LRFD Bridge Design

AASHTO LRFD Bridge Design Specifications

Loading and General Information

Created July 2007

LRFD Bridge Design

AASHTO LRFD Bridge Design Specification

Loads and General Information

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AASHTO LRFD Bridge Design Specifications

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Richard A Miller

- **AASHTO-LRFD Specification, 4th Ed., 2007**

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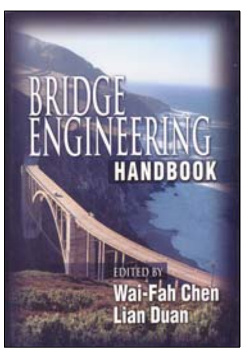

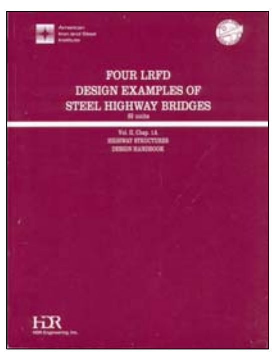

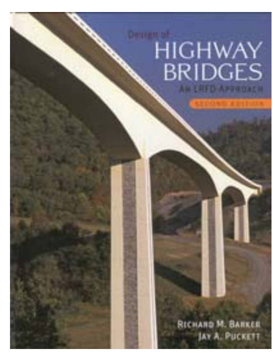

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- "Four LRFD Design Examples of Steel Highway Bridges," Vol. II, Chapter 1A Highway Structures Design Handbook, Published by American Iron and Steel Institute in cooperation with HDR Engineering, Inc. Available at http://www.aisc.org/
- "Design of Highway Bridges," Richard Barker and Jay Puckett, 1977, Wiley & Sons (0-471-30434-4)

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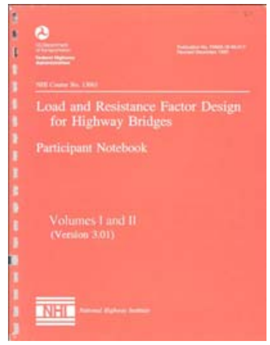

- AASHTO Web Site: http://bridges.transportation.org/
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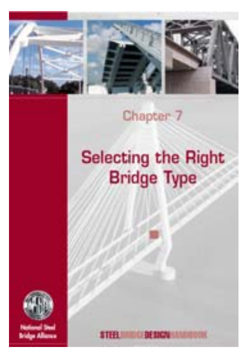
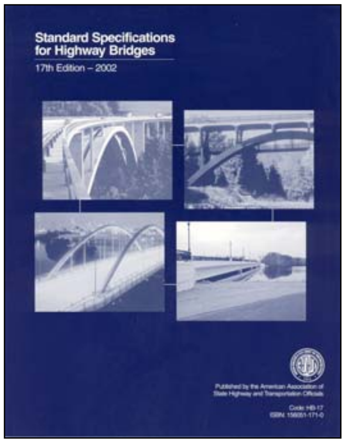
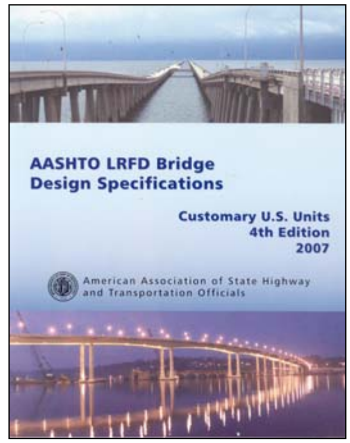


- AISC / National Steel Bridge Alliance Web Site: [http://www.steelbridges. org/](http://www.steelbridges.org/)
- "Steel Bridge Design Handbook"

References





- “AASHTO Standard Specification for Highway Bridges,” 17th Edition, 1997, 2003
- “AASHTO LRFD Bridge Design Specifications,” 4th Edition, 2007
- “AASHTO Guide Specification for Distribution of Loads for Highway Bridges”

Philosophies of Design

- **ASD** - Allowable Stress Design
- **LFD** - Load Factor Design
- **LRFD** - Load and Resistance Factor Design

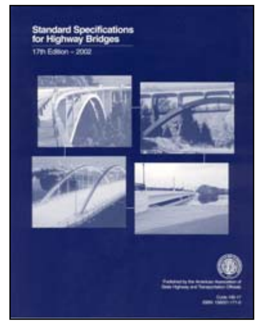
Philosophies of Design

ASD: Allowable Stress Design

- **For Safety:**

$$f \le F_A = \frac{F_y}{F.S.}$$

  - *f - computed stress*
  - *F_A - Allowable Stress*



- **In terms of bending moment…**

$$\frac{\sum M}{S} \le \frac{F_y}{1.82}$$

ASD does not recognize different variabilities of different load types.

Philosophies of Design

LFD: Load Factor Design

- **For Safety:**

$$\sum \gamma Q \le R_n$$

  - Q - Load Effect
  - R - Component Resistance
  - γ - Load Factor

- **In terms of bending moment…**

$$1.30M_D + 2.17M_{(L+I)} \le \phi M_n$$

  - ϕ - Strength Reduction Factor

In LFD, load and resistance are not considered simultaneously.

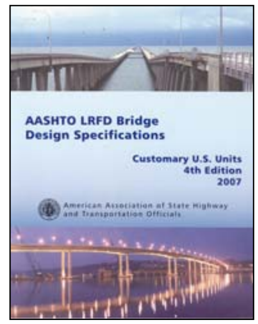
Philosophies of Design

LRFD: Load & Resistance Factor Design

- **For Safety:**

$$\sum \gamma Q \leq \phi R_n$$

- Q - Load Effect
- R - Component Resistance
- γ - Load Factor
- ϕ - Resistance Factor



The LRFD philosophy provides a more uniform, systematic, and rational approach to the selection of load factors and resistance factors than LFD.

Philosophies of Design - LRFD Fundamentals

Variability of Loads and Resistances:

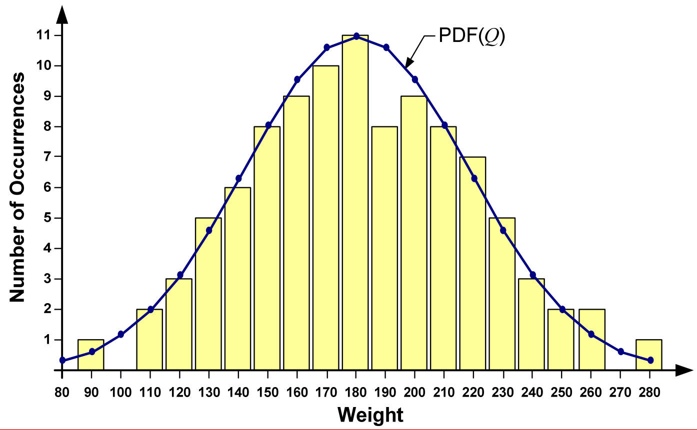
- **Suppose that we measure the weight of 100 students…**

Weight	Number of Samples	Weight	Number of Samples
70	0	180	11
80	0	190	8
90	1	200	9
100	0	210	8
110	2	220	7
120	3	230	5
130	5	240	3
140	6	250	2
150	8	260	2
160	9	270	0
170	10	280	1

Average = 180^{lbs} St Deviation = 38^{lbs}

Philosophies of Design - LRFD Fundamentals

Variability of Loads and Resistances:



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Philosophies of Design - LRFD Fundamentals

Variability of Loads and Resistances:

- **Now suppose that we measure the strength of 100 ropes…**

Weight	Number of Samples	Weight	Number of Samples
210	0	320	15
220	0	330	14
230	0	340	11
240	0	350	8
250	1	360	5
260	1	370	3
270	3	380	2
280	5	390	0
290	7	400	1
300	11	410	0
310	13	420	0

Average = 320^{lbs} St Deviation = 28^{lbs}

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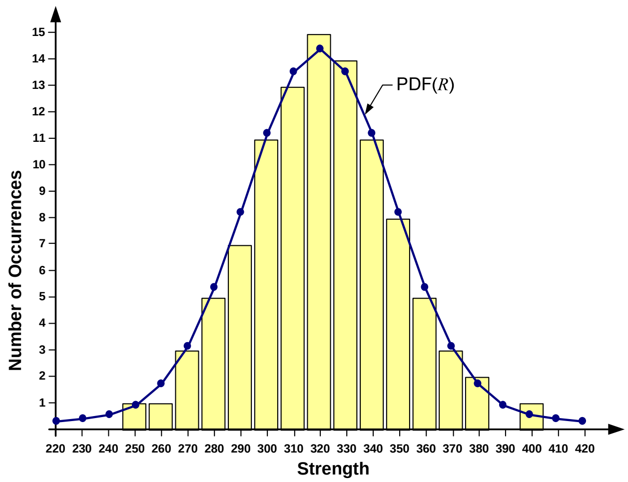
Variability of Loads and Resistances:



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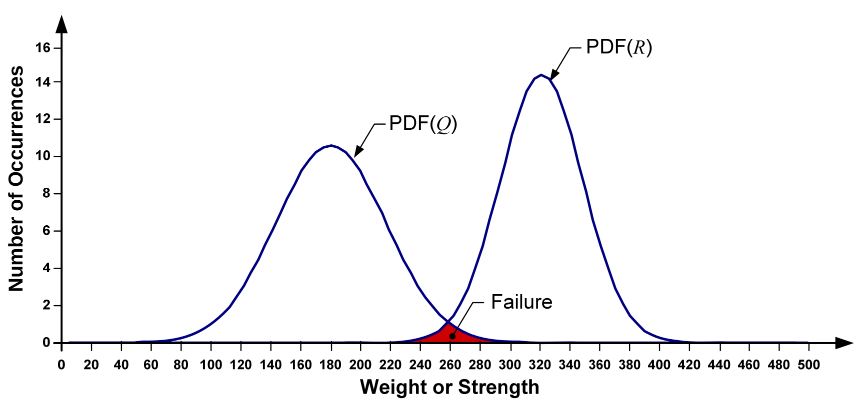
Variability of Loads and Resistances:



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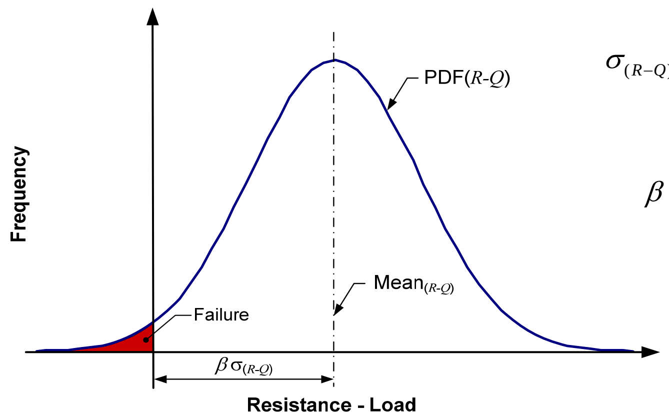
Variability of Loads and Resistances:



$$\sigma_{(R-Q)} = \sqrt{\sigma_R{}^2 + \sigma_Q{}^2}$$

$$\beta = \frac{\text{Mean}_{(R-Q)}}{\sigma_{(R-Q)}}$$

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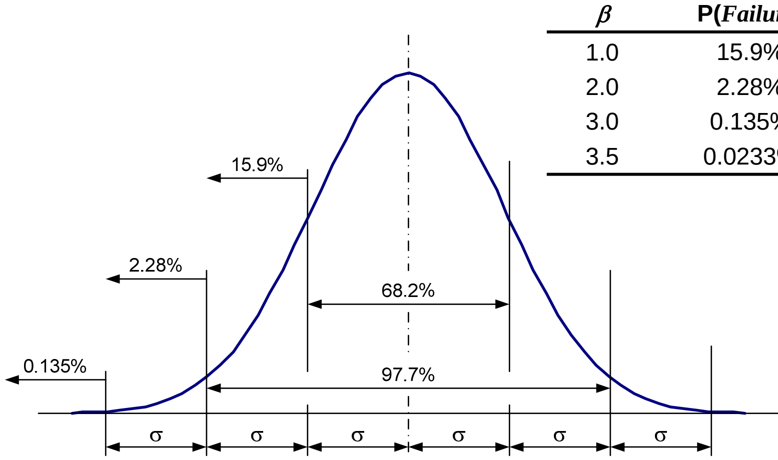
Philosophies of Design - LRFD Fundamentals

Reliability Index:

β	P(*Failure*)
1.0	15.9%
2.0	2.28%
3.0	0.135%
3.5	0.0233%



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Reliability Index:

- **AISC:**

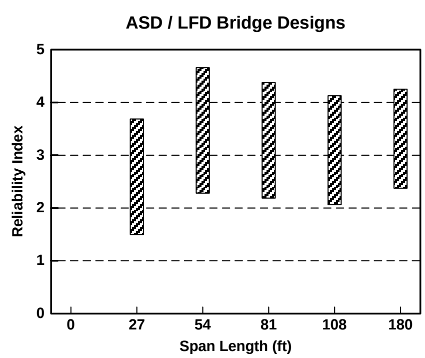
β	D+(L or S)	D+L+W	D+L+E
Members	3.0	2.5	1.75
Connections	4.5	4.5	4.5

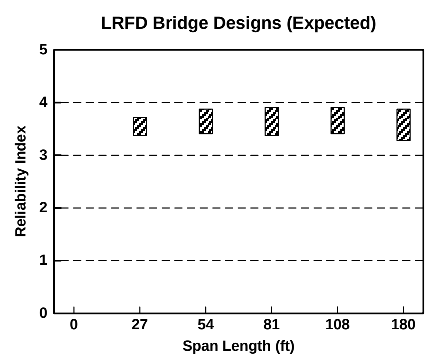
- **AASHTO:**

β= 3.5 Super/Sub Structures
β= 2.5 Foundations

Philosophies of Design - LRFD Fundamentals

Reliability Index:





Chen & Duan

Philosophies of Design - LRFD Fundamentals

- **Resistance Factor:**

$$\phi = \frac{R_m}{R_n} e^{[-0.55\beta \ \mathrm{COV}(R_m)]}$$

  - R_m - Mean Value of R (from experiments)
  - R_n - Nominal Value of R
  - β - Reliability Index
  - COV(R_m) - Coeff. of Variation of R

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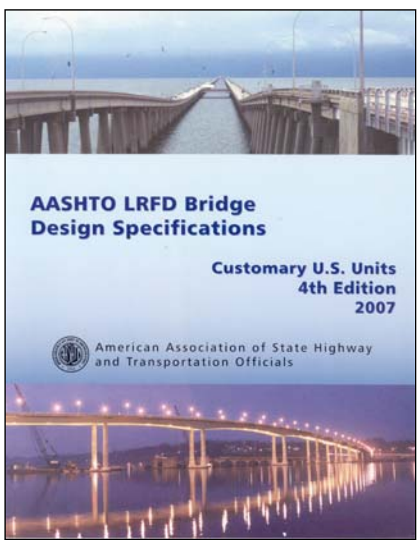

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AASHTO-LRFD
Chapter 1: Introduction

- AASHTO-LRFD Specification, 4th, 2007

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Chapter 1 – Introduction

§1.3.2: Limit States

- **Service:**
  - Deals with restrictions on stress, deformation, and crack width under regular service conditions.
  - Intended to ensure that the bridge performs acceptably during its design life.
- **Strength:**
  - Intended to ensure that strength and stability are provided to resist statistically significant load combinations that a bridge will experience during its design life.
  - Extensive distress and structural damage may occur at strength limit state conditions, but overall structural integrity is expected to be maintained.
- **Extreme Event:**
  - Intended to ensure structural survival of a bridge during an earthquake, vehicle collision, ice flow, or foundation scour.
- **Fatigue:**
  - Deals with restrictions on stress range under regular service conditions reflecting the number of expected cycles.

Pg 1.4-5; Chen & Duan

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Chapter 1 – Introduction

§1.3.2: Limit States

$$Q = \sum \eta_i \gamma_i Q_i \quad \textbf{(1.3.2.1-1)}$$

γ_i - Load Factor
Q_i - Load Effect
η_i - Load Modifier

When the maximum value of γ_i is appropriate

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \quad \textbf{(1.3.2.1-2)}$$

When the minimum value of γi is appropriate

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.00 \quad \textbf{(1.3.2.1-3)}$$

Pg 1.3 AASHTO-LRFD 2007

Chapter 1 – Introduction

§1.3.2: Limit States - Load Modifiers

Applicable only to the Strength Limit State

- η_D – Ductility Factor:
  - η_D = 1.05 for nonductile members
  - η_D = 1.00 for conventional designs and details complying with specifications
  - η_D = 0.95 for components for which additional ductility measures have been taken
- η_R – Redundancy Factor:
  - η_R = 1.05 for nonredundant members
  - η_R = 1.00 for conventional levels of redundancy
  - η_R = 0.95 for exceptional levels of redundancy
- η_I – Operational Importance:
  - η_I = 1.05 for important bridges
  - η_I = 1.00 for typical bridges
  - η_I = 0.95 for relatively less important bridges

These modifiers are applied at the element level, not the entire structure.

Pgs. 1.5-7; Chen & Duan AASHTO-LRFD 2007

§ 3.4 - Load Factors and Combinations

§1.3.2: ODOT Recommended Load Modifiers

For the Strength Limit States

- η_D – Ductility Factor:
  - Use a ductility load modifier of η_D = 1.00 for all strength limit states
- η_R – Redundancy Factor:
  - Use η_R = 1.05 for "non-redundant" members
  - Use η_R = 1.00 for "redundant" members
  - Bridges with 3 or fewer girders should be considered "non-redundant."
  - Bridges with 4 girders with a spacing of 12' or more should be considered "non-redundant."
  - Bridges with 4 girders with a spacing of less than 12' should be considered "redundant."
  - Bridge with 5 or more girders should be considered "redundant."

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§ 3.4 - Load Factors and Combinations

§1.3.2: ODOT Recommended Load Modifiers

For the Strength Limit States

- η_R – Redundancy Factor:
  - Use η_R = 1.05 for "non-redundant" members
  - Use η_R = 1.00 for "redundant" members
  - Single and two column piers should be considered non-redundant.
  - Cap and column piers with three or more columns should be considered redundant.
  - T-type piers with a stem height to width ratio of 3-1 or greater should be considered non-redundant.
  - For information on other substructure types, refer to NCHRP Report 458 *Redundancy in Highway Bridge Substructures.*
  - η_R does NOT apply to foundations. Foundation redundancy is included in the resistance factor.

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§1.3.2: ODOT Recommended Load Modifiers

For the Strength Limit States

- η_I – Operational Importance:
  - In General, use η_I = 1.00 unless one of the following applies
  - Use η_I = 1.05 if any of the following apply
    - Design ADT ≥ 60,000
    - Detour length ≥ 50 miles
    - Any span length ≥ 500’
  - Use η_I = 0.95 if both of the following apply
    - Design ADT ≤ 400
    - Detour length ≤ 10 miles

Detour length applies to the shortest, emergency detour route.

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Chapter 2: General Design and Location Features

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Chapter 2 – General Design and Location Features

Contents

§ 2.5.2 - Serviceability

§2.5.2.6.2 Criteria for Deflection

ODOT requires the use of Article 2.5.2.6.2 and 2.5.2.6.3 for limiting deflections of structures.

ODOT prohibits the use of "the stiffness contribution of railings, sidewalks and median barriers in the design of the composite section."

§ 2.5.2 - Serviceability

§2.5.2.6.2 Criteria for Deflection

- **Principles which apply**
  - When investigating absolute deflection, load all lanes and assume all components deflect equally.
  - When investigating relative deflection, choose the number and position of loaded lanes to maximize the effect.
  - The live load portion of Load Combination Service I (plus impact) should be used.
  - The live load is taken from Article 3.6.1.1.2 (covered later).
  - For skewed bridges, a right cross-section may be used, for curved bridges, a radial cross section may be used.

ODOT prohibits the use of "the stiffness contribution of railings, sidewalks and median barriers in the design of the composite section."

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§ 2.5.2 - Serviceability

§2.5.2.6.2 Criteria for Deflection

In the absence of other criteria, these limits may be applied to steel, aluminum and/or concrete bridges:

Load	Limit
General vehicular load	Span/800
Vehicular and/or pedestrian load	Span/1000
Vehicular load on cantilever arms	Span/300
Vehicular and/or pedestrian load on cantilever arms	Span/375

For steel I girders/beams, the provisions of Arts. 6.10.4.2 and 6.11.4 regarding control of deflection through flange stress controls shall apply.

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§ 2.5.2 - Serviceability

§2.5.2.6.2 Criteria for Deflection

For wood construction:

Load	Limit
Vehicular and pedestrian loads	Span/425
Vehicular loads on wood planks and panels: extreme relative deflection between adjacent edges	0.10 in

Pg 2.10-14

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§ 2.5.2 - Serviceability

§2.5.2.6.2 Criteria for Deflection

For orthotropic plate decks:

Load	Limit
Vehicular loads on deck plates	Span/300
Vehicular loads on ribs of orthotropic metal decks	Span/1000
Vehicular loads on ribs of orthotropic metal decks: extreme relative deflection between adjacent ribs	0.10 in

Pg 2.10-14

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§ 2.5.2 - Serviceability

§2.5.2.6.3 Optional Criteria for Span-to-Depth ratios

Table 2.5.2.6.3-1 Traditional Minimum Depths for Constant Depth Superstructures

Superstructure		Minimum Depth (Including Deck) When variable depth members are used, values may be adjusted to account for changes in relative stiffness of positive and negative moment sections	
Material	Type	Simple Spans	Continuous Spans
Reinforced concrete	Slabs with main reinforcement parallel to traffic	$\frac{1.2(S+10)}{30}$	$\frac{S+10}{30} \geq 0.54\ ft.$
	T-Beams	$0.070L$	$0.065L$
	Box Beams	$0.060L$	$0.055L$
	Pedestrian Structure Beams	$0.035L$	$0.033L$
Prestressed Concrete	Slabs	$0.030L \geq 6.5\ in.$	$0.027L \geq 6.5\ in.$
	CIP Box Beams	$0.045L$	$0.040L$
	Precast I-Beams	$0.045L$	$0.040L$
	Pedestrian Structure Beams	$0.033L$	$0.030L$
	Adjacent Box Beams	$0.030L$	$0.025L$
Steel	Overall Depth of Composite I-Beam	$0.040L$	$0.032L$
	Depth of I-Beam Portion of Composite I-Beam	$0.033L$	0.027
	Trusses	$0.100L$	$0.100L$

ODOT states that "designers shall apply the span-to-depth ratios shown."

