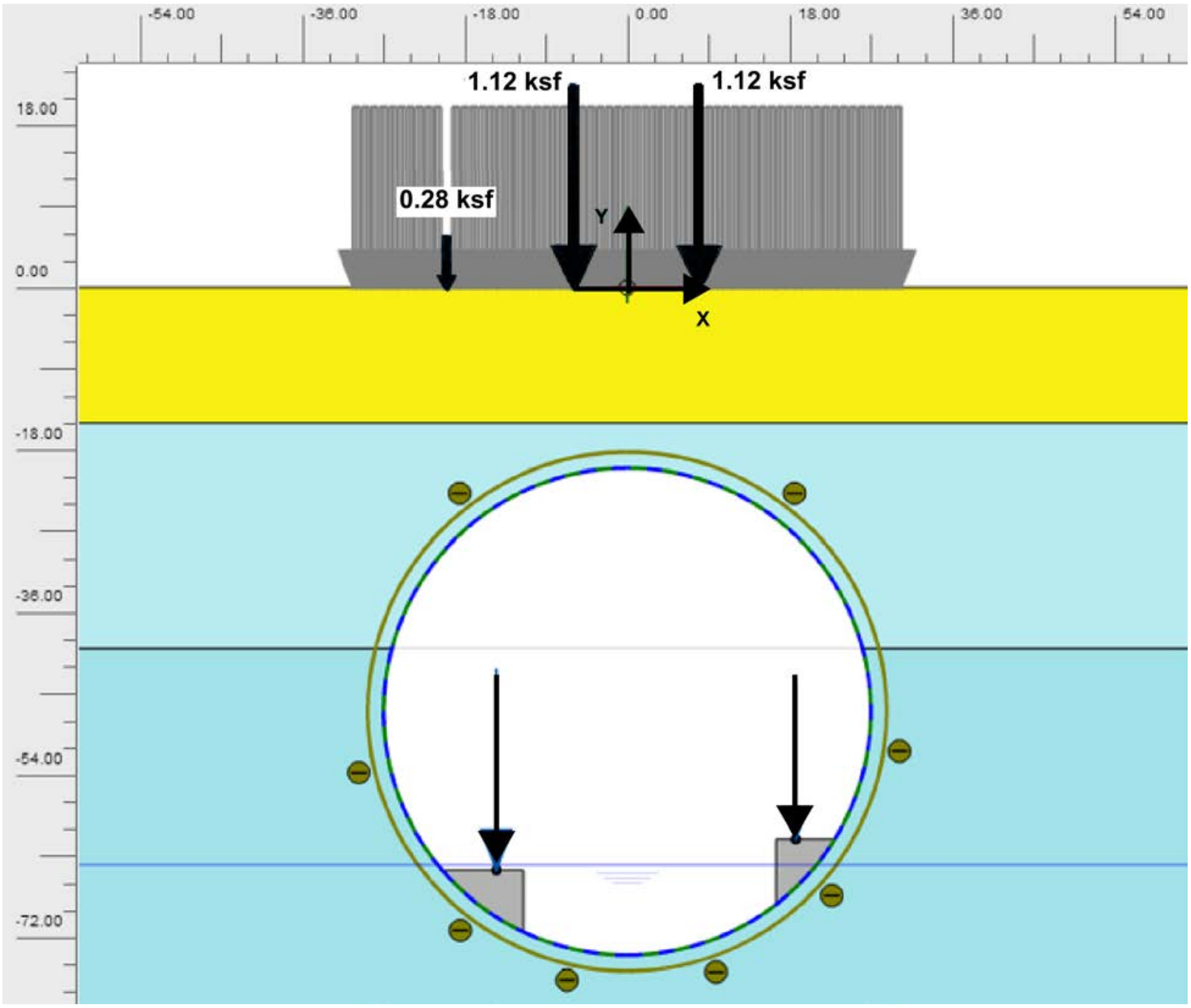
$$p_{soil} = \gamma_{soil} H = (0.125 \text{ kcf})(20) = 2.50 \text{ ksf } ? \quad p_{lane} = 0.02 \text{ ksf}$$

Per LRFD BDS Article 3.6.2.2, the dynamic load allowance at depth of 20 feet is 0% (no increase in wheel loads). Additionally, per LRFD BDS Article 3.6.1.2.6a, the single lane multiple presence factor of 1.20 should be applied to traffic parallel to the span (perpendicular to longitudinal direction of tunnel). The 1.20 factor is applied to the FE results prior to computing load factors.

2.4.2.5.2 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 more appropriate.

A relatively simple, but time consuming method is to consider all relevant vehicle locations one at a time in separate FE analyses. Figure 14 presents an example for the HL-93 Truck at one specific location. The three activated loads (dark arrows) 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 14. 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 Example 4 of the Reference Guide for Load Rating of Tunnel Structures (FHWA, 2019) using the Dynamics feature of Plaxis (2019). Moving load analyses were performed to determine the location of each live load that results in the maximum moment in the tunnel liner (at the crown) for this example.

2.4.2.6 Load Factors

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 unfactored permanent and live loads. Load factors should be applied to the resulting demands later in the load rating procedure.

2.4.2.7 Volume Loss

During earth pressure balanced tunnel boring machine (EPB TBM) tunnel construction, there is typically a small volume of soil that is over-excavated in excess of the theoretical volume of the excavation, leading to ground or volume loss. This volume loss can lead to ground settlements at the surface and can also impact the tunnel liner forces. Volume loss is typically lumped into three groups (FHWA, 2009): face loss, shield loss, and tail loss. Face loss consists of ground movements into the shield face. Shield loss occurs between the cutting edge and the shield tail, due in part to the small amount of overcut that occurs for maneuverability. Tail loss occurs behind the tail as the ground support mechanism moves to the liner itself and the grout that fills the annular space between the cut and the liner. In the tunneling industry, the volume loss (i.e., amount of over-excavation) is typically expressed as a percentage of the tunnel size.

EPB TBMs are able to minimize the magnitude of volume loss by applying pressure to the tunnel face, minimizing the overcut, and grouting the tail void (FHWA, 2009). However, the volume loss is not likely to be eliminated entirely. Ultimately, the amount of volume loss is dependent on the quality of workmanship during construction. As a result, it is good practice to consider a range of volume loss in the load rating analysis in order to address this uncertainty. However, if ground loss levels were measured and recorded during the construction of a tunnel, a more precise value can be used for the load rating.

Modern EPB TBM techniques have significantly reduced the ground loss on recent projects. For example, the LA Metro Regional Connector, Sao Paulo Metro Line 4, Heathrow Airside Road Tunnel, and Madrid South Bypass M-30 Tunnels have reported ground loss in the range of 0.1% to 0.4% for tunnel diameters of approximately 20 to 50 feet (Metro, 2019).

Volume loss is applied in this example using the line contraction feature in Plaxis (2019). The circumference of the liner plate element is reduced to achieve the specified volume loss. As such, it is a simplification of the actual volume loss mechanism described above. In this example, two volume loss scenarios are considered as lower and upper bounds in order to address uncertainties and numerical simplifications: zero volume loss; and 0.5% volume loss.

2.4.2.8 Sensitivity Scenarios

FE analyses were performed for the following scenarios to address uncertainties discussed in the previous sections:

- LE soil properties with no volume loss.
- LE soil properties with 0.5% volume loss.
- UE soil properties with no volume loss.
- UE soil properties with 0.5% volume loss.

The load rating analysis for this example was based on the highest structure demands from these four scenarios.

2.4.3 FE Analysis Procedure

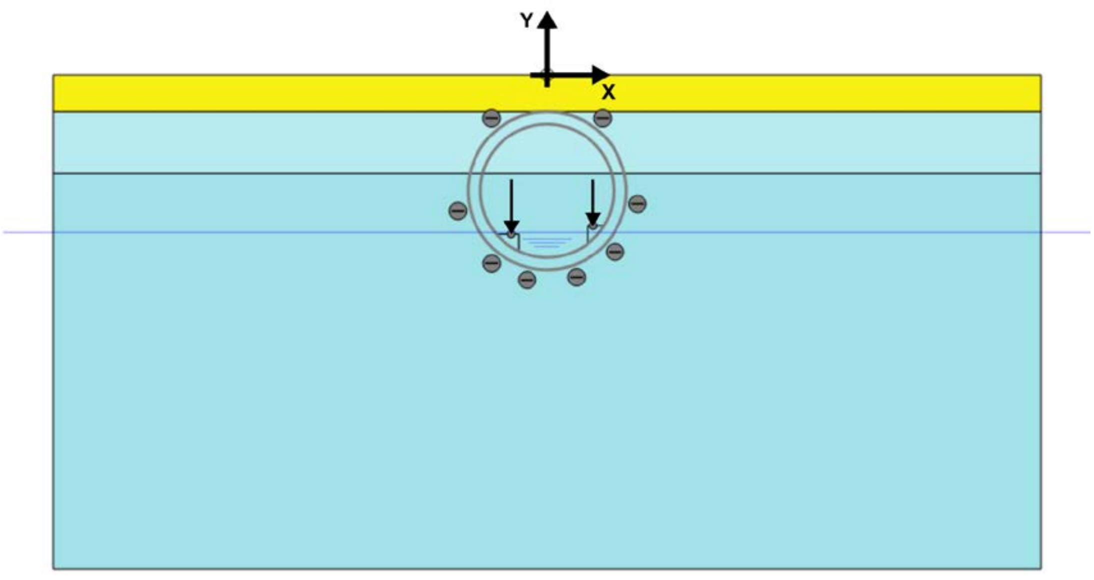
The FE analysis was performed in phases in order to consider a realistic construction sequence and to separately evaluate demands under permanent loads only as well as permanent plus live loads. An initial phase was performed with level ground conditions (without the tunnel present) to generate stresses in the ground prior to the start of construction, as illustrated in Figure 15. 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 following staged construction phases were performed next:

Tunnel Construction (Figure 16): Excavate tunnel, activate liner and interface, and dewater inside tunnel by setting soil clusters inside tunnel to dry.

Volume Loss (not shown): Activate the line contraction to apply the volume loss (for scenarios that include non-zero volume loss).

Interior Dead Loads (Figure 17): Activate corbels and point loads representing the dead load from the structures inside the tunnel lining. All unfactored permanent loads (DC, EV, and EH) are accounted for at the end of this phase, with the exception of the building surcharge which is discussed in Section 2.4.4.1.2.

Live Loads (Figure 13 and Figure 14): Activate vehicle live load. All unfactored permanent loads and live loads (DC, EV, EH, and LL) are accounted for at the end of this phase, with the exception of the building surcharge which is discussed in Section 2.4.4.1.2.



Source: FHWA

Figure 15. Illustration. Generate Stresses Under Original Level Ground Condition.

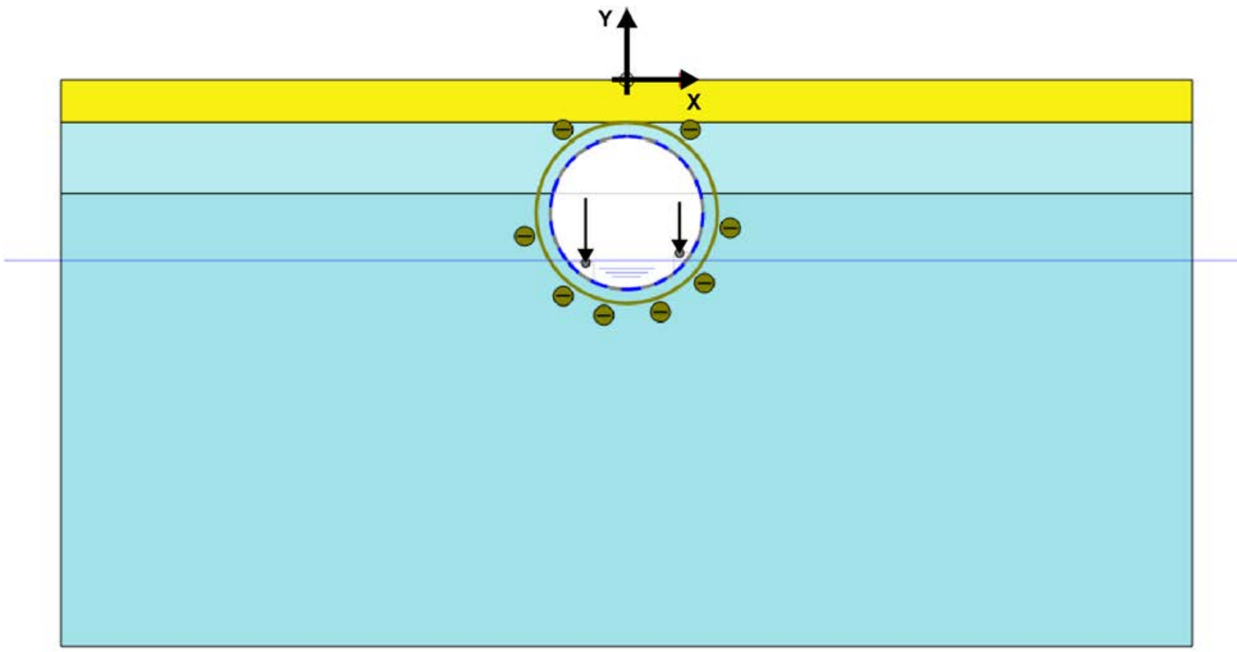


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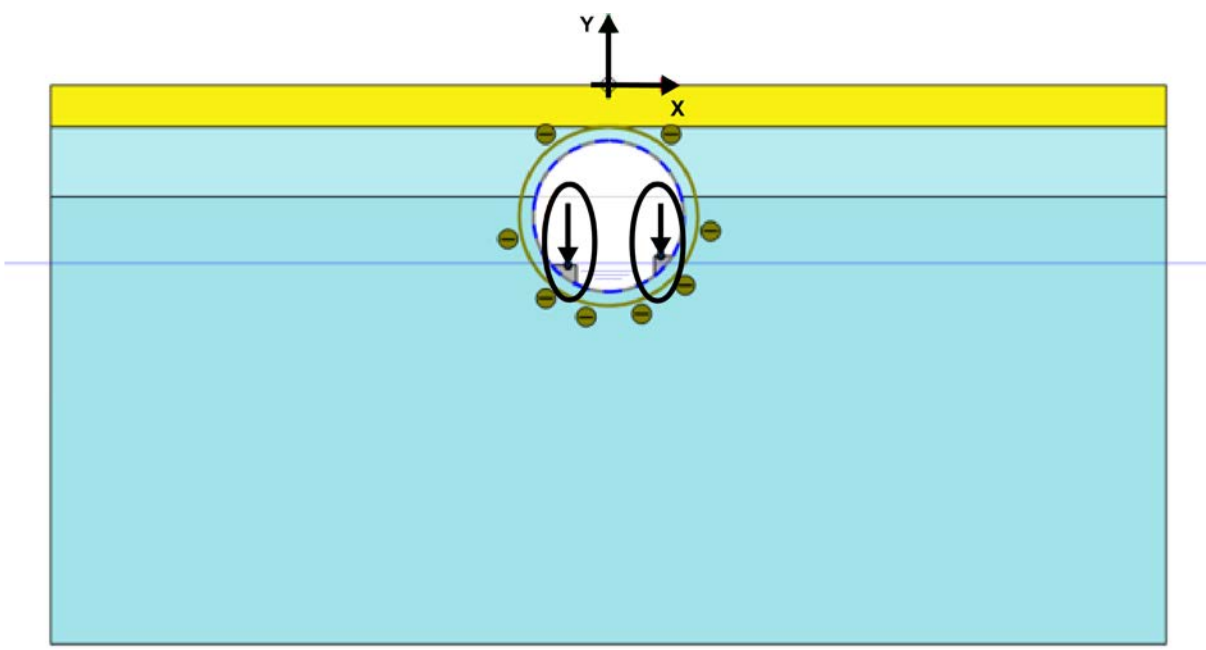
Figure 16. Illustration. Excavate Tunnel, Activate Liner and Interface, Dewater Inside.



