LRFD Bridge Design

AASHTO LRFD Bridge Design Specifications

Loading and General Information

Created July 2007
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July 31, 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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References

- “Bridge Engineering Handbook,” Wai-Faf Chen and Lian Duan, 1999, CRC Press (0-8493-7434-0)
- “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/
References

- AASHTO Web Site: [http://bridges.transportation.org/](http://bridges.transportation.org/)

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References

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


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:
  \[ f \leq F_A = \frac{F_y}{F_S}. \]
  - \( f \) - computed stress
  - \( F_A \) - Allowable Stress

- In terms of bending moment...
  \[ \sum \frac{M}{S} \leq \frac{F_y}{1.82} \]

**ASD does not recognize different variabilities of different load types.**

**LFD: Load Factor Design**

- For Safety:
  \[ \sum \gamma Q \leq R_n \]
  - \( Q \) - Load Effect
  - \( R \) - Component Resistance
  - \( \gamma \) - Load Factor

- In terms of bending moment...
  \[ 1.30M_D + 2.17M_{(L+1)} \leq \phi M_n \]
  - \( \phi \) - Strength Reduction Factor

*In LFD, load and resistance are not considered simultaneously.*
Philosophies of Design

LRFD: Load & Resistance Factor Design

- For Safety:
  \[ \sum \gamma Q \leq \phi R \]

- \( Q \) - Load Effect
- \( R \) - Component Resistance
- \( \gamma \) - Load Factor
- \( \phi \) - Resistance Factor

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

Chen & Duan

Variability of Loads and Resistances:

- Suppose that we measure the weight of 100 students...

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

Average = 180lbs  St Deviation = 38lbs
Philosophies of Design - LRFD Fundamentals

Variability of Loads and Resistances:

Now suppose that we measure the strength of 100 ropes...

<table>
<thead>
<tr>
<th>Weight</th>
<th>Number of Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>210</td>
<td>0</td>
</tr>
<tr>
<td>220</td>
<td>0</td>
</tr>
<tr>
<td>230</td>
<td>0</td>
</tr>
<tr>
<td>240</td>
<td>0</td>
</tr>
<tr>
<td>250</td>
<td>1</td>
</tr>
<tr>
<td>260</td>
<td>1</td>
</tr>
<tr>
<td>270</td>
<td>3</td>
</tr>
<tr>
<td>280</td>
<td>5</td>
</tr>
<tr>
<td>290</td>
<td>7</td>
</tr>
<tr>
<td>300</td>
<td>11</td>
</tr>
<tr>
<td>310</td>
<td>13</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Weight</th>
<th>Number of Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>320</td>
<td>15</td>
</tr>
<tr>
<td>330</td>
<td>14</td>
</tr>
<tr>
<td>340</td>
<td>11</td>
</tr>
<tr>
<td>350</td>
<td>8</td>
</tr>
<tr>
<td>360</td>
<td>5</td>
</tr>
<tr>
<td>370</td>
<td>3</td>
</tr>
<tr>
<td>380</td>
<td>2</td>
</tr>
<tr>
<td>390</td>
<td>0</td>
</tr>
<tr>
<td>400</td>
<td>1</td>
</tr>
<tr>
<td>410</td>
<td>0</td>
</tr>
<tr>
<td>420</td>
<td>0</td>
</tr>
</tbody>
</table>

Average = 320 lbs
St Deviation = 28 lbs
Philosophies of Design - LRFD Fundamentals

Variability of Loads and Resistances:

![Diagram showing variability of loads and resistances with a histogram and distribution curves.]
Variability of Loads and Resistances:

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

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

Reliability Index:

<table>
<thead>
<tr>
<th>( \beta )</th>
<th>( P(\text{Failure}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>15.9%</td>
</tr>
<tr>
<td>2.0</td>
<td>2.28%</td>
</tr>
<tr>
<td>3.0</td>
<td>0.135%</td>
</tr>
<tr>
<td>3.5</td>
<td>0.0233%</td>
</tr>
</tbody>
</table>
Philosophies of Design - LRFD Fundamentals

Reliability Index:

- **AISC:**

  \[
  \beta \\
  \begin{array}{ccc}
  \beta & D+(L \text{ or } S) & D+L+W & D+L+E \\
  \hline
  \text{Members} & 3.0 & 2.5 & 1.75 \\
  \text{Connections} & 4.5 & 4.5 & 4.5 \\
  \end{array}
  \]

- **AASHTO:**

  \[\beta = 3.5 \text{ Super/Sub Structures} \]
  \[\beta = 2.5 \text{ Foundations}\]

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Philosophies of Design - LRFD Fundamentals

Reliability Index:

- **ASD / LFD Bridge Designs**
- **LRFD Bridge Designs (Expected)**
Philosophies of Design - LRFD Fundamentals

- Resistance Factor:

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

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

AASHTO-LRFD Specification
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<th></th>
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</tbody>
</table>
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.
### §1.3.2: Limit States

\[ Q = \sum \eta_i \gamma_i Q_i \]  

- \( \gamma_i \): Load Factor  
- \( Q_i \): Load Effect  
- \( \eta_i \): Load Modifier

When the maximum value of \( \gamma_i \) is appropriate

\[ \eta_i = \eta_D \eta_R \eta_I \geq 0.95 \]  

When the minimum value of \( \gamma_i \) is appropriate

\[ \eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.00 \]

### §1.3.2: Limit States - Load Modifiers

**Applicable only to the Strength Limit State**

- \( \eta_D \): Ductility Factor:  
  - \( \eta_D = 1.05 \) for nonductile members  
  - \( \eta_D = 1.00 \) for conventional designs and details complying with specifications  
  - \( \eta_D = 0.95 \) for components for which additional ductility measures have been taken

- \( \eta_R \): Redundancy Factor:  
  - \( \eta_R = 1.05 \) for nonredundant members  
  - \( \eta_R = 1.00 \) for conventional levels of redundancy  
  - \( \eta_R = 0.95 \) for exceptional levels of redundancy

- \( \eta_I \): Operational Importance:  
  - \( \eta_I = 1.05 \) for important bridges  
  - \( \eta_I = 1.00 \) for typical bridges  
  - \( \eta_I = 0.95 \) for relatively less important bridges

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

- 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.
§ 3.4 - Load Factors and Combinations

§1.3.2: ODOT Recommended Load Modifiers

For the Strength Limit States

- \( \eta_I \) = Operational Importance:
  - In General, use \( \eta_I = 1.00 \) unless one of the following applies

  - Use \( \eta_I = 1.05 \) if any of the following apply
    - Design ADT \( \geq 60,000 \)
    - Detour length \( \geq 50 \) miles
    - Any span length \( \geq 500' \)

  - Use \( \eta_I = 0.95 \) if both of the following apply
    - Design ADT \( \leq 400 \)
    - Detour length \( \leq 10 \) miles

Detour length applies to the shortest, emergency detour route.
AASHTO-LRFD
Chapter 2: General Design and Location Features

Contents

2.1 – Scope

2.2 – Definitions

2.3 – Location Features
   - 2.3.1 – Route Location
   - 2.3.2 – Bridge Site Arrangement
   - 2.3.3 – Clearances
   - 2.3.4 – Environment

2.4 – Foundation Investigation
   - 2.4.1 – General
   - 2.4.2 – Topographic Studies
§ 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.”