Pg 2.10-14 **AASHTO-LRFD 2007**

AASHTO-LRFD
Bridge Design Specification
Section 3: Loads and Load Factors

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§ 3.4 - Loads and Load Factors

§3.4.1: Load Factors and Load Combinations

Permanent Loads

- *DD* - Downdrag
- *DC* - Structural Components and Attachments
- *DW* - Wearing Surfaces and Utilities
- *EH* - Horizontal Earth Pressure
- *EL* - Locked-In Force Effects Including Pretension
- *ES* - Earth Surcharge Load
- *EV* - Vertical Pressure of Earth Fill

§ 3.4 - Loads and Load Factors

§3.4.1: Load Factors and Load Combinations

Transient Loads

- *BR* – Veh. Braking Force
- *CE* – Veh. Centrifugal Force
- *CR* - Creep
- *CT* - Veh. Collision Force
- *CV* - Vessel Collision Force
- *EQ* - Earthquake
- *FR* - Friction
- *IC* - Ice Load
- *LL* - Veh. Live Load
- *IM* - Dynamic Load Allowance
- *LS* - Live Load Surcharge
- *PL* - Pedestrian Live Load
- *SE* - Settlement
- *SH* - Shrinkage
- *TG* - Temperature Gradient
- *TU* - Uniform Temperature
- *WA* - Water Load
- *WL* - Wind on Live Load
- *WS* - Wind Load on Structure

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§ 3.4 - Loads and Load Factors

§3.4.1: Load Factors and Load Combinations

Table 3.4.1-1 Load Combinations and Load Factors

Load Combination	*DC DD DW EH EV ES EL*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU CR SH*	*TG*	*SE*	Use One of These at a Time *EQ*	*IC*	*CT*	*CV*
STRENGTH I (unless noted)	γ_p	1.75	1.00	--	--	1.00	0.50/1.20	γ_{TG}	γ_{SE}	--	--	--	--
STRENGTH II	γ_p	1.35	1.00	--	--	1.00	0.50/1.20	γ_{TG}	γ_{SE}	--	--	--	--
STRENGTH III	γ_p		1.00	1.40	--	1.00	0.50/1.20	γ_{TG}	γ_{SE}	--	--	--	--
STRENGTH IV	γ_p		1.00	--	--	1.00	0.50/1.20	--	--	--	--	--	--
STRENGTH V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	--	--	--	--

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§3.4.1: Load Factors and Load Combinations

Table 3.4.1-1 Load Combinations and Load Factors (cont.)

Load Combination	*DC DD DW EH EV ES EL*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU CR SH*	*TG*	*SE*	Use One of These at a Time *EQ*	*IC*	*CT*	*CV*
EXTREME EVENT I	γ_p	γ_{EQ}	1.00	--	--	1.00	--	--	--	1.00	--	--	--
EXTREME EVENT II	γ_p	0.50	1.00	--	--	1.00	--	--	--	--	1.00	1.00	1.00
FATIGUE – *LL, IM, & CE ONLY*	--	0.75	--	--	--	--	--	--	--	--	--	--	--

§3.4.1: Load Factors and Load Combinations

Table 3.4.1-1 Load Combinations and Load Factors (cont.)

Load Combination	*DC DD DW EH EV ES EL*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU CR SH*	*TG*	*SE*	Use One of These at a Time *EQ*	*IC*	*CT*	*CV*
SERVICE I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	--	--	--	--
SERVICE II	1.00	1.30	1.00	--	--	1.00	1.00/1.20	--	--	--	--	--	--
SERVICE III	1.00	0.80	1.00	--	--	1.00	1.00/1.20	γ_{TG}	γ_{SE}	--	--	--	--
SERVICE IV	1.00	--	1.00	0.70	--	1.00	1.00/1.20	--	1.0	--	--	--	--

§3.4.1: Load Factors and Load Combinations

- **Strength I:** Basic load combination relating to the normal vehicular use of the bridge without wind.
- **Strength II:** Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both, without wind.
- **Strength III:** Load combination relating to the bridge exposed to wind in excess of 55 mph.
- **Strength IV:** Load combination relating to very high dead load to live load force effect ratios. (Note: In commentary it indicates that this will govern where the DL/LL >7, spans over 600', and during construction checks.)
- **Strength V:** Load combination relating to normal vehicular use with a wind of 55 mph.

§3.4.1: Load Factors and Load Combinations

- **Extreme Event I:** Load combination including earthquakes.
- **Extreme Event II:** Load combination relating to ice load, collision by vessels and vehicles, and certain hydraulic events with a reduced live load.
- **Fatigue:** Fatigue and fracture load combination relating to repetitive gravitational vehicular live load and dynamic responses under a single design truck.

§ 3.4 - Loads and Load Factors

§3.4.1: Load Factors and Load Combinations

- **Service I:** Load combination relating to normal operational use of the bridge with a 55 mph wind and all loads at nominal values. Compression in precast concrete components.
- **Service II:** Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular load.
- **Service III:** Load combination relating only to tension in prestressed concrete superstructures with the objective of crack control.
- **Service IV:** Load combination relating only to tension in prestressed concrete columns with the objective of crack control.

§ 3.4 - Loads and Load Factors

§3.4.1: Load Factors and Load Combinations

Table 3.4.1-2 Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor Maximum	Load Factor Minimum
DC: Component and Attachments		1.25	0.90
***DC*: Strength IV only**		**1.50**	**0.90**
DD: Downdrag	**Piles, αTomlinson Method**	**1.4**	**0.25**
	Plies, λ Method	**1.05**	**0.30**
	Drilled Shafts, O'Neill and Reese (*1999*) Method	**1.25**	**0.35**
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
EL: Locked in Erections Stresses		1.00	1.00

§ 3.4 - Loads and Load Factors

§3.4.1: Load Factors and Load Combinations

- **An important note about γ_p: The purpose of γ_p is to account for the fact that sometimes certain loads work opposite to other loads.**
  - If the load being considered works in a direction to increase the critical response, the maximum γ_p is used.
  - If the load being considered would decrease the maximum response, the minimum γ_p is used.
- **The minimum value of γ_p is used when the permanent load would increase stability or load carrying capacity**

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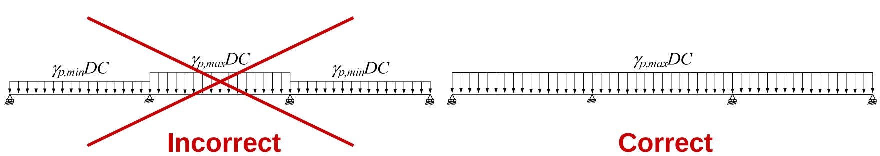
§ 3.4 - Loads and Load Factors

§3.4.1: Load Factors and Load Combinations

- **Sometimes, a permanent load both contributes to and mitigates a critical load effect.**
- For example, in the three span continuous bridge shown, DC in the first and third spans would mitigate the positive moment in the middle span. However, it would be incorrect to use a different γ_p for the two end spans. In this case, γ_p would be 1.25 for DC for all three spans (Commentary C3.4.1 – paragraph 20).



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§3.4.1: Load Factors and Load Combinations

- **Table 3.4.1-1 "Load Combinations and Load Factors" gives two separate values for the load factor for *TU* (uniform temperature), *CR* (creep), and *SH* (shrinkage). The larger value is used for deformations. The smaller value is used for all other effects.**

- ***TG* (temperature gradient), γ_{TG} should be determined on a project-specific basis. In lieu of project-specific information to the contrary, the following values may be used:**

  - 0.0 for strength and extreme event limit states,
  - 1.0 for service limit state where live load is NOT considered,
  - 0.5 for service limit state where live load is considered.

§3.4.1: Load Factors and Load Combinations

- **For *SE* (settlement), γ_{SE} should be based on project specific information. In lieu of project specific information, γ_{SE} may be taken as 1.0.**
  - Load combinations which include settlement shall also be applied without settlement.

- **The load factor for live load in Extreme Event I, γ_{EQ}, shall be determined on a project specific basis.**

ODOT Exception: Assume that the Extreme Event I Load Factor for Live Load is Equal to 0.0. (γ_{EQ} = 0.0)

§3.4.1: Load Factors and Load Combinations

- **When prestressed components are used in conjunction with steel girders, the following effects shall be considered as construction loads (*EL*):**
  - If a deck is prestressed BEFORE being made composite, the friction between the deck and the girders.
  - If the deck is prestressed AFTER being made composite, the additional forces induced in the girders and shear connectors.
  - Effects of differential creep and shrinkage.
  - Poisson effect.

§3.4.2: Load Factors for Construction Loads

At the Strength Limit State Under Construction Loads:

- For Strength Load Combinations I, III and V, the factors for *DC* and *DW* shall not be less than 1.25.
- For Strength Load Combination I, the load factor for construction loads and any associated dynamic effects shall not be less than 1.5.
- For Strength Load Combination III, the load factor for wind shall not be less than 1.25.

§3.4.3: Load Factors for Jacking and Post-Tensioning Forces

- **Jacking Forces**
  - The design forces for in-service jacking shall be not less than 1.3 times the permanent load reaction at the bearing adjacent to the point of jacking (unless otherwise specified by the Owner).
  - The live load reaction must also consider maintenance of traffic if the bridge is not closed during the jacking operation.

- **PT Anchorage Zones**
  - The design force for PT anchorage zones shall be 1.2 times the maximum jacking force.

Common Load Combinations for Steel Design

- **Strength I:** $1.25DC + 1.50DW + 1.75(LL+IM)$
- **Service II:** $1.00DC + 1.00DW + 1.30(LL+IM)$
- **Fatigue:** $0.75(LL+IM)$

§ 3.4 - Loads and Load Factors

Common Load Combinations for Prestressed Concrete

- **Strength I:** $1.25DC + 1.50DW + 1.75(LL+IM)$
- **Strength IV:** $1.50DC + 1.50DW$
- **Service I:** $1.00DC + 1.00DW + 1.00(LL+IM)$
- **Service III:** $1.00DC + 1.00DW + 0.80(LL+IM)$
- **Service IV:** $1.00DC + 1.00DW + 1.00WA + 0.70WS + 1.00FR$
- **Fatigue:** $0.75(LL+IM)$

Note: Fatigue rarely controls for prestressed concrete

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§ 3.4 - Loads and Load Factors

Common Load Combinations for Reinforced Concrete

- **Strength I:** $1.25DC + 1.50DW + 1.75(LL+IM)$
- **Strength IV:** $1.50DC + 1.50DW$
- **Fatigue:** $0.75(LL+IM)$

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§3.5.1 Dead Loads: *DC*, *DW*, and *EV*

- ***DC*** **is the dead load of the structure and components present at construction. These have a lower load factor because they are known with more certainty.**
- ***DW*** **are future dead loads, such as future wearing surfaces. These have a higher load factor because they are known with less certainty.**
- ***EV*** **is the vertical component of earth fill.**
- **Table 3.5.1-1 gives unit weight of typical components which may be used to calculate DC, DW and EV.**

§3.5.1 Dead Loads: *DC*, *DW*, and *EV*

- ***DC*** **is the dead load of the structure and components present at construction. These have a lower load factor because they are known with more certainty.**
- ***DW*** **are future dead loads, such as future wearing surfaces. These have a higher load factor because they are known with less certainty.**
- ***EV*** **is the vertical component of earth fill.**
- **Table 3.5.1-1 gives unit weight of typical components which may be used to calculate *DC*, *DW* and *EV*.**

§ 3.5 – Permanent Loads

§3.5.1 Dead Loads: *DC*, *DW*, and *EV*

If a beam slab bridge meets the requirements of Article 4.6.2.2.1, then the permanent loads of and on the deck may be distributed uniformly among the beams and/or stringers.

Article 4.6.2.2.1 basically lays out the conditions under which approximate distribution factors for live load can be used.

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§ 3.6 - Live Loads

§3.6.1.1.1: Lane Definitions

- **# Design Lanes** = **INT(w/12.0 ft)**
  - *w* is the clear roadway width between barriers.
- **Bridges 20 to 24 ft wide shall be designed for two traffic lanes, each ½ the roadway width.**
- **Examples:**
  - A 20 ft. wide bridge would be required to be designed as a two lane bridge with 10 ft. lanes.
  - A 38 ft. wide bridge has 3 design lanes, each 12 ft. wide.
  - A 16 ft. wide bridge has one design lane of 12 ft.

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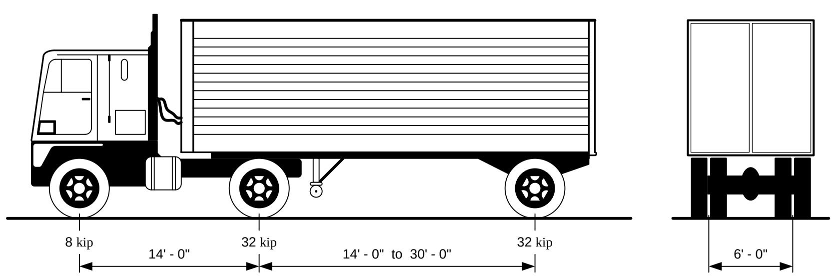
§ 3.6 - Live Loads

§3.6.1.3.1: Application of Design Vehicular Loads

- **The governing force effect shall be taken as the larger of the following:**
  - The effect of the design tandem combined with the design lane load
  - The effect of one design truck (HL-93) combined with the effect of the design lane load
  - For negative moment between inflection points, 90% of the effect of two design trucks (HL-93 with 14 ft. axle spacing) spaced at a minimum of 50 ft. combined with 90% of the design lane load.

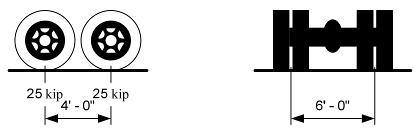
§ 3.6 - Live Loads

§3.6.1.2.2: Design Truck



§ 3.6 - Live Loads

§3.6.1.2.3: Design Tandem



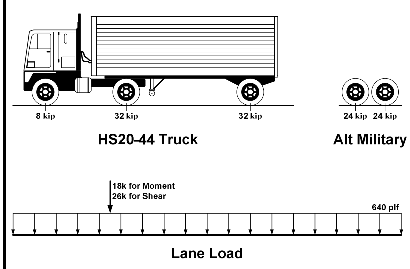
§ 3.6 - Live Loads

§3.6.1.2.4: Design Lane Load

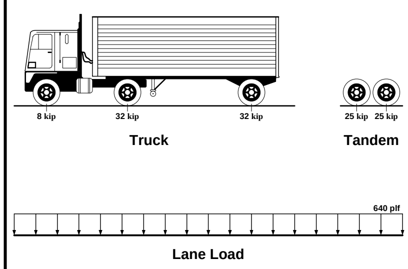
- **0.640$^{kip}/_{ft}$ is applied <u>SIMULTANEOUSLY</u> with the design truck or design tandem over a width of 10 ft. within the design lane.**
- **NOTE: the impact factor, *IM*, is NOT applied to the lane load. It is only applied to the truck or tandem load.**
- **This is a big change from the *Standard Specifications…***

§ 3.6 - Live Loads

AASHTO Standard Spec vs LRFD Spec:



Old Std Spec Loading:
- HS20 Truck, or
- Alternate Military, or
- Lane Load



New LRFD Loading:
- HL-93 Truck <u>and</u> Lane Load, or
- Tandem <u>and</u> Lane Load, or
- 90% of 2 Trucks <u>and</u> Lane Load

§ 3.6 - Live Loads

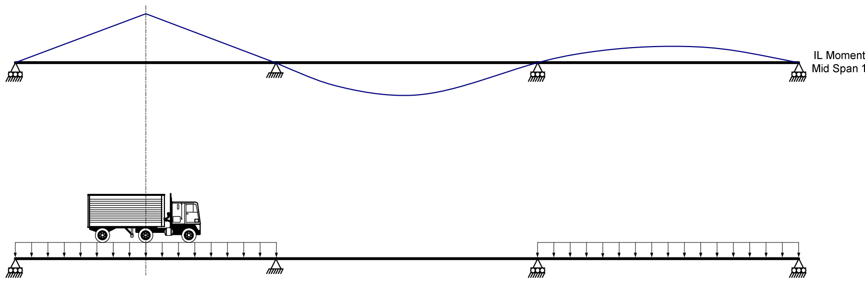
§3.6.1.3.1: Application of Design Vehicular Loads

- **The lane load is applied, without impact, to any span, or part of a span, as needed to maximize the critical response.**
- **A single truck, with impact, is applied as needed to maximize the critical response (except for the case of negative moment between inflection points).**
  - The Specification calls for a single truck to be applied, regardless of the number of spans.
  - The exception is for the case of negative moment between inflection points where 2 trucks are used.
- **If an axle or axles do not contribute to the critical response, they are ignored.**

Pg 3.25

§ 3.6 - Live Loads

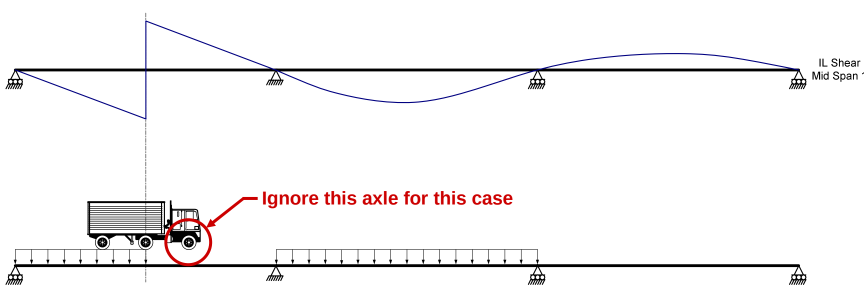
Live Loads for Maximum Positive Moment in Span 1



- The impact factor is applied only to the truck, not the lane load
- Although a truck in the third span would contribute to maximum response, by specification only <u>one</u> truck is used.

§ 3.6 - Live Loads

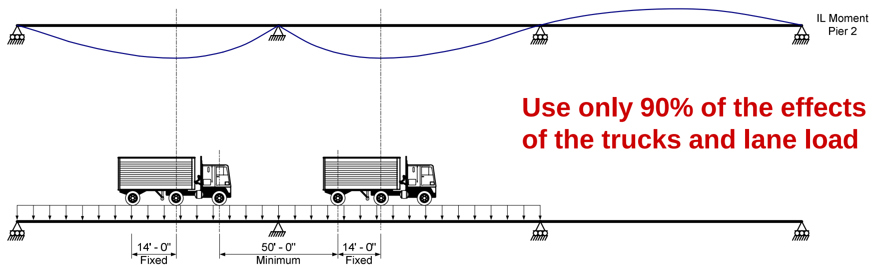
Live Loads for Shear at Middle of Span 1



- Impact is applied only to the truck.
- In this case, the front axle is ignored as it does not contribute to the maximum response.

§ 3.6 - Live Loads

Live Loads for Maximum Moment Over Pier 1



Use only 90% of the effects of the trucks and lane load

- **Impact is applied to the trucks only.**
- **The distance between rear axles is fixed at 14 ft.**
- **The distance between trucks is a minimum of 50 ft.**

- **This applies for negative moment between points of contraflexure and reactions at interior piers**

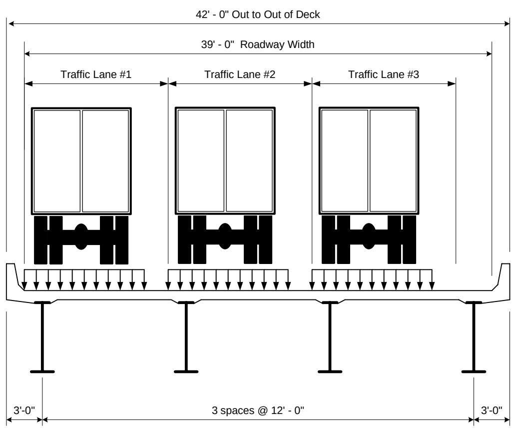
§ 3.6 - Live Loads

§3.6.1.3: Application of Design Vehicular Live Loads

- **In cases where the transverse position of the load must be considered:**
  - The design lanes are positioned to produce the extreme force effect.
  - The design lane load is considered to be 10 ft. wide. The load is positioned to maximize the extreme force effect.
  - The truck/tandem is positioned such that the center of any wheel load is not closer than:
    - 1.0 ft. from the face of the curb/railing for design of the deck overhang.
    - 2.0 ft. from the edge of the design lane for design of all other components.

§ 3.6 - Live Loads

Both the Design Lanes and 10' Loaded Width in each lane shall be positioned to produce extreme force effects.



- Center of truck wheels must be at least 2' from the edge of a design lane
- The lane load may be at the edge of a design lane.

§ 3.6 - Live Loads

§3.6.1.3.3: Design Loads for Decks, Deck Systems, and the Top Slabs of Box Culverts

When the Approximate Strip Method is Used:

- **Where the slab spans primarily in the transverse direction:**
  - only the axles of the design truck or design tandem of shall be applied to the deck slab or the top slab of box culverts.
- **Where the slab spans primarily in the longitudinal direction:**
  - For top slabs of box culverts of all spans and for all other cases (including slab-type bridges where the span does not exceed 15.0 ft.) only the axle loads of the design truck or design tandem shall be applied.
  - For all other cases (including slab-type bridges where the span exceeds 15.0 ft.) the entire HL-93 loading shall be applied.

§ 3.6 - Live Loads

§3.6.1.3.3: Design Loads for Decks, Deck Systems, and the Top Slabs of Box Culverts

When Refined Methods of Analysis are Used:

- **Where the slab spans primarily in the transverse direction**
  - only the axles of the design truck or design tandem shall be applied to the deck slab.
- **Where the slab spans primarily in the longitudinal direction (including slab-type bridges)**
  - the entire HL-93 loading shall be applied.

Centrifugal and Braking Forces need not be considered for deck design.