Source: FHWA

Figure 17. Illustration. Activate Corbels and Apply Interior Structure Dead Loads.

All of the analysis phases listed above were performed for each of the four sensitivity scenarios described in Section 2.4.2.8.

2.4.4 FE Analysis Results

Results are presented in the following sections for use in the load rating analysis in the form of moment, shear, and axial force plots along the liner. The results are plotted as a function of the angle from the horizontal, with zero degrees being at the right-most point on the liner (3 o'clock position), 90 degrees at the crown (12 o'clock position), and 270 degrees at the invert (6 o'clock position), as illustrated in Figure 18. Axial compression loads are negative.

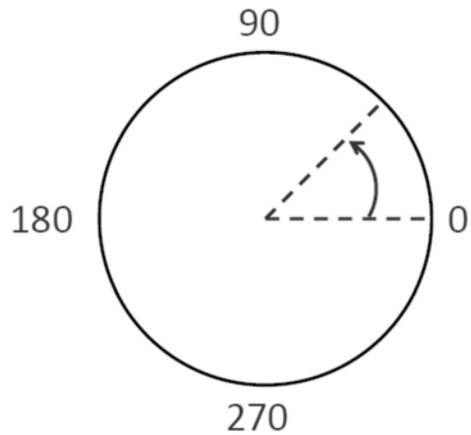


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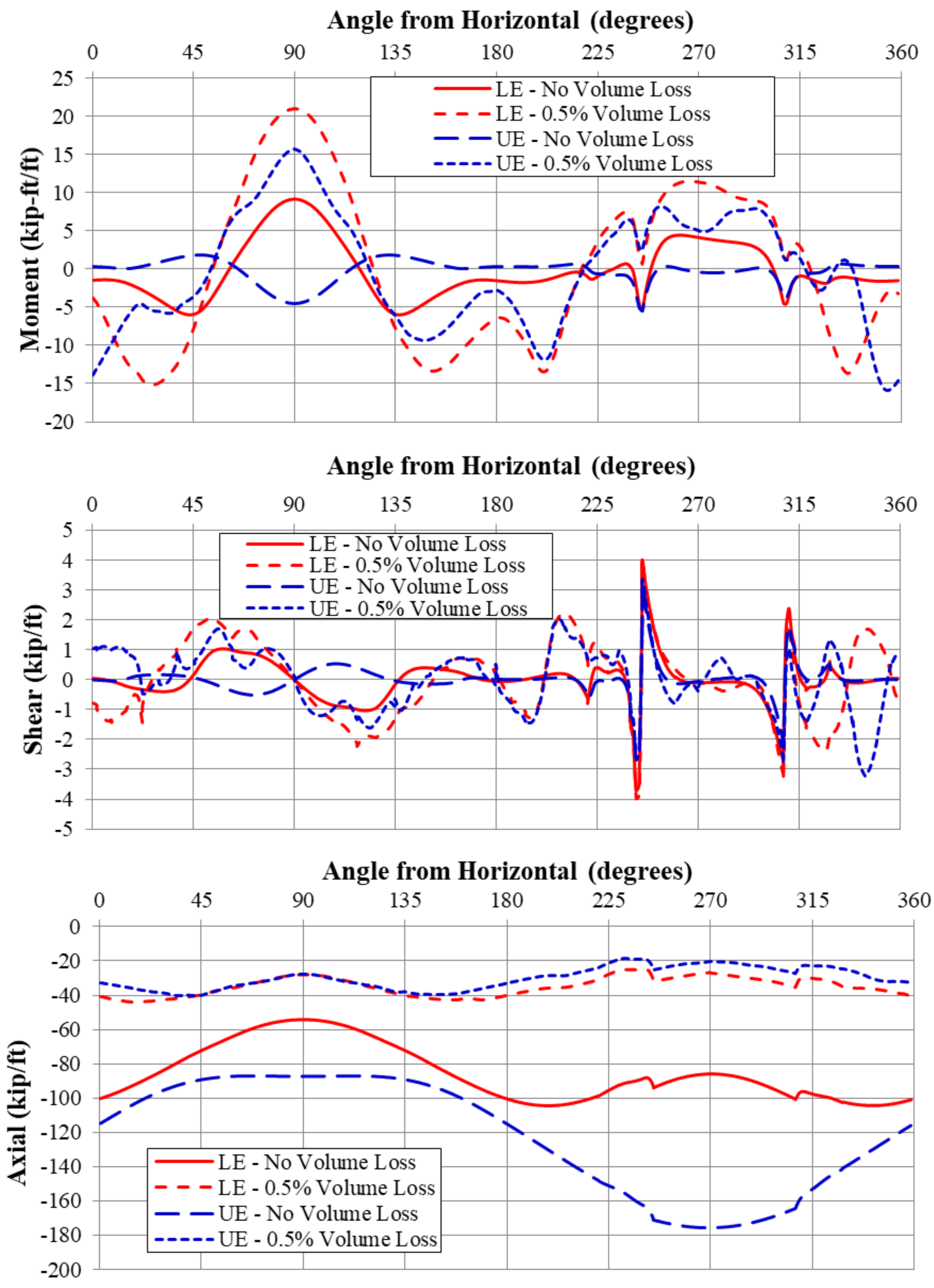
Figure 18. Illustration. Convention Used for Angle from Horizontal to Present the Results.

2.4.4.1 Structure Demands due to Permanent Loads

2.4.4.1.1 Without Building Surcharge

Results of the analysis including all of the permanent loads except for the building surcharge are presented in Figure 19 for each of the four sensitivity scenarios described in Section 2.4.2.8.



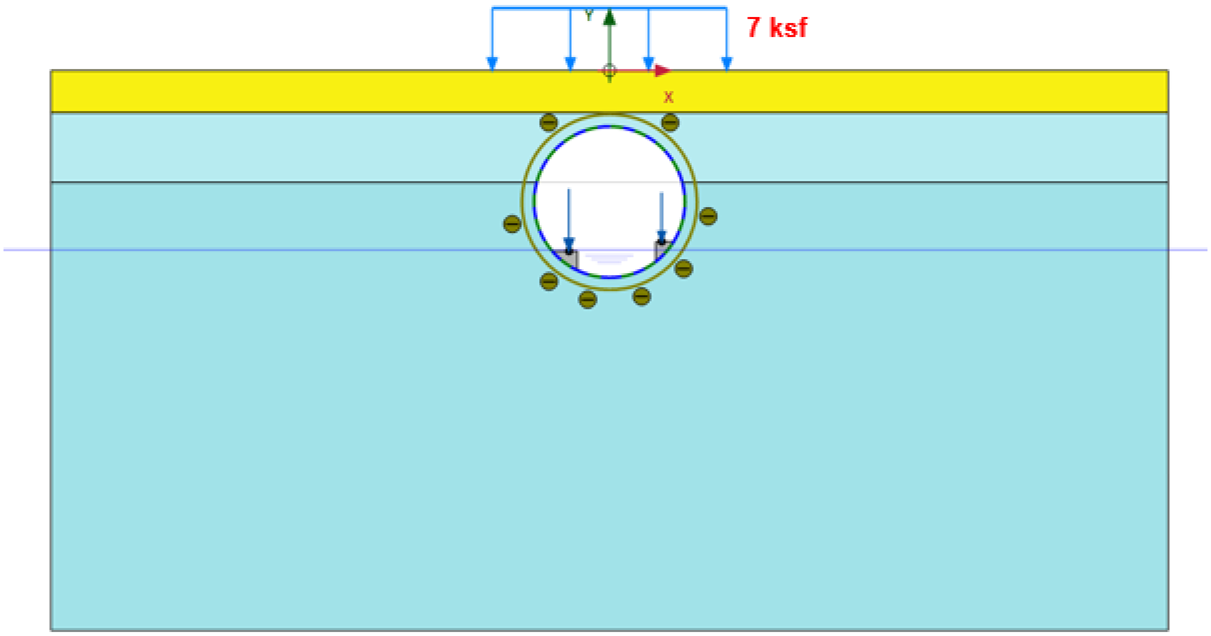
Source: FHWA

Figure 19. Analysis Results. Permanent Loads Without the Building Surcharge.

2.4.4.1.2 With Building Surcharge

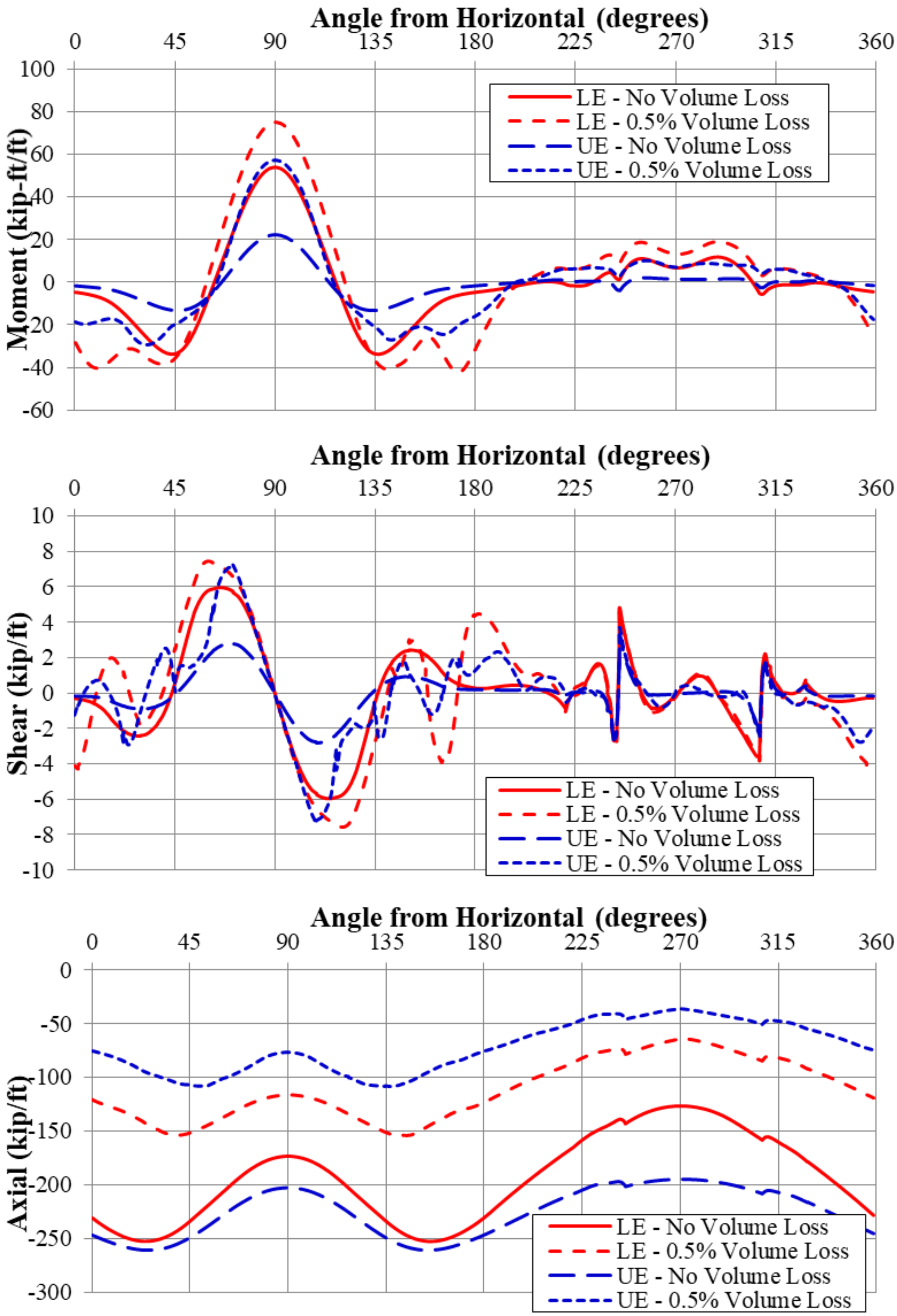
The presence of large buildings at the surface above the tunnel was considered by applying a vertical surcharge of 7 ksf across a width of 84 feet centered about the tunnel centerline, as illustrated in Figure 20. The 7 ksf is a defined building load prescribed within the project parameters. Based on the assumption that the buildings were in place before the tunnel, the building surcharge was activated prior to tunnel excavation and construction. Results of the analysis including all of the permanent loads plus the building surcharge are presented in Figure 21 for each of the four sensitivity scenarios described in Section 2.4.2.8.