---

§ 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:

<table>
<thead>
<tr>
<th>Load</th>
<th>Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>General vehicular load</td>
<td>Span/800</td>
</tr>
<tr>
<td>Vehicular and/or pedestrian load</td>
<td>Span/1000</td>
</tr>
<tr>
<td>Vehicular load on cantilever arms</td>
<td>Span/300</td>
</tr>
<tr>
<td>Vehicular and/or pedestrian load on cantilever arms</td>
<td>Span/375</td>
</tr>
</tbody>
</table>

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.
§ 2.5.2 - Serviceability

§2.5.2.6.2 Criteria for Deflection

For wood construction:

<table>
<thead>
<tr>
<th>Load</th>
<th>Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vehicular and pedestrian loads</td>
<td>Span/425</td>
</tr>
<tr>
<td>Vehicular loads on wood planks and panels: extreme relative deflection between adjacent edges</td>
<td>0.10 in</td>
</tr>
</tbody>
</table>

For orthotropic plate decks:

<table>
<thead>
<tr>
<th>Load</th>
<th>Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vehicular loads on deck plates</td>
<td>Span/300</td>
</tr>
<tr>
<td>Vehicular loads on ribs of orthotropic metal decks</td>
<td>Span/1000</td>
</tr>
<tr>
<td>Vehicular loads on ribs of orthotropic metal decks: extreme relative deflection between adjacent ribs</td>
<td>0.10 in</td>
</tr>
</tbody>
</table>
§ 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

<table>
<thead>
<tr>
<th>Material</th>
<th>Superstructure</th>
<th>Minimum Depth (Including Deck)</th>
<th>Simple Spans</th>
<th>Continuous Spans</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td>Slabs with main reinforcement parallel to traffic</td>
<td>0.035L ≥ 6.5 in.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>T-Beams</td>
<td>0.04L</td>
<td>0.04L</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Box Beams</td>
<td>0.033L</td>
<td>0.033L</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pedestrian Structure Beams</td>
<td>0.035L</td>
<td>0.035L</td>
<td></td>
</tr>
<tr>
<td>Prestressed Concrete</td>
<td>Slabs</td>
<td>0.027L ≥ 6.5 in.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>CIP Box Beams</td>
<td>0.04L</td>
<td>0.04L</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Precast I-Beams</td>
<td>0.04L</td>
<td>0.04L</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pedestrian Structure Beams</td>
<td>0.033L</td>
<td>0.033L</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Adjacent Box Beams</td>
<td>0.030L</td>
<td>0.030L</td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>Overall Depth of Composite I-Beam</td>
<td>0.04L</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Depth of I-Beam Portion of Composite I-Beam</td>
<td>0.033L</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Trusses</td>
<td>0.10L</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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

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ODOT Short Course Created July 2007

Loads & Analysis: Slide #37
§ 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

---

#### Table 3.4.1-1 Load Combinations and Load Factors

| Load Combination       | DC | DD | DW | EH | EV | LL | IM | CR | BR | PL | LS | WA | WS | WL | FR | TG | SE | EQ | IC | CT | CV |
|------------------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| **STRENGTH I** (unless noted) | $\gamma_p$ | 1.75 | 1.00 | -- | -- | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |
| **STRENGTH II**       | $\gamma_p$ | 1.35 | 1.00 | -- | -- | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |
| **STRENGTH III**      | $\gamma_p$ | 1.00 | 1.40 | -- | -- | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |
| **STRENGTH IV**       | $\gamma_p$ | 1.00 | -- | -- | 1.00 | 0.50/1.20 | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |
| **STRENGTH V**        | $\gamma_p$ | 1.35 | 1.00 | 0.40 | 1.0 | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |
§ 3.4 - Loads and Load Factors

§ 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 LL IM CE PL ES EL WS WL FR TU CR SH TG SE EQ IC CT CV |
|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|
| EXTREME EVENT I  | γ₀ 1.00          | --              | --              | 1.00             | --              | --              | 1.00             | --              | --              | 1.00             | --              | --              |
| EXTREME EVENT II | γ₀ 0.50          | 1.00            | --              | 1.00             | --              | --              | 1.00             | 1.00            | 1.00            |
| FATIGUE – LL, IM, & CE ONLY | γ₀ 0.75 | --              | --              | --              | --              | --              | --              | --              | --              | --              | --              |
§ 3.4 - Loads and Load Factors

§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 - Loads and Load Factors

§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.

### Table 3.4.1-2 Load Factors for Permanent Loads, $\gamma_p$

<table>
<thead>
<tr>
<th>Type of Load, Foundation Type, and Method Used to Calculate Downdrag</th>
<th>Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum</td>
</tr>
<tr>
<td>$DC$: Component and Attachments</td>
<td>1.25</td>
</tr>
<tr>
<td>$DC$: Strength IV only</td>
<td>1.50</td>
</tr>
<tr>
<td>$DD$: Downdrag</td>
<td>1.4</td>
</tr>
<tr>
<td>$DD$: Piles, $\lambda$ Tomlinson Method</td>
<td>1.05</td>
</tr>
<tr>
<td>$DD$: Drilled Shafts, O'Neill and Reese (1999) Method</td>
<td>1.25</td>
</tr>
<tr>
<td>$DH$: Wearing Surfaces and Utilities</td>
<td>1.50</td>
</tr>
<tr>
<td>$EH$: Horizontal Earth Pressure</td>
<td>1.50</td>
</tr>
<tr>
<td>$EH$: At-Res</td>
<td>1.35</td>
</tr>
<tr>
<td>$EL$: Locked in Erections Stresses</td>
<td>1.00</td>
</tr>
</tbody>
</table>
§ 3.4 - Loads and Load Factors

§ 3.4.1: Load Factors and Load Combinations

- An important note about $\gamma_p$: The purpose of $\gamma_p$ 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 $\gamma_p$ is used.
  - If the load being considered would decrease the maximum response, the minimum $\gamma_p$ is used.