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§ 3.6 - Live Loads

§3.6.1.3.4: Deck Overhang Load

For design of a deck overhang with a cantilever $\leq$ 6 ft. measured from the centerline of the exterior girder to the face of a structurally continuous concrete railing…

…the outside row of wheel loads may be replaced by a 1.0 klf line load located 1 ft. from the face of the railing. (Article 3.6.1.3.4)

ODOT Exception!!! This method is not permitted!!! Deck overhangs are designed according to Section 302.2.2 in the ODOT Bridge Design Manual.

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§ 3.6 - Live Loads

§3.6.2.2: Buried Components

The dynamic load allowance for culverts and other buried structures covered by Section 12, in percent shall be taken as:

$$IM = 33(1.0\text{-}0.125D_E) \geq 0\% \qquad \text{(4.6.2.2.1-1)}$$

where : D_E = minimum depth of earth cover above the structure (ft.)

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§ 3.6 - Live Loads

§3.6.1.1.2: Multiple Presence of Live Load

- **Multiple Presence Factor**

# of Loaded Lanes	*MP Factor*
1	*1.20*
2	*1.00*
3	*0.85*
>3	*0.65*

- **These factors are based on an assumed ADTT of 5,000 trucks**
  - If the ADTT is less than 100, 90% of the specified force may be used
  - If the ADTT is less than 1,000, 95% of the specified force may be used

Multiple Presence Factors are NOT used with the Distribution Factors

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§ 3.6 - Live Loads

§3.6.2: Dynamic Load Allowance

- **Impact Factors, *IM***
  - *Deck Joints* *75%* ***ODOT EXCEPTION***
    - *125% of static design truck or 100% of static design tandem*
  - *Fatigue* *15%*
  - *All other cases* *33%*

- **The Dynamic Load Allowance is applied only to the truck load (including fatigue trucks), not to lane loads or pedestrian loads.**

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§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- **Each fatigue detail shall satisfy,**

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad \textbf{(6.6.1.2.2-1)}$$

where,

γ - load factor specified in Table 3.4.1-1 for fatigue ($\gamma_{fatigue} = 0.75$)

(Δf) - live load stress range due to the passage of the fatigue load specified in §3.6.1.4

η and ϕ are taken as 1.00 for the fatigue limit state

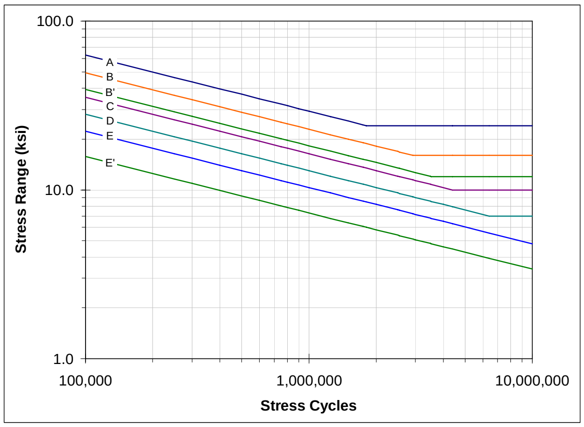
The live-load stress due to the passage of the fatigue load is approximately one-half that of the heaviest truck expected in 75 years.

Pgs 6.29-6.31,6.42 AASHTO-LRFD 2007

§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- **This is based on the typical S-N diagram:**



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§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \geq \frac{(\Delta F)_{TH}}{2} \qquad \textbf{(6.6.1.2.5-1)}$$

- A - Fatigue Detail Category Constant - Table 6.6.1.2.5-1
- $N = (365)\,(75)\,n\,(ADTT)_{SL}$ (75 Year Design Life) **(6.6.1.2.5-2)**
- n - # of stress ranges per truck passage - Table 6.6.1.2.5-2
- $(ADTT)_{SL}$ - Single-Lane ADTT from §3.6.1.4
- $(\Delta F)_{TH}$ - Constant amplitude fatigue threshold - Table 6.6.1.2.5-3

ODOT is planning to simply design for infinite life on Interstate Structures

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§6.6 - Fatigue and Fracture Considerations

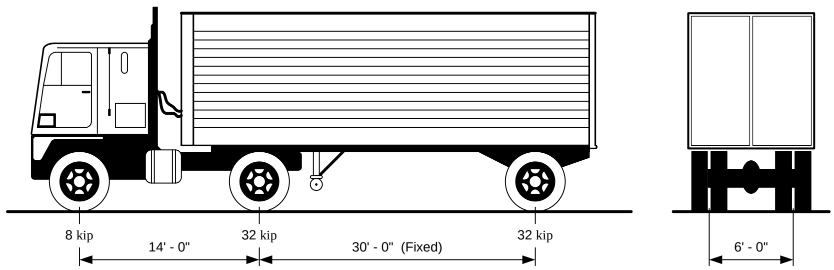
§6.6.1.2: Load Induced Fatigue

Tables 6.6.1.2.5-1&3 Fatigue Constant and Threshold Stress Range

Detail Category	A x 10^8 (ksi^3)	$(\Delta F)_{TH}$ (ksi)
A	250	24.0
B	120	16.0
B'	61.0	12.0
C	44.0	10.0
C'	44.0	12.0
D	22.0	7.0
E	11.0	4.5
E'	3.9	2.6
M164 Bolts	17.1	31.0
M253 Bolts	31.5	38.0

§6.6 - Fatigue and Fracture Considerations

§3.6.1.4.1: Fatigue Truck



The fatigue truck is applied alone – lane load is NOT used. The dynamic allowance for fatigue is *IM* = 15%. The load factor for fatigue loads is 0.75 for *LL*, *IM* and *CE* ONLY.

No multiple presence factors are used in the Fatigue Loading, the distribution factors are based on one lane loaded, and load modifiers (η) are taken as 1.00.

§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

Table 6.6.1.2.5-2 Cycles per Truck Passage

	Span Length	
	> 40 ft.	≤ 40 ft.
Simple Span Girders	1.0	2.0
Continuous Girders		
- Near Interior Supports	1.5	2.0
- Elsewhere	1.0	2.0
Cantilever Girders	5.0	
Trusses	1.0	

	Spacing	
	> 20 ft.	≤ 20 ft.
Transverse Members	1.0	2.0

Fatigue details located within L/10 of a support are considered to be "near" the support.

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§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- In the absence of better information,

$$(ADTT)_{SL} = p\ ADTT \qquad \text{(3.6.1.4.2-1)}$$

where,

p - The fraction of truck traffic in a single lane

Table 3.6.1.4.2-1 Single Lane Truck Fraction

# Lanes Available to Trucks	p
1	1.00
2	0.85
3 or more	0.80

Must consider the number of lanes available to trucks <u>in each direction!</u>

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§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- In the absence of better information,

$$ADTT = (TF)\ ADT$$

where,

TF - The fraction trucks in the average daily traffic

Table C3.6.1.4.2-1 ADT Truck Fraction

Class of Highway	TF
Rural Interstate	0.20
Urban Interstate	0.15
Other Rural	0.15
Other Urban	0.10

ODOT is suggesting that the $ADTT$ be taken as 4 x 20-year-avg ADT

Pgs 3.27-3.28

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§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- **Consider the Following:**
  - **A fatigue detail near the center of a span of 4-lane, urban interstate highway with an ADT of 30,000 vehicles.**
  - **$ADTT$ = (TF) (ADT) = (0.15) (30,000 Vehicles) = 4,500 Trucks**
  - **$(ADTT)_{SL}$ = p $ADTT$ = (0.80) (4,500 Trucks) = 3,600 Trucks**
  - **N = (365) (75) n $(ADTT)_{SL}$**
  - **= (365) (75) (1) (3,600 Trucks) = 98.55M Cycles**

Since this is a structure on an interstate, it is assume that the ADT value given is for traffic traveling in one direction only.

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§ 3.6 - Live Loads

§3.6.1.6: Pedestrian Loads

- **Pedestrian load = 0.075 $^{kip}/_{ft^2}$ applied to sidewalks wider than 2 ft.**
  - Considered simultaneous with vehicle loads.

- **If the bridge is ONLY for pedestrian and/or bicycle traffic, the load is 0.085 $^{kip}/_{ft^2}$.**

- **If vehicles can mount the sidewalk, sidewalk pedestrian loads are not considered concurrently.**

ODOT Exception - If a pedestrian bridge can accommodate service vehicles use Section 301.4.1 of the ODOT Bridge Design Manual (H15-44).

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§ 3.6 - Live Loads

§3.6.3: Centrifugal Force - *CE*

For the purpose of computing the radial force or the overturning effect on wheel loads, the centrifugal effect on live load shall be taken as the product of the axle weights of the design truck or tandem and the factor, C taken as:

$$C = f\frac{v^2}{gR} \qquad (3.6.3\text{-}1)$$

v = highway design speed ($^{ft}/_{sec}$)
f = 4/3 for all load combinations except fatigue and 1.0 for fatigue
g = gravitational constant = 32.2 $^{ft}/_{sec^2}$.
R = radius of curvature for the traffic lane (ft).

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§ 3.6 - Live Loads

§3.6.3: Centrifugal Force - *CE*

- **Highway design speed shall not be taken to be less than the value specified in the current edition of the AASHTO publication, *A Policy of Geometric Design of Highways and Streets.***

- **The multiple presence factors shall apply.**

- **Centrifugal forces shall be applied horizontally at a distance 6.0 ft above the roadway surface. A load path to carry the radial force to the substructure shall be provided.**

- **The effect of superelvation in reducing the overturning effect of centrifugal force on vertical wheel leads may be considered.**

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§ 3.6 - Live Loads

§3.6.4: Braking Force - *BR*

- **The braking force shall be taken as the greater of:**
  - 25% of the axle weights of the design truck or design tandem
  - or 5% of the design truck plus lane load
  - or 5% of the design tandem plus lane load

  This braking force shall be placed in all design lanes which are considered to be loaded in accordance with Article 3.6.1.1.1 (defines number of design lanes) and which are carrying traffic headed in the same direction. These forces shall be assumed to act horizontally at a distance of 6.0 ft above the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future.

- **The multiple presence factors shall apply.**

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§ 3.6 - Live Loads

§3.6.5: Vehicular Collision Force - *CT*

- **The provisions of Article 3.6.5.2 need not be considered for structures which are protected by:**
  - An embankment
  - A structurally independent, crashworthy ground mounted 54.0 in high barrier located within 10.0 ft from the component being protected
  - A 42.0 in high barrier located at more than 10.0 ft from the component being protected
- **In order to qualify for this exemption, such barrier shall be structurally and geometrically capable of surviving the crash test for Test Level 5, as specified in Section 13.**

ODOT: This section does not apply to redundant substructure units.

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§ 3.8 - Wind Loads

§3.8: Wind Loads *WL* and *WS* - General

- ***WS* is the wind load on the structure.**
- ***WL* is the wind load on the live load.**
- **Both horizontal and vertical wind loads must be considered.**

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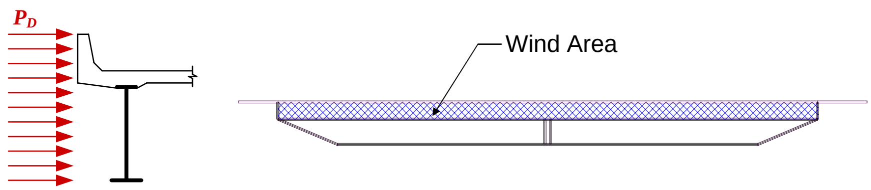
§ 3.8 - Wind Loads

§3.8: Wind Loads *WL* and *WS* - General

- **The pressures are assumed to be caused by a base wind velocity, V_B = 100 mph.**
- **The wind is assumed to be a uniformly distributed load applied to the sum area of all components of the structure, as seen in elevation taken perpendicular to the wind direction. The direction is varied to produce the extreme force effect. Areas which do not contribute to the extreme force effect may be ignored.**



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§ 3.8 - Wind Loads

§3.8: Wind Loads *WL* and *WS* - General

For both *WS* and *WL*, the first step is to find the design wind velocity, V_{DZ}, at a particular elevation, Z. For bridges more than 30 ft. above low ground or water level:

$$V_{DZ} = 2.5 V_0 \left(\frac{V_{30}}{V_B}\right) \ln\left(\frac{Z}{Z_0}\right) \qquad \textbf{(3.8.1.1-1)}$$

V_{30} = wind velocity at 30 ft. above low ground (mph).
V_b = base wind velocity = 100 mph
Z = height of structure at which the winds are being calculated > 30 ft. above low ground or water level.
Z_0 = Friction length of upstream fetch (ft)
V_0 = Friction velocity (mph)

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§3.8: Wind Loads *WL* and *WS* - General

Table 3.8.1.1-1 Values of V_0 and Z_0 Various Upstream Surface Conditions

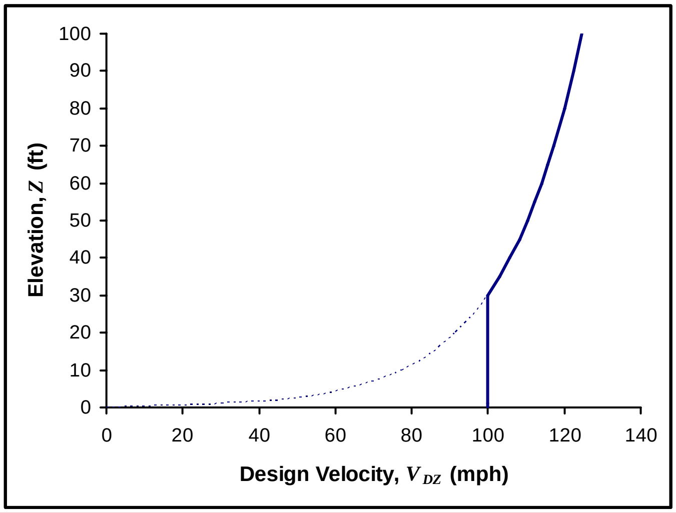
Condition	Open Country	Suburban	City
V_0 (mph)	8.20	10.90	12.00
Z_0 (ft)	0.23	3.28	8.20

§3.8: Wind Loads *WL* and *WS* - General

- **V_{30} may be estimated by:**
  - Fastest-mile-of-wind charts available in ASCE 7 for various recurrence intervals.
  - By site specific investigations
  - In lieu of a better criterion use 100 mph
- **For bridges less than 30 ft. above low ground or water level, use V_{DZ} = 100 mph.**

§ 3.8 - Wind Loads

§3.8: Wind Loads *WL* and *WS* - General



§ 3.8 - Wind Loads

§3.8.1.2: Wind Pressure on the Structure - *WS*

The wind pressure on the structure can be found from:

$$P_D = P_B\left(\frac{V_{DZ}}{V_B}\right)^2 = P_B\frac{V_{DZ}^2}{10,000} \quad \left(\frac{\text{kip}}{\text{ft}^2}\right) \qquad \textbf{(3.8.1.2.1-1)}$$

P_B = Base wind pressure specified in Table 3.8.1.2.1-1 (ksf)

Table 3.8.1.2.1-1 Values of P_B corresponding to V_B = 100 mph

Superstructure Component	Windward Load (ksf)	Leeward Load (ksf)
Trusses, Columns, and Arches	0.050	0.025
Beams	0.050	N/A
Large Flat Surfaces	0.040	N/A

§ 3.8 - Wind Loads

§3.8.1.2: Wind Pressure on the Structure - *WS*

- **If justified by local conditions, a different base velocity can be used for combinations not involving wind on *LL*.**
- **Unless required by Article 3.8.3 (aeroelastic instability), the wind direction is assumed horizontal.**
- **More precise data may be used in place of equation 3.8.1.2.1-1.**
- **Total wind loading shall not be less than:**
  - 0.30 $^{kip}/_{ft}$ on the plane of the windward chord of trusses or arches.
  - 0.15 $^{kip}/_{ft}$ on the plane of the leeward chord of trusses or arches
  - 0.30 $^{kip}/_{ft}$ on beam or girder spans.

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§ 3.8 - Wind Loads

§3.8.1.2: Wind Pressure on the Structure - *WS*

- **If the wind angle is not perpendicular, the table on the next slide is used for P_B .**
- **The skew angle is measured from a perpendicular to the longitudinal axis.**
- **The direction shall be that which produces the extreme force effect.**
- **Longitudinal and transverse pressures are considered simultaneously.**

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§ 3.8 - Wind Loads

§3.8.1.2: Wind Pressure on the Structure - *WS*

Table 3.8.1.2.2-1 P_b for various angles of attack with V_B = 100 mph

Skew Angle	Trusses/Columns/ Arches		Girders	
	Lateral Load	Longitudinal Load	Lateral Load	Longitudinal Load
(degrees)	(ksf)	(ksf)	(ksf)	(ksf)
0	0.075	0.000	0.050	0.000
15	0.070	0.012	0.044	0.006
30	0.065	0.028	0.041	0.012
45	0.047	0.041	0.033	0.016
60	0.024	0.050	0.017	0.019

§ 3.8 - Wind Loads

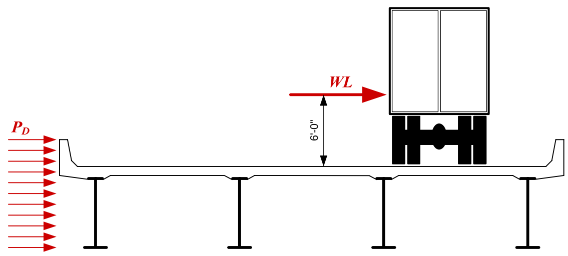
§3.8.1.2: Wind Pressure on the Structure - *WS*

- Longitudinal and transverse forces are calculated from an assumed base wind pressure of 0.040 $^{kip}/_{ft^2}$.
- If the wind angle is skewed, the wind pressure is resolved into components.
- The component perpendicular to the end acts on the area as seen from the end elevation.
- The component perpendicular to the front elevation acts on the area seen from the front elevation and is applied simultaneous with the superstructure wind load.

§ 3.8 - Wind Loads

§3.8.1.3: Wind Pressure on Vehicles - *WL*

- **Wind pressure on vehicles**
  - Movable, interruptible force of 0.10 klf applied at 6 ft above the roadway. The force shall be transmitted to the structure.



- **If the force is not perpendicular, the table on the following slide is used.**

Pg 3.40-42

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§ 3.8 - Wind Loads

§3.8.1.3: Wind Pressure on Vehicles - *WL*

Table 3.8.1.3-1 Wind Components on Live Load

Skew Angle (degrees)	Normal Component (klf)	Parallel Component (klf)
0	0.100	0.000
15	0.088	0.012
30	0.082	0.024
45	0.066	0.032
60	0.034	0.038

Pg 3.41

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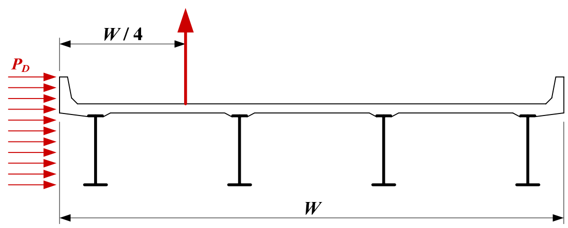
§ 3.8 - Wind Loads

§3.8.2: Vertical Wind Pressure

- **Wind uplift force of 0.020 $^{kip}/_{ft^2}$ times the width of the deck + sidewalk + parapet.**
- **Applied as a longitudinal line load at the windward quarter point of the deck width.**
- **Applied in conjunction with the horizontal wind loads**
- **Applied only to Service IV and Strength III limit states, in combinations which do NOT include wind on live load (*WL*) and only when the wind direction is perpendicular to the longitudinal axis.**



Pg 3.41

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§ 3.10 Earthquake Effects: EQ

ODOT Exception

- ***All bridges in Ohio fall in Seismic Zone I***
- ***Acceleration co-efficient is assumed above 0.025, but less than 0.09.***
- ***Design the connection between the superstructure and sub-structure to resist 0.2 times the vertical reaction due to tributary permanent load.***
  - *Tributary area refers to the uninterrupted segment of the superstructure contributing to load on the seismic restraint.*
  - *Restrained direction is typically transverse.*
  - *Tributary permanent load includes allowance for future wearing surface.*

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§ 3.10 Earthquake Effects: EQ

ODOT Exception

- ***The Extreme Event I load factor for live load, γ_{EQ} is taken as 0.0.***
- ***Standard integral and semi-integral type abutments supply suitable resistance to seismic forces.***
  - *No additional restraint at these abutments should be provided.*
  - *Restraints should be provided at the piers for multi-span bridges.*
  - *Bearing guides are required for semi-integral abutments with a skew of 30^o or more.*
- ***If seismic restraints are provided, EQ for substructures at Extreme Event I Limit State = 0.2 times tributary live and dead loads applied in the restrained direction resulting in maximum effect.***

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§ 3.12 –Effects Due to Superimposed Deformations: TU, TG, SH, CR, SE

§3.12.1: Uniform Temperature

Movements due to uniform temperature are calculated using the following temperature limits:

Table 3.12.2.1-1 Procedure A Temperature Ranges (Partial)

Climate	Steel or Aluminum	Concrete	Wood
Cold	-30^o to 120^o F	0^o to 80^o F	0^o to 75^o F

ODOT requires the use of Cold Climate, Procedure A.