It should be noted that for tunnels at this depth below soil, building loads as shown in Figure 20 and live loads over the top are not coincident. As a result, the ratings are not calculated including the building surcharge. At increased depths where adjacent building surcharges would overlap with overhead vehicular traffic, the live load effects would be nonexistent.



Source: FHWA

Figure 20. Illustration. Building Surcharge Included at the Ground Surface.



Source: FHWA

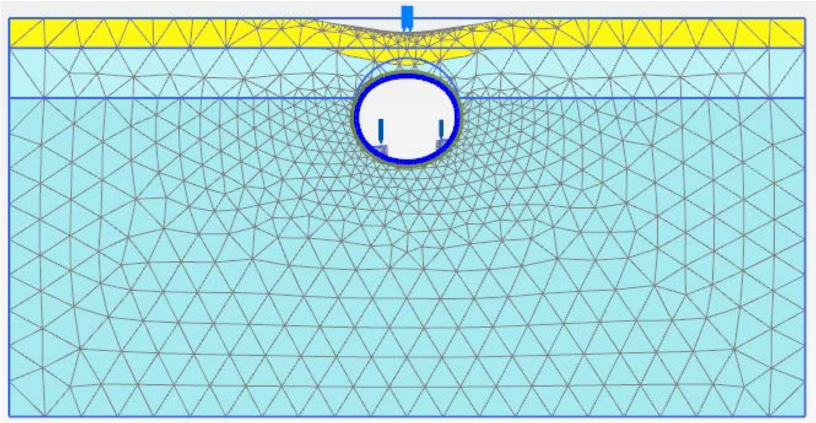
Figure 21. Analysis Results. Permanent Loads Including the 7 ksf Building Surcharge.

2.4.4.2 Structure Demands due to Permanent and Live Loads

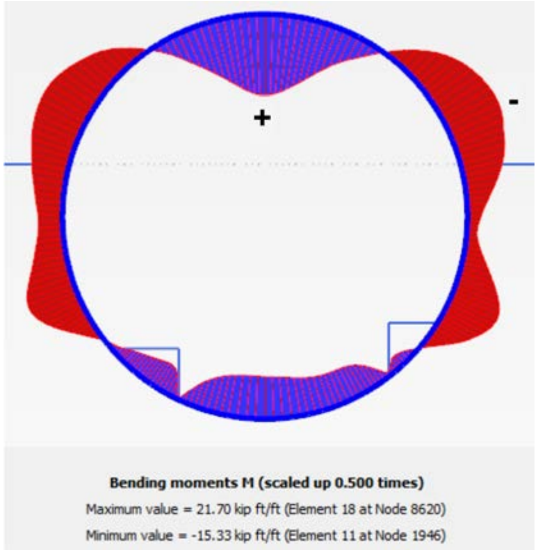
Live load analysis results are presented in this section. The results indicate that the impacts of surface live loads on the tunnel liner are relatively small, and the difference between each vehicle is hardly noticeable. As explained in Section 2.4.2.5.1, the wheel loads are distributed horizontally through the overlying soil before reaching the tunnel, which significantly lessens their impact on the liner as compared with the weight of the soil itself.

2.4.4.2.1 HL-93 Tandem

The deformed mesh and liner moments for the HL-93 Tandem live load plus permanent loads (excluding the building surcharge) are illustrated in Figure 22 for the LE with 0.5% volume loss scenario. Moment, shear, and axial force diagrams are presented in Figure 23 for each of the four sensitivity scenarios described in Section 2.4.2.8.



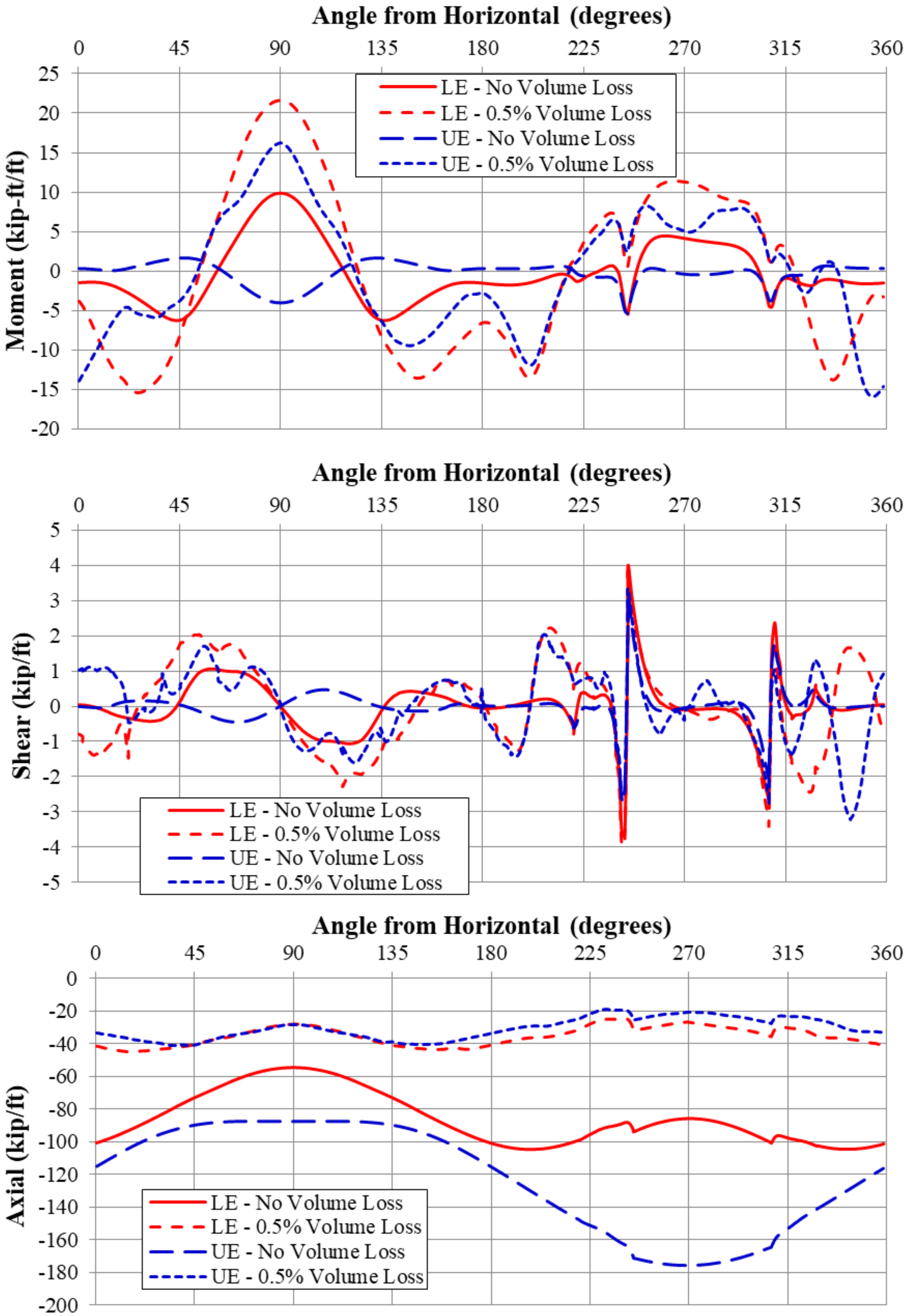
a) Exaggerated (50x) Deformed Mesh (LE, 0.5% Volume Loss)



b) Liner Moments (LE, 0.5% Volume Loss)

Source: FHWA

Figure 22. Analysis Results. Permanent Loads + HL-93 Tandem (No Building Surcharge).

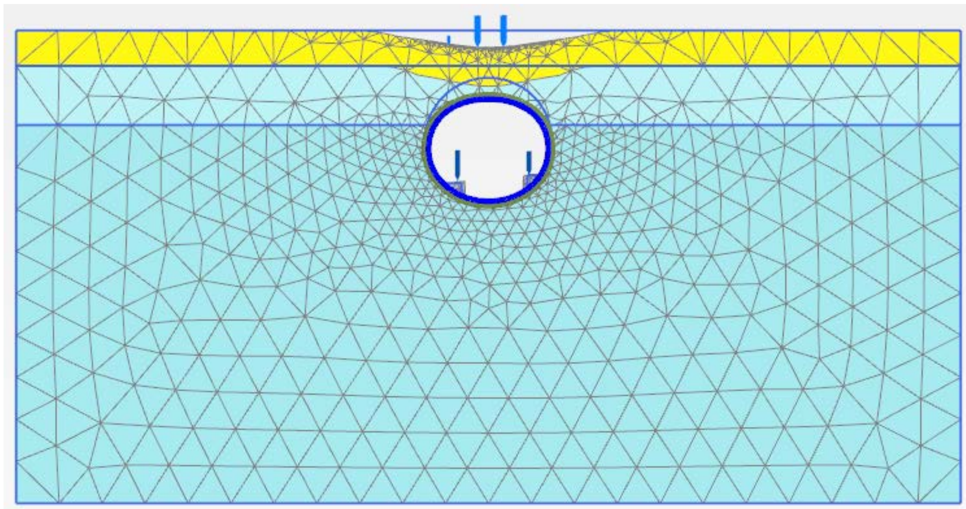


Source: FHWA

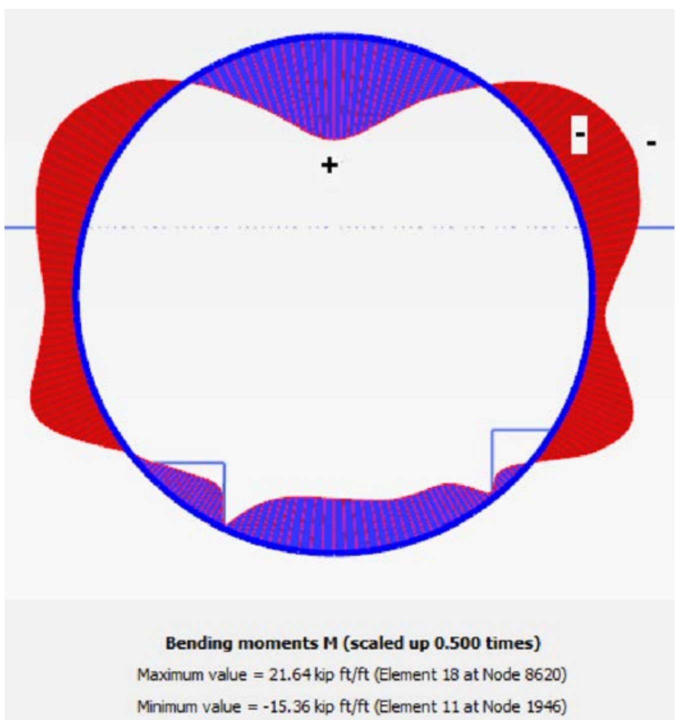
Figure 23. Analysis Results. Permanent Loads + HL-93 Tandem (No Building Surcharge).

2.4.4.2.2 HL-93 Truck

The deformed mesh and liner moments for the HL-93 Truck live load plus permanent loads (excluding the building surcharge) are illustrated in Figure 24 for the LE with 0.5% volume loss scenario. Moment, shear, and axial force diagrams are presented in Figure 25 for each of the four sensitivity scenarios described in Section 2.4.2.8.



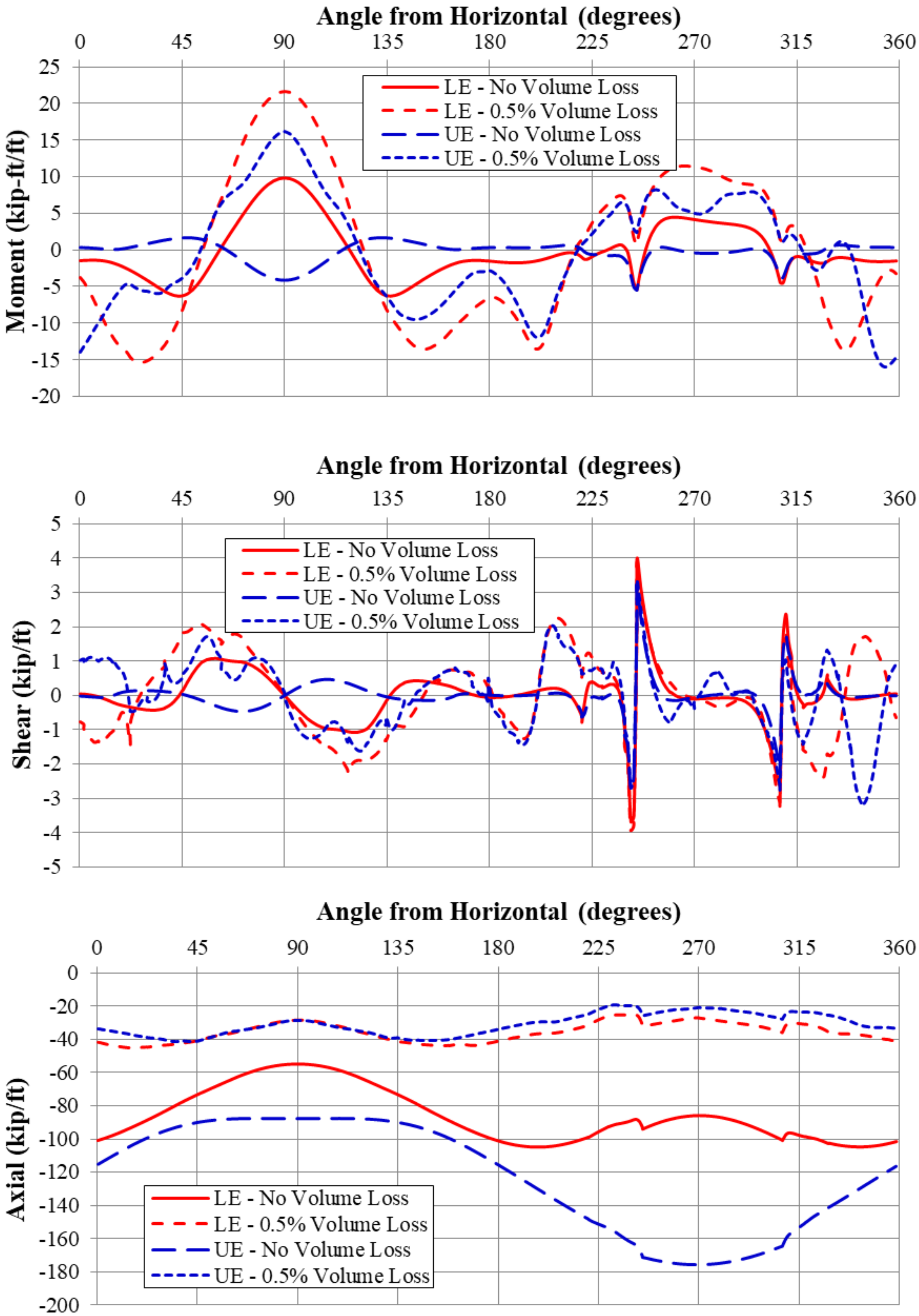
a) Exaggerated (50x) Deformed Mesh (LE, 0.5% Volume Loss)



b) Liner Moments (LE, 0.5% Volume Loss)

Source: FHWA

Figure 24. Analysis Results. Permanent Loads + HL-93 Truck (No Building Surcharge).