- The minimum value of $\gamma_p$ is used when the permanent load would increase stability or load carrying capacity

---

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 $\gamma_p$ for the two end spans. In this case, $\gamma_p$ would be 1.25 for DC for all three spans (Commentary C3.4.1 – paragraph 20).

Incorrect

Correct

---
§ 3.4 - Loads and Load Factors

§ 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), $\gamma_{TG}$, 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.

---

$\gamma_{SE}$ should be based on project specific information. In lieu of project specific information, $\gamma_{SE}$ 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, $\gamma_{EQ}$, 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. ($\gamma_{EQ} = 0.0$)**
§ 3.4 - Loads and Load Factors

§ 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 - Loads and Load Factors

§ 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 - Loads and Load Factors

§ 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.

---

§ 3.4 - Loads and Load Factors

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

Note: Fatigue rarely controls for prestressed concrete

§ 3.4 - Loads and Load Factors

Common Load Combinations for Reinforced Concrete

- Strength I: $1.25 DC + 1.50 DW + 1.75(LL+IM)$
- Strength IV: $1.50 DC + 1.50 DW$
- Fatigue: $0.75(LL+IM)$
§ 3.5 – Permanent Loads

§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.

§ 3.6 - Live Loads

§3.6.1.1: 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 $\frac{1}{2}$ 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.
§ 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

[Diagram of a truck with loads and axle spacings]
§ 3.6 - Live Loads

§3.6.1.2.3: Design Tandem

§ 3.6 - Live Loads

§3.6.1.2.4: Design Lane Load

- 0.640kip/lb is applied SIMULTANEOUSLY 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 and Lane Load, or
- Tandem and Lane Load, or
- 90% of 2 Trucks and 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.
§ 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 one truck is used.

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.

---

§ 3.6.1.3.3: 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.

§ 3.6 - Live Loads

§3.6.1.3.4: Deck Overhang Load

For design of a deck overhang with a cantilever ≤ 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.
§ 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:

\[ IM = 33(1.0 - 0.125D_E) \geq 0\% \] (4.6.2.1.1-1)

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

§ 3.6 - Live Loads

§3.6.1.1.2: Multiple Presence of Live Load

- **Multiple Presence Factor**
  
<table>
<thead>
<tr>
<th># of Loaded Lanes</th>
<th>MP Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.20</td>
</tr>
<tr>
<td>2</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>0.85</td>
</tr>
<tr>
<td>&gt;3</td>
<td>0.65</td>
</tr>
</tbody>
</table>

- **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*
§ 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.

---

§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

- Each fatigue detail shall satisfy,

\[
\gamma (\Delta f) \leq (\Delta f)_{n}
\]

(6.6.1.2.2-1)

where,
- \(\gamma\) - load factor specified in Table 3.4.1-1 for fatigue \((\gamma_{\text{fatigue}} = 0.75)\)
- \((\Delta f)\) - live load stress range due to the passage of the fatigue load specified in §3.6.1.4

\(\eta\) and \(\phi\) 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.*
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

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

\[
N = \left(\frac{365}{75}\right) n (\text{ADTT})_{\text{SL}} \quad (\text{75 Year Design Life})
\]

\[
\Delta F = \left(\frac{A}{N}\right)^{1/2} \geq \frac{\Delta F_{\text{TH}}}{2}
\]

- \(A\) - Fatigue Detail Category Constant - Table 6.6.1.2.5-1
- \(N\) - # of stress ranges per truck passage - Table 6.6.1.2.5-2
- \((\text{ADTT})_{\text{SL}}\) - Single-Lane ADTT from §3.6.1.4
- \((\Delta F)_{\text{TH}}\) - Constant amplitude fatigue threshold - Table 6.6.1.2.5-3

**ODOT is planning to simply design for infinite life on Interstate Structures**
§6.6 - Fatigue and Fracture Considerations

§6.6.1.2: Load Induced Fatigue

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

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 ($\eta$) 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

<table>
<thead>
<tr>
<th>Span Length</th>
<th>&gt; 40 ft.</th>
<th>≤ 40 ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple Span Girders</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Continuous Girders</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Near Interior Supports</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td>- Elsewhere</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Cantilever Girders</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>Trusses</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

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

In the absence of better information,

\[(ADTT)_{SL} = p \cdot ADTT\]  

(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

<table>
<thead>
<tr>
<th># Lanes Available to Trucks</th>
<th>(p)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.00</td>
</tr>
<tr>
<td>2</td>
<td>0.85</td>
</tr>
<tr>
<td>3 or more</td>
<td>0.80</td>
</tr>
</tbody>
</table>

Must consider the number of lanes available to trucks in each direction!
§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>
<thead>
<tr>
<th>Class of Highway</th>
<th>TF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural Interstate</td>
<td>0.20</td>
</tr>
<tr>
<td>Urban Interstate</td>
<td>0.15</td>
</tr>
<tr>
<td>Other Rural</td>
<td>0.15</td>
</tr>
<tr>
<td>Other Urban</td>
<td>0.10</td>
</tr>
</tbody>
</table>

Table C3.6.1.4.2-1 ADT Truck Fraction

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

---

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.

\[ ADTT = (TF) (ADT) = (0.15) (30,000 \text{ Vehicles}) = 4,500 \text{ Trucks} \]

\[ (ADTT)_{SL} = p \cdot ADTT = (0.80) (4,500 \text{ Trucks}) = 3,600 \text{ Trucks} \]

\[ N = (365) (75) n (ADTT)_{SL} \]

\[ = (365) (75) (1) (3,600 \text{ Trucks}) = 98.55M \text{ Cycles} \]

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

§3.6.1.6: Pedestrian Loads

- Pedestrian load = 0.075kip/ft² 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/ft².

- 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).

---

§ 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:

\[
C = f \frac{v^2}{gR}
\]

(3.6.3-1)

- \(v\) = 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/sec²
- \(R\) = radius of curvature for the traffic lane (ft).
§ 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.

§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.
§ 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.

---

§ 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.
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.