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AASHTO-LRFD
Bridge Design specification
Section 4: Structural Analysis and Evaluation

- AASHTO-LRFD Specification, 4th Ed., 2007

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§4.4 – Acceptable Methods of Structural Analysis

- **Simplified Analysis**
  - Distribution Factor
- **Refined Analysis**
  - Finite Element Modeling

Pg 4.9 – 4.10

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§4.6.2 - Approximate Methods of Analysis – Dist Factors

§ 4.6.2.2 Lateral Load Distribution Beam and Slab Bridges

- **Design live load bending moment or shear force is the product of a lane load on a beam model and the appropriate distribution factor.**

$$M_{U,LL} = (DF)(M_{Beam\ Line})$$

- **The following Distribution Factors are applicable to Reinforced Concrete Decks on Steel Girders, CIP Concrete Girders, and Precast Concrete I or Bulb-Tee sections.**

- **Also applies to Precast Concrete Tee and Double Tee Sections when sufficient connectivity is present.**

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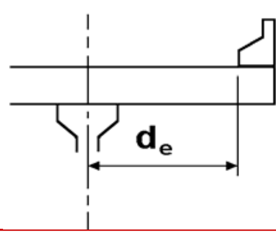
§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2 Lateral Load Distribution Beam and Slab Bridges

The simplified distribution factors may be used if:

- Width of the slab is constant
- Number of beams, $N_b \geq 4$
- Beams are parallel and of similar stiffness
- Roadway overhang $d_e \leq 3$ ft*
- Central angle < 4^0
- Cross section conforms to AASHTO Table 4.6.2.2.1-1

***ODOT Exception: The roadway overhang $d_e \leq 3$ ft. does not apply to interior DFs for sections (a) and (k).**



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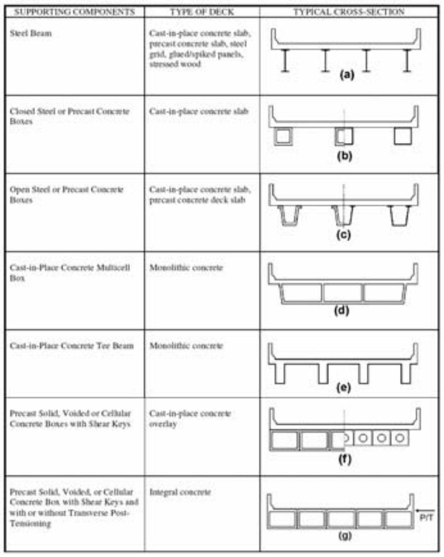
§4.6.2 - Approximate Methods of Analysis – Distribution Factors



SUPPORTING COMPONENTS	TYPE OF DECK	TYPICAL CROSS-SECTION
Steel Beam	Cast-in-place concrete slab, precast concrete slab, steel grid, glued/spiked panels, stressed wood	(a)
Closed Steel or Precast Concrete Boxes	Cast-in-place concrete slab	(b)
Open Steel or Precast Concrete Boxes	Cast-in-place concrete slab, precast concrete deck slab	(c)
Cast-in-Place Concrete Multicell Box	Monolithic concrete	(d)
Cast-in-Place Concrete Tee Beam	Monolithic concrete	(e)
Precast Solid, Voided or Cellular Concrete Boxes with Shear Keys	Cast-in-place concrete overlay	(f)
Precast Solid, Voided, or Cellular Concrete Box with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	P/T (g)

This is part of Table 4.6.2.2.1-1 showing common bridge types.

The letter below the diagram correlates to a set of distribution factors.

Slab-on-Steel-Girder bridges qualify as type (a) cross sections.

Pg 4.31-32

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§4.6.2 - Approximate Methods of Analysis – Distribution Factors

Typical ODOT Bridge Type	Table 4.6.2.2.1-1 Cross Section
Steel Beam/Girder	(a)
Concrete "I" beam	(k)
Composite Box or Non-composite with Transverse PT	(f)
Non-composite Box w/o Transverse PT	(g) Using DF that assume beams are connected only enough to prevent relative displacement at interface.

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§4.6.2 - Approximate Methods of Analysis – Distribution Factors

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Distribution Factors	Range of Applicability
Wood Deck on Wood or Steel Beams	a, l	See Table 4.6.2.2.2a-1	
Concrete Deck on Wood Beams	l	One Design Lane Loaded: $S/12.0$ Two or More Design Lanes Loaded: $S/10.0$	$S \leq 6.0$
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$	$3.5 \leq S \leq 16.0$ $4.5 \leq t_s \leq 12.0$ $20 \leq L \leq 240$ $N_b \geq 4$ $10{,}000 \leq K_g \leq 7{,}000{,}000$
		use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$
Cast-in-Place Concrete Multicell Box	d	One Design Lane Loaded: $\left(1.75+\frac{S}{3.6}\right)\left(\frac{1}{L}\right)^{0.35}\left(\frac{1}{N_c}\right)^{0.45}$ Two or More Design Lanes Loaded: $\left(\frac{13}{N_c}\right)^{0.3}\left(\frac{S}{5.8}\right)\left(\frac{1}{L}\right)^{0.25}$	$7.0 \leq S \leq 13.0$ $60 \leq L \leq 240$ $N_c \geq 3$ If $N_c > 8$ use $N_c = 8$
Concrete Deck on Concrete Spread Box Beams	b, c	One Design Lane Loaded: $\left(\frac{S}{3.0}\right)^{0.35}\left(\frac{Sd}{12.0L^2}\right)^{0.25}$ Two or More Design Lanes Loaded: $\left(\frac{S}{6.3}\right)^{0.6}\left(\frac{Sd}{12.0L^2}\right)^{0.125}$	$6.0 \leq S \leq 18.0$ $20 \leq L \leq 140$ $18 \leq d \leq 65$ $N_b \geq 3$
		Use Lever Rule	$S > 18.0$
Concrete Beams used in Multibeam Decks	f	One Design Lane Loaded: $k\left(\frac{b}{33.3L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25}$ where: $k = 2.5(N_b)^{-0.2} \geq 1.5$	$35 \leq b \leq 60$ $20 \leq L \leq 120$ $5 \leq N_b \leq 20$
	g if sufficiently connected to act as a unit	Two or More Design Lanes Loaded: $k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12.0L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}$	

This is a part of Table 4.6.2.2.2b-1 showing distribution factors for moment. A similar table exists for shear distribution factors.

The table give the DF formulae and the limits on the specific terms. If a bridge does NOT meet these requirements or the requirements on the previous slide, refined analysis must be used.

§4.6.2 - Approximate Methods of Analysis – Distribution Factors

Table C4.6.2.2.1-1 *L* for Use in Live Load Distribution Factor Equations.

Force Effect	*L* (ft)
Positive Moment	Length of the span for which the moment is being calculated.
Negative Moment – Near interior supports of continuous spans from point of contraflexure to point of contraflexure under a uniform load in all spans.	Average length of two adjacent spans.
Negative moment other than near interior supports of continuous spans	Length of the span for which the moment is being calculated.
Shear	Length of the span for which the shear is being calculated.
Exterior reaction	Length of exterior span
Interior reaction of a continuous span.	Average length of two adjacent spans.

§4.6.2 - Approximate Methods of Analysis

Lateral Load Distribution Beam and Slab Bridges

- **For the purpose of further explanation, a single case of distribution factors will be used as an example.**
- **The following Distribution Factors are applicable to Reinforced Concrete Decks on Steel Girders, CIP Concrete Girders, and Precast Concrete I or Bulb-Tee sections. These are types a, e and k.**
- **Also applies to Precast Concrete Tee and Double Tee Sections when sufficient connectivity is present. These are types i and j.**

§4.6.2 - Approximate Methods of Analysis – Distribution Factors

Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,Lt_s^3}\right)^{0.1}$	$3.5 \le S \le 16.0$ $4.5 \le t_s \le 12.0$ $20 \le L \le 240$ $N_b \ge 4$ $10{,}000 \le K_g \le 7{,}000{,}000$
		use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$

§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.2 Moment Distribution - Interior Girders

- **Interior Girders:**
  - One Lane Loaded:

$$DF_{M,Int} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

This term may be taken as 1.00 for prelim design

  - Two or More Lanes Loaded:

$$DF_{M,Int} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

Pg 4.35 - Table 4.6.2.2.2b-1

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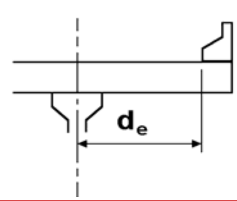
§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2 Beam-Slab Bridges

- **Parameter Definitions & Limits of Applicability:**

S - Beam or girder spacing (ft.)	$3.5 \le S \le 16.0$
L - Span length of beam or girder (ft.)	$20 \le L \le 240$
K_g - Longitudinal stiffness parameter (in^4)	$10\text{k} \le K_g \le 7\text{M}$
t_s - Thickness of concrete slab (in)	$4.5 \le t_s \le 12.0$
d_e - Distance from exterior beam to interior edge of curb (ft.) (Positive if the beam is "inside" of the curb.)	$-1.0 \le d_e \le 5.5$



Pgs 4.29 and 4.35

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§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2 Beam-Slab Bridges

- **Parameter Definitions & Limits of Applicability:**

$$K_g = n\left(I + Ae_g^2\right) \qquad \textbf{(4.6.2.2.1-1)}$$

- n - Modular ratio, E_{Beam} / E_{Deck} (See Section 6.10.1.1.1b, Pg 6.70)
- I - Moment of inertia of beam (in^4)
- A - Area of beam (in^2)
- e_g - Distance between CG steel and CG deck (in)

ODOT Exception: For interior beam DF, include monolithic wearing surface and haunch in e_g and K_g when this increases the DF.

Pg 4.30 AASHTO-LRFD 2007

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§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.2d Moment Distribution - Exterior Beams

- **Exterior Girders:**
  - One Lane Loaded:

    Lever Rule

  - Two or More Lanes Loaded:

$$DF_{ext} = e\ DF_{int}$$

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

Pg 4.38 - Table 4.6.2.2.2d-1 AASHTO-LRFD 2007

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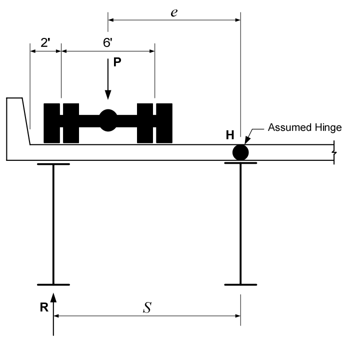
§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.2d Moment Distribution - Exterior Beams

- **Lever Rule:**
  - Assume a hinge develops over each interior girder and solve for the reaction in the exterior girder as a fraction of the truck load.



This example is for one lane loaded. Multiple Presence Factors apply 1.2 is the MPF

$$\sum M_H \rightarrow 1.2Pe - RS = 0$$

$$R = \frac{1.2Pe}{S} \quad \therefore \quad DF = \frac{1.2e}{S}$$

In the diagram, P is the axle load.

Pg 4.38 - Table 4.6.2.2.2d-1

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§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.2e Moment Distribution - Skewed Bridges

- **Correction for Skewed Bridges:**
  - The bending moment <u>may</u> be reduced in bridges with a skew of 30° ≤ θ ≤ 60°

$$DF_M^{'} = \left(1 - C_1 (Tan\theta)^{1.5}\right) DF_M$$

$$C_1 = 0.25\left(\frac{K_g}{12Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

  - When the skew angle is greater than 60°, take θ = 60°

Pg 4.39 - Table 4.6.2.2.2e-1

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§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.3a Shear Distribution - Interior Beams

- **Interior Girders:**
  - One Lane Loaded:

$$DF_{V,Int} = 0.36 + \frac{S}{25.0}$$

  - Two or More Lanes Loaded:

$$DF_{V,Int} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2$$

Pg 4.41 - Table 4.6.2.2.3a-1

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§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.3b Shear Distribution - Exterior Beams

- **Exterior Girders:**
  - One Lane Loaded:

Lever Rule

  - Two or More Lanes Loaded:

$$DF_{Ext} = e\ DF_{Int}$$

$$e = 0.60 + \frac{d_e}{10}$$

Pg 4.43 - Table 4.6.2.2.3b-1

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§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.3c Shear Distribution - Skewed Bridges

- **Correction for Skewed Bridges:**
  - The shear forces in beams of skewed bridges shall be adjusted with a skew of 0° ≤ θ ≤ 60°

$$DF_V' = \left(1.0 + 0.20\left(\frac{12Lt_s^3}{K_g}\right)^{0.3} Tan\theta\right) DF_V$$

  - Note that this adjustment is for SUPPORT shear at the obtuse corner of the exterior beam, except in multibeam bridges when it is applied to all beams (Article 4.6.2.2.3c).

Note: an adjacent box girder is an example of a multibeam bridge.

Pg 4.44 - Table 4.6.2.2.3c-1 AASHTO-LRFD 2007

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§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.2d Exterior Beams

- **Minimum Exterior DF: (Rigid Body Rotation of Bridge Section)**

$$DF_{Ext,Min} = \frac{N_L}{N_b} + \frac{X_{Ext}\sum^{N_L} e}{\sum^{N_b} x^2} \qquad \text{(C4.6.2.2.2d-1)}$$

  - N_L - Number of loaded lanes under consideration
  - N_b - Number of beams or girders
  - e - Eccentricity of design truck or load from CG of pattern of girders (ft.)
  - x - Distance from CG of pattern of girders to each girder (ft.)
  - X_{Ext} - Distance from CG of pattern of girders to exterior girder (ft.)

Pg 4.37 AASHTO-LRFD 2007

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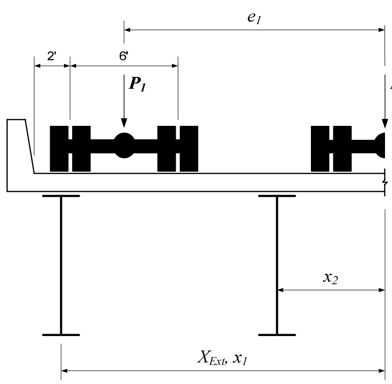
§ 4.6.2.2.2d Exterior Beams

- **Minimum Exterior DF: (Rigid Body Rotation of Bridge Section)**



$$DF_{Ext,Min} = \frac{N_L}{N_b} + \frac{X_{Ext}\sum^{N_L} e}{\sum^{N_b} x^2}$$ (C4.6.2.2.2d-1)

N_L - Number of loaded lanes under consideration
N_b - Number of beams or girders
e - Eccentricity of design truck or load from CG of pattern of girders (ft.)
x - Distance from CG of pattern of girders to each girder (ft.)
X_{Ext} - Distance from CG of pattern of girders to exterior girder (ft.)

§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.1 Dead Load Distribution

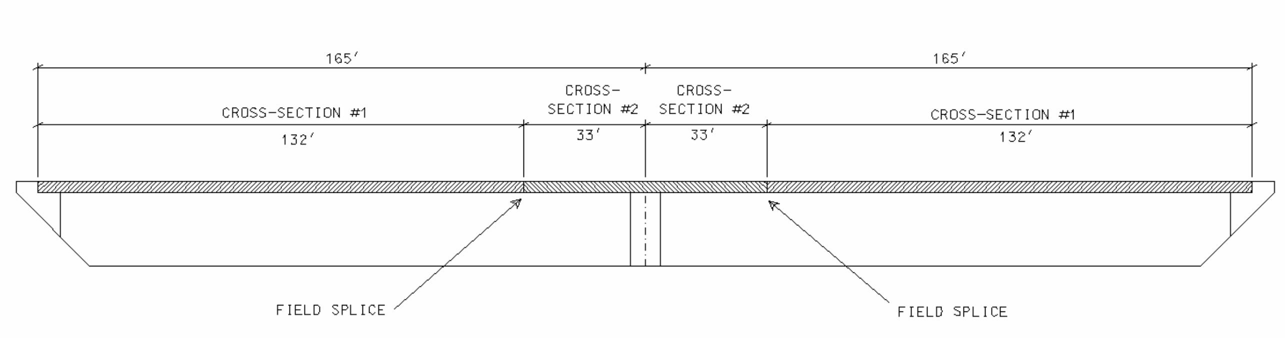
"Where bridges meet the conditions specified herein, permanent loads of and on the deck may be distributed uniformly among the beams and/or stringers. For this type of bridge, the conditions are:"

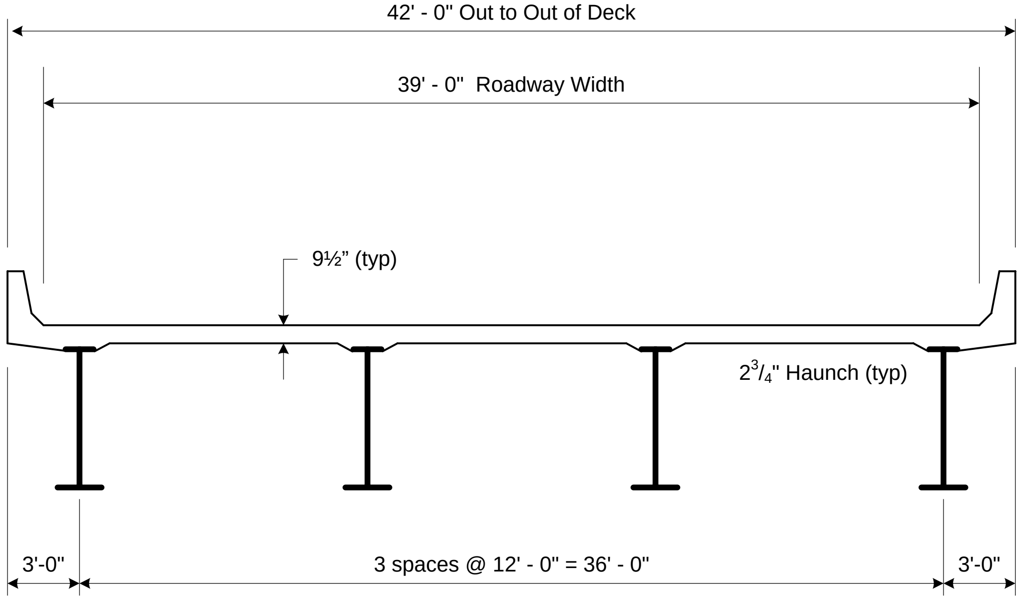
- **Width of deck is constant**
- **Unless otherwise specified, the number of beams is not less than four**
- **Beams are parallel and have approximately the same stiffness**
- **Unless otherwise specified, the roadway part of the overhang, d_e, does not exceed 3.0 ft**
- **Curvature in plan is less then the limit specified in Article 4.6.1.2**
- **Cross-section is consistent with one of the cross-sections shown Table 4.6.2.2.1-1**

1. PROBLEM STATEMENT AND ASSUMPTIONS:

A two-span continuous composite I-girder bridge has two equal spans of 165' and a 42' deck width. The steel girders have $F_y = 50^{ksi}$ and all concrete has a 28-day compressive strength of $f'_c = 4.5^{ksi}$. The concrete slab is $9^1/_2$" thick. A typical 2¾" haunch was used in the section properties. Concrete barriers weighing 640^{plf} and an asphalt wearing surface weighing 60^{psf} have also been applied as a composite dead load.

HL-93 loading was used per AASHTO (2004), including dynamic load allowance.



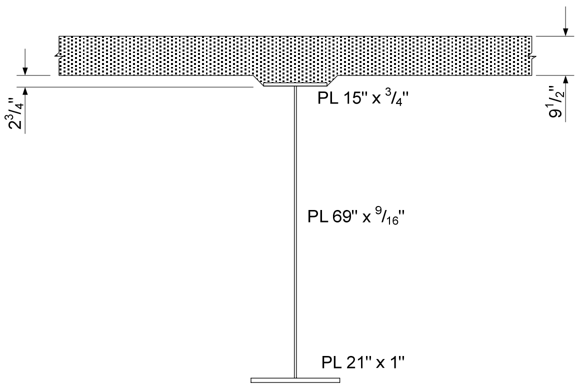


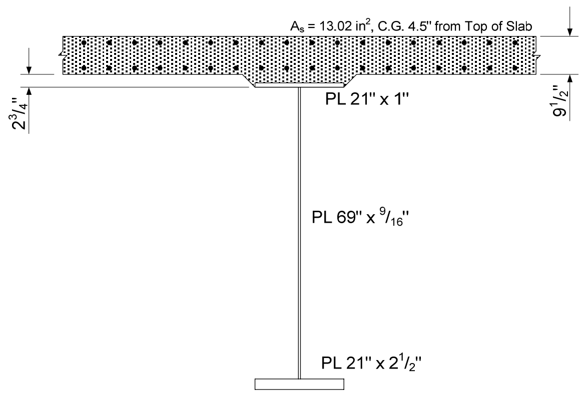
References:

Barth, K.E., Hartnagel, B.A., White, D.W., and Barker, M.G., 2004, "Recommended Procedures for Simplified Inelastic Design of Steel I-Girder Bridges," ASCE Journal of Bridge Engineering, May/June Vol. 9, No. 3

"Four LRFD Design Examples of Steel Highway Bridges," Vol. II, Chapter 1A Highway Structures Design Handbook, Published by American Iron and Steel Institute in cooperation with HDR Engineering, Inc. Available at http://www.aisc.org/

Positive Bending Section (Section 1) **Negative Bending Section (Section 2)**





2. LOAD CALCULATIONS:

DC dead loads (structural components) include:

- Steel girder self weight (DC1)
- Concrete deck self weight (DC1)
- Haunch self weight (DC1)
- Barrier walls (DC2)

DW dead loads (structural attachments) include:

- Wearing surface (DW)

2.1: Dead Load Calculations

Steel Girder Self-Weight (DC1): (Add 15% for Miscellaneous Steel)

(a) Section 1 (Positive Bending)

A = (15")(3/4") + (69")(9/16") + (21")(1") = 71.06 in^2

$$W_{section1} = 71.06\ \text{in}^2\left(\frac{490\ \text{pcf}}{\left(12\,\frac{\text{in}}{\text{ft}}\right)^2}\right)(1.15) = 278.1\,\frac{\text{Lb}}{\text{ft}}\quad \text{per girder}$$

(b) Section 2 (Negative Bending)

A = (21")(1") + (69")(9/16") + (21")(2-1/2") = 112.3 in^2

$$W_{section2} = 112.3\ \text{in}^2\left(\frac{490\ \text{pcf}}{\left(12\,\frac{\text{in}}{\text{ft}}\right)^2}\right)(1.15) = 439.5\,\frac{\text{Lb}}{\text{ft}}\quad \text{per girder}$$

Deck Self-Weight (DC1):

$$W_{deck} = (9.5")(144")\left(\frac{150 \text{ pcf}}{\left(12\,\frac{\text{in}}{\text{ft}}\right)^2}\right) = 1{,}425\,\tfrac{\text{Lb}}{\text{ft}} \quad \text{per girder}$$

Haunch Self-Weight (DC1):

Average width of flange: $\left(\dfrac{21"(66') + 15"(264')}{66' + 264'}\right) = 16.2"$