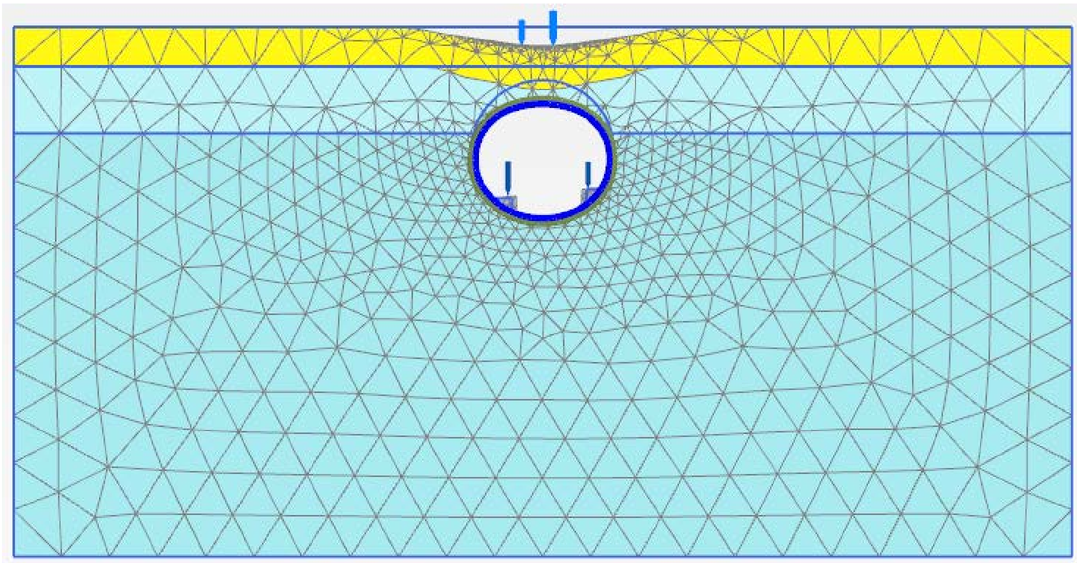


Source: FHWA

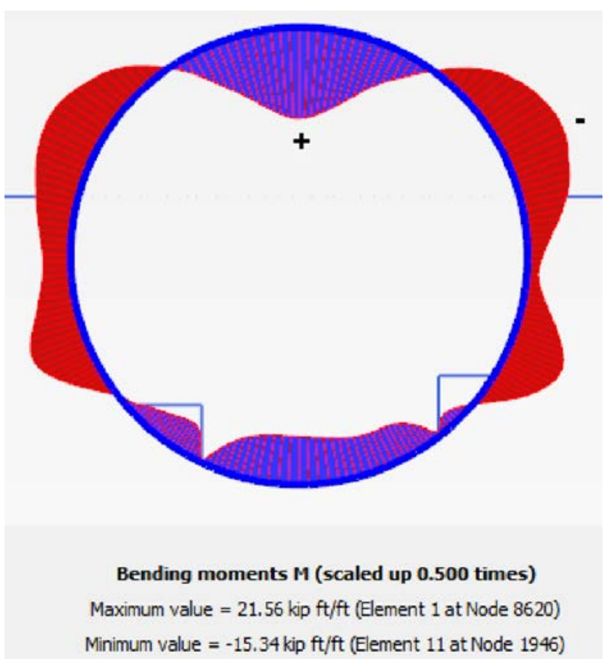
Figure 25. Analysis Results. Permanent Loads + HL-93 Truck (No Building Surcharge).

2.4.4.2.3 EV-2

The deformed mesh and liner moments for the EV-2 live load plus permanent loads (excluding the building surcharge) are illustrated in Figure 26 for the LE with 0.5% volume loss scenario. Moment, shear, and axial force diagrams are presented in Figure 27 for each of the four sensitivity scenarios described in Section 2.4.2.8.



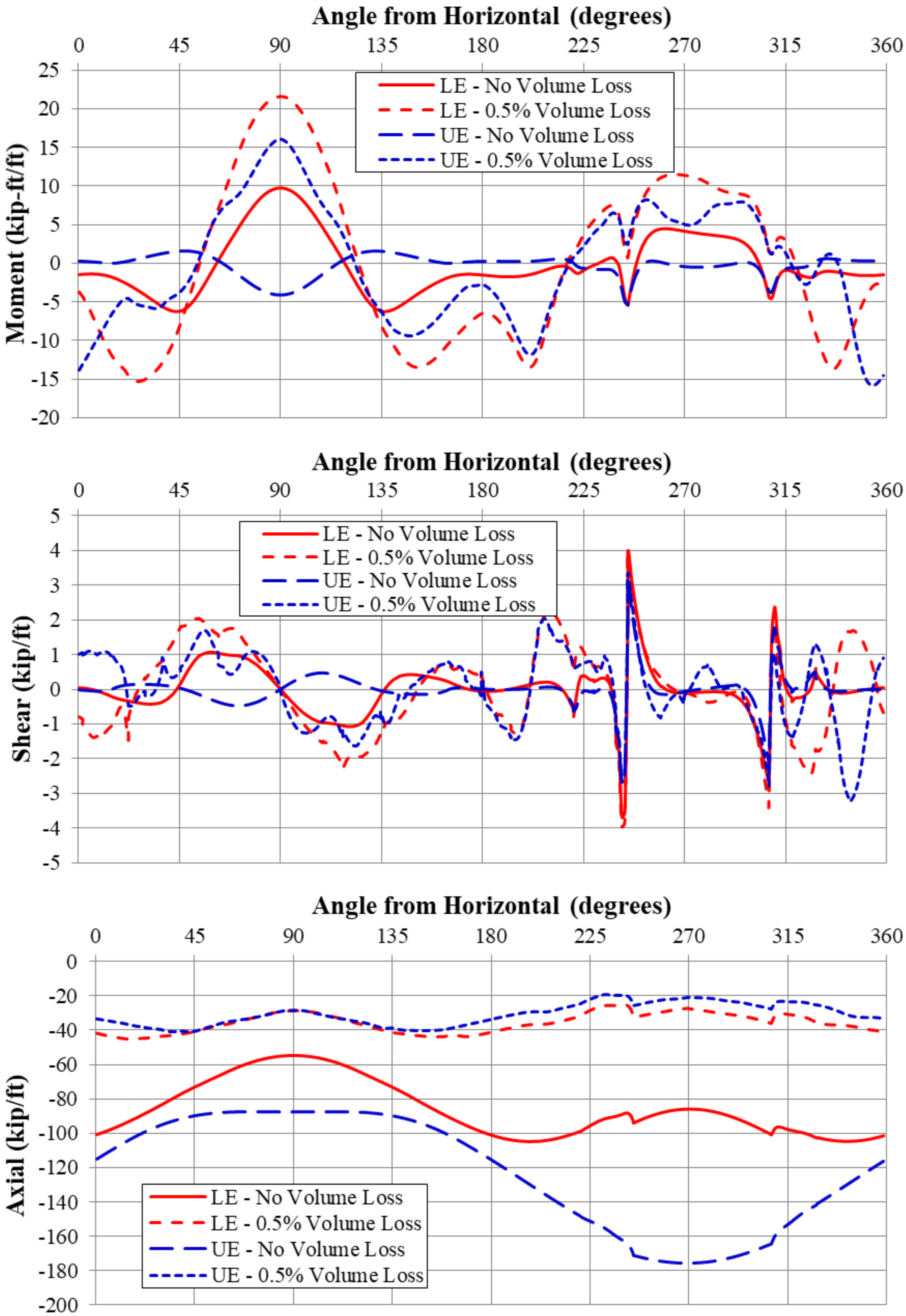
a) Exaggerated (50x) Deformed Mesh (LE, 0.5% Volume Loss)



b) Liner Moments (LE, 0.5% Volume Loss)

Source: FHWA

Figure 26. Analysis Results. Permanent Loads + EV-2 (No Building Surcharge).

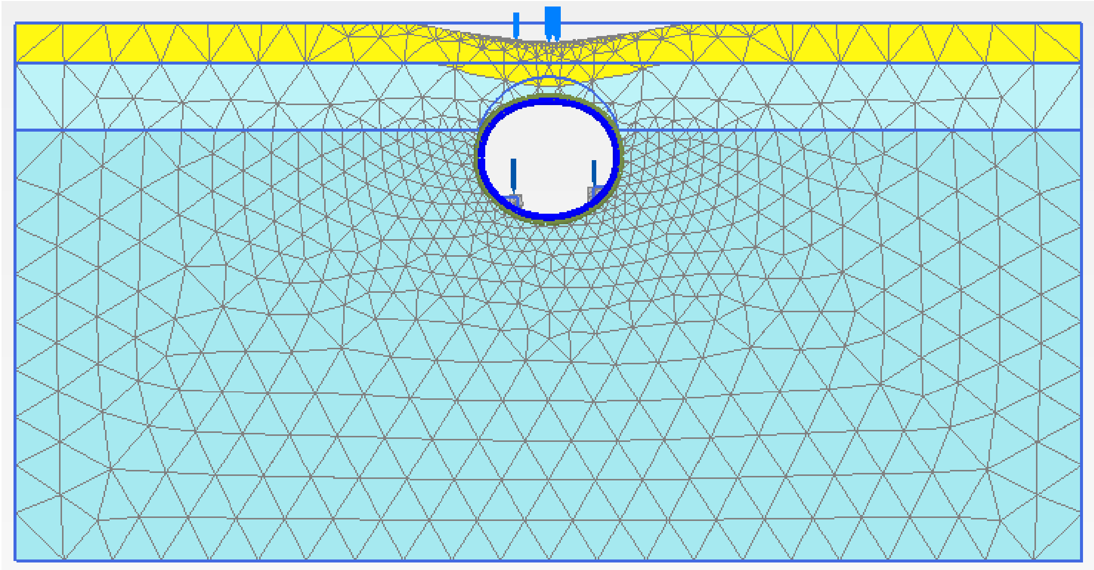


Source: FHWA

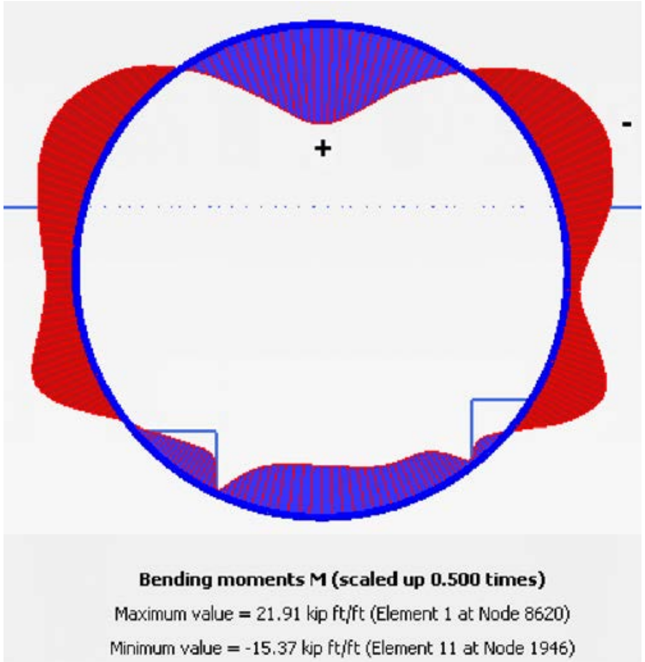
Figure 27. Analysis Results. Permanent Loads + EV-2 (No Building Surcharge).

2.4.4.2.4 EV-3

The deformed mesh and liner moments for the EV-3 live load plus permanent loads (excluding the building surcharge) are illustrated in Figure 28 for the LE with 0.5% volume loss scenario. Moment, shear, and axial force diagrams are presented in Figure 29 for each of the four sensitivity scenarios described in Section 2.4.2.8.