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:

$$V_{DZ} = 2.5V_0 \left( \frac{V_{30}}{V_B} \right) \ln \left( \frac{Z}{Z_0} \right)$$

$(3.8.1.1-1)$

$V_0$ = 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)
§ 3.8 - Wind Loads

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

Table 3.8.1.1 Values of $V_0$ and $Z_0$ Various Upstream Surface Conditions

<table>
<thead>
<tr>
<th>Condition</th>
<th>Open Country</th>
<th>Suburban</th>
<th>City</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_0$ (mph)</td>
<td>8.20</td>
<td>10.90</td>
<td>12.00</td>
</tr>
<tr>
<td>$Z_0$ (ft)</td>
<td>0.23</td>
<td>3.28</td>
<td>8.20</td>
</tr>
</tbody>
</table>

§3.8 - Wind Loads

§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

The wind pressure on the structure can be found from:

$$P_D = P_B \left( \frac{V_{DZ}}{V_B} \right)^2 = P_B \left( \frac{V_{DZ}}{10,000} \right)^2 \left( \frac{\text{kip}}{\text{ft}} \right)$$

(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

<table>
<thead>
<tr>
<th>Superstructure Component</th>
<th>Windward Load (ksf)</th>
<th>Leeward Load (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trusses, Columns, and Arches</td>
<td>0.050</td>
<td>0.025</td>
</tr>
<tr>
<td>Beams</td>
<td>0.050</td>
<td>N/A</td>
</tr>
<tr>
<td>Large Flat Surfaces</td>
<td>0.040</td>
<td>N/A</td>
</tr>
</tbody>
</table>
§ 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:
  - $0.30 \frac{kip}{ft}$ on the plane of the windward chord of trusses or arches.
  - $0.15 \frac{kip}{ft}$ on the plane of the leeward chord of trusses or arches
  - $0.30 \frac{kip}{ft}$ on beam or girder spans.

§ 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.
§ 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 $V_o = 100$ mph

<table>
<thead>
<tr>
<th>Skew Angle (degrees)</th>
<th>Lateral Load (ksf)</th>
<th>Longitudinal Load (ksf)</th>
<th>Lateral Load (ksf)</th>
<th>Longitudinal Load (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.075</td>
<td>0.000</td>
<td>0.050</td>
<td>0.000</td>
</tr>
<tr>
<td>15</td>
<td>0.070</td>
<td>0.012</td>
<td>0.044</td>
<td>0.006</td>
</tr>
<tr>
<td>30</td>
<td>0.065</td>
<td>0.028</td>
<td>0.041</td>
<td>0.012</td>
</tr>
<tr>
<td>45</td>
<td>0.047</td>
<td>0.041</td>
<td>0.033</td>
<td>0.016</td>
</tr>
<tr>
<td>60</td>
<td>0.024</td>
<td>0.050</td>
<td>0.017</td>
<td>0.019</td>
</tr>
</tbody>
</table>

§ 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/ft².
- 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.

<table>
<thead>
<tr>
<th>Skew Angle (degrees)</th>
<th>Normal Component (klf)</th>
<th>Parallel Component (klf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.100</td>
<td>0.000</td>
</tr>
<tr>
<td>15</td>
<td>0.088</td>
<td>0.012</td>
</tr>
<tr>
<td>30</td>
<td>0.082</td>
<td>0.024</td>
</tr>
<tr>
<td>45</td>
<td>0.066</td>
<td>0.032</td>
</tr>
<tr>
<td>60</td>
<td>0.034</td>
<td>0.038</td>
</tr>
</tbody>
</table>
§ 3.8 - Wind Loads

§ 3.8.2: Vertical Wind Pressure

- Wind uplift force of 0.020 kip/ft² 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.

![Wind uplift force diagram](attachment:wind uplift force.png)

§ 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 substructure 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.
§ 3.10 Earthquake Effects: EQ

ODOT Exception

- *The Extreme Event I load factor for live load, $\gamma_{EQ}$ 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° 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.*

§ 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>
<thead>
<tr>
<th>Climate</th>
<th>Steel or Aluminum</th>
<th>Concrete</th>
<th>Wood</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cold</td>
<td>-30° to 120° F</td>
<td>0° to 80° F</td>
<td>0° to 75° F</td>
</tr>
</tbody>
</table>

*ODOT requires the use of Cold Climate, Procedure A.*
§4.4 – Acceptable Methods of Structural Analysis

- Simplified Analysis
  - Distribution Factor

- Refined Analysis
  - Finite Element Modeling
§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.

§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 \( d_e \leq 3 \text{ ft} \)
- Central angle \(< 40^\circ \)
- Cross section conforms to AASHTO Table 4.6.2.2.1-1

*ODOT Exception: The roadway overhang \( d_e \leq 3 \text{ ft} \) does not apply to interior DFs for sections (a) and (k).
This is part of Table 4.6.2.1-1 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.
§4.6.2 - Approximate Methods of Analysis – Distribution Factors

This is a part of Table 4.6.2.2b-1 showing distribution factors for moment. A similar table exists for shear distribution factors.

The table gives 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.

---

Table C4.6.2.2.1-1  \( I \) for Use in Live Load Distribution Factor Equations.

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

---
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.

---

<table>
<thead>
<tr>
<th>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</th>
<th>Distribution Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>n, e, k and also i, j if sufficiently connected to act as a unit</td>
<td>One Design Lane Loaded:</td>
</tr>
<tr>
<td></td>
<td>0.06 + ( \frac{S}{14} ) ( \frac{S}{L} ) ( \frac{K_L}{12L_d^2} ) ( \lambda )</td>
</tr>
<tr>
<td>Two or More Design Lanes Loaded:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.075 + ( \frac{S}{9.5} ) ( \frac{S}{L} ) ( \frac{K_L}{12L_d^2} ) ( \lambda )</td>
</tr>
<tr>
<td>use lesser of the values obtained from the equation above with ( N_t = 3 ) or the lever rule</td>
<td>( N_t = 3 )</td>
</tr>
</tbody>
</table>

\( 3.5 \leq S \leq 16.0 \)
\( 4.5 \leq t_f \leq 12.0 \)
\( 20 \leq L \leq 240 \)
\( N_t \geq 4 \)
\( 10,000 \leq K_L \leq 7,000,000 \)
§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.2 Moment Distribution - Interior Girders

- **Interior Girders:**
  - **One Lane Loaded:**
    \[
    DF_{M,int} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.1} \frac{K_g}{12L^2}\]
    This term may be taken as 1.00 for prelim design
  
  - **Two or More Lanes Loaded:**
    \[
    DF_{M,int} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \frac{K_g}{12L^2}\]

---

§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.) \[ 3.5 \leq S \leq 16.0 \]
  - \( L \) - Span length of beam or girder (ft.) \[ 20 \leq L \leq 240 \]
  - \( K_g \) - Longitudinal stiffness parameter (in\(^4\)) \[ 10k \leq K_g \leq 7M \]
  - \( t_s \) - Thickness of concrete slab (in) \[ 4.5 \leq t_s \leq 12.0 \]
  - \( d_e \) - Distance from exterior beam to interior edge of curb (ft.) (Positive if the beam is “inside” of the curb.) \[ -1.0 \leq d_e \leq 5.5 \]
§ 4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2 Beam-Slab Bridges

- Parameter Definitions & Limits of Applicability:

\[ K_g = n \left( I + A e_g^2 \right) \]  
(4.6.2.2.1-1)

- \( n \) - Modular ratio, \( E_{\text{Beam}} / E_{\text{Deck}} \)  
(See Section 6.10.1.1b, 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.