Average width of haunch: $\left(\tfrac{1}{2}\right)\left[\left(16.2" + (2)(9")\right) + 16.2"\right] = 25.2"$

$$W_{haunch} = \left(\frac{(2")(25.2")}{\left(12\,\frac{\text{in}}{\text{ft}}\right)^2}\right)(150 \text{ pcf}) = 52.5\,\tfrac{\text{Lb}}{\text{ft}} \quad \text{per girder}$$

Barrier Walls (DC2):

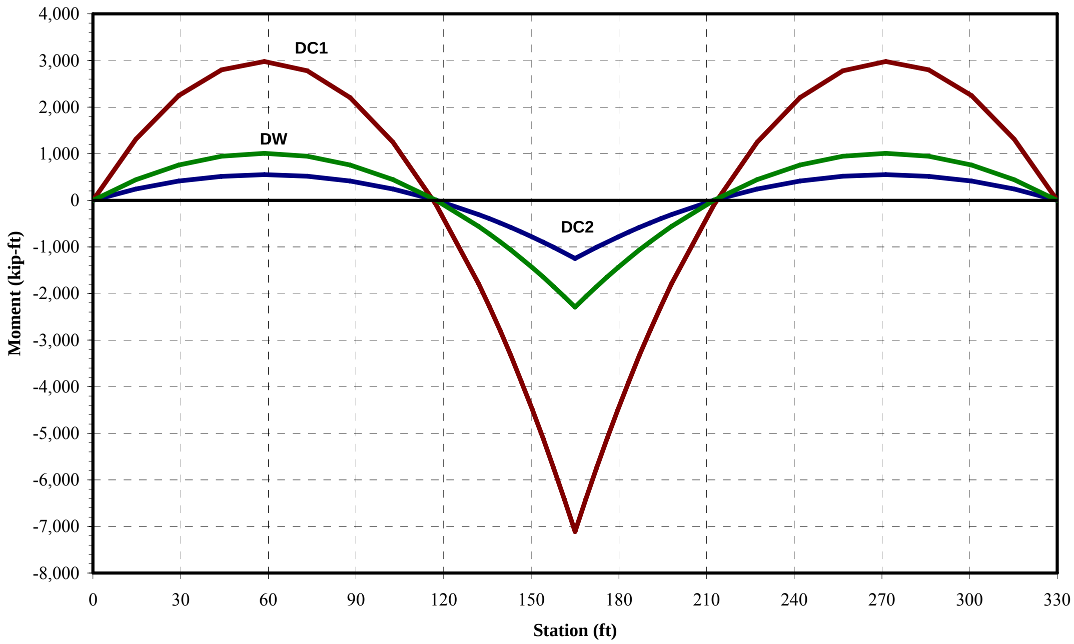
$$W_{barriers} = \left(\frac{(2 \text{ each})(640 \text{ plf})}{4 \text{ girders}}\right) = 320.0\,\text{Lb}/\text{ft} \quad \text{per girder}$$

Wearing Surface (DW):

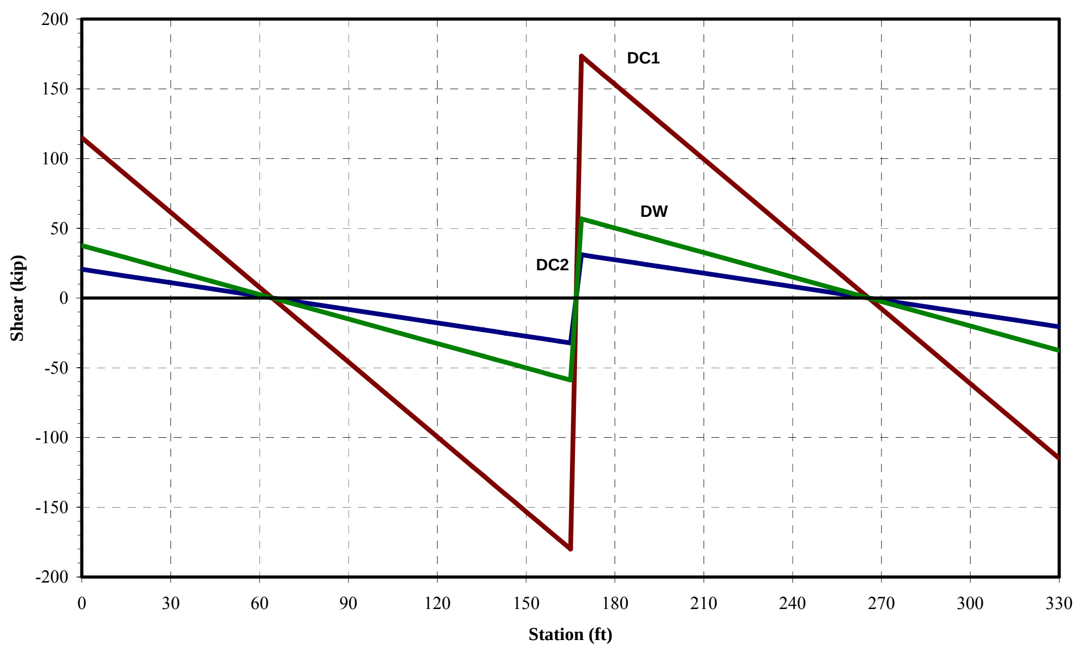
$$W_{fws} = \frac{(39')(60 \text{ psf})}{4 \text{ girders}} = 585\,\tfrac{\text{Lb}}{\text{ft}} \quad \text{per girder}$$

The moment effect due to dead loads was found using an FE model composed of four frame elements. This data was input into Excel to be combined with data from moving live load analyses performed in SAP 2000. DC1 dead loads were applied to the non-composite section (bare steel). All live loads were applied to the short-term composite section ($1n$ = 8). DW (barriers) and DC2 (wearing surface) dead loads were applied to the long-term composite section ($3n$ = 24).

Unfactored Dead Load Moment Diagrams from SAP



Unfactored Dead Load Shear Diagrams from SAP



The following Dead Load results were obtained from the FE analysis:

- The maximum positive live-load moments occur at stations 58.7’ and 271.3’
- The maximum negative live-load moments occur over the center support at station 165.0’

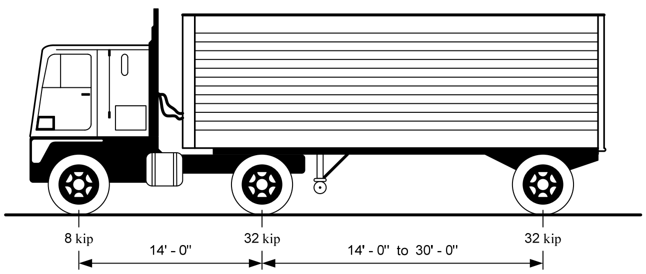
	Max (+) Moment Stations 58.7’ and 271.3’	**Max (-) Moment Station 165.0’**
DC1 - Steel:	$475^{k\text{-}ft}$	$-1{,}189^{k\text{-}ft}$
DC1 - Deck:	$2{,}415^{k\text{-}ft}$	$-5{,}708^{k\text{-}ft}$
DC1 - Haunch:	$89^{k\text{-}ft}$	$-210^{k\text{-}ft}$
DC1 - Total:	$2{,}979^{k\text{-}ft}$	$-7{,}107^{k\text{-}ft}$
DC2:	$553^{k\text{-}ft}$	$-1{,}251^{k\text{-}ft}$
DW	$1{,}011^{k\text{-}ft}$	$-2{,}286^{k\text{-}ft}$

2.2: Live Load Calculations

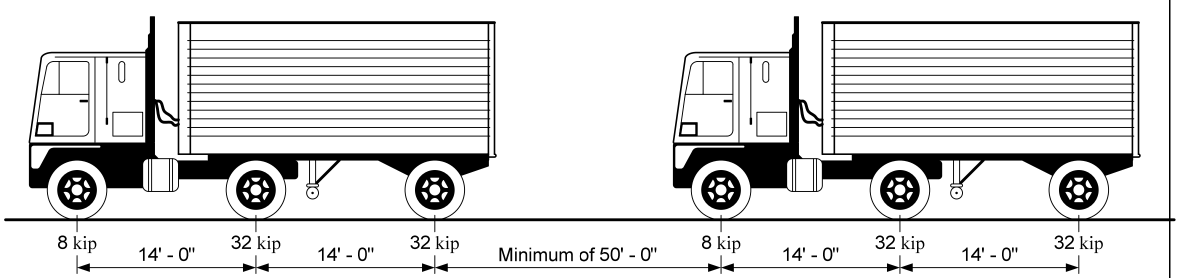
The following design vehicular live load cases described in AASHTO-LRFD are considered:

1) The effect of a design tandem combined with the effect of the lane loading. The design tandem consists of two 25^{kip} axles spaced 4.0’ apart. The lane loading consists of a 0.64^{klf} uniform load on all spans of the bridge. (HL-93M in SAP)

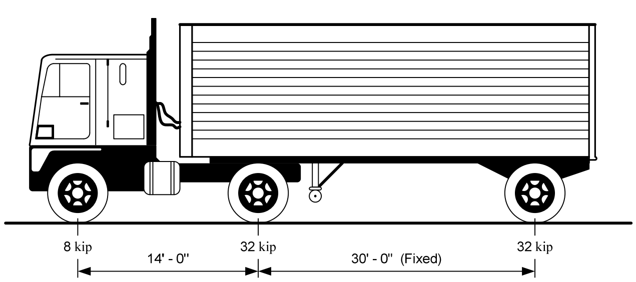
2) The effect of one design truck with variable axle spacing combined with the effect of the 0.64^{klf} lane loading. (HL-93K in SAP)



3) For negative moment *between points of contraflexure* only: 90% of the effect of a truck-train combined with 90% of the effect of the lane loading. The truck train consists of two design trucks (shown below) spaced a minimum of 50’ between the lead axle of one truck and the rear axle of the other truck. The distance between the two 32^{kip} axles should be taken as 14’ for each truck. The points of contraflexure were taken as the field splices at 132’ and 198’ from the left end of the bridge. (HL-93S in SAP)

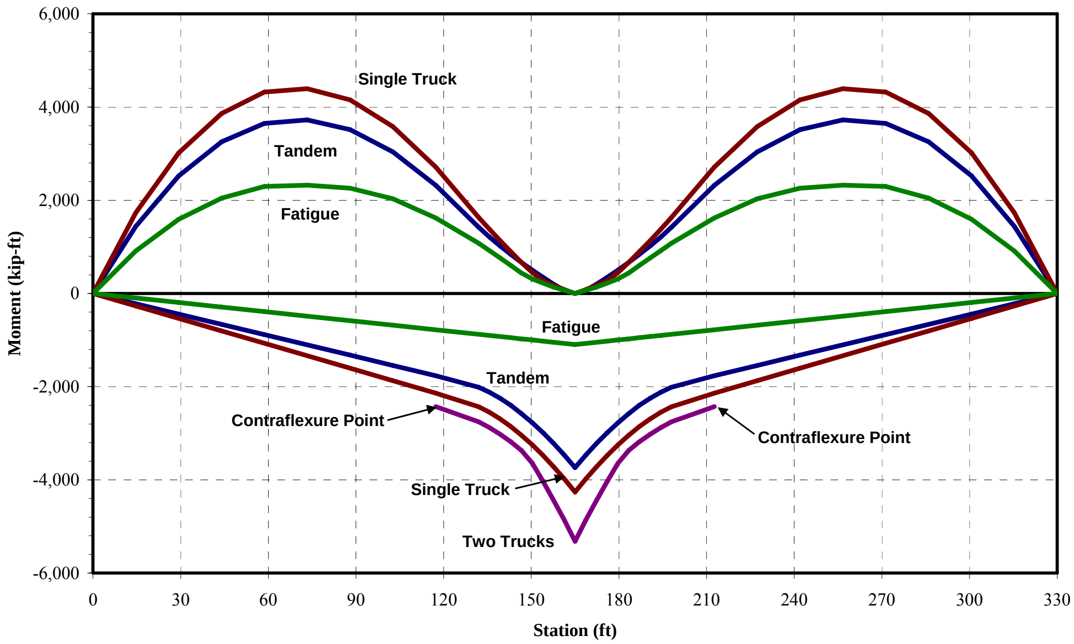


4) The effect of one design truck with fixed axle spacing used for fatigue loading.

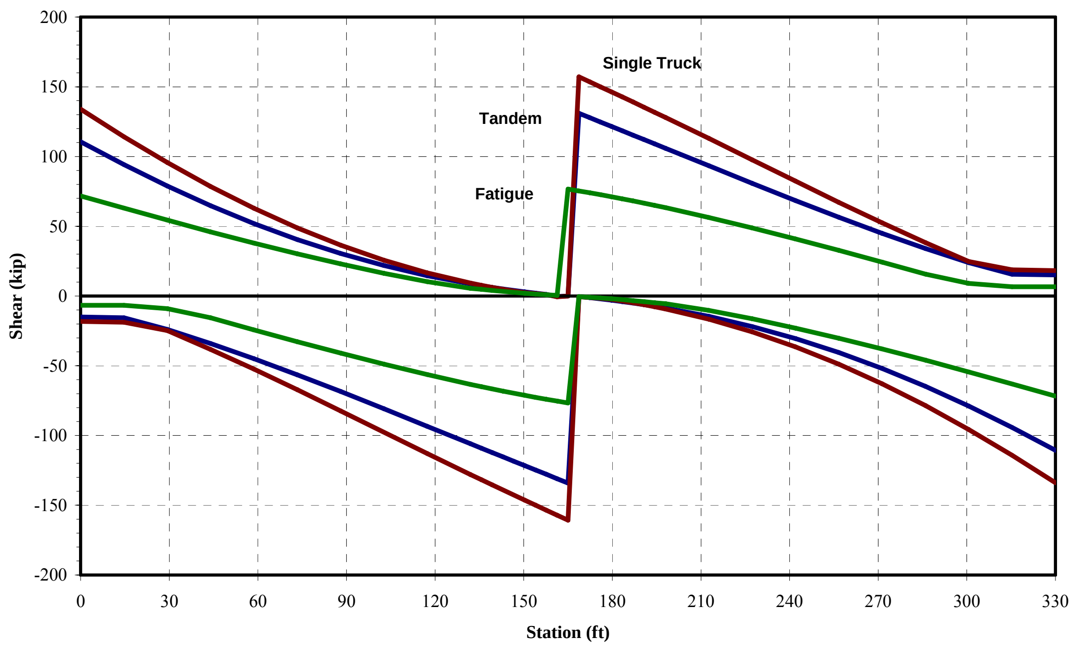


All live load calculations were performed in SAP 2000 using a beam line analysis. The nominal moment data from SAP was then input into Excel. An Impact Factor of 1.33 was applied to the truck and tandem loads and an impact factor of 1.15 was applied to the fatigue loads within SAP.

Unfactored Moving Load Moment Envelopes from SAP



Unfactored Moving Load Shear Envelopes from SAP



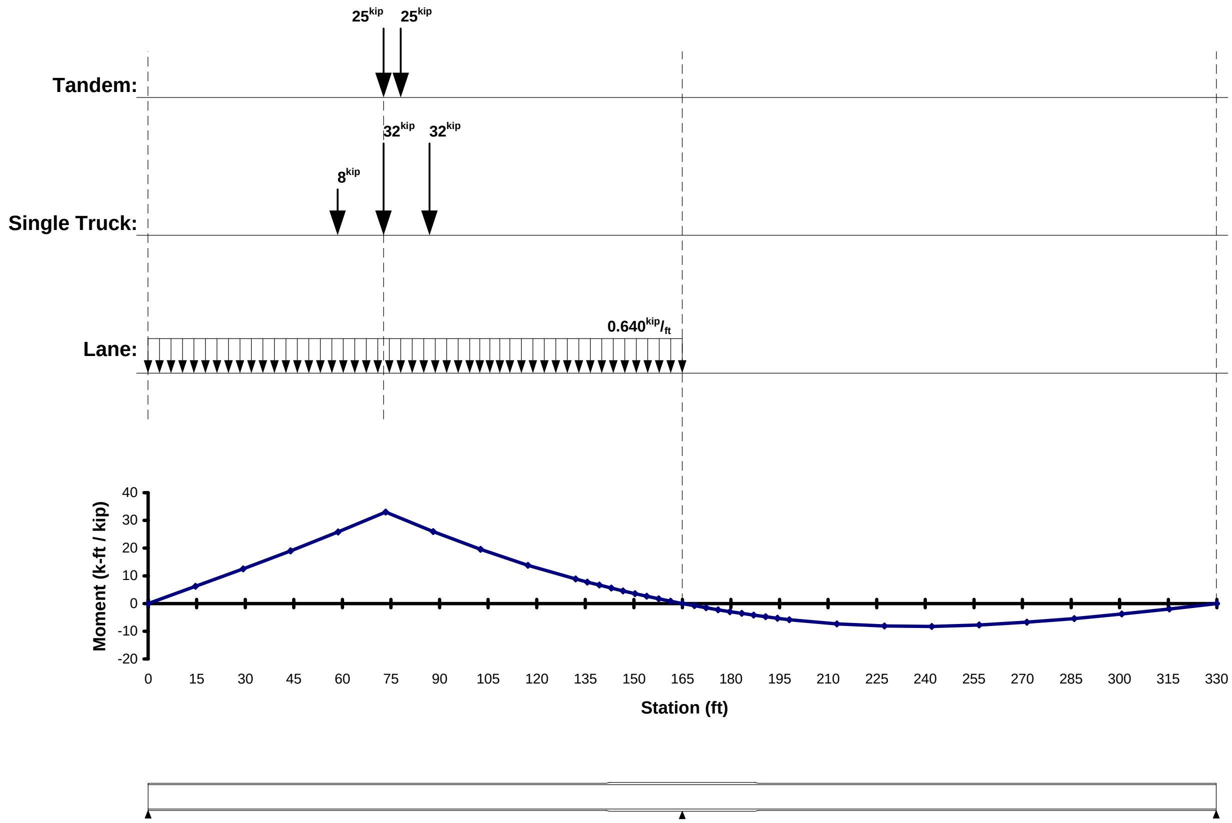
The following Live Load results were obtained from the SAP analysis:

- The maximum positive live-load moments occur at stations 73.3' and 256.7'
- The maximum negative live-load moments occur over the center support at station 165.0'

	Max (+) Moment Stations 73.3' and 256'	**Max (-) Moment Station 165'**
HL-93M	$3{,}725^{k\text{-}ft}$	$-3{,}737^{k\text{-}ft}$
HL-93K	$4{,}396^{k\text{-}ft}$	$-4{,}261^{k\text{-}ft}$
HL-93S	N/A	$-5{,}317^{k\text{-}ft}$
Fatigue	$2{,}327^{k\text{-}ft}$	$-1{,}095^{k\text{-}ft}$

Before proceeding, these live-load moments will be confirmed with an influence line analysis.

2.2.1: Verify the Maximum Positive Live-Load Moment at Station 73.3':



Tandem: $\left(25^{\text{kip}}\right)\left(33.00\,\frac{\text{k-ft}}{\text{kip}}\right)+\left(25^{\text{kip}}\right)\left(31.11\,\frac{\text{k-ft}}{\text{kip}}\right)=1{,}603^{\text{k-ft}}$

Single Truck: $\left(8^{\text{kip}}\right)\left(26.13\,\frac{\text{k-ft}}{\text{kip}}\right)+\left(32^{\text{kip}}\right)\left(33.00\,\frac{\text{k-ft}}{\text{kip}}\right)+\left(32^{\text{kip}}\right)\left(26.33\,\frac{\text{k-ft}}{\text{kip}}\right)=2{,}108^{\text{k-ft}}$

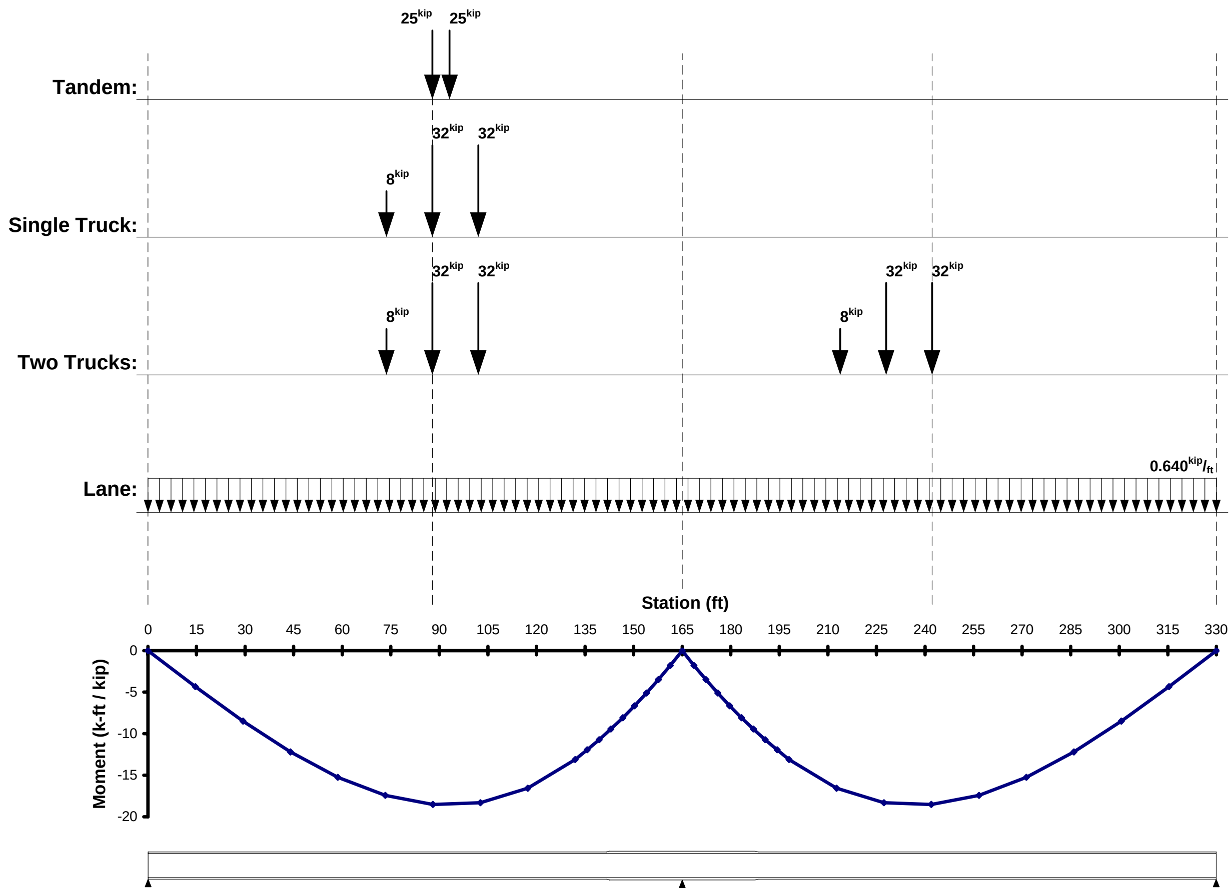
Lane Load: $\left(0.640\,\frac{\text{kip}}{\text{ft}}\right)\left(2{,}491\,\frac{\text{k-ft}^2}{\text{kip}}\right)=1{,}594^{\text{k-ft}}$

(IM)(Tandem) + Lane: $\left(1.33\right)\left(1{,}603^{\text{k-ft}}\right)+1{,}594^{\text{k-ft}}=3{,}726^{\text{k-ft}}$

(IM)(Single Truck) + Lane: $\left(1.33\right)\left(2{,}108^{\text{k-ft}}\right)+1{,}594^{\text{k-ft}}=4{,}397^{\text{k-ft}}$ **GOVERNS**

The case of two trucks is not considered here because it is only used when computing negative moments.