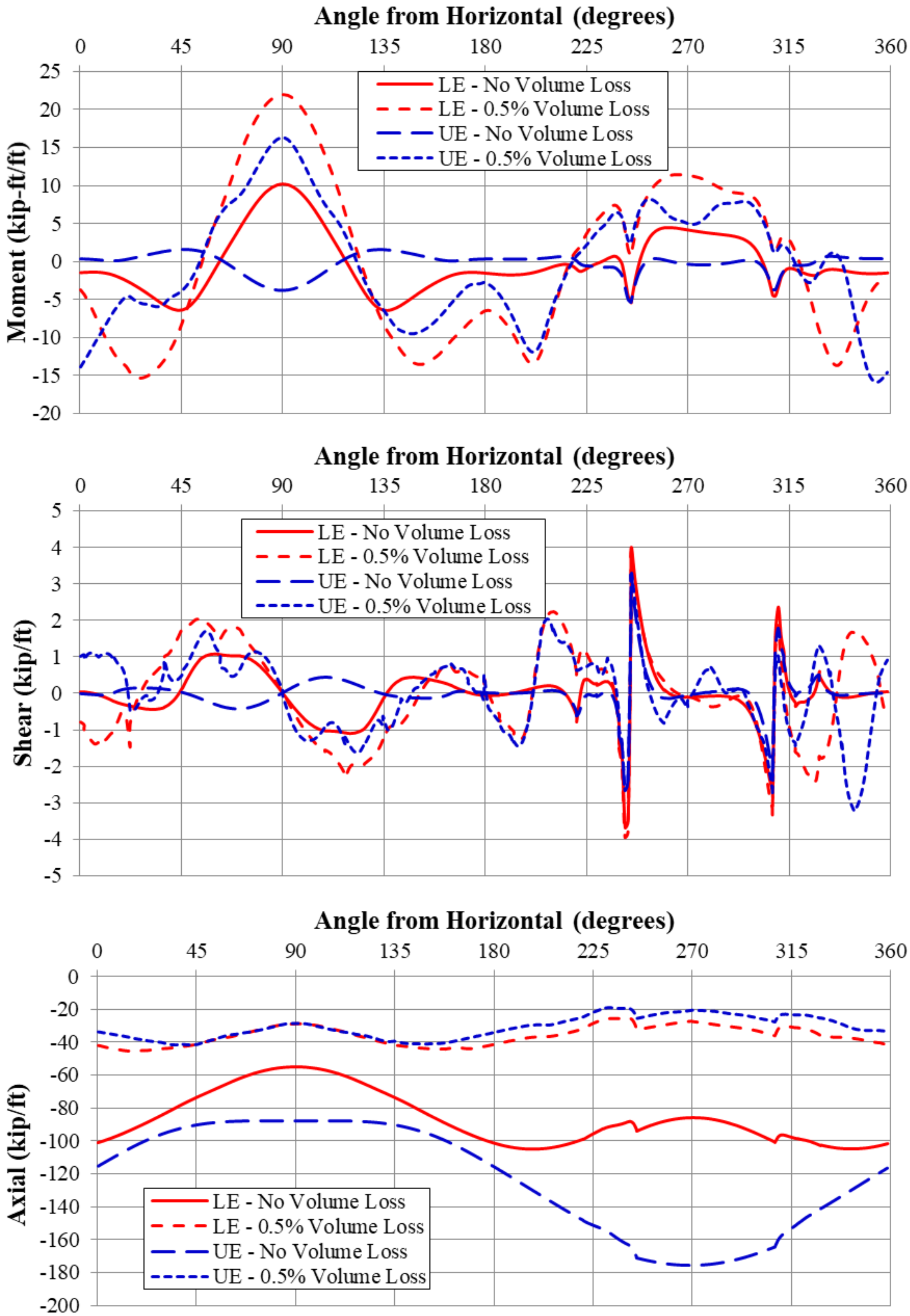
a) Exaggerated (50x) Deformed Mesh (LE, 0.5% Volume Loss)



b) Liner Moments (LE, 0.5% Volume Loss)

Source: FHWA

Figure 28. Analysis Results. Permanent Loads + EV-3 (No Building Surcharge).



Source: FHWA

Figure 29. Analysis Results. Permanent Loads + EV-3 (No Building Surcharge).

2.4.4.3 Critical Forces

The critical load forces were extracted from the shear, moment and axial force diagrams. Factored loads were reviewed around the perimeter to find the maximum load locations. The load combination and location for the maximum and minimum moments, shears and axial forces are identified and the subsequent forces are extracted and reported in Table 32 through Table 35. These loads are unfactored. Live load results were obtained by subtracting the demands due to permanent loads only in Section 2.4.4.1 from the corresponding permanent plus live load demands in Section 2.4.4.2. Live load values in the following tables include the 1.20 single lane multiple presence factor.

The maximum moment shown in Table 32 was obtained when subjected to the *LE-Contraction* soil condition at *90°* with respect to Figure 18, which occurs at the top of the tunnel. Positive moment causes tension on the inside face of the tunnel liner (in the absence of axial compression). It can be seen that the shear is effectively zero at this location and the live load components of the moment and axial thrust is small in relation to dead load.

Table 32. Maximum Moment (kip-ft) & Corresponding Shear (kip) & Axial (kip).

Force	Perm	Perm +Bldg	Truck	Tandem	EV-2	EV-3
Moment	20.95	74.80	0.82	0.89	0.61	0.96
Shear	0.02	0.10	0.00	0.00	0.00	0.01
Axial	-27.91	-116.23	-0.71	-0.50	-0.48	-0.71

The minimum moment as shown in Table 33 was obtained when subjected to the *UE-Contraction* soil condition at *354°* with respect to Figure 18, which occurs at the right side of the tunnel and is further from the surface than the positive moments above. It can be seen that shear live load forces at this location are essentially zero. Although this location is the maximum factored negative (or minimum) moment location, only negligible live loads moments, which are positive, exist there. To compare against the maximum positive moments above, maximum negative live load moments from around the perimeter are shown in Table 33 to envelope the results. The maximum negative live load moments occur in the *LE-Contraction* soil condition at *52°*. It will be shown later the reinforcement is the same on both faces and the concrete cover on each face is similar (2 inches on outside versus 1.5 inches on inside) so the larger positive moments above will control the rating. As such, no negative moment rating factors will be calculated.

Table 33. Minimum Moment (kip-ft) & Corresponding Shear (kip) & Axial (kip).

Force	Perm	Perm +Bldg	Truck	Tandem	EV-2	EV-3
Moment*	-15.93	-12.47	-0.27	-0.27	-0.21	-0.32
Shear	0.07	-2.73	0.02	0.01	0.01	0.02
Axial	-31.94	-72.06	-0.82	-0.64	-0.57	-0.77

*Note: Live load moments are minimums from entire perimeter.

The location of maximum factored shear, whether it be positive or negative, as shown in Table 34 was obtained when subjected to the *LE-No Contraction* soil condition at *245°* with respect to Figure 18, which is located near the bottom of the tunnel liner at the left corbel. It can be seen that live load forces (moment, shear or axial) at this location are essentially zero. This location, at the bottom of the tunnel is not influenced by live load above. Other locations do have live load shears but they are not the maximum factored shear location. It will be shown later, that the shear capacity is much greater than the factored maximum shear (1.35 times dead load) so ratings will not be calculated. The shear resistance will be conservatively based on maximum positive or negative moments and axial thrust to ensure capacity envelopes all possible combinations. For reference, the maximum HL-93 Truck shear is ±0.12 kips for the *LE-Contraction* soil condition on either side of the tunnel crown (*90°*).

Table 34. Maximum Shear (kip) & Corresponding Moment (kip-ft) & Axial (kip).

Force	Perm	Perm +Bldg	Truck	Tandem	EV-2	EV-3
Moment	-5.50	1.23	0.06	0.05	0.04	0.06
Shear	4.00	4.83	0.03	0.03	0.02	0.03
Axial	-93.99	-143.16	-0.06	-0.01	-0.04	-0.07

The minimum axial thrust (negative axial load) as shown in Table 35 was obtained when subjected to the *UE-No Contraction* soil condition at *270°* with respect to Figure 18, which is located at the bottom of the tunnel. As expected, there are no factored axial tension forces in the liner. For similar reasons to the maximum shear location above, it can be seen that live load forces (shear or axial) at this location are essentially zero. This location, at the bottom of the tunnel is not influenced by live load above. Similar to the minimum (negative) moment location, the maximum factored thrust location is not subjected to compression live load. The live load values show in Table 35 are the maximum values around the perimeter to envelope the results. The maximum live load axial thrust is for the *LE-Contraction* soil condition on the left and right sides of the tunnel. It will be shown later, that the axial capacity is much greater than the factored maximum axial thrust (1.35 times dead load) so ratings will not be calculated.

Table 35. Maximum Axial (kip) & Corresponding Moment (kip-ft) & Shear (kip).

Force	Perm	Perm +Bldg	Truck	Tandem	EV-2	EV-3
Moment	-0.43	1.64	0.02	0.01	0.01	0.02
Shear	-0.07	-0.07	0.00	0.00	0.00	0.00
Axial*	-175.57	-195.19	-1.32	-1.06	-0.92	1.35

*Note: Live load thrusts are minimums from entire perimeter.

2.4.5 Resistance Calculations

The tunnel liner was constructed from 24-inch thick precast concrete sections that are 77.86 inches wide at the outside. These sections have 2 inches of clear to the ASTM A496 deformed steel wire size D-31 transverse bars on the outside face and 1.5 inches on the inside face. The size D-30 longitudinal bars, which resist the bending moments, are inside the transverse reinforcement. Structural resistances are calculated assuming the 16 longitudinal bars in each section equally resist the force demands in that precast section. This results in 2.47 bars per linear foot of the tunnel liner. The deformed steel wire size D-30 longitudinal bars have a diameter of 0.618 inch while the size D-31 transverse bars have a diameter of 0.628 inch. These sections are constructed utilizing 7.0 ksi concrete and 75 ksi reinforcing steel.

Concrete Properties:

f'_c	=	7.0 ksi	
α_1	=	0.85	LRFD BDS 5.6.2.2
β_1	=	0.70 for f'_c = 7.0 ksi	LRFD BDS 5.6.2.2
λ	=	1.0 (normal weight concrete)	LRFD BDS 5.4.2.8
γ_3	=	0.75 (AASHTO M31 Grade 75)	LRFD BDS 5.6.3.3
γ_1	=	1.6	LRFD BDS 5.6.3.3

Modulus of rupture:

$$f_r = 0.24\lambda\sqrt{f'_c} = 0.24(1.0)\sqrt{7.0} = 0.635 \text{ ksi}$$ LRFD BDS 5.4.2.6

Compression reinforcement in flexural capacity calculations is conservatively ignored. The same reinforcement is used for both faces. Compute minimum moment resistance based on direction with maximum reinforcement cover (outside face). Calculated results are based on a per foot analysis width.

2.4.5.1 Moment Resistance

Rectangular section height:	h	=	24.00 inch	
Rectangular section width:	b	=	12.00 inch	
Clear distance to rebar from tension face:	clr	=	2.00 inch	
Area of single rebar:	A_{s_bar}	=	0.300 inch^2	
Diameter of rebar:	dia_{bar}	=	0.618 inch	
Spacing of rebar:	s	=	4.87 inch	
Flexural resistance factor:	ϕ_f	=	TBD	LRFD BDS 5.5.4.2

Moment Resistance:

Determine equivalent area of reinforcing bar:

$$A_s = A_{s\_bar} No.Bar = 0.300(12''/4.87'' \text{ bar spacing}) = 0.740 \text{ inch}^2$$

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

$$d_s = h - \frac{dia_{bar}}{2} - d_{trans} - clr = 24.00 - \frac{0.618}{2} - 0.628 - 2.00 = 21.06 \text{ inches}$$

Determine distance from the equivalent stress block for tension controlled, non-prestressed tension reinforcement only:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.740(75)}{0.85(7.0)(12.00)} = 0.78 \text{ inches}$$ LRFD BDS Eq. 5.6.3.1.1-4 and 5.6.2.2

$$\beta_1 = 0.7 \Rightarrow c = \frac{a}{\beta_1} = \frac{0.78}{0.7} = 1.11 \text{ inches}$$ LRFD BDS 5.6.2.2

$$\varepsilon_s = \varepsilon_c \left(\frac{d_s}{c} - 1 \right) = 0.003 \left(\frac{21.06}{1.11} - 1 \right) = 0.054 > 0.005$$
$$\therefore \phi_f = 0.90$$ LRFD BDS 5.5.4.2, 5.6.2.1

Accounting for the level of applied axial compression at the location of maximum factored moment in the equation above would not shift the strain out of the tension-controlled zone.

Calculate nominal moment resistance:

$$\phi M_n = \phi_f A_s f_y \left(d_s - \frac{a}{2} \right) =$$
$$0.9(0.740)(75) \left(21.06 - \frac{0.78}{2} \right) \left(\frac{1 \text{ ft}}{12 \text{ inch}} \right)$$ LRFD BDS Eq. 5.6.3.2.2-1
$$= 86.02 \text{ kip-ft}$$

The calculation of moment resistance conservatively ignores the presence of axial thrust, which generally increase the moment resistance up to the levels of axial compression present in the analysis. This can be seen in Figure G6A-1 in Appendix G6A of the MBE (AASHTO, 2018). If the resulting moment ratings were inadequate, the refined moment-axial interaction procedure could be employed. However, as can be seen in the figure, very high loads of axial compression may reduce the moment resistance.

Axial Resistance:

Tunnel liner need to be checked for axial thrust and flexural interaction. Check flexure/axial interaction:

$$0.1 \phi_c f_c' A_g = 0.1(0.7)(7.0)(24)(12) = 141.12 \text{ kip}$$ LRFD BDS 5.6.4.5

Since the axial forces associated with the maximum factored moments (permanent dead load plus live load) do not exceed $0.1\phi_c f_c' A_g$, the axial thrust can be neglected. Furthermore, at the locations where the dead load effects are approaching this limit, the live load moment effect is approaching zero; as a result, the load rating can be neglected when axial forces are in excess of this limit.