---

§ 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:
    \[ DF_{\text{ext}} = e \cdot DF_{\text{int}} \]
    \[ e = 0.77 + \frac{d}{9.1} \]
### §4.6.2.2d 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

\[
\sum M_H = 1.2 Pe - RS = 0
\]

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

In the diagram, \( P \) is the axle load.

---

### §4.6.2.2e Moment Distribution - Skewed Bridges

**Correction for Skewed Bridges:**

- The bending moment may be reduced in bridges with a skew of \( 30^\circ \leq \theta \leq 60^\circ \)

\[
DF_M' = \left(1 - C_i (tan \theta)^{0.5}\right)DF_M
\]

\[
C_i = 0.25 \left(\frac{K}{12Lt^3}\right)^{0.25} S^{0.5} L^{0.5}
\]

- When the skew angle is greater than \( 60^\circ \), take \( \theta = 60^\circ \)
§ 4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.3a Shear Distribution - Interior Beams

- **Interior Girders:**
  - One Lane Loaded:
    \[
    DF_{\text{Int}} = 0.36 + \frac{S}{25.0}
    \]
  - Two or More Lanes Loaded:
    \[
    DF_{\text{Int}} = 0.2 + \frac{S}{12} - \left( \frac{S}{35} \right)^2
    \]

§ 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:
    \[
    DF_{\text{Ext}} = e \cdot DF_{\text{Int}}
    \]
    \[
    e = 0.60 + \frac{d_e}{10}
    \]
§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 $0^\circ \leq \theta \leq 60^\circ$
    \[
    DF' = 1.0 + 0.20 \left( \frac{12kL}{K_e} \right)^{0.3} \tan \theta \cdot DF_i
    \]
  - 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.

---

§4.6.2 - Approximate Methods of Analysis

§ 4.6.2.2.2d Exterior Beams

- **Minimum Exterior DF: (Rigid Body Rotation of Bridge Section)**
  \[
  DF_{Ext,abs} = \frac{N_L}{N_b} \sum_{i=1}^{N_b} X_{Ext} \frac{e_i}{x_i^2} \quad (C4.6.2.2d-1)
  \]

- $N_L$ - 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

§ 4.6.2.2d Exterior Beams

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

\[ DF_{\text{ext}, \text{rot}} = \frac{N_L}{N_b} \frac{X_{\text{ext}}}{\sum x^2} \]  

\(N_L\) - 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.)
\(X_{\text{ext}}\) - Distance from CG of pattern of girders to exterior girder (ft.)

“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, \(d_o\), does not exceed 3.0 ft
- Curvature in plan is less than 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 $F_y = 50^{\text{ksi}}$ and all concrete has a 28-day compressive strength of $f'_c = 4.5^{\text{ksi}}$. The concrete slab is 9 1/2” thick. A typical 2 3/4” 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:

“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/
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)

\[
A = (15\text{'}\text{'})(3/4\text{')}) + (69\text{'}\text{'})(9/16\text{')}) + (21\text{'}\text{'})(1\text{}) = 71.06 \text{ in}^2
\]

\[
W_{section1} = 71.06 \text{ in}^2 \left( \frac{490 \text{ pcf}}{\left( \frac{12 \text{ in}}{\text{ft}} \right)^2} \right) \left(1.15\right) = 278.1 \frac{\text{lb}}{\text{ft}} \text{ per girder}
\]

(b) Section 2 (Negative Bending)

\[
A = (21\text{'}\text{'})(1\text{'}\text{')}) + (69\text{'}\text{'})(9/16\text{')}) + (21\text{'}\text{'})(2-1/2\text{')}) = 112.3 \text{ in}^2
\]

\[
W_{section2} = 112.3 \text{ in}^2 \left( \frac{490 \text{ pcf}}{\left( \frac{12 \text{ in}}{\text{ft}} \right)^2} \right) \left(1.15\right) = 439.5 \frac{\text{lb}}{\text{ft}} \text{ per girder}
\]
Deck Self-Weight (DC1):

\[ W_{\text{deck}} = (9.5\text{"})(144\text{")} \left( \frac{150 \text{pcf}}{(12 \text{ in}^2)} \right) = 1,425 \text{lb per girder} \]

Haunch Self-Weight (DC1):

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

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

\[ W_{\text{haunch}} = \left( \frac{(2\text{")}(25.2\text{")}}{(12 \text{ in}^2)} \right) (150 \text{pcf}) = 52.5 \text{lb per girder} \]

Barrier Walls (DC2):

\[ W_{\text{barriers}} = \left( \frac{(2 \text{ each})(640 \text{ plf})}{4 \text{ girders}} \right) = 320.0 \text{ lb per girder} \]

Wearing Surface (DW):

\[ W_{\text{fws}} = \left( \frac{(39\text{'})(60 \text{ psf})}{4 \text{ girders}} \right) = 585 \text{ lb per girder} \]

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’

<table>
<thead>
<tr>
<th></th>
<th>Max (+) Moment</th>
<th>Max (-) Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stations 58.7’ and 271.3’</td>
<td>Station 165.0’</td>
</tr>
<tr>
<td>DC1 - Steel:</td>
<td>475 k-ft</td>
<td>-1,189 k-ft</td>
</tr>
<tr>
<td>DC1 - Deck:</td>
<td>2,415 k-ft</td>
<td>-5,708 k-ft</td>
</tr>
<tr>
<td>DC1 - Haunch:</td>
<td>89 k-ft</td>
<td>-210 k-ft</td>
</tr>
<tr>
<td>DC1 - Total:</td>
<td>2,979 k-ft</td>
<td>-7,107 k-ft</td>
</tr>
<tr>
<td>DC2:</td>
<td>553 k-ft</td>
<td>-1,251 k-ft</td>
</tr>
<tr>
<td>DW</td>
<td>1,011 k-ft</td>
<td>-2,286 k-ft</td>
</tr>
</tbody>
</table>
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 25kip axles spaced 4.0’ apart. The lane loading consists of a 0.64klf 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 0.64klf 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 32kip 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’

