New Hampshire DOT Department of Transportation

# Bridge Design Manual

## Chapter 4

## Loads and Load Factors

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*(Revised March 2016)*

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## 4.1 Design Criteria

1) All ***new*** bridges in the State of New Hampshire shall be designed to the following current specifications:
    - American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications and Interim Revisions
    - ASTM International
    - American Welding Society (AWS)
    - NHDOT Standard Specifications for Road and Bridge Construction

2) All ***rehabilitation*** bridges in the State of New Hampshire shall be designed to the following current specifications:
    - American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications and Interim Revisions. See Chapter 1, Section 1.1.3 Design Methods.
    - American Association of State Highway and Transportation Officials (AASHTO) Manual for Bridge Evaluation (MBE)
    - ASTM International
    - American Welding Society (AWS)
    - NHDOT Standard Specifications for Road and Bridge Construction

3) All ***railroad*** bridges shall be designed to the current specifications of the American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering and the specifications of the railroad involved.

Note, the material in this Bridge Design Manual is supplemental to the specifications listed and takes precedence over them.

If a computer program is used in the design/analysis of the bridge, the name, version, and release date of the software shall be noted on the design calculations per *AASHTO LRFD Bridge Design Specifications, Section 4.4, Acceptable Methods of Structural Analysis*.

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# 4.2 General

## *4.2.1 Limit States*

Designers must ensure the structure has been checked for adequacy in carrying all appropriate load combinations at any possible construction stage. In general the following is a summary of the design limit states:

1) Strength: Strength and Stability
   - I Basic Load Combination
   - II Special or Permit
   - III Wind greater than 55-mph (88.5-kph)
   - IV High DL/LL ratio
   - V LL + (Wind = 55-mph [88.5-kph])

2) Extreme Event: Long Return Period Events
   - I Earthquake
   - II Ice, Vessel and Vehicular Collision

3) Service: Stresses, Deformations, Cracking or Crack Control
   - I Normal Service Load Combination
   - II Steel Structures
   - III Tension in Prestressed Concrete

4) Fatigue:
   - I Fatigue and Fracture Load Combination (Infinite Load-induced Fatigue Life)
   - II Fatigue and Fracture Load Combination (Finite Load-induced Fatigue Life)

The basic limit state equation is as follows (*AASHTO LRFD 1.3.2.1):*

$$Q = \Sigma\eta_i \gamma_i Q_i \leq \phi R_n = R_r$$

Where:

$\eta_i$ = Load modifier (a function of $\eta_D$, $\eta_R$, and $\eta_I$)
$\gamma_i$ = Load factor
$Q_i$ = Force effect: moment, shear, stress range or deformation caused by applied loads
$Q$ = Total factored force effect
$\phi$ = Resistance factor
$R_n$ = Nominal resistance: resistance of a component to force effects
$R_r$ = Factored resistance = $\phi R_n$

This equation states that the force effects are multiplied by factors to account for uncertainty in loading, structural ductility, operational importance, and redundancy. These effects must be less than or equal to the available resistance multiplied by factors to account for variability and uncertainty in the materials and construction.

### *4.2.2 Load Modifiers*

For most structures, each of the load modifiers will be 1.00. For a limited number of bridges, load modifiers with values different from 1.00 will need to be used, as determined by the Design Chief. Table 4.2.2-1 summarizes NHDOT's policy for load modifiers. *(AASHTO LRFD 1.3)*

| NHDOT Load Modifiers | | |
|---|---|---|
| **Modifier** | **Value** | **Condition** |
| Ductility ($\eta_D$) | 1.00 | Steel structures, timber bridges, ductile concrete structures |
| | 1.05 | Non-ductile concrete structures |
| Redundancy ($\eta_R$) | 1.00 | Redundant |
| | 1.05 | Non-redundant |
| Importance ($\eta_I$) | 1.00 | ADT ≤ 40,000 |
| | 1.05 | Major river crossing *or* ADT > 40,000 *or* Interstate Bridge |

**NHDOT Load Modifiers**

*Table 4.2.2-1*

### *4.2.3 Load Factors*

The load factor, $\gamma_i$, is used to adjust force effects on a structural element. This factor accounts for variability of loads, lack of accuracy in analysis and the probability of simultaneous occurrence of different loads. For the design limit states, the values of $\gamma_i$ for different types of loads are found in *AASHTO LRFD Tables 3.4.1-1, 3.4.1-2 and 3.4.1-3*.