Minimum Steel:

Determine minimum reinforcement, beginning with the section modulus:

$$S_c = \frac{bh^2}{6} = \frac{12.00(24.00)^2}{6} = 1152 \text{ inch}^3$$ LRFD BDS 5.6.3.3

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

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c = 0.75(1.6)(0.635)(1152)\left(\frac{1 \text{ ft}}{12 \text{ inch}}\right)$$ LRFD BDS Eq. 5.6.3.3-1

$$= 73.15 \text{ kip-ft}$$

Since $\phi_f M_n > M_{cr}$, minimum reinforcement at this section is satisfied

2.4.5.2 Shear Resistance

The shear resistance of the liner is also evaluated. The liner does have shear bars dispersed through the precast sections. However, for the purpose of simplification, only the concrete shear capacity will be utilized for the shear resistance. If the resistance is not sufficient with just the concrete shear resistance, the shear bars can be included in the resistance calculations.

The shear depth, d_v, in accordance with LRFD BDS 5.7.3.2.

$$d_v = \max\begin{Bmatrix} 0.72h \\ 0.9d \\ d - a/2 \end{Bmatrix} = \begin{Bmatrix} 0.72(24) = 17.28 \\ 0.9(21.06) = 18.95 \\ 21.06 - 0.78/2 = 20.67 \end{Bmatrix} = 20.67 \text{ inches}$$ LRFD BDS 5.7.2.8

Determining the nominal shear resistance involves the use of the applied shears and moments at the section of interest. To envelope the possible combinations of forces, use the non-concurrent maximum factored HL-93 Inventory loadings.:

$$M_u = 29.85 \text{ kip-ft}$$
$$V_u = 5.38 \text{ kip}$$
$$\mathrm{N}_u = 236.96 \text{ kip}$$

The shear resistance of the liner 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 not less than 16 inches and the shear steel is not being considered in the resistance calculations.

$$M_u \geq V_u d_v \Rightarrow$$
$$29.85(12) = 358.2 \text{kip-in} \geq 5.38(20.67) = 111.2 \text{ kip-inch}$$ LRFD BDS5.7.3.4.2

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u|\right)}{E_s A_s}$$

LRFD BDS Eq. 5.7.3.4.2-4

$$= \frac{\left(\frac{|358.2|}{20.67} + 0.5(239.96) + 5.38\right)}{0.740(29000)} = 0.0066$$

$$0 \le \varepsilon_s \le 0.006$$

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500(0.006) = 50.00^o$$

$$s_x = d_v \le h - cl - 2d_{trans} - d_b = 20.67 \text{ in} \le 24 - (2.00 + 1.50) - 2(0.628) - 0.618 = 18.63 \text{ inch}$$

Where *cl* is the clear cover;

$$s_{xe} = 12 \text{ in} \le s_x \frac{1.38}{a_g + 0.63} \le 80 \text{ inch}$$

LRFD BDS Eq. 5.7.3.4.2-7

$$s_{xe} = 12 \text{ in} \le 18.63 \frac{1.38}{0.75 + 0.63} \le 80 \text{ in} \Rightarrow 18.63 \text{ inch}$$

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \frac{51}{(39 + s_{xe})}$$

LRFD BDS Eq. 5.7.3.4.2-2

$$= \frac{4.8}{[1 + 750(0.006)]} \frac{51}{[39 + 18.63]} = 0.88$$

$$\phi V_n = \phi_v \left[0.0316\beta\lambda\sqrt{f_c^{'}} \le 0.25 f_c^{'}\right] b_v d_v$$

$$= 0.85\left[0.0316(0.88)(1.0)\sqrt{7.0} \le 0.25(7.0)\right] 12(20.67)$$

LRFD BDS 5.7.3.3

$$= 15.51 \text{ kip}$$

It can be seen that even with very conservative calculation of shear resistance using non-concurrent loads and ignoring shear reinforcement, the resistance is significantly higher than the maximum factored HL-93 Inventory shear force. The following LRFD BDS checks have not been performed for this example but should be checked for actual rating calculations: regions requiring transverse reinforcement (5.7.2.3) and longitudinal reinforcement checks (5.7.3.5) [23 CFR 625.4(d)(1)(v)].

2.4.6 LRFR Rating Calculations

The structural condition of the tunnel liner 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 Section 5 and specified in the calculations of Section 2.4.5 of this document.

The equation for calculating the rating factor is based on MBE Equation 6A.5.12.4-1, which has been simplified for the load types being applied.

$$RF = \frac{C \pm \gamma_{DC} DC \pm \gamma_{DW} DW \pm \gamma_{EV} EV \pm \gamma_{EH} EH \pm \gamma_{ES} ES}{\gamma_{LL}(LL + IM) \pm \gamma_{LS} LS}$$ 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 37 though Table 39 show the results of the overall capacity based on MBE Equation 6A.5.12.4-2 as well as the rating factors based on MBE Equation 6A.5.12.4-1. Ratings are performed at the critical section. As discussed in Section 2.4.4.3, only ratings for maximum positive moment will be calculated. It was shown that the total factored loads and low levels of live load forces indicate the negative moment, shear and axial loads do not need to be investigated.

Maximum and minimum load factors are selectively used to obtain the maximum absolute value of the force effects. Dead loads and earth loads are not able to be de-coupled due to the nature of the FEM. Therefore, a conservative load factor of 1.35 is used for all permanent loads. This load factor is the larger of the DC, EV and EH load factors. The live load factor for the EV vehicles is 2.0 as identified in the FHWA Emergency Vehicle Memorandum (FHWA, 2016) for buried structures. The load factors used are shown in Table 36. The moments, shears and their corresponding capacities in Table 37 to Table 39 are in kip-ft and kip, respectively. The load effects are unfactored, but the capacities are factored resistances. Note that Table 39 is for the EV-3 vehicle because it controls over EV-2.

Table 36. Load Factors.

Permanent Loads	HL-93 LL Inventory	HL-93 LL Operating	EV Legal
1.35	1.75	1.35	2.0

Table 37. HL-93 Inventory Level Rating Factors.

Section	Perm.	Live Load	Capacity	R.F.
M_{pos}	20.95	0.89	86.02	37.1

Table 38. HL-93 Operating Level Rating Factors.

Section	Perm.	Live Load	Capacity	R.F.
M_{pos}	20.95	0.89	86.02	48.1

Table 39. EV-3 Rating Factors.

Section	Perm.	Live Load	Capacity	R.F.
M_{pos}	20.95	0.96	86.02	30.07

The rating factors are all well above 1 indicating that the tunnel liner is sufficient for all the evaluated live loads. In addition, it is observed that the live load effects induce a minor effect on the tunnel liner force demands in relation to permanent loads.

The flexural and shear resistance of the joints between panels around the circumference of the liner are not investigated in this example but should be reviewed for an actual rating. This includes resistance to crushing and opening of the joint under axial thrust and bending. In addition to the thrust due to external loads, the panels are attached to each other with torqued bolts creating additional compression across the joint providing additional resistance to joint opening. The shear resistance of the interface also needs to be investigated. Shear is resisted through friction and steel guide rods and the previously mentioned bolts.

A structural adequacy evaluation under a 7 ksf building load is investigated at other locations in addition to the rating of the liner under live load effects. This loading does not include a live load effect and therefore cannot produce a rating factor; however, a performance ratio can be evaluated by dividing the resistance by the demand. This ratio will therefore correlate in the same manner as the rating factor in which a ratio greater than 1 indicates adequate strength while a ratio less than 1 indicates insufficient strength. The moments, shears and their corresponding capacities in Table 40 are in kip-ft and kip, respectively. The permanent loads including the building load are factored per Table 36 and the capacities are factored resistances.

Table 40. Building Performance Ratio.

Force	Perm.	Capacity	P.R.
M_{pos}	72.82	86.02	1.18
M_{neg}	45.64	86.02	1.88
Shear	8.05	15.51	1.93

The performance ratio is greater than 1, indicating this tunnel liner has sufficient resistance to support the 7 ksf building load when not subjected to concurrent live load at this depth below the surface.

CHAPTER 3 - EXAMPLE 2 – BOX TUNNEL ROOF GIRDER

Example 2 presents a double cell tunnel consisting of simply supported, composite, steel roof girders supported by walls and a slab-on-grade bottom slab. The roof girder supports surface roadway loads. The lower roadway slab is on-grade and therefore does not need to be rated. The walls are soldier pile tremie concrete walls, which resist the lateral earth loads and the vertical loads from the roof girders. The tunnel roof girder is subjected to vertical dead loads, earth loads and live loads. It is assumed that little or no lateral loads are imparted to the roof girders due to the simple shear connections of the girders to the soldier piles. The box tunnel roof girder is rated with the LRFR method for the design vehicles (HL-93 Inventory and Operating Level) and emergency vehicles (EV-2 and EV-3) at the legal load level.

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

1. Structure data
2. Example notes
3. Rating approach/assumptions
4. Load application
5. Structural analysis
6. Resistance calculations
7. LRFR rating calculations

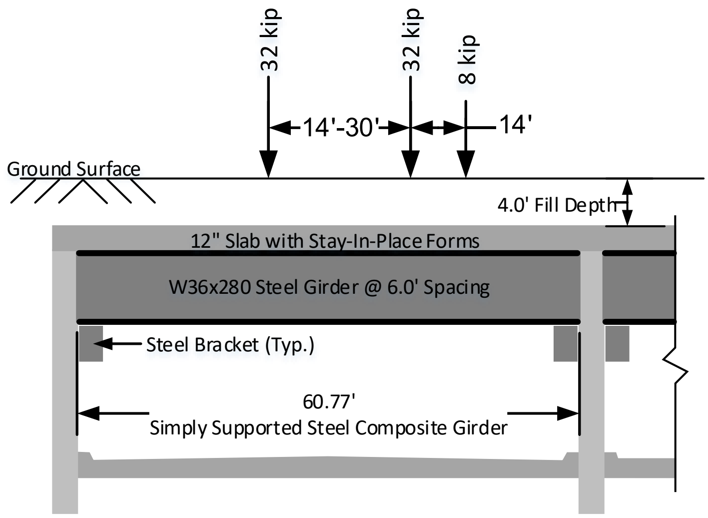
3.1 STRUCTURE DATA

3.1.1 Materials

Materials are known, otherwise use MBE Articles 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, or drained, soil above the roof girder.

Concrete:	f'_c	=	4.0 ksi
Reinforcing Steel:	f_y	=	60.0 ksi
Structural Steel:	F_y	=	50.0 ksi
Soil:	γ_{soil}	=	0.125 kcf
	ϕ_{soil}	=	30°

3.1.2 Dimensions



Source: FHWA

Figure 30. Illustration. Cross-Section Showing Tunnel Geometry.

3.1.3 Example Notes

- This cross-section has been selected from this project as representative to demonstrate rating of the tunnel roof girder under external loading.
- The live load carrying member considered in this example is the simply supported composite steel I- girder.
- If the roadway above the tunnel has 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 roof girder.
- The focus of this example is LRFR.
- This example focuses on the load rating of a critical and typical roof girder. A section taken near the ends of the tunnel would need to account for increases in loading due to edge of slab effects. Additionally, critical connection details may need to be rated in a full rating project.
- The rating Engineer should review the record drawings and inspection reports carefully to properly identify the support condition (pinned, expansion, fixed). Typical conditions, assumed for this example are simply supported conditions for the roof girder.
- Rating is performed only at maximum shear and moment locations. The capacity of the bracket is also evaluated. Typical ratings include evaluation at all tenth points and full connection evaluation.

3.1.4 Rating Approach/Assumptions

- LRFR evaluation is performed for a single girder (perpendicular to direction of traffic inside the tunnel and parallel to the direction of traffic above the box tunnel).
- The steel girder is composite with the concrete roof slab.
- The pavement on the ground above 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 sides of the tunnel with soil parameter γ = 125 pcf. Additionally, this box tunnel is in a dry soil condition above the roof girder.
- LRFR live load ratings are evaluated for HL-93 design loading (Truck or Tandem) and the EV-2 and EV-3 emergency vehicles per the associated FHWA Memo (FHWA, 2016).

3.2 STEEL ROOF GIRDER

3.2.1 Load Calculations

The roof girder is subject to self-weight, air pressure, soil/pavement, and live load. There is no wearing surface applied and is therefore neglected in this analysis. Other forces such as TU are neglected due to the simply supported boundary conditions of the composite girder.

3.2.1.1 Dead Load Component, DC

Dead load components include the self-weight of the reinforced concrete slab, steel girder and stay-in-place (SIP) forms.

Normal weight of reinforced concrete:

LRFD BDS Table 3.5.1-1 and Article C3.5.1

$$\gamma_c = 0.145\text{ kcf} + 0.005\text{ kcf} = 0.150\text{ kcf}$$

$$\mathrm{w}_c = \gamma_c t_s b_{eff} = 0.150(1.0)(6.0) = 0.900\text{ klf}$$

Steel girder weight:

$$w_{W36x280} = 0.280\text{ klf}$$

Stay-in-place forms:

$$w_{SIP} = 0.015\text{ ksf}\,(6.0\text{ ft Spa.}) = 0.090\text{ klf}$$

3.2.1.2 Wearing Surface, DW

This tunnel assumed no wearing surfaces were applied and therefore were not included in the load calculations.