<table>
<thead>
<tr>
<th>Max (+) Moment</th>
<th>Max (-) Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stations 73.3’ and 256’</td>
<td>Station 165’</td>
</tr>
<tr>
<td>HL-93M</td>
<td>$3,725^{k-ft}$</td>
</tr>
<tr>
<td>HL-93K</td>
<td>$4,396^{k-ft}$</td>
</tr>
<tr>
<td>HL-93S</td>
<td>N/A</td>
</tr>
<tr>
<td>Fatigue</td>
<td>$2,327^{k-ft}$</td>
</tr>
</tbody>
</table>

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

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

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

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

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

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: 
\[
\left( 25^{kip} \right) \left( 18.51 \frac{kip}{ft} \right) + \left( 25^{kip} \right) \left( 18.45 \frac{kip}{ft} \right) = 924.0^{kip} 
\]

Single Truck: 
\[
\left( 8^{kip} \right) \left( 17.47 \frac{kip}{ft} \right) + \left( 32^{kip} \right) \left( 18.5 \frac{kip}{ft} \right) + \left( 32^{kip} \right) \left( 18.3 \frac{kip}{ft} \right) = 1,318^{kip} 
\]

Two Trucks: 
\[
\left( 8^{kip} \right) \left( 17.47 \frac{kip}{ft} \right) + \left( 32^{kip} \right) \left( 18.5 \frac{kip}{ft} \right) + \left( 32^{kip} \right) \left( 18.3 \frac{kip}{ft} \right) + \ldots 
\]
\[
\ldots + \left( 8^{kip} \right) \left( 16.72 \frac{kip}{ft} \right) + \left( 32^{kip} \right) \left( 18.31 \frac{kip}{ft} \right) + \left( 32^{kip} \right) \left( 18.51 \frac{kip}{ft} \right) = 2,630^{kip} 
\]

Lane Load: 
\[
\left( 0.640 \frac{kip}{ft} \right) \left( 3,918 \frac{kip}{ft} \right) = 2,508^{kip} 
\]

\[
(\text{IM})(\text{Tandem}) + \text{Lane} : \quad (1.33) \left( 924.0^{kip} \right) + 2,508^{kip} = 3,737^{kip} 
\]

\[
(\text{IM})(\text{Single Truck}) + \text{Lane} : \quad (1.33) \left( 1,318^{kip} \right) + 2,508^{kip} = 4,261^{kip} 
\]

\[
(0.90) \{ (\text{IM})(\text{Two Trucks}) + \text{Lane} \} : \quad (0.90) \left[ (1.33) \left( 2,630^{kip} \right) + 2,508^{kip} \right] = 5,405^{kip} \quad \text{GOVERNS} 
\]
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
  
  
  $$BR_{Single\ Lane} = (0.25) \left( 8^{kip} + 32^{kip} + 32^{kip} \right) = 18.00^{kip}$$

- **B)** 25% of the Design Tandem

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

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

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

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

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

Case (A) Governs:

$$BR_{Net} = \left( BR_{Single\ Lane} \right) \left( \#\ Lanes \right) \left( MPF \right)$$

$$= \left( 18.00^{kip} \right) (3)(0.85) = 45.90^{kip}$$

This load has not 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, \( C \).

\[
C = f \cdot \frac{v^2}{gR}
\]

where:
- \( v \) - Highway design speed (\( \frac{ft}{sec} \))
- \( f \) - \( \frac{4}{3} \) for all load combinations except for Fatigue, in which case it is 1.0
- \( g \) - The acceleration of gravity (\( \frac{ft}{sec^2} \))
- \( R \) - The radius of curvature for the traffic lane (ft).

Suppose that we have a radius of \( R = 600’ \) and a design speed of \( v = 65 \text{mph} = 95.33 \frac{ft}{sec} \).

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

\[
CE = (\text{Axle Loads})(C)(\# \text{ Lanes})(MPF)
\]
\[
= (72 \text{kip})(0.6272)(3)(0.85) = 115.2 \text{kip}
\]

This force has not been factored...

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  
$V_o = 8.20$ mph  
$Z_o = 0.23$ ft

**Horizontal Wind Load on Structure:** $(WS)$

**Design Pressure:**

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

- $P_B$ - Base Pressure  
- For beams, $P_B = 50$ psf when $V_B = 100$ 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:**

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

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

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

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

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

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

$$W = (55.92 \text{psf})(10.44') = 583.7 \text{ lbs ft}$$

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

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

**For Center Pier:**  
$WS_{H,Pier} = (583.7 \text{ lbs ft})(2)(\frac{1}{2})(165') = 96.31 \text{ kip}$
**Vertical Wind Load on Structure:** \((WS)\)

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

\[ P_v = (20 \text{psf})(w) = (20 \text{psf})(42') = 840 \frac{\text{lbs}}{\text{ft}} \]

Total Uplift:
\[ (840 \frac{\text{lbs}}{\text{ft}})(2)(165') = 277.2 \text{kip} \]

For End Abutments:
\[ (840 \frac{\text{lbs}}{\text{ft}})(\frac{1}{2})(165') = 69.30 \text{kip} \]

For Center Pier:
\[ (840 \frac{\text{lbs}}{\text{ft}})(2)(\frac{1}{2})(165') = 138.6 \text{kip} \]

**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, \( b_{\text{eff}} \):

For an interior beam, \( b_{\text{eff}} \) is the lesser of:

\[
\begin{align*}
\frac{L_{\text{eff}}}{4} &= \frac{132'}{4} = 33' = 396'' \\
12t_s + \frac{b_f}{2} &= (12)(8.5'') + \frac{15''}{2} = 109.5'' \\
S &= (12')(12''/6') = 144''
\end{align*}
\]

For an exterior beam, \( b_{\text{eff}} \) is the lesser of:

\[
\begin{align*}
\frac{L_{\text{eff}}}{4} &= \frac{132'}{4} = 33' = 198.0'' \\
12t_s + \frac{b_f}{2} &= (12)(8.5'') + \frac{15''}{2} = 109.5'' \\
\frac{S}{2} + d_c &= \left(\frac{12'}{2} + 3'\right)(12''/\pi) = 108.0''
\end{align*}
\]

Note that \( L_{\text{eff}} \) 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, \( A_s = 13.02 \text{ in}^2 \) 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