Several of the loads have variable load factors (e.g., $\gamma_P$, $\gamma_{EQ,}$ $\gamma_{TG}$, $\gamma_{SE}$ ). The load factors for permanent loads ($\gamma_P$) typically have two values, a maximum value and a minimum value. When analyzing a structure it will often be necessary to use both values. The objective is to envelop the maximum load effects on various elements for design. See Section 4.3 for the variable load factors to be applied to the corresponding loading.

When assembling load combinations do not use more than one load factor concurrently for any load component. For example, when checking uplift, a load factor of 0.90 or 1.25 should be used for the dead load on all spans. Designers should not use 0.9 on the span adjacent to the uplift point and 1.25 on the next span. Also, when breaking a load into vertical and horizontal components (e.g. earth pressure), the designer should not use different load factors for the components. Either the maximum or minimum load factors shall be used for both components.

Designers must ensure the structure has been checked for adequacy in carrying all appropriate load combinations at any possible construction stage. For example, a tall abutment should be checked for any permissible construction case in addition to the final condition. If the construction sequence requires the abutment to be completely constructed prior to placement of the beams (a case which maximizes the horizontal earth pressure load with a minimum of vertical load) or the abutment is constructed such that the superstructure is completed prior to backfilling. This latter case would maximize vertical load without horizontal earth pressure load. This would be an exception to NHDOT policy which requires the abutment backfilled prior to placing the superstructure.

Substructure design routinely uses the maximum and minimum permanent-load load factors. An illustrative yet simple example is a spread footing supporting a cantilever retaining wall.

- When checking bearing, the weight of the soil (EV) over the heel is factored by the maximum load factor, 1.35 for retaining walls and abutments, because greater EV increases the bearing pressure, $q_{ult}$, making the limit state more critical.
- When checking sliding, EV is factored by the minimum load factor, 1.0 for retaining walls and abutments, because lesser EV decreases the resistance to sliding, $Q_\tau$, again making the limit state more critical.
- When checking sliding and overturning with the backfill compacted and the superstructure not in place, use the minimum load factor for EV and DC (substructure) because this decreases the resistance to sliding and overturning making the limit state more critical.

When checking deck pouring sequence or phased construction, load factors shall be applied to achieve maximum effect. Bearings shall be checked for uplift using the Strength I load combination with the minimum load factor for dead load and maximum load factor for live load as noted in *AASHTO LRFD C3.4.1*.

LRFD factored design loads may produce a small calculated uplift under worst-case factors. However, a line-girder analysis does not account for the net downward force of the adjacent beams with the cross-frame connections, and the stiff end diaphragms, and end-of-deck haunch. New Hampshire's existing beams and bearing assemblies have had many years of service with no signs of distress or displacement from uplift. If uplift is a concern, the designer shall perform a second analysis of the uplift using the LFD Design methodology or service loads. The results, along with engineering recommendations, shall be provided to the Design Chief for approval.

A construction load case shall be considered by designers. Use a load factor ≥ 1.5 for construction live loads and a load factor ≥ 1.25 for dead loads (including formwork, falsework and stockpiled materials or machinery) as noted in *AASHTO LRFD 3.4.2*.

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# 4.3 Loads

## 4.3.1 General

In LRFD design, structural materials must be able to resist their applied design loads. There are two general categories of design loads as follows:

1) Permanent Loads:
   - Loads that are always present in or on the bridge and do not change in magnitude during the life of the bridge.
   - Includes dead load and earth load.

2) Transient Loads:
   - Loads that are *not* always present in or on the bridge or *change* in magnitude during the life of the bridge.
   - Includes live loads, wind, temperature, braking force and centrifugal force.