3.2.1.3 Vertical Earth Pressure, EV

The construction method is assumed to be an embankment installation. Therefore, the vertical earth pressure can utilize the "embankment installation" in LRFD BDS equations 12.11.2.2.1-1 and 12.11.2.2.1-2.

$$F_e = 1 + 0.20\frac{H}{B_c} = 1 + 0.2\frac{4.0}{60.77} = 1.013 \qquad \text{LRFD BDS Eq. 12.11.2.2.1-1}$$

Where H is the fill depth of 4 feet and B_c is the supported width of 60.77 feet.

$$W_E = F_e \gamma_s B_c H = 1.013(0.125)(B_c)(4.0) = 0.507B_c \text{ klf} \qquad \text{LRFD BDS Eq. 12.11.2.2.1-1}$$

Convert W_E to a distributed earth load, w_{ev}, along the length of girder:

$$w_{ev} = \frac{W_E S_{girder}}{B_c} = \frac{0.507B_c(6.0)}{B_c} = 3.042 \text{ klf}$$

3.2.1.4 Live Load Application, LL

The live load travels parallel to the span of the steel girder over a 4 feet fill and therefore the wheel load distributes through the earth fill. The transverse live load distribution should be calculated in accordance to LRFD BDS Article 3.6.1.2.6b for traffic parallel to span. The LLDF is specified as 1.15 from LRFD BDS Table 3.6.1.2.6a-1.

$$H = 4.0 \text{ ft}$$

$$H_{\text{int}} = \frac{s_w - \frac{w_t}{12} - \frac{0.06D_j}{12}}{LLDF} = \frac{6 - \frac{20}{12} - \frac{0.06(60.77\times 12)}{12}}{1.15} \qquad \text{LRFD BDS Eq. 3.6.1.2.6b-1}$$

$$= 0.598 \text{ ft}$$

The effective width the load is distributed over can be calculated by LRFD BDS Eq. 3.6.1.2.6b-3 since H is greater than H_{int}.

$$w_w = \frac{w_t}{12} + s_w + LLDF(H) + 0.06\frac{D_i}{12} = \frac{20}{12} + 6.0 + 1.15(4.0) + 0.06\left(\frac{60.77\times 12}{12}\right) = 15.913 \text{ ft}$$

The longitudinal live load distribution should be calculated in accordance to LRFD BDS Article 3.6.1.2.6b for traffic parallel to the span. Compute the distribution widths for the various axle spacings of the HL-93 and EV vehicles (4' for HL-93 Tandem and EV-3 vehicle, 14 feet for HL-93 Truck, and 15' for EV-2 and EV-3 vehicles). Note the HL-93 Truck produces the critical force effects when the rear axle is at 14-foot spacing. Additionally, where ratings are adequate, the longitudinal distribution of wheel loads may conservatively and conveniently be ignored by applying the axle point loads directly to the beam.

$$H_{\text{int}-p(4')} = \frac{s_a - \frac{l_t}{12}}{LLDF} = \frac{4 - \frac{10}{12}}{1.15} = 2.754 \text{ ft}$$

LRFD BDS Eq. 3.6.1.2.6b-4 & 5

$$l_w = \frac{l_t}{12} + s_a + LLDF(H) = \frac{10}{12} + 4 + 1.15(4) = 9.43 \text{ ft}$$

$$H_{\text{int}-p(14')} = \frac{s_a - \frac{l_t}{12}}{LLDF} = \frac{14 - \frac{10}{12}}{1.15} = 11.449 \text{ ft}$$

LRFD BDS Eq. 3.6.1.2.6b-4 & 5

$$l_w = \frac{l_t}{12} + LLDF(H) = \frac{10}{12} + 1.15(4) = 5.43 \text{ ft}$$

$$H_{\text{int}-p(15')} = \frac{s_a - \frac{l_t}{12}}{LLDF} = \frac{15 - \frac{10}{12}}{1.15} = 12.319 \text{ ft}$$

LRFD BDS Eq. 3.6.1.2.6b-4 & 5

$$l_w = \frac{l_t}{12} + LLDF(H) = \frac{10}{12} + 1.15(4) = 5.43 \text{ ft}$$

Each axle load is distributed over the transverse and longitudinal areas and normalized over the girder spacing. For calculation simplicity, a unit load pressure is calculated then multiplied by the axle loads of the various vehicles.

$$p_{LL-1\text{kip}} = \frac{1 \text{ axle}}{15.913 l_w} = \left(\frac{0.063}{l_w}\right)\frac{\text{axle}}{\text{ft}^2}$$

$$\text{w}_{LL-1\text{kip}} = \left(\frac{0.063}{l_w}\right)\frac{\text{axle}}{\text{ft}^2}\left(6 \text{ ft girder spacing}\right) = \left(\frac{0.377}{l_w}\right)\frac{\text{axle}}{\text{ft}}$$

This distributed load (axle) per length along the girder can then be multiplied by the various axle loads. However, the live load also needs to be multiplied by the dynamic allowance for buried structures specified in LRFD BDS Article 3.6.2.2 and a single multiple presence factor of 1.2 as specified in LRFD BDS Article 3.6.1.1.2. For this example, a multiple presence factor of 1.0 was used for the EV legal loads based on MBE Article 6A5.12.10.3 for buried structures with traffic traveling parallel to the span. Where the axle loads are 14 feet or 15 feet, the axle loads shall be distributed over 5.43 feet. When the axle spacing is 4 feet, the axle loads shall be combined and distributed over 9.43 feet.

$$\text{I.M.} = 33(1.0 - 0.125 D_E) \geq 0\% = 33(1 - 0.125 \times 4) = 16.5\%$$

LRFD BDS Eq. 3.6.2.2-1

$$P_{HL93-Truck} = (8\text{ kip/axle})\left(\frac{0.377\text{ axle}}{5.43\text{ ft}}\right)(1.165)(1.2) = 0.776\text{ klf per girder}$$

$$P_{HL93-Truck} = (32\text{ kip/axle})\left(\frac{0.377\text{ axle}}{5.43\text{ ft}}\right)(1.165)(1.2) = 3.106\text{ klf per girder}$$

$$P_{HL93-Tandem} = (2)(25\text{ kip/axle})\left(\frac{0.377\text{ axle}}{9.43\text{ ft}}\right)(1.165)(1.2) = 2.795\text{ klf per girder}$$

$$P_{EV2} = (24\text{ kip/axle})\left(\frac{0.377\text{ axle}}{5.43\text{ ft}}\right)(1.165)(1.0) = 1.941\text{ klf per girder}$$

$$P_{EV2} = (33.5\text{ kip/axle})\left(\frac{0.377\text{ axle}}{5.43\text{ ft}}\right)(1.165)(1.0) = 2.710\text{ klf per girder}$$

$$P_{EV3} = (24\text{ kip/axle})\left(\frac{0.377\text{ axle}}{5.43\text{ ft}}\right)(1.165)(1.0) = 1.941\text{ klf per girder}$$

$$P_{EV3} = (2)(31\text{ kip/axle})\left(\frac{0.377\text{ axle}}{9.43\text{ ft}}\right)(1.165)(1.0) = 2.888\text{ klf per girder}$$

For this example, only axle loads are applied based on MBE Article 6A5.12.10.3 for reinforced concrete box culverts,. However, this is a steel roof girder with a span in excess of 60 feet; therefore, it is prudent to include the lane load for HL-93 ratings. As a result, the distribution of the lane load also needs to be determined, assuming the load distributes at a 1.15 factor as specified in LRFD BDS Table 3.6.1.2.6a-1. Therefore, the bottom width of the load application at the structure is 1.15 times wider than the width at the ground surface.

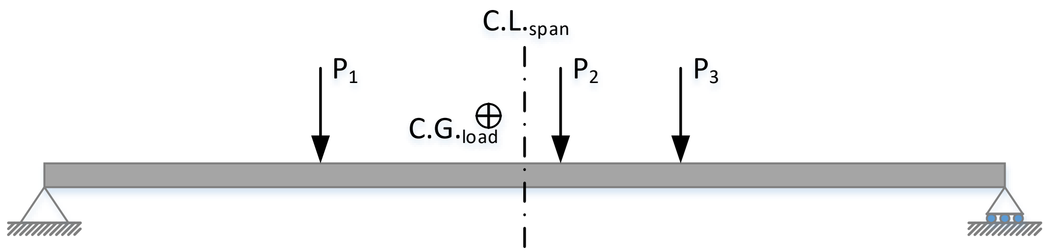
$$W_{lane-buried} = W_{lane} + LLDF(H) = 10 + 2(1.15)(4) = 19.2\text{ feet}$$

$$w_{LL} = w_{HL93-Lane}\left(\frac{s_{girder}}{W_{lane-buried}}\right)M.P. = (0.64\text{ klf})\left(\frac{6\text{ feet}}{19.2\text{ feet}}\right)(1.2) = 0.240\text{ klf per girder}$$

3.2.2 Structural Analysis

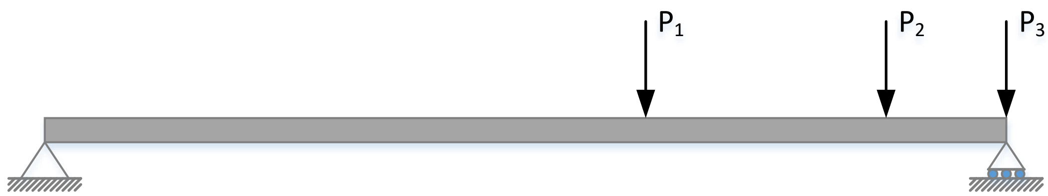
A beam analysis is utilized for the structural analysis. All the load effects and member resistances are calculated using the tributary, one-dimensional analysis. The composite girder is assumed to be simply supported and therefore can be evaluated with simple hand calculations or by a structural analysis program.

The maximum moment can be obtained by positioning the axles such that the centerline of the span is midway between the center of gravity of the load and the nearest wheel load, as depicted in Figure 31. The maximum shear is obtained when the axle loads are placed adjacent to the end of the span, as depicted in Figure 32.



Source: FHWA

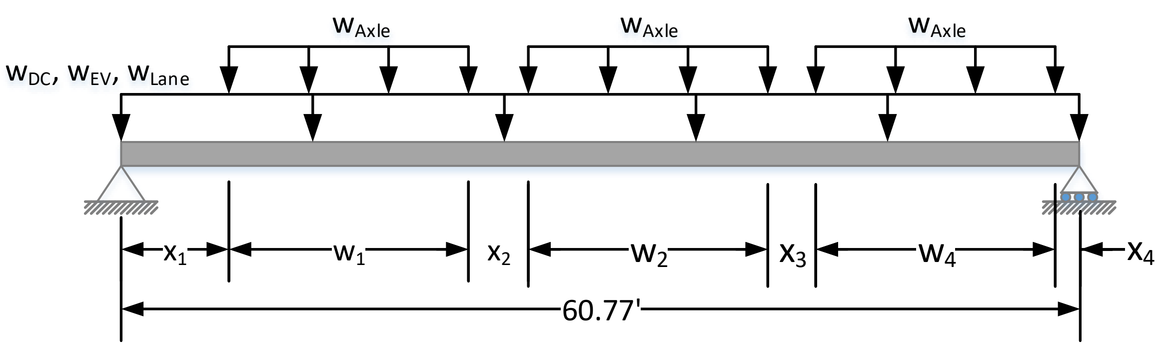
Figure 31. Illustration. Live Load Placement for Maximum Moment.



Source: FHWA

Figure 32. Illustration. Live Load Placement for Maximum Shear.

Using this load placement theory, the distributed dead and live loads are placed on the beam as depicted in Figure 33.



Source: FHWA

Figure 33. Illustration. Load Placement.

Where the variables x_i and w_i are defined in Table 41. Results show the Tandem controls the HL-93 loading due to the convergence of the axle load effects over such a concentrated portion of the girder.

Table 41. Moment Load Placement Dimensions.

Load	x_1 (ft)	w_1 (ft)	x_2 (ft)	w_2 (ft)	x_3 (ft)	w_3 (ft)	x_4 (ft)
HL-93 Truck	9.00	5.43	8.57	5.43	8.57	5.43	18.34
HL-93 Tandem	25.67	9.43	25.67	N/A	N/A	N/A	N/A
EV-2	18.93	5.43	9.57	5.43	21.41	N/A	N/A
EV-3	15.41	5.43	9.57	9.43	20.93	N/A	N/A

Table 42. Shear Load Placement Dimensions.

Load	x_1 (ft)	w_1 (ft)	x_2 (ft)	w_2 (ft)	x_3 (ft)	w_3 (ft)	x_4 (ft)
HL-93 Truck	27.34	5.43	8.57	5.43	8.57	5.43	0.00
HL-93 Tandem	51.34	9.43	0.00	N/A	N/A	N/A	N/A
EV-2	40.34	5.43	9.57	5.43	0.00	N/A	N/A
EV-3	36.34	5.43	9.57	9.43	0.00	N/A	N/A

LRFD Road Tunnel Design and Construction Guide Specifications, 1st Ed. Table 3.4-1 identifies the applicable minimum and maximum load factors for each load specified above. Since the girder is simply supported, only maximum load factors were necessary to produce critical forces. The live load factors for the HL-93 are identified in Table 3.4-1; however, the live load factor for the EV vehicles are 2.0 as identified in the FHWA Emergency Vehicle Memorandum. The forces are given in Table 43.

Table 43. Unfactored Girder Critical Forces.

Load	Shear (kip)	Moment (k-ft)
DC	39	586
Earth Load, EV	92	1404
HL-93 Lane	7	111
HL-93 Truck	30	424
HL-93 Tandem	24	369
EV-2	22	300
EV-3	32	457

3.2.3 Resistance Calculations

The depth, analysis section width, and reinforcement for each of the critical sections are tabulated below in Table 44 and Table 45.

3.2.3.1 Material Properties

f'_c = 4.0 ksi
F_y = 50.0 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

Slab reinforcement, which is in compression, is conservatively ignored in the flexural capacity calculations.

3.2.3.2 General Properties

Beam section: *W36x280*
Beam spacing: $s = b_{eff}$ = 72.00 inch

Slab thickness:	t_{slab}	=	12.00 inch	
Flexural resistance factor:	ϕ_f	=	1.0	LRFD BDS 6.5.4.2
Shear resistance factor:	ϕ_v	=	1.0	LRFD BDS 6.5.4.2
Modulus of Elasticity:				LRFD BDS 5.4.2.4 & 6.4.1

$$E_c = 120,000(\gamma_c)^2 f_c'^{0.33} = 120,000(0.145)^2(4.0)^{0.33} = 3987 \text{ ksi}$$

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

n=8.0 (assumed)

3.2.3.3 Section Properties

Non-Composite Section:

$$A_{W36x280} = 82.4 \text{ inch}^2$$

$$y_b = y_t = 18.26 \text{ inches}$$

$$I_x = 18900 \text{ inch}^4$$

$$S_{x-top} = S_{x-bot} = 1030 \text{ inch}^3$$

Short Term Properties (assume n=8.0):

Table 44. Short Term Composite Properties.