<table>
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<tr>
<th></th>
<th>t</th>
<th>b</th>
<th>A</th>
<th>y</th>
<th>Ay</th>
<th>Ix</th>
<th>d</th>
<th>Ad^e</th>
<th>Ix</th>
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<tr>
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<td>1.75</td>
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\[
71.06 \quad 2,180.06 \quad I_{\text{total}} = 53,157
\]

\[
Y = 30.68 \quad S_{\text{BST1,top}} = 1,327 \quad S_{\text{BST1,bot}} = 1,733
\]

#### Short-Term Composite (n = 8)

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<tr>
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<th>t</th>
<th>b</th>
<th>A</th>
<th>y</th>
<th>Ay</th>
<th>Ix</th>
<th>d</th>
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<tbody>
<tr>
<td>Slab</td>
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<td>791.72</td>
<td>0.53</td>
<td>-12.18</td>
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<td>1.75</td>
<td>57.69</td>
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<td>69,901</td>
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</table>

\[
187.41 \quad 10,905.84 \quad I_{\text{total}} = 140,521 \quad n: 8.00
\]

\[
Y = 58.19 \quad S_{\text{STT1,top}} = 11,191 \quad S_{\text{STT1,bot}} = 2,415
\]

#### Long-Term Composite (n = 24)

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\[
109.84 \quad 5,088.66 \quad I_{\text{total}} = 102,676 \quad n: 24.00
\]

\[
Y = 46.33 \quad S_{\text{LTT1,top}} = 4,204 \quad S_{\text{LTT1,bot}} = 2,216
\]

#### Cracked Section

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<td>Rebar</td>
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\[
84.08 \quad 3,159.82 \quad I_{\text{total}} = 193,764
\]

\[
Y = 37.58 \quad S_{\text{CRT1,top}} = 5,842 \quad S_{\text{CRT1,bot}} = 5,156
\]

These section properties do NOT include the haunch or sacrificial wearing surface.
3.3: *Section 2 Flexural Properties*

### Bare Steel

<table>
<thead>
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<th>t</th>
<th>b</th>
<th>A</th>
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\[ I_{\text{total}} = 96,642 \]

\[ Y = 26.83 \]

\[ S_{\text{BS2, top}} = 2,116 \]

\[ S_{\text{BS2, bot}} = 3,602 \]

### Short Term Composite \((n = 8)\)

<table>
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<tr>
<th></th>
<th>t</th>
<th>b</th>
<th>A</th>
<th>y</th>
<th>Ay</th>
<th>Iₓ</th>
<th>d</th>
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<th>Iₓ</th>
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<tbody>
<tr>
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<td>76.75</td>
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\[ I_{\text{total}} = 239,734 \]

\[ n: 8.00 \]

\[ Y = 52.23 \]

\[ S_{\text{ST2, top}} = 11,828 \]

\[ S_{\text{ST2, bot}} = 4,590 \]

### Long-Term Composite \((n = 24)\)

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<th>t</th>
<th>b</th>
<th>A</th>
<th>y</th>
<th>Ay</th>
<th>Iₓ</th>
<th>d</th>
<th>Adf</th>
<th>Iₓ</th>
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\[ I_{\text{total}} = 168,704 \]

\[ n: 24.00 \]

\[ Y = 39.65 \]

\[ S_{\text{LT2, top}} = 5,135 \]

\[ S_{\text{LT2, bot}} = 4,255 \]

### Cracked Section

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\[ I_{\text{total}} = 126,006 \]

\[ Y = 32.04 \]

\[ S_{\text{CR2, top}} = 3,115 \]

\[ S_{\text{CR2, bot}} = 3,932 \]

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:

\[ DF_{\text{M1,Int+}} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12L^2} \right)^{0.1} \]

\[ K_g = n(J + Ae^2) \]

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

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

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

\[ DF_{\text{M1,Int+}} = 0.5021 \]

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:

\[ DF_{\text{M2,Int+}} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12L^2} \right)^{0.1} \]

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

\[ DF_{\text{M2,Int+}} = 0.7781 \]

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.

\[ DF_{\text{M1,Ext+}} = \frac{8.5}{12} = 0.7083 \]

Multiple Presence: \( DF_{\text{M1,Ext+}} = (1.2)(0.7083) = 0.8500 \)
Two or More Lanes Loaded:

\[ DF_{M2,Ext+} = e \cdot DF_{M2,Int+} \]

\[ e = 0.77 + \frac{d_e}{9.1} \]

\[ = 0.77 + \frac{1.5}{9.1} = 0.9348 \]

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

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:

\[ DF_{M1,Int-} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12Ls^2} \right)^{0.1} \]

\[ K_g = n(J + Ae_{eg}^2) \]

\[ K_g = 8(96,642 \text{ in}^4 + (112.3 \text{ in}^2)(52.17''^2) \]

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

\[ DF_{M1,Int-} = 0.06 + \left( \frac{12' \cdot 0.165}{14} \right)^{0.4} \left( \frac{12' \cdot 0.165}{165'} \right)^{0.3} \left( \frac{3,218,000 \text{ in}^4}{(12')(165')(8.5''^3)} \right)^{0.1} \]

\[ DF_{M1,Int-} = 0.5321 \]

Two or More Lanes Loaded:

\[ DF_{M2,Int-} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12Ls^2} \right)^{0.1} \]

\[ DF_{M2,Int-} = 0.075 + \left( \frac{12' \cdot 0.165}{9.5} \right)^{0.6} \left( \frac{12' \cdot 0.165}{165'} \right)^{0.2} \left( \frac{3,218,000 \text{ in}^4}{(12')(165')(8.5''^3)} \right)^{0.1} \]

\[ DF_{M2,Int-} = 0.8257 \]
Exterior Girder –

One Lane Loaded:

Same as for the positive moment section: \( DF_{M1,Ext} = 0.8500 \)

Two or More Lanes Loaded:

\[
DF_{M2,Ext} = e \cdot DF_{M2,Int}
\]

\[e = 0.77 + \frac{d_o}{9.1}\]

\[= 0.77 + \frac{1.5}{9.1} = 0.9348\]

\[
DF_{M2,Ext} = (0.9348)(0.8257) = 0.7719
\]

4.3: Minimum Exterior Girder Distribution Factor:

\[
DF_{M1,Ext,Min} = \frac{N_L}{N_b} + \frac{X_{Ext} \sum N_e}{\sum x^2}
\]

One Lane Loaded:

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

Multiple Presence:

\[
DF_{M1,Ext,Min} = (1.2)(0.6125) = 0.7350
\]
Two Lanes Loaded:

\[
DF_{M,3,Ext,Min} = \frac{2}{4} \frac{(18.0')(14.5'+2.5')}{(2)[(18')^2+(6')^2]} = 0.9250
\]

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:

<table>
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<tr>
<th></th>
<th>Positive Moment</th>
<th>Negative Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interior</td>
<td>Exterior</td>
</tr>
<tr>
<td>1 Lane Loaded:</td>
<td>0.5021</td>
<td>0.8500 ≥ 0.7350</td>
</tr>
<tr>
<td>2 Lanes Loaded:</td>
<td>0.7781</td>
<td>0.7274 ≥ 0.9250</td>
</tr>
</tbody>
</table>

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.