2.2.2: Verify the Maximum Negative Live-Load Moment at Station 165.0':



Tandem: $\left(25^{kip}\right)\left(18.51\frac{k\text{-}ft}{kip}\right)+\left(25^{kip}\right)\left(18.45\frac{k\text{-}ft}{kip}\right)=924.0^{k\text{-}ft}$

Single Truck: $\left(8^{kip}\right)\left(17.47\frac{k\text{-}ft}{kip}\right)+\left(32^{kip}\right)\left(18.51\frac{k\text{-}ft}{kip}\right)+\left(32^{kip}\right)\left(18.31\frac{k\text{-}ft}{kip}\right)=1{,}318^{k\text{-}ft}$

Two Trucks:

$$\left(8^{kip}\right)\left(17.47\frac{k\text{-}ft}{kip}\right)+\left(32^{kip}\right)\left(18.51\frac{k\text{-}ft}{kip}\right)+\left(32^{kip}\right)\left(18.31\frac{k\text{-}ft}{kip}\right)+\ldots$$
$$\ldots+\left(8^{kip}\right)\left(16.72\frac{k\text{-}ft}{kip}\right)+\left(32^{kip}\right)\left(18.31\frac{k\text{-}ft}{kip}\right)+\left(32^{kip}\right)\left(18.51\frac{k\text{-}ft}{kip}\right)=2{,}630^{k\text{-}ft}$$

Lane Load: $\left(0.640\frac{kip}{ft}\right)\left(3{,}918\frac{k\text{-}ft^2}{kip}\right)=2{,}508^{k\text{-}ft}$

(IM)(Tandem) + Lane: $\left(1.33\right)\left(924.0^{k\text{-}ft}\right)+2{,}508^{k\text{-}ft}=3{,}737^{k\text{-}ft}$

(IM)(Single Truck) + Lane: $\left(1.33\right)\left(1{,}318^{k\text{-}ft}\right)+2{,}508^{k\text{-}ft}=4{,}261^{k\text{-}ft}$

(0.90){(IM)(Two Trucks) + Lane}: $\left(0.90\right)\left[\left(1.33\right)\left(2{,}630^{k\text{-}ft}\right)+2{,}508^{k\text{-}ft}\right]=5{,}405^{k\text{-}ft}$ **GOVERNS**

Based on the influence line analysis, we can say that the moments obtained from SAP appear to be reasonable and will be used for design.

Before these Service moments can be factored and combined, we must compute the distribution factors. Since the distribution factors are a function of K_g, the longitudinal stiffness parameter, we must first compute the sections properties of the girders.

2.3: Braking Force

The Breaking Force, *BR,* is taken as the maximum of:

A) 25% of the Design Truck

$$BR_{Single\ Lane} = (0.25)\left(8^{\text{kip}} + 32^{\text{kip}} + 32^{\text{kip}}\right) = 18.00^{\text{kip}}$$

B) 25% of the Design Tandem

$$BR_{Single\ Lane} = (0.25)\left(25^{\text{kip}} + 25^{\text{kip}}\right) = 12.50^{\text{kip}}$$

C) 5% of the Design Truck with the Lane Load.

$$BR_{Single\ Lane} = (0.05)\left[\left(8^{\text{kip}} + 32^{\text{kip}} + 32^{\text{kip}}\right) + (2)(165')\left(0.640\,\tfrac{\text{kip}}{\text{ft}}\right)\right] = 14.16^{\text{kip}}$$

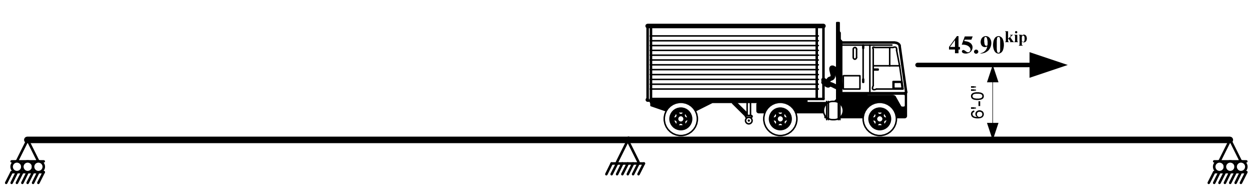
D) 5% of the Design Tandem with the Lane Load.

$$BR_{Single\ Lane} = (0.05)\left[\left(25^{\text{kip}} + 25^{\text{kip}}\right) + (2)(165')\left(0.640\,\tfrac{\text{kip}}{\text{ft}}\right)\right] = 13.06^{\text{kip}}$$

Case (A) Governs:

$$BR_{Net} = \left(BR_{Single\ Lane}\right)(\#\ Lanes)(MPF)$$
$$= \left(18.00^{\text{kip}}\right)(3)(0.85) = 45.90^{\text{kip}}$$

This load has <u>not</u> been factored…



2.4: Centrifugal Force

A centrifugal force results when a vehicle turns on a structure. Although a centrifugal force doesn't apply to this bridge since it is straight, the centrifugal load that would result from a hypothetical horizontal curve will be computed to illustrate the procedure.

The centrifugal force is computed as the product of the axle loads and the factor, *C*.

$$C = f\,\frac{v^2}{gR} \qquad (3.6.3\text{-}1)$$

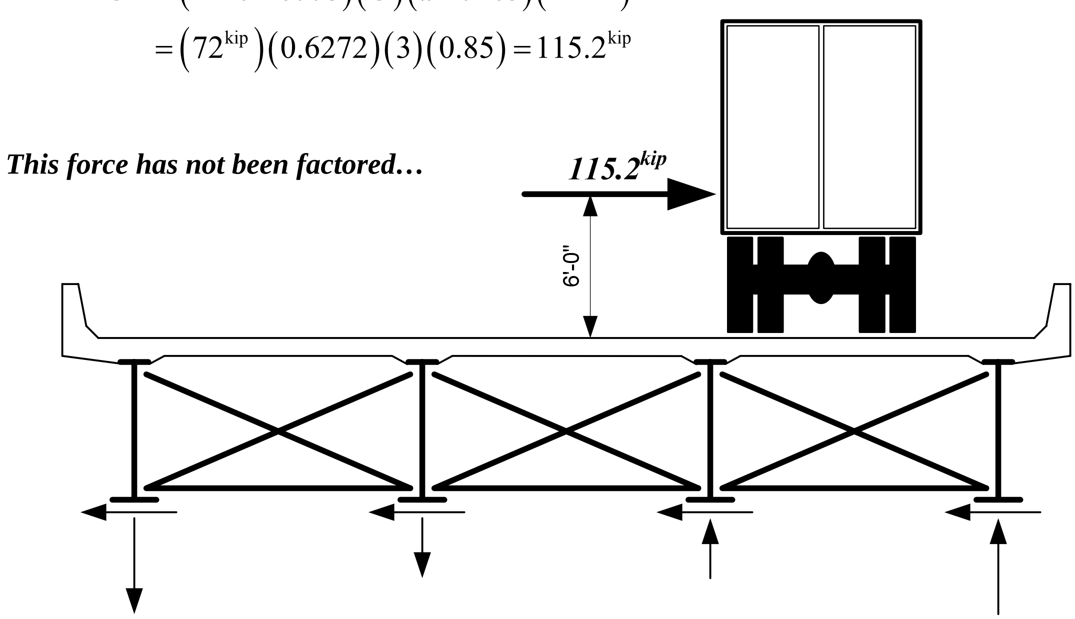
where:

- v - Highway design speed $\left(\frac{\text{ft}}{\text{sec}}\right)$
- f - $^4/_3$ for all load combinations except for Fatigue, in which case it is 1.0
- g - The acceleration of gravity $\left(\frac{\text{ft}}{\text{sec}^2}\right)$
- R - The radius of curvature for the traffic lane (ft).

Suppose that we have a radius of R = 600' and a design speed of $v = 65^{\text{mph}} = 95.33^{\text{ft}}/_{\text{sec}}$.

$$C = \left(\frac{4}{3}\right)\left[\frac{\left(95.33\,\frac{\text{ft}}{\text{sec}}\right)^2}{\left(32.2\,\frac{\text{ft}}{\text{sec}^2}\right)\left(600'\right)}\right] = 0.6272$$

$$\begin{aligned} CE &= \left(Axle\ Loads\right)\left(C\right)\left(\#\,Lanes\right)\left(MPF\right) \\ &= \left(72^{\text{kip}}\right)\left(0.6272\right)\left(3\right)\left(0.85\right) = 115.2^{\text{kip}} \end{aligned}$$



The centrifugal force acts horizontally in the direction pointing away from the center of curvature and at a height of 6' above the deck. Design the cross frames at the supports to carry this horizontal force into the bearings and design the bearings to resist the horizontal force and the resulting overturning moment.

2.5: Wind Loads

For the calculation of wind loads, assume that the bridge is located in the "open country" at an elevation of 40' above the ground.

Take Z = 40' Open Country $V_o = 8.20^{\text{mph}}$

$Z_o = 0.23^{\text{ft}}$

Horizontal Wind Load on Structure: *(WS)*

Design Pressure:

$$P_D = P_B\left(\frac{V_{DZ}}{V_B}\right)^2 = P_B\frac{V_{DZ}^{\;2}}{10{,}000^{\text{mph}^2}} \qquad (3.8.1.2.1\text{-}1)$$

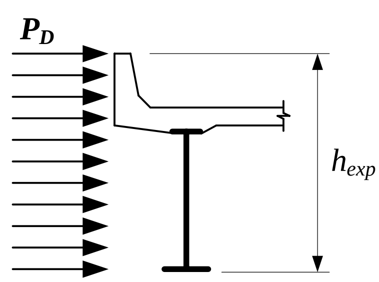
P_B - Base Pressure - For beams, $P_B = 50^{\text{psf}}$ when $V_B = 100^{\text{mph}}$. (Table 3.8.1.2.1-1)
V_B - Base Wind Velocity, typically taken as 100^{mph}.
V_{30} - Wind Velocity at an elevation of Z = 30' (mph)
V_{DZ} - Design Wind Velocity (mph)

Design Wind Velocity:

$$V_{DZ} = 2.5V_o\left(\frac{V_{30}}{V_B}\right)\ln\left(\frac{Z}{Z_o}\right) \qquad (3.8.1.1\text{-}1)$$

$$= (2.5)\left(8.20^{\text{mph}}\right)\left(\frac{100}{100}\right)\text{Ln}\left(\frac{40^{\text{ft}}}{0.23^{\text{ft}}}\right) = 105.8^{\text{mph}}$$

$$P_D = \left(50^{\text{psf}}\right)\frac{\left(105.8^{\text{mph}}\right)^2}{\left(10{,}000^{\text{mph}^2}\right)} = 55.92^{\text{psf}}$$



The height of exposure, h_{exp}, for the finished bridge is computed as

$$h_{exp} = 71.5\text{"} + 11.75\text{"} + 42\text{"} = 125.3\text{"} = 10.44'$$

The wind load per unit length of the bridge, W, is then computed as:

$$W = \left(55.92^{\text{psf}}\right)\left(10.44'\right) = 583.7\,\tfrac{\text{lbs}}{\text{ft}}$$

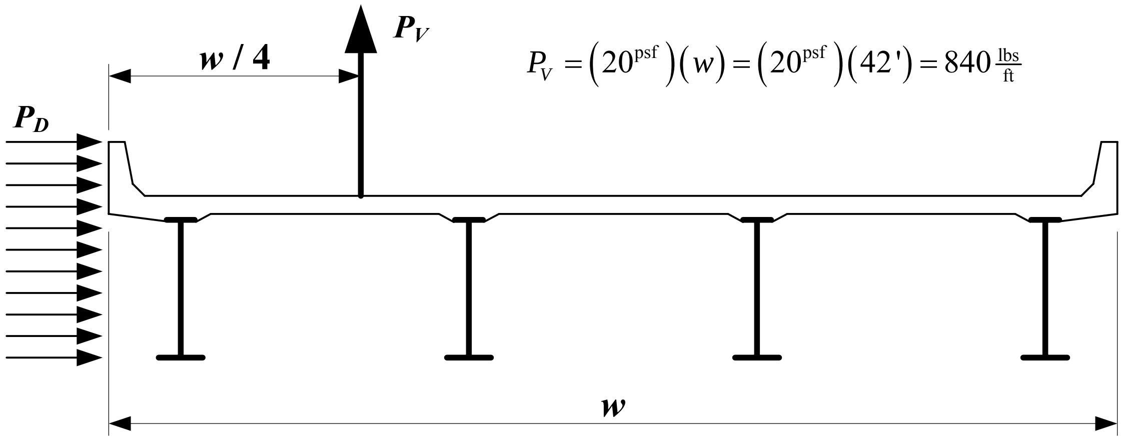
Total Wind Load: $WS_{H,Total} = \left(583.7\,\tfrac{\text{lbs}}{\text{ft}}\right)(2)\left(165'\right) = 192.6^{\text{kip}}$

For End Abutments: $WS_{H,Abt} = \left(583.7\,\tfrac{\text{lbs}}{\text{ft}}\right)\left(\tfrac{1}{2}\right)\left(165'\right) = 48.16^{\text{kip}}$

For Center Pier: $WS_{H,Pier} = \left(583.7\,\tfrac{\text{lbs}}{\text{ft}}\right)(2)\left(\tfrac{1}{2}\right)\left(165'\right) = 96.31^{\text{kip}}$

Vertical Wind Load on Structure: (*WS*)

When no traffic is on the bridge, a vertical uplift (a line load) with a magnitude equal to 20^{psf} times the overall width of the structure, *w*, acts at the windward quarter point of the deck.



$$P_V = \left(20^{psf}\right)\left(w\right) = \left(20^{psf}\right)\left(42'\right) = 840\,\tfrac{lbs}{ft}$$

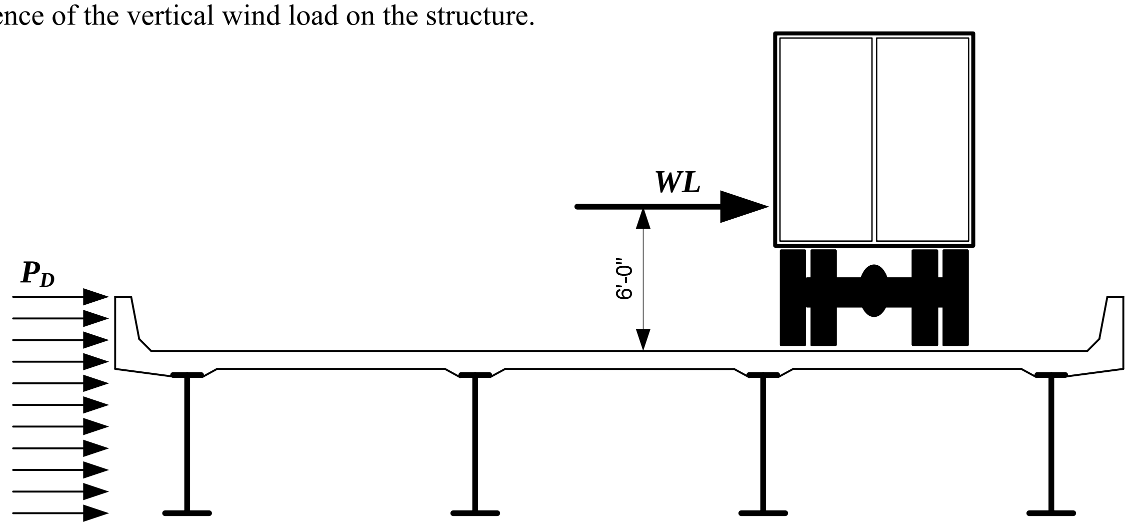
Total Uplift: $\left(840\,\tfrac{lbs}{ft}\right)\left(2\right)\left(165'\right) = 277.2^{kip}$

For End Abutments: $\left(840\,\tfrac{lbs}{ft}\right)\left(\tfrac{1}{2}\right)\left(165'\right) = 69.30^{kip}$

For Center Pier: $\left(840\,\tfrac{lbs}{ft}\right)\left(2\right)\left(\tfrac{1}{2}\right)\left(165'\right) = 138.6^{kip}$

Wind Load on Live Load: (*WL*)

The wind acting on live load is applied as a line load of 100 lbs/ft acting at a distance of 6' above the deck, as is shown below. This is applied along with the horizontal wind load on the structure but in the absence of the vertical wind load on the structure.



3. SECTION PROPERTIES AND CALCULATIONS:

3.1: Effective Flange Width, b_{eff}:

For an interior beam, b_{eff} is the lesser of:

$$\begin{cases} \bullet\ \dfrac{L_{eff}}{4} = \dfrac{132'}{4} = 33' = 396" \\ \bullet\ 12t_s + \dfrac{b_f}{2} = (12)(8.5") + \dfrac{15"}{2} = 109.5" \\ \bullet\ S = (12')(12\ ^{\text{in}}\!/_{\text{ft}}) = 144" \end{cases}$$

For an exterior beam, b_{eff} is the lesser of:

$$\begin{cases} \bullet\ \dfrac{L_{eff}}{4} = \dfrac{132'}{4} = 33' = 198.0" \\ \bullet\ 12t_s + \dfrac{b_f}{2} = (12)(8.5") + \dfrac{15"}{2} = 109.5" \\ \bullet\ \dfrac{S}{2} + d_e = \left(\dfrac{12'}{2} + 3'\right)\left(12\ \tfrac{\text{in}}{\text{ft}}\right) = 108.0" \end{cases}$$

Note that L_{eff} was taken as 132.0’ in the above calculations since for the case of effective width in continuous bridges, the span length is taken as the distance from the support to the point of dead load contra flexure.

For computing the section properties shown on the two pages that follow, reinforcing steel in the deck was ignored for short-term and long-term composite calculations but was included for the cracked section. The properties for the cracked Section #1 are not used in this example, thus the amount of rebar included is moot. For the properties of cracked Section #2, A_s = 13.02 in^2 located 4.5” from the top of the slab was taken from an underlying example problem first presented by Barth (2004).

3.2: Section 1 Flexural Properties

Bare Steel

	t	b	A	y	Ay	I_x	d	Ad^2	I_X
Top Flange	0.7500	15.00	11.25	70.38	791.72	0.53	-39.70	17,728	17,729
Web	0.5625	69.00	38.81	35.50	1,377.84	15,398.86	-4.82	902	16,301
Bot Flange	1.0000	21.00	21.00	0.50	10.50	1.75	30.18	19,125	19,127
			71.06		2,180.06			**I_{Total} =**	**53,157**
				Y =	30.68			**$S_{BS1,top}$ =**	**1,327**
								$S_{BS1,bot}$ =	**1,733**

Short-Term Composite (*n* = 8)

	t	b	A	y	Ay	I_x	d	Ad^2	I_X
Slab	8.5000	109.50	116.34	75.00	8,725.78	700.49	-16.81	32,862	33,562
Haunch	0.0000	15.00	0.00	70.75	0.00	0.00	-12.56	0	0
Top Flange	0.7500	15.0000	11.25	70.38	791.72	0.53	-12.18	1,669	1,670
Web	0.5625	69.0000	38.81	35.50	1,377.84	15,398.86	22.69	19,988	35,387
Bot Flange	1.0000	21.0000	21.00	0.50	10.50	1.75	57.69	69,900	69,901
			187.41		10,905.84			**I_{Total} =**	**140,521**
	n :	8.00							
				Y =	58.19			**$S_{ST1,top}$ =**	**11,191**
								$S_{ST1,bot}$ =	**2,415**

Long-Term Composite (*n* = 24)

	t	b	A	y	Ay	I_x	d	Ad^2	I_X
Slab	8.5000	109.50	38.78	75.00	2,908.59	233.50	-28.67	31,885	32,119
Haunch	0.0000	15.00	0.00	70.75	0.00	0.00	-24.42	0	0
Top Flange	0.7500	15.0000	11.25	70.38	791.72	0.53	-24.05	6,506	6,507
Web	0.5625	69.0000	38.81	35.50	1,377.84	15,398.86	10.83	4,549	19,948
Bot Flange	1.0000	21.0000	21.00	0.50	10.50	1.75	45.83	44,101	44,103
			109.84		5,088.66			**I_{Total} =**	**102,676**
	n :	24.00							
				Y =	46.33			**$S_{LT1,top}$ =**	**4,204**
								$S_{LT1,bot}$ =	**2,216**

Cracked Section

	t	b	A	y	Ay	I_x	d	Ad^2	I_X
Rebar	4.5000		13.02	75.25	979.76		-75.25	73,727	73,727
Top Flange	0.7500	15.0000	11.25	70.38	791.72	0.53	-70.38	55,717	55,718
Web	0.5625	69.0000	38.81	35.50	1,377.84	15,398.86	-35.50	48,913	64,312
Bot Flange	1.0000	21.0000	21.00	0.50	10.50	1.75	-0.50	5	7
			84.08		3,159.82			**I_{Total} =**	**193,764**
				Y =	37.58			**$S_{CR1,top}$ =**	**5,842**
								$S_{CR1,bot}$ =	**5,156**

These section properties do NOT include the haunch or sacrificial wearing surface.