Member	A (inch^2)	Y_{bot} (inches)	AY_{bot} (inch^3)	$\bar{Y}$ (inches)	$A\bar{Y}^2$ (inch^4)	I_{member} (inch^4)
Slab	108	42.52	4592	-10.44	11782	1296
Top Flange	26.1	35.74	931	-3.66	349	5
Web	29.5	18.26	539	13.82	5638	2743
Bottom Flange	26.1	0.79	20	31.29	25509	5
Σ	190		6083		43278	4050

$$A_{ST} = 190 \text{ inch}^2$$

$$y_{bot} = \frac{\Sigma A\bar{y}_{bot}}{\Sigma A} = \frac{6083}{190} = 32.08 \text{ inches}$$

$$y_{top} = d_{grd} + t_{slab} - y_{bot} = 36.52 + 12 - 32.08 = 16.64 \text{ inches}$$

$$I_x = \Sigma A\bar{y}^2 + \Sigma I_{member} = 43278 + 4050 = 47328 \text{ inch}^4$$

$$S_{x-bot} = \frac{I_x}{y_{bot}} = \frac{47328}{32.08} = 1476 \text{ inch}^3$$

$$S_{x-top} = \frac{I_x}{y_{top}} = \frac{47328}{16.64} = 2844 \text{ inch}^3$$

Long Term Properties (assume 3*n*=24.0):

Table 45. Long Term Composite Properties.

Member	A ($inch^2$)	Y_{bot} (inches)	AY_{bot} ($inch^3$)	$\bar{Y}$ (inches)	$A\bar{Y}^2$ ($inch^4$)	I_{member} ($inch^4$)
Slab	36	42.52	1531	-16.84	10205	432
Top Flange	26.1	35.74	931	-10.05	2632	5
Web	29.5	18.26	539	7.42	1628	2743
Bottom Flange	26.1	0.79	20	24.90	16152	5
Σ	118		3022		30617	3186

$$A_{LT} = 118\,\text{inch}^2$$

$$y_{bot} = \frac{\Sigma A\bar{y}_{bot}}{\Sigma A} = \frac{3022}{118} = 25.68\,\text{inches}$$

$$\text{y}_{top} = d_{grd} + t_{slab} - y_{bot} = 36.52 + 12 - 25.68 = 22.84\,\text{inch}$$

$$I_x = \Sigma A\bar{y}^2 + \Sigma I_{member} = 30617 + 3186 = 33803\,\text{inch}^4$$

$$S_{x-bot} = \frac{I_x}{y_{bot}} = \frac{33803}{25.68} = 1316\,\text{inch}^3$$

$$S_{x-top} = \frac{I_x}{y_{top}} = \frac{33803}{22.84} = 1480\,\text{inch}^3$$

3.2.3.4 Plastic Moment

The plastic moment of the composite section is calculated in accordance to LRFD BDS Article D6.1. To accomplish this, the rolled W36x280 is discretized into top flange, web and bottom flange areas.

$$b_{ft} = b_{fb} = 16.595\,\text{inches}$$

$$t_{ft} = t_{fb} = 1.57\,\text{inches}$$

$$D = 33.38\,\text{inches}$$

$$t_w = 0.885\,\text{inches}$$

The section plastic forces are then calculated.

$$P_s = 0.85 f_c^{'} b_{eff} t_s = 0.85(4.0)(72)(12) = 2938\,\text{kip}$$

$$P_c = P_t = F_y b_f t_f = 50(16.595)(1.57) = 1303\,\text{kip}$$

$$P_w = F_y D t_w = 50(33.38)(0.885) = 1477\,\text{kip}$$

The relative magnitudes of these plastic forces are compared as outlined in LRFD BDS Table D6.1-1. This reveals this example is Case II. The corresponding plastic section properties are then calculated.

$$P_t + P_w + P_c \geq P_s \Rightarrow 1303 + 1477 + 1303 = 4082 \geq 2938 \text{ kip}$$

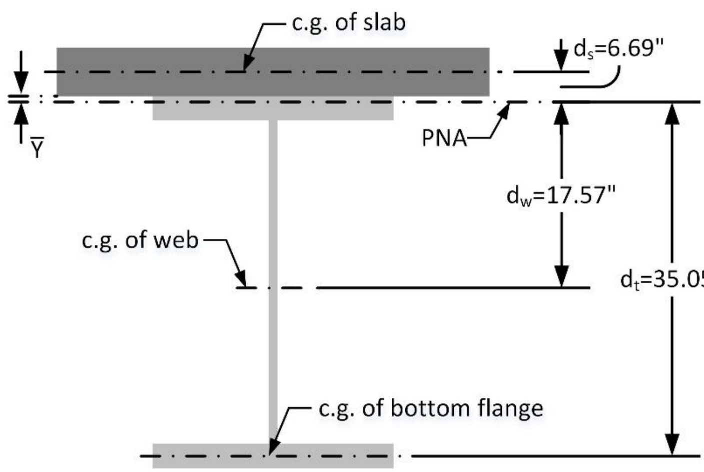
$$\bar{Y} = \left(\frac{t_c}{2}\right)\left[\frac{P_w + P_t - P_s}{P_c} + 1\right] = \left(\frac{1.57}{2}\right)\left[\frac{1477 + 1303 - 2938}{1303} + 1\right] = 0.69 \text{ inches}$$

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

$$= \left[\left(\frac{1303}{2 \times 1.57}\right)\left[0.69^2 + \left(1.57 - 0.69\right)^2\right] + \left[2938 \times 6.69 + 1477 \times 17.57 + 1303 \times 35.05\right]\right]\left(\frac{1 \text{ ft}}{12 \text{ inch}}\right)$$

$$= 7648 \text{ kip-ft}$$

$$D_p = \bar{Y} + t_s = 0.69 + 12 = 12.69 \text{ inches}$$



Source: FHWA

Figure 34. Illustration: Plastic Neutral Axis Offset Dimensions.

3.2.3.5 Moment Resistance

The section has been determined to be compact for composite positive flexure per LRFD BDS Article 6.10.6.2.2. The plastic depth is compared against the total depth to ensure the concrete deck will not crush prior to steel plastic capacity is reached per LRFD BDS Article 6.10.7.12.

$$D_t = h_{grd} + t_s = 36.52 + 12 = 48.52 \text{ inches}$$

$$D_p \leq 0.1 D_t \Rightarrow 12.69 \leq 0.1\left(48.52\right) = 4.85 \text{ inches}$$

Since the plastic compressive depth is greater than 10% of the total depth and this is a simply supported section, the plastic moment capacity needs to be determined per LRFD BDS Eq. 6.10.7.1.2-2.

$$\phi_f M_n = \phi_f M_p \left(1.07 - 0.7\frac{D_p}{D_t}\right) = 1.0\left(7648\right)\left(1.07 - 0.7\frac{12.69}{48.52}\right) = 6783 \text{ kip-ft}$$

3.2.3.6 Shear Resistance

The shear resistance also needs to be determined. The maximum shear force of this simply supported girder is within the end panel of the girder. Therefore, tension field action cannot be included in the shear resistance of the girder.

The slenderness of the web first needs to be determined to calculate the shear buckling coefficient per LRFD BDS Article 6.10.9.3.2. The panel length, d_o, is 15.19 feet since the cross frames with transverse stiffeners are located at quarter points of the span.

$$\frac{D}{t_w} = \frac{33.38}{0.885} = 37.72$$

$$k = 5$$

$$1.12\sqrt{\frac{Ek}{F_y}} = 1.12\sqrt{\frac{29000 \times 5}{50}} = 60.32$$

Since the web slenderness ratio, D/t_w, is less than the web compact limit, C=1.0. Therefore, the shear resistance is the plastic shear limit per LRFD BDS Article 6.10.9.3.3.

$$\phi_v V_n = \phi_v C 0.58 F_y D t_w = 1.0(1.0)(0.58)(50)(33.38)(0.885) = 857 \text{ kip}$$

3.2.3.7 Bracket Resistance

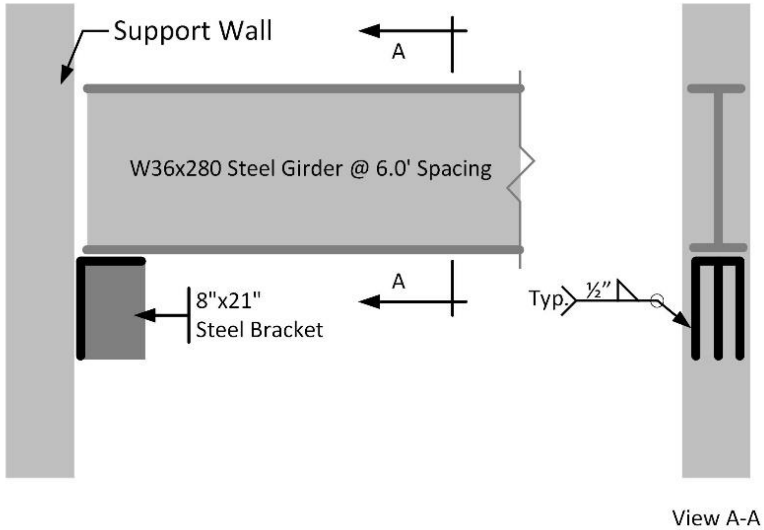
The bracket connection needs to be rated along with the girder capacities. The bracket buckling resistance is often prudent to rate along with the connection resistance. However, for simplicity of this example, the weld connection will only be shown as that is the controlling limit state on this particular bracket plate.

The bracket is composed of 3~8-inch x 21-inch vertical plates with vertical ½-inch fillet welds on both sides of each plate. The weld is composed of E70 electrodes.

$$\begin{aligned} l_w &= 6(21 \text{ inches}) = 126 \text{ inches} \\ a_w &= 0.707(w_{weld}) = 0.707(0.5") = 0.354 \text{ inches} \\ \phi R_n &= 0.6\phi_{e2} F_{exx} l_w a_w = 0.6(0.80)(70)(126)(0.354) \\ &= 1497 \text{ kip} \end{aligned}$$

BDS Article 6.13.3.2.4 & 6.5.4.2

It can be seen that the capacity of the bracket exceeds the shear capacity of the girder; therefore, the girder will control the ratings.



Source: FHWA

Figure 35. Illustration. Bracket Details.

3.2.4 LRFR Rating Calculations

The structural condition of the steel 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

Resistance factors are based on LRFD BDS Article 6.5.4.2:

Flexure resistance factor:	ϕ_f	=	1.00	LRFD BDS 6.5.4.2
Shear resistance factor:	ϕ_v	=	1.00	LRFD BDS 6.5.4.2

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.

$$RF = \frac{C \pm \gamma_{DC} DC \pm \gamma_{EV} EV}{\gamma_{LL}(LL + IM)}$$ 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 47 shows the results of the overall capacity based on MBE Equation 6A.4.2.1-2 as well as the rating factors based on MBE Equation 6A.4.2.1-1. Maximum and minimum load factors are selectively used to obtain the maximum absolute value of the force effects. The load factors used are shown in Table 46. Moments, shears and their corresponding capacities in Table 47 are in kip-ft and kip, respectively.

Table 46. Load Factors.

DC	Earth Load, EV	HL-93 LL Inventory	HL-93 LL Operating	Emergency Vehicle, EV LL Legal
1.25	1.35	1.75	1.35	2.00

Table 47. Girder Resistance and Rating Factors.

Force	Rating	DC	Earth Load, EV	LL	C	RF
Moment	HL-93 Inventory	586	1404	535	6783	4.44
Moment	HL-93 Operating	586	1404	535	6783	5.75
Moment	EV-2	586	1404	300	6783	6.92
Moment	EV-3	586	1404	457	6783	4.55
Shear	HL-93 Inventory	39	92	37	857	10.57
Shear	HL-93 Operating	39	92	37	857	13.70
Shear	EV-2	39	92	22	857	15.56
Shear	EV-3	39	92	32	857	10.70
Bracket	HL-93 Inventory	39	92	37	1497	20.46
Bracket	HL-93 Operating	39	92	37	1497	26.52
Bracket	EV-2	39	92	22	1497	30.10
Bracket	EV-3	39	92	32	1497	20.70

REFERENCES

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

AASHTO. (2017b). *LRFD Road Tunnel Design and Construction Guide Specifications*, 1st Edition, LRFDTUN-1. American Association of State Highway and Transportation Officials, Washington, DC.

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

FHWA. (2004). *FHWA Road Tunnel Design Guidelines,* FHWA-IF-05-023. Federal Highway Administration, Washington, DC.

FHWA. (2009). *Technical Manual for Design and Construction of Road Tunnel—Civil Elements,* FHWA-NHI-10-034. Federal Highway Administration, Washington, DC.

FHWA. (2013). *Information: Load Rating of Specialized Hauling Vehicles*. Federal Highway Administration, Washington, DC.

FHWA. (2015a). *Specifications for the National Tunnel Inventory,* FHWA-HIF-15-006. Federal Highway Administration, Washington, DC.

FHWA. (2015b). *Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual,* FHWA-HIF-15-005. Federal Highway Administration, Washington, DC.

FHWA. (2016). *Memorandum: Load Rating for the FAST Act's Emergency Vehicles*. Federal Highway Administration, Washington, DC.

FHWA. (2019). *Reference Guide for Load Rating of Tunnel Structures*, FHWA-HIF-19-010. Federal Highway Administration, Washington, DC.

Los Angeles County Metropolitan Transit Authority (Metro). (2019). *Westside Purple Line Extension,* Section 3, Structure Impact Assessment Report for Tunnels and Cross Passages. Prepared by ALDEA, November 26.

Plaxis. (2019). *Plaxis 2D Reference Manual.* Plaxis BV. The Netherlands.