<table>
<thead>
<tr>
<th></th>
<th>Positive Moment</th>
<th>Negative Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interior</td>
<td>Exterior</td>
</tr>
<tr>
<td>1 Lane Loaded:</td>
<td>0.4184</td>
<td>0.7083 ≥ 0.6125</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Station (ft)</th>
<th>(LL+IM)+ (k-ft)</th>
<th>(LL+IM)- (k-ft)</th>
<th>Fat+ (k-ft)</th>
<th>Fat- (k-ft)</th>
<th>DC1 (k-ft)</th>
<th>DC2 (k-ft)</th>
<th>DW (k-ft)</th>
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<td>0.0</td>
<td>0.0</td>
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<td>755.6</td>
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<td>-206.8</td>
<td>2799.9</td>
<td>515.0</td>
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<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
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:**

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

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

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

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

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:

<table>
<thead>
<tr>
<th>Shear Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
</tr>
<tr>
<td>1 Lane Loaded:</td>
</tr>
<tr>
<td>2 Lanes Loaded:</td>
</tr>
</tbody>
</table>

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.

<table>
<thead>
<tr>
<th>Shear Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
</tr>
<tr>
<td>1 Lane Loaded:</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Station (ft)</th>
<th>(LL+IM)+ (kip)</th>
<th>(LL+IM)- (kip)</th>
<th>Fat+ (kip)</th>
<th>Fat- (kip)</th>
<th>DC1 (kip)</th>
<th>DC2 (kip)</th>
<th>DW (kip)</th>
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<td>-20.6</td>
<td>-37.6</td>
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</table>
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: \[0.75(LL + IM)\]

\[(IM = 15\% \text{ for Fatigue}; \ IM = 33\% \text{ otherwise})\]

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, \(\eta\), was applied as is shown below.

\[Q = \sum \eta_i \gamma_i Q_i\]

\(\eta\) is taken as the product of \(\eta_D\), \(\eta_R\), and \(\eta_I\), and is taken as not less than 0.95. For this example, \(\eta_D\) and \(\eta_I\) are taken as 1.00 while \(\eta_R\) 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 \(\eta\) 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 Girder Moments for Design

| Station (ft) | Strength I Total + (k-ft) | Total - (k-ft) | Strength IV Total + (k-ft) | Total - (k-ft) | Service II Total + (k-ft) | Total - (k-ft) | Fatigue Strength Total + (k-ft) | Total - (k-ft) |
|-------------|-----------------|----------------|-----------------|----------------|-----------------|----------------|----------------|----------------|----------------|
| 0.0         | 0.0             | 0.0            | 0.0             | 0.0            | 0.0             | 0.0            | 0.0             | 0.0            |
| 14.7        | 5677.1          | -515.7         | 3134.6          | 0.0            | 4280.7          | -383.1         | 0.2             | 0.0            |
| 29.3        | 9806.0          | -1031.5        | 5374.1          | 0.0            | 7393.0          | -766.2         | 0.0             | 0.0            |
| 44.0        | 12403.3         | -1547.2        | 6708.8          | 0.0            | 9349.1          | -1149.4        | 0.0             | 0.0            |
| 58.7        | 13567.8         | -2062.9        | 7145.1          | 0.0            | 10222.6         | -1532.5        | 0.0             | 0.0            |
| 73.3        | 13287.4         | -2578.7        | 6679.8          | 0.0            | 10004.2         | -1915.6        | 0.0             | 0.0            |
| 88.0        | 11687.1         | -3094.4        | 5312.9          | 0.0            | 8787.0          | -2298.7        | 0.0             | 0.0            |
| 102.7       | 8740.0          | -3610.2        | 3047.7          | 0.0            | 6551.1          | -2681.8        | 0.0             | 0.0            |
| 117.3       | 4621.6          | -4237.1        | 11.2            | -133.5         | 3432.8          | -3153.9        | 0.0             | 0.0            |
| 132.0       | 2772.1          | -8317.8        | 0.0             | -4187.3        | 2059.3          | -6268.9        | 0.0             | 0.0            |
| 135.7       | 2342.0          | -9533.2        | 0.0             | -5347.3        | 1739.8          | -7195.8        | 0.0             | 0.0            |
| 139.3       | 1926.4          | -10838.2       | 0.0             | -6566.4        | 1431.1          | -8190.4        | 0.0             | 0.0            |
| 143.0       | 1522.6          | -12230.6       | 0.0             | -7845.7        | 1131.1          | -9251.2        | 0.0             | 0.0            |
| 146.7       | 1131.6          | -13707.1       | 0.0             | -9184.5        | 840.6           | -10375.8       | 0.0             | 0.0            |
| 150.3       | 851.2           | -15392.8       | 0.0             | -10582.9       | 632.3           | -11657.1       | 0.0             | 0.0            |
| 154.0       | 588.9           | -17317.3       | 0.0             | -12041.3       | 437.4           | -13117.1       | 0.0             | 0.0            |
| 157.7       | 340.9           | -19328.3       | 0.0             | -13559.1       | 253.3           | -14642.7       | 0.0             | 0.0            |
| 161.3       | 140.4           | -21420.1       | 0.0             | -15137.1       | 104.3           | -16229.6       | 0.0             | 0.0            |
| 165.0       | 0.0             | -23617.1       | 0.0             | -16774.1       | 0.0             | -17895.9       | 0.0             | 581.7          |
| 168.7       | 140.4           | -21420.1       | 0.0             | -15137.1       | 104.3           | -16229.6       | 3.1             | 578.8          |
| 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        | 319.8             | 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          | -8317.8        | 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             | 151.5          |
| 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

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<th>Strength I Total + (kip)</th>
<th>Strength I Total - (kip)</th>
<th>Strength IV Total + (kip)</th>
<th>Strength IV Total - (kip)</th>
<th>Service II Total + (kip)</th>
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