3.3: Section 2 Flexural Properties

Bare Steel

	t	b	A	y	Ay	I_x	d	Ad^2	I_X
Top Flange	1.0000	21.00	21.00	72.00	1,512.00	1.75	-45.17	42,841	42,843
Web	0.5625	69.00	38.81	37.00	1,436.06	15,398.86	-10.17	4,012	19,411
Bot Flange	2.5000	21.00	52.50	1.25	65.63	27.34	25.58	34,361	34,388
			112.31		3,013.69			I_{Total} =	**96,642**
				Y =	26.83			$S_{BS2,top}$ =	**2,116**
								$S_{BS2,bot}$ =	**3,602**

Short Term Composite (*n* = 8)

	t	b	A	y	Ay	I_x	d	Ad^2	I_X
Slab	8.5000	109.50	116.34	76.75	8,929.38	700.49	-24.52	69,941	70,641
Haunch	0.0000	21.00	0.00	72.50	0.00	0.00	-20.27	0	0
Top Flange	1.0000	21.0000	21.00	72.00	1,512.00	1.75	-19.77	8,207	8,208
Web	0.5625	69.0000	38.81	37.00	1,436.06	15,398.86	15.23	9,005	24,403
Bot Flange	2.5000	21.0000	52.50	1.25	65.63	27.34	50.98	136,454	136,481
			228.66		11,943.07			I_{Total} =	**239,734**
	n :	8.00							
				Y =	52.23			$S_{ST2,top}$ =	**11,828**
								$S_{ST2,bot}$ =	**4,590**

Long-Term Composite (*n* = 24)

	t	b	A	y	Ay	I_x	d	Ad^2	I_X
Slab	8.5000	109.50	38.78	76.75	2,976.46	233.50	-37.10	53,393	53,626
Haunch	0.0000	15.00	0.00	72.50	0.00	0.00	-32.85	0	0
Top Flange	1.0000	21.0000	21.00	72.00	1,512.00	1.75	-32.35	21,983	21,985
Web	0.5625	69.0000	38.81	37.00	1,436.06	15,398.86	2.65	272	15,670
Bot Flange	2.5000	21.0000	52.50	1.25	65.63	27.34	38.40	77,395	77,423
			151.09		5,990.15			I_{Total} =	**168,704**
	n :	24.00							
				Y =	39.65			$S_{LT2,top}$ =	**5,135**
								$S_{LT2,bot}$ =	**4,255**

Cracked Section

	t	b	A	y	Ay	I_x	d	Ad^2	I_X
Rebar	4.5000		13.02	77.00	1,002.54		-44.96	26,313	26,313
Top Flange	1.0000	21.0000	21.00	72.00	1,512.00	1.75	-39.96	33,525	33,527
Web	0.5625	69.0000	38.81	37.00	1,436.06	15,398.86	-4.96	953	16,352
Bot Flange	2.5000	21.0000	52.50	1.25	65.63	27.34	30.79	49,786	49,813
			125.33		4,016.23			I_{Total} =	**126,006**
				Y =	32.04			$S_{CR2,top}$ =	**3,115**
								$S_{CR2,bot}$ =	**3,932**

These section properties do NOT include the haunch or sacrificial wearing surface.

4. DISTRIBUTION FACTOR FOR MOMENT

4.1: Positive Moment Region (Section 1):

Interior Girder –

One Lane Loaded:

$$DF_{M1,Int+} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$K_g = n(I + Ae_g^2)$$

$$K_g = 8(53{,}157 \text{ in}^4 + (71.06 \text{ in}^2)(46.82")^2)$$

$$K_g = 1{,}672{,}000 \text{ in}^4$$

$$DF_{M1,Int+} = 0.06 + \left(\frac{12'}{14}\right)^{0.4}\left(\frac{12'}{165'}\right)^{0.3}\left(\frac{1{,}672{,}000 \text{ in}^4}{(12)(165')(8.5")^3}\right)^{0.1}$$

$$DF_{M1,Int+} = 0.5021$$

In these calculations, the terms e_g and K_g include the haunch and sacrificial wearing surface since doing so increases the resulting factor. Note that t_s in the denominator of the final term excludes the sacrificial wearing surface since excluding it increases the resulting factor.

Two or More Lanes Loaded:

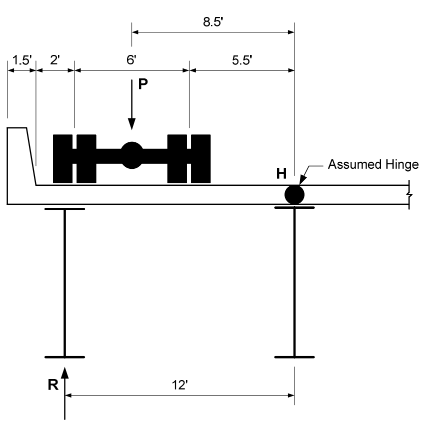
$$DF_{M2,Int+} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$DF_{M2,Int+} = 0.075 + \left(\frac{12'}{9.5}\right)^{0.6}\left(\frac{12'}{165'}\right)^{0.2}\left(\frac{1{,}672{,}000 \text{ in}^4}{12(165')(8.5")^3}\right)^{0.1}$$

$$DF_{M2,Int+} = 0.7781$$

Exterior Girder –

One Lane Loaded:



The lever rule is applied by assuming that a hinge forms over the first interior girder as a truck load is applied near the parapet. The resulting reaction in the exterior girder is the distribution factor.

$$DF_{M1,Ext+} = \frac{8.5}{12} = 0.7083$$

Multiple Presence: $DF_{M1,Ext+}$ = (1.2) (0.7083) = 0.8500

Two or More Lanes Loaded:

$DF_{M2,Ext+} = e\, DF_{M2,Int+}$

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

$$= 0.77 + \frac{1.5}{9.1} = 0.9348$$

$DF_{M2,Ext+} = (0.9348)\,(0.7781) = 0.7274$

4.2: Negative Moment Region (Section 2):

The span length used for negative moment near the pier is the average of the lengths of the adjacent spans. In this case, it is the average of 165.0' and 165.0' = 165.0'.

Interior Girder –

One Lane Loaded:

$$DF_{M1,Int-} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$K_g = n(I + Ae_g^2)$$

$$K_g = 8(96,642\text{ in}^4 + (112.3\text{ in}^2)(52.17")^2)$$

$$K_g = 3,218,000\text{ in}^4$$

$$DF_{M1,Int-} = 0.06 + \left(\frac{12'}{14}\right)^{0.4}\left(\frac{12'}{165'}\right)^{0.3}\left(\frac{3,218,000\text{ in}^4}{(12)(165')(8.5")^3}\right)^{0.1}$$

$$DF_{M1,Int-} = 0.5321$$

Two or More Lanes Loaded:

$$DF_{M2,Int-} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$DF_{M2,Int-} = 0.075 + \left(\frac{12'}{9.5}\right)^{0.6}\left(\frac{12'}{165'}\right)^{0.2}\left(\frac{3,218,000\text{ in}^4}{(12)(165')(8.5")^3}\right)^{0.1}$$

$$DF_{M2,Int-} = 0.8257$$

Exterior Girder –

One Lane Loaded:

Same as for the positive moment section: $DF_{M1,Ext-} = 0.8500$

Two or More Lanes Loaded:

$DF_{M2,Ext-} = e\ DF_{M2,Int-}$

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

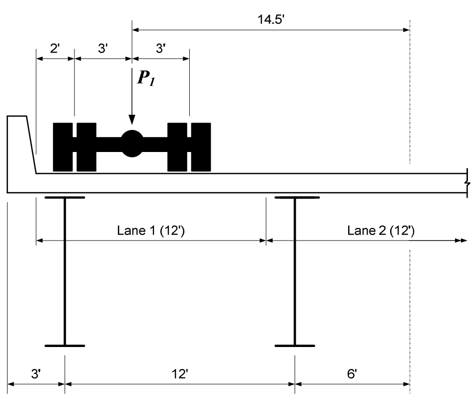
$$= 0.77 + \frac{1.5}{9.1} = 0.9348$$

$DF_{M2,Ext-} = (0.9348)\ (0.8257) = 0.7719$

4.3: Minimum Exterior Girder Distribution Factor:

$$DF_{Ext,Min} = \frac{N_L}{N_b} + \frac{X_{Ext}\sum^{N_L} e}{\sum^{N_b} x^2}$$

One Lane Loaded:

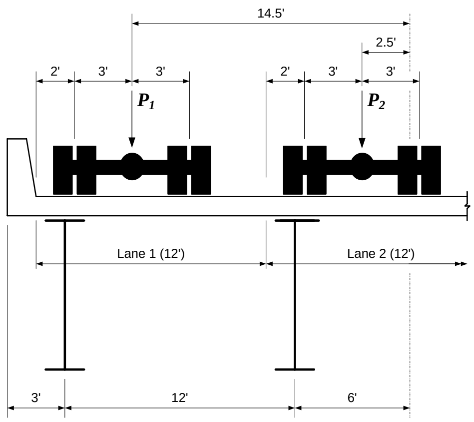


$$DF_{M1,Ext,Min} = \frac{1}{4} + \frac{(18.0')(14.5')}{(2)\left[(18')^2 + (6')^2\right]} = 0.6125$$

Multiple Presence:

$DF_{M1,Ext,Min} = (1.2)\ (0.6125) = 0.7350$

Two Lanes Loaded:



$$DF_{M2,Ext,Min} = \frac{2}{4} + \frac{(18.0')(14.5'+2.5')}{(2)\left[(18')^2+(6')^2\right]} = 0.9250$$

Multiple Presence:

$DF_{M2,Ext,Min}$ = (1.0) (0.9250) = 0.9250

Three Lanes Loaded:

The case of three lanes loaded is not considered for the minimum exterior distribution factor since the third truck will be placed to the right of the center of gravity of the girders, which will stabilize the rigid body rotation effect resulting in a lower factor.

4.4: Moment Distribution Factor Summary

Strength and Service Moment Distribution:

	Positive Moment		Negative Moment	
	Interior	**Exterior**	**Interior**	**Exterior**
1 Lane Loaded:	0.5021	0.8500 ≥ 0.7350	0.5321	0.8500 ≥ 0.7350
2 Lanes Loaded:	0.7781	0.7274 ≥ 0.9250	0.8257	0.7719 ≥ 0.9250

For Simplicity, take the Moment Distribution Factor as 0.9250 everywhere for the Strength and Service load combinations.

Fatigue Moment Distribution:

For Fatigue, the distribution factor is based on the one-lane-loaded situations with a multiple presence factor of 1.00. Since the multiple presence factor for 1-lane loaded is 1.2, these factors can be obtained by divided the first row of the table above by 1.2.

	Positive Moment		Negative Moment	
	Interior	**Exterior**	**Interior**	**Exterior**
1 Lane Loaded:	0.4184	0.7083 ≥ 0.6125	0.4434	0.7083 ≥ 0.6125

For Simplicity, take the Moment Distribution Factor as 0.7083 everywhere for the Fatigue load combination

Multiplying the live load moments by this distribution factor of 0.9250 yields the table of "nominal" girder moments shown on the following page.

Nominal Girder Moments for Design

			Nominal Moments				
Station	**(LL+IM)+**	**(LL+IM)-**	**Fat+**	**Fat-**	**DC1**	**DC2**	**DW**
(ft)	**(k-ft)**	**(k-ft)**	**(k-ft)**	**(k-ft)**	**(k-ft)**	**(k-ft)**	**(k-ft)**
0.0	0.0	0.0	0.2	0.0	0.0	0.0	0.0
14.7	1605.1	-280.7	645.6	-68.9	1309.9	240.0	440.3
29.3	2791.4	-561.3	1127.9	-137.9	2244.5	412.0	755.6
44.0	3572.6	-842.0	1449.4	-206.8	2799.9	515.0	944.7
58.7	3999.4	-1122.7	1626.1	-275.8	2978.6	549.7	1008.3
73.3	4066.7	-1403.4	1647.9	-344.7	2779.3	515.8	946.1
88.0	3842.5	-1684.0	1599.4	-413.7	2202.1	413.2	757.9
102.7	3310.8	-1964.7	1439.3	-482.6	1248.4	242.3	444.4
117.3	2509.4	-2245.4	1148.6	-551.6	-84.8	2.5	4.6
132.0	1508.6	-2547.5	763.6	-620.5	-1793.1	-305.4	-560.2
135.7	1274.6	-2660.0	651.3	-637.8	-2280.8	-393.2	-721.2
139.3	1048.4	-2793.3	539.1	-655.0	-2794.0	-485.2	-890.0
143.0	828.6	-2945.6	425.3	-672.2	-3333.2	-581.5	-1066.7
146.7	615.8	-3115.6	310.8	-689.5	-3898.1	-682.1	-1251.3
150.3	463.3	-3371.3	221.9	-706.7	-4488.6	-787.0	-1443.7
154.0	320.5	-3728.6	158.6	-724.0	-5105.1	-896.2	-1643.9
157.7	185.5	-4105.0	98.8	-741.2	-5747.2	-1009.7	-1852.1
161.3	76.4	-4496.9	49.4	-758.4	-6415.3	-1127.5	-2068.1
165.0	0.0	-4918.1	0.1	-775.6	-7108.8	-1249.5	-2291.9
168.7	76.4	-4496.9	49.4	-758.4	-6415.3	-1127.5	-2068.1
172.3	185.5	-4105.0	98.8	-741.2	-5747.2	-1009.7	-1852.1
176.0	320.5	-3728.6	158.6	-724.0	-5105.1	-896.2	-1643.9
179.7	463.3	-3371.3	221.9	-706.7	-4488.6	-787.0	-1443.7
183.3	615.8	-3115.6	310.8	-689.5	-3898.1	-682.1	-1251.3
187.0	828.6	-2945.6	425.3	-672.2	-3333.2	-581.5	-1066.7
190.7	1048.4	-2793.3	539.1	-655.0	-2794.0	-485.2	-890.0
194.3	1274.6	-2660.0	651.3	-637.8	-2280.8	-393.2	-721.2
198.0	1508.6	-2547.5	763.2	-620.6	-1793.1	-305.4	-560.2
212.7	2509.4	-2245.4	1148.6	-551.6	-84.8	2.5	4.6
227.3	3310.8	-1964.7	1439.3	-482.6	1248.4	242.3	444.4
242.0	3842.5	-1684.0	1599.4	-413.7	2202.1	413.2	757.9
256.7	4066.7	-1403.4	1647.9	-344.7	2779.3	515.8	946.1
271.3	3999.4	-1122.7	1626.1	-275.8	2978.6	549.7	1008.3
286.0	3572.6	-842.0	1449.4	-206.8	2799.9	515.0	944.7
300.7	2791.4	-561.3	1127.9	-137.9	2244.5	412.0	755.6
315.3	1605.1	-280.7	645.6	-68.9	1309.9	240.0	440.3
330.0	0.0	0.0	0.2	0.0	0.0	0.0	0.0

5. DISTRIBUTION FACTOR FOR SHEAR

The distribution factors for shear are independent of the section properties and span length. Thus, the only one set of calculations are need - they apply to both the section 1 and section 2

5.1: Interior Girder –

One Lane Loaded:

$$DF_{V1,Int} = 0.36 + \frac{S}{25.0}$$
$$= 0.36 + \frac{12'}{25.0} = 0.8400$$

Two or More Lanes Loaded:

$$DF_{V2,Int} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2$$
$$= 0.2 + \frac{12'}{12} - \left(\frac{12'}{35}\right)^2 = 1.082$$

5.2: Exterior Girder –

One Lane Loaded:

Lever Rule, which is the same as for moment: $DF_{V1,Ext} = 0.8500$

Two or More Lanes Loaded:

$DF_{V2,Ext} = e\ DF_{V2,Int}$

$$e = 0.60 + \frac{d_e}{10}$$
$$= 0.60 + \frac{1.5'}{10} = 0.7500$$

$DF_{V2,Ext}$ = (0.7500) (1.082) = 0.8115

5.3: Minimum Exterior Girder Distribution Factor -

The minimum exterior girder distribution factor applies to shear as well as moment.

$DF_{V1,Ext,Min} = 0.7350$

$DF_{V2,Ext,Min} = 0.9250$

5.4: Shear Distribution Factor Summary

Strength and Service Shear Distribution:

	Shear Distribution	
	Interior	**Exterior**
1 Lane Loaded:	0.8400	0.8500 ≥ 0.7350
2 Lanes Loaded:	1.082	0.6300 ≥ 0.9250

For Simplicity, take the Shear Distribution Factor as 1.082 everywhere for Strength and Service load combinations.

Fatigue Shear Distribution:

For Fatigue, the distribution factor is based on the one-lane-loaded situations with a multiple presence factor of 1.00. Since the multiple presence factor for 1-lane loaded is 1.2, these factors can be obtained by divided the first row of the table above by 1.2.

	Shear Distribution	
	Interior	**Exterior**
1 Lane Loaded:	0.7000	0.7083 ≥ 0.6125

For Simplicity, take the Shear Distribution Factor as 0.7083 everywhere for the Fatigue load combination.

Multiplying the live load shears by these distribution factors yields the table of "nominal" girder shears shown on the following page.

Nominal Girder Shears for Design

			Nominal Shears				
Station	**(LL+IM)+**	**(LL+IM)-**	**Fat+**	**Fat-**	**DC1**	**DC2**	**DW**
(ft)	**(kip)**	**(kip)**	**(kip)**	**(kip)**	**(kip)**	**(kip)**	**(kip)**
0.0	144.9	-19.7	50.8	-4.7	115.0	20.6	37.6
14.7	123.5	-20.3	44.6	-4.7	88.8	15.9	29.0
29.3	103.5	-26.8	38.5	-6.4	62.5	11.2	20.5
44.0	85.0	-41.4	32.6	-11.1	36.3	6.5	11.9
58.7	68.1	-56.7	26.9	-17.2	10.1	1.8	3.3
73.3	52.8	-72.7	21.4	-23.2	-16.1	-2.9	-5.3
88.0	39.4	-89.1	16.3	-29.0	-42.3	-7.6	-13.9
102.7	27.8	-105.7	11.5	-34.6	-68.6	-12.3	-22.4
117.3	18.0	-122.3	7.3	-39.9	-94.8	-17.0	-31.0
132.0	10.0	-138.6	3.9	-44.9	-121.0	-21.7	-39.6
135.7	8.3	-142.5	3.4	-46.0	-127.6	-22.8	-41.7
139.3	6.7	-146.5	2.8	-47.2	-134.1	-24.0	-43.9
143.0	5.5	-150.5	2.3	-48.3	-140.7	-25.2	-46.0
146.7	4.3	-154.5	1.8	-49.4	-147.2	-26.4	-48.2
150.3	3.2	-158.4	1.4	-50.4	-153.8	-27.5	-50.3
154.0	2.2	-162.3	1.0	-51.5	-160.3	-28.7	-52.5
157.7	1.3	-166.2	0.6	-52.4	-166.9	-29.9	-54.6
161.3	0.0	-170.1	0.3	-53.4	-173.4	-31.0	-56.8
165.0	0.0	-173.9	54.3	-54.3	-180.0	-32.2	-58.9
168.7	170.1	-0.5	53.4	-0.3	173.4	31.0	56.8
172.3	166.2	-1.3	52.4	-0.6	166.9	29.9	54.6
176.0	162.3	-2.2	51.5	-1.0	160.3	28.7	52.5
179.7	158.4	-3.2	50.4	-1.4	153.8	27.5	50.3
183.3	154.5	-4.3	49.4	-1.8	147.2	26.4	48.2
187.0	150.5	-5.5	48.3	-2.3	140.7	25.2	46.0
190.7	146.5	-6.7	47.2	-2.8	134.1	24.0	43.9
194.3	142.5	-8.3	46.0	-3.4	127.6	22.8	41.7
198.0	138.6	-10.0	44.9	-3.9	121.0	21.7	39.6
212.7	122.3	-18.0	39.9	-7.3	94.8	17.0	31.0
227.3	105.7	-27.8	34.6	-11.5	68.6	12.3	22.4
242.0	89.1	-39.4	29.0	-16.3	42.3	7.6	13.9
256.7	72.7	-52.8	23.2	-21.4	16.1	2.9	5.3
271.3	56.7	-68.1	17.2	-26.9	-10.1	-1.8	-3.3
286.0	41.4	-85.0	11.1	-32.6	-36.3	-6.5	-11.9
300.7	26.8	-103.5	6.4	-38.5	-62.5	-11.2	-20.5
315.3	20.3	-123.5	4.7	-44.6	-88.8	-15.9	-29.0
330.0	19.7	-144.9	4.7	-50.8	-115.0	-20.6	-37.6

6. FACTORED SHEAR AND MOMENT ENVELOPES

The following load combinations were considered in this example:

Strength I:	$1.75(LL + IM) + 1.25DC1 + 1.25DC2 + 1.50DW$
Strength IV:	$1.50DC1 + 1.50DC2 + 1.50DW$
Service II:	$1.3(LL + IM) + 1.0DC1 + 1.0DC2 + 1.0DW$
Fatigue:	$0.75(LL + IM)$ (IM = 15% for Fatigue; IM = 33% otherwise)

Strength II is not considered since this deals with special permit loads. Strength III and V are not considered as they include wind effects, which will be handled separately as needed. Strength IV is considered but is not expected to govern since it addresses situations with high dead load that come into play for longer spans. Extreme Event load combinations are not included as they are also beyond the scope of this example. Service I again applies to wind loads and is not considered (except for deflection) and Service III and Service IV correspond to tension in prestressed concrete elements and are therefore not included in this example.

In addition to the factors shown above, a load modifier, η, was applied as is shown below.

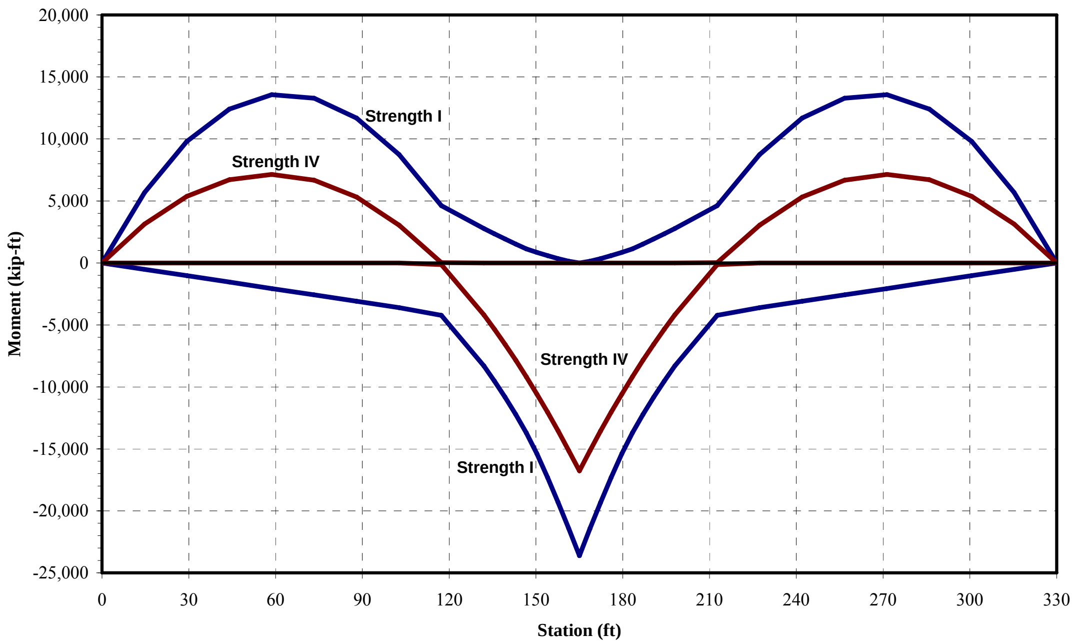
$$Q = \sum \eta_i \gamma_i Q_i$$

η is taken as the product of η_D, η_R, and η_I, and is taken as not less than 0.95. For this example, η_D and η_I are taken as 1.00 while η_R is taken as 1.05 since the bridge has 4 girders with a spacing greater than or equal to 12'.

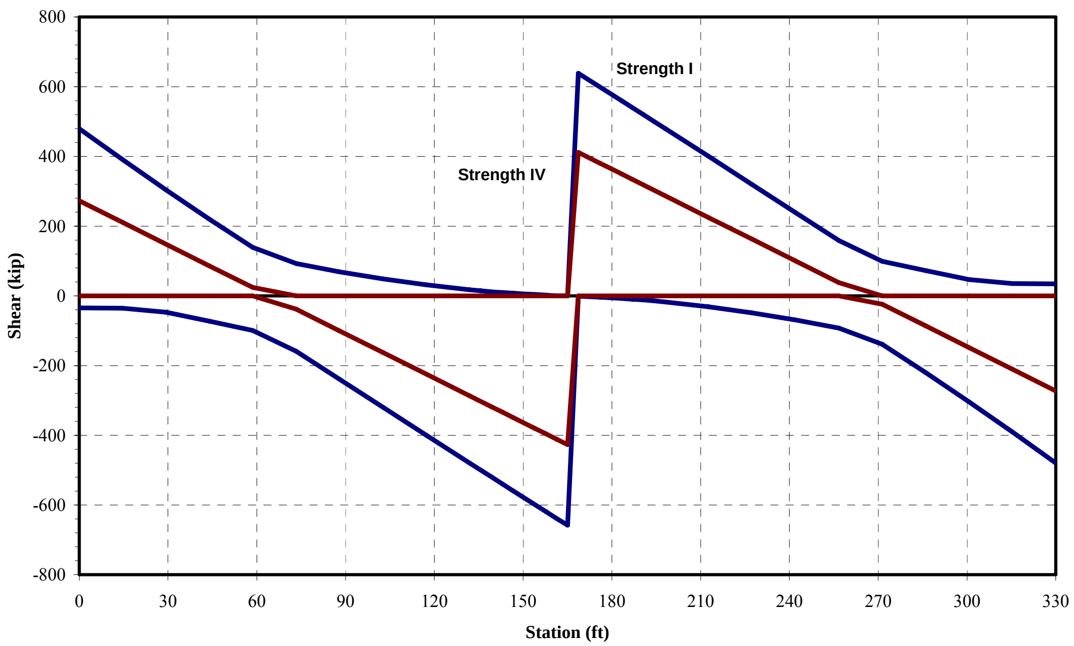
Using these load combinations, the shear and moment envelopes shown on the following pages were developed.

Note that for the calculation of the Fatigue moments and shears that η is taken as 1.00 and the distribution factor is based on the one-lane-loaded situations with a multiple presence factor of 1.00 (AASHTO Sections 6.6.1.2.2, Page 6-29 and 3.6.1.4.3b, Page 3-25).

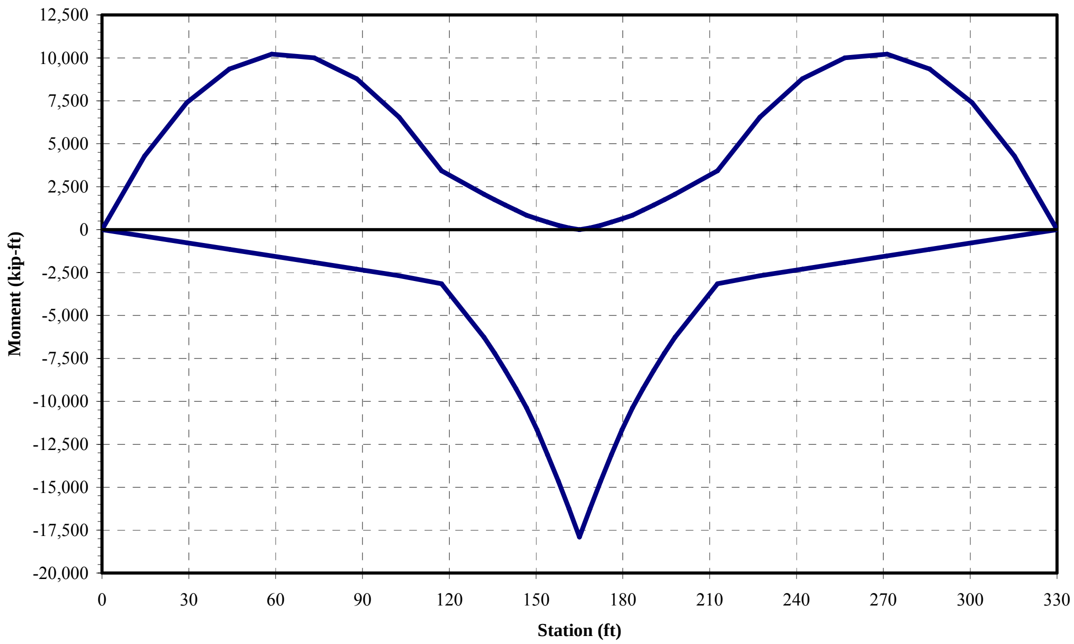
Strength Limit Moment Envelopes



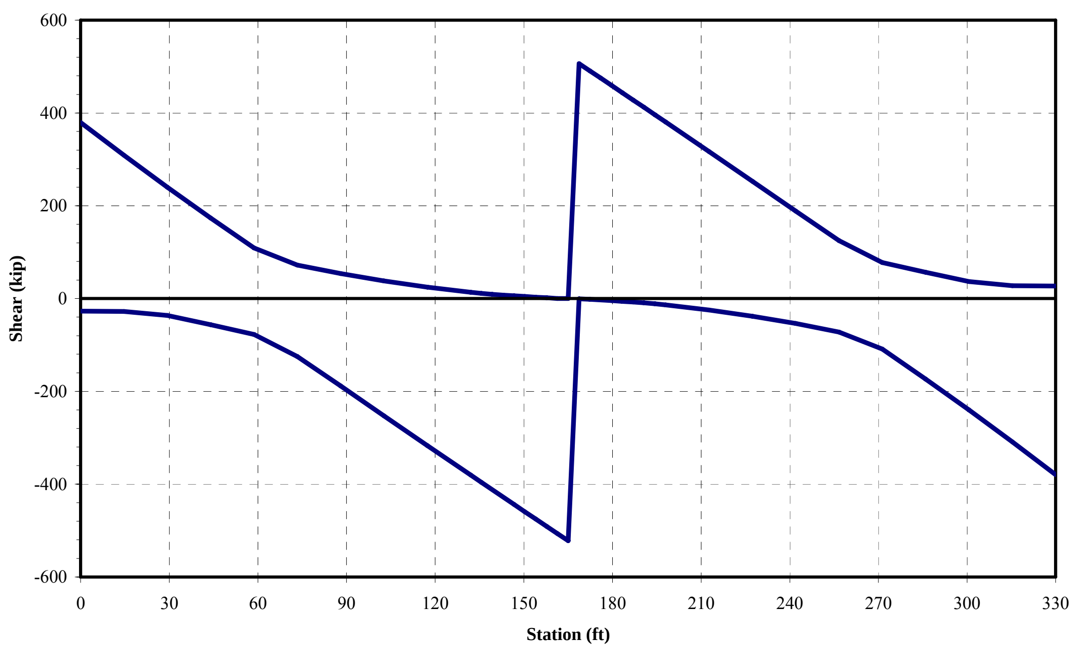
Strength Limit Shear Force Envelope



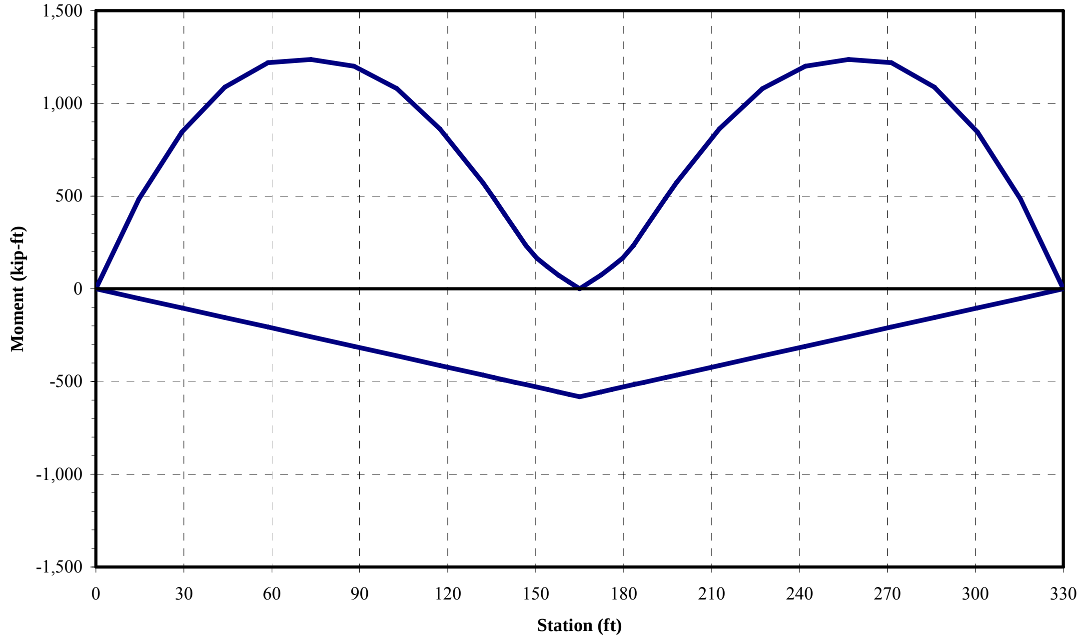
Service II Moment Envelope



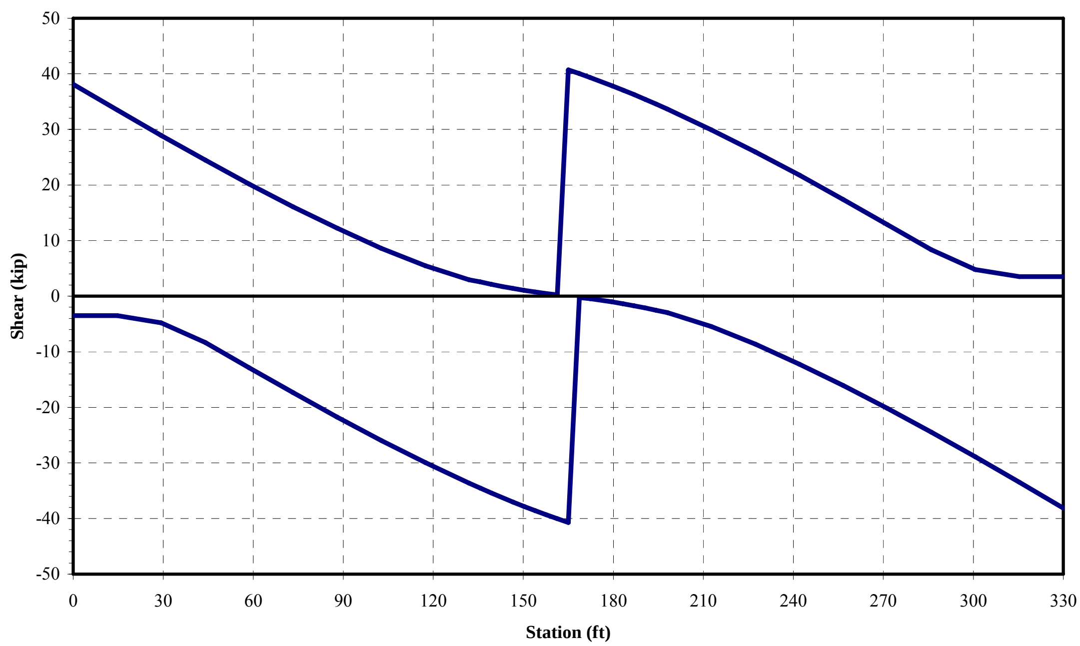
Service II Shear Envelope



Factored Fatigue Moment Envelope



Factored Fatigue Shear Envelope



Factored Girder Moments for Design

Station	Strength I		Strength IV		Service II		Fatigue	
	Total +	Total -	Total +	Total -	Total +	Total -	Total +	Total -
(ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0
14.7	5677.1	-515.7	3134.6	0.0	4280.7	-383.1	484.2	-51.7
29.3	9806.0	-1031.5	5374.1	0.0	7393.0	-766.2	845.9	-103.4
44.0	12403.3	-1547.2	6708.8	0.0	9349.1	-1149.4	1087.1	-155.1
58.7	13567.8	-2062.9	7145.1	0.0	10222.6	-1532.5	1219.6	-206.8
73.3	13287.4	-2578.7	6679.8	0.0	10004.2	-1915.6	1235.9	-258.6
88.0	11687.1	-3094.4	5312.9	0.0	8787.0	-2298.7	1199.5	-310.3
102.7	8740.0	-3610.2	3047.7	0.0	6551.1	-2681.8	1079.5	-362.0
117.3	4621.6	-4237.1	11.2	-133.5	3432.8	-3153.9	861.5	-413.7
132.0	2772.1	-8317.5	0.0	-4187.3	2059.3	-6268.9	572.7	-465.4
135.7	2342.0	-9533.2	0.0	-5347.3	1739.8	-7195.8	488.5	-478.3
139.3	1926.4	-10838.2	0.0	-6566.4	1431.1	-8190.4	404.3	-491.3
143.0	1522.6	-12230.6	0.0	-7845.7	1131.1	-9251.2	318.9	-504.2
146.7	1131.6	-13707.1	0.0	-9184.5	840.6	-10375.8	233.1	-517.1
150.3	851.2	-15392.8	0.0	-10582.9	632.3	-11657.1	166.5	-530.0
154.0	588.9	-17317.3	0.0	-12041.3	437.4	-13117.1	119.0	-543.0
157.7	340.9	-19328.3	0.0	-13559.1	253.3	-14642.7	74.1	-555.9
161.3	140.4	-21420.1	0.0	-15137.1	104.3	-16229.6	37.1	-568.8
165.0	0.0	-23617.1	0.0	-16774.1	0.0	-17895.9	0.1	-581.7
168.7	140.4	-21420.1	0.0	-15137.1	104.3	-16229.6	37.1	-568.8
172.3	340.9	-19328.3	0.0	-13559.1	253.3	-14642.7	74.1	-555.9
176.0	588.9	-17317.3	0.0	-12041.3	437.4	-13117.1	119.0	-543.0
179.7	851.2	-15392.8	0.0	-10582.9	632.3	-11657.1	166.5	-530.0
183.3	1131.6	-13707.1	0.0	-9184.5	840.6	-10375.8	233.1	-517.1
187.0	1522.6	-12230.6	0.0	-7845.7	1131.1	-9251.2	318.9	-504.2
190.7	1926.4	-10838.2	0.0	-6566.4	1431.1	-8190.4	404.3	-491.3
194.3	2342.0	-9533.2	0.0	-5347.3	1739.8	-7195.8	488.5	-478.3
198.0	2772.1	-8317.5	0.0	-4187.3	2059.3	-6268.9	572.4	-465.4
212.7	4621.6	-4237.1	11.2	-133.5	3432.8	-3153.9	861.5	-413.7
227.3	8740.0	-3610.2	3047.7	0.0	6551.1	-2681.8	1079.5	-362.0
242.0	11687.1	-3094.4	5312.9	0.0	8787.0	-2298.7	1199.5	-310.3
256.7	13287.4	-2578.7	6679.8	0.0	10004.2	-1915.6	1235.9	-258.6
271.3	13567.8	-2062.9	7145.1	0.0	10222.6	-1532.5	1219.6	-206.8
286.0	12403.3	-1547.2	6708.8	0.0	9349.1	-1149.4	1087.1	-155.1
300.7	9806.0	-1031.5	5374.1	0.0	7393.0	-766.2	845.9	-103.4
315.3	5677.1	-515.7	3134.6	0.0	4280.7	-383.1	484.2	-51.7
330.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0

Factored Girder Shears for Design

Station	Strength I		Strength IV		Service II		Fatigue	
	Total +	Total -	Total +	Total -	Total +	Total -	Total +	Total -
(ft)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)
0.0	479.5	-34.5	272.8	0.0	379.7	-26.9	38.1	-3.5
14.7	390.5	-35.5	210.6	0.0	309.0	-27.7	33.5	-3.5
29.3	304.0	-46.9	148.4	0.0	240.2	-36.6	28.9	-4.8
44.0	220.1	-72.4	86.2	0.0	173.4	-56.5	24.5	-8.3
58.7	138.9	-99.3	24.0	0.0	108.9	-77.5	20.2	-12.9
73.3	92.5	-158.9	0.0	-38.2	72.1	-124.8	16.1	-17.4
88.0	68.9	-239.1	0.0	-100.4	53.8	-188.6	12.2	-21.8
102.7	48.6	-319.7	0.0	-162.6	37.9	-252.7	8.6	-26.0
117.3	31.5	-400.1	0.0	-224.8	24.6	-316.8	5.5	-29.9
132.0	17.5	-480.2	0.0	-287.0	13.7	-380.5	3.0	-33.7
135.7	14.5	-500.0	0.0	-302.6	11.3	-396.3	2.5	-34.5
139.3	11.7	-519.8	0.0	-318.1	9.2	-412.1	2.1	-35.4
143.0	9.6	-539.7	0.0	-333.7	7.5	-427.9	1.7	-36.2
146.7	7.6	-559.6	0.0	-349.2	5.9	-443.7	1.4	-37.0
150.3	5.7	-579.3	0.0	-364.8	4.4	-459.4	1.0	-37.8
154.0	3.9	-599.0	0.0	-380.3	3.0	-475.1	0.8	-38.6
157.7	2.2	-618.7	0.0	-395.9	1.7	-490.8	0.5	-39.3
161.3	0.0	-638.3	0.0	-411.4	0.0	-506.4	0.2	-40.0
165.0	0.0	-657.9	0.0	-427.0	0.0	-522.0	40.7	-40.7
168.7	638.3	-0.9	411.4	0.0	506.4	-0.7	40.0	-0.2
172.3	618.7	-2.2	395.9	0.0	490.8	-1.7	39.3	-0.5
176.0	599.0	-3.9	380.3	0.0	475.1	-3.0	38.6	-0.8
179.7	579.3	-5.7	364.8	0.0	459.4	-4.4	37.8	-1.0
183.3	559.6	-7.6	349.2	0.0	443.7	-5.9	37.0	-1.4
187.0	539.7	-9.6	333.7	0.0	427.9	-7.5	36.2	-1.7
190.7	519.8	-11.7	318.1	0.0	412.1	-9.2	35.4	-2.1
194.3	500.0	-14.5	302.6	0.0	396.3	-11.3	34.5	-2.5
198.0	480.2	-17.5	287.0	0.0	380.5	-13.7	33.7	-2.9
212.7	400.1	-31.5	224.8	0.0	316.8	-24.6	29.9	-5.5
227.3	319.7	-48.6	162.6	0.0	252.7	-37.9	26.0	-8.6
242.0	239.1	-68.9	100.4	0.0	188.6	-53.8	21.8	-12.2
256.7	158.9	-92.5	38.2	0.0	124.8	-72.1	17.4	-16.1
271.3	99.3	-138.9	0.0	-24.0	77.5	-108.9	12.9	-20.2
286.0	72.4	-220.1	0.0	-86.2	56.5	-173.4	8.3	-24.5
300.7	46.9	-304.0	0.0	-148.4	36.6	-240.2	4.8	-28.9
315.3	35.5	-390.5	0.0	-210.6	27.7	-309.0	3.5	-33.5
330.0	34.5	-479.5	0.0	-272.8	26.9	-379.7	3.5	-38.1