## 4.3.2 Dead Loads

Superstructures must be designed to resist dead load effects. In LRFD, dead load components consist of DC and DW dead loads. DC dead loads include structural components and nonstructural attachments, and DW dead loads include wearing surfaces and utilities. Different load factors are used for DC and DW dead loads to account for the differences in the predictability of dead loads that are resisted by either the non-composite or composite sections. Table 4.3.2-1 summarizes the various material unit weights that are commonly used. Table 4.3.2-2 summarizes the unit weights of various traffic barriers commonly used.

| NHDOT Unit Weights | |
|---|---|
| **Material** | **Unit Weight ($lb/ft^3$)** |
| Bituminous Wearing Course<br>(No provisions for future overlay is required) | 140 |
| Reinforced Concrete | 150 |
| Steel | 490 |
| Timber | 50-60 |
| Granular Backfill (Bridge) | 120 |
| Stone Masonry | 170 |

**NHDOT Unit Weights**

*Table 4.3.2-1*

| NHDOT Traffic Barrier Unit Weights | |
|---|---|
| **Traffic Barrier** | **Unit Weight (lb/ft)** |
| T2 Steel Bridge Rail (8-ft. rail spacing) | 55 |
| T3 Steel Bridge Rail (8-ft. rail spacing) | 71 |
| T4 Steel Bridge Rail (8-ft. rail spacing) | 87 |
| 2-Bar Aluminum Bridge Rail (6'-9" rail spacing) | 17 |
| 3-Bar Aluminum Bridge Rail (7'-5" rail spacing) | 22 |
| 4-Bar Aluminum Bridge Rail (8-ft. rail spacing) | 26 |
| Aluminum Balusters | 5 |
| Steel Protective Screening (8-ft. rail spacing) | 17 |
| Steel Snow Screen (5-ft. high) (8-ft. rail spacing) | 12 |
| T101 Bridge Rail | 44 |
| Precast Concrete Barrier Double-Faced Single Slope (45-in.) (median barrier) | 867 |
| Temp. Portable Concrete Barrier – Anchored (20-ft.) | 400 |
| Temp. Portable Concrete Barrier - Braced | 416 |
| Temp. H-pile Steel Barrier | 93 |

**NHDOT Traffic Barrier Unit Weights**

*Table 4.3.2-2*

A. Distribution to Superstructure

For the distribution of the weight of plastic concrete to the girders, including that of an integral sacrificial wearing surface, assume that the formwork is simply supported between interior girders and tributary area over the exterior girders. Superimposed dead loads (e.g., curbs, barriers, sidewalks, parapets, railings, future wearing surfaces) placed after the deck slab has cured may be distributed equally to all girders (*AASHTO LRFD 4.6.2.2* "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 wider bridges with more than six girders, assume that the superimposed dead loads of sidewalks, parapets or railings are carried by the three girders immediately under and adjacent to the load. In some cases, such as staged construction and heavier utilities, the bridge designer should conduct a more refined analysis to determine a more accurate distribution of superimposed dead loads.

### 4.3.3 Earth Loads

A. Lateral Earth Pressure

Lateral earth pressure shall be assumed to be linearly proportional to the depth of earth and taken as:

$p = k \gamma_s z$ *(AASHTO LRFD Eq. 3.11.5.1-1)*

Where:

$p$ = lateral earth pressure (ksf)
$k$ = coefficient of lateral earth pressure ($k_0$, $k_a$, $k_p$)
$\gamma_s$ = unit weight of soil (kcf)
$z$ = depth below the surface of earth (ft)

B. At-Rest Lateral Earth Pressure Coefficient ($k_0$)

For normally consolidated soils and a vertical back of wall, the at-rest lateral earth pressure coefficient may be taken as:

$$k_0 = (1 - \sin \phi'_f)\,(1 + \sin \beta)$$

Where:

$\phi'_f$ = effective angle of internal friction of the soil (deg.)
$\beta$ = angle of back slope to the horizontal (deg.)

The resultant lateral earth force is assumed to act parallel to the backfill.

Most retaining walls are not designed for the at-rest earth pressure condition. Minimum wall movements required to reach the active pressure condition are given in *AASHTO LRFD Table C3.11.1-1*. Walls that can tolerate little or no movement should be designed for the at-rest lateral earth pressure condition. Even if the retaining wall is bearing on bedrock, it may still be able to rotate and meet the minimum wall movement to reach active pressure. The Geotechnical Engineer will recommend if an At-Rest condition should be used.

C. Active Lateral Earth Pressure Coefficient ($k_a$)

The active lateral earth pressure coefficient shall be calculated using the applicable earth pressure theories described herein.

1) Rankine Theory:

The Rankine theory is intended to be used for determining earth pressure on a frictionless vertical plane within a soil mass and not directly against the wall. The resultant lateral earth force acts on the vertical plane at an angle β that is assumed to be parallel to the backfill.

The Rankine lateral active earth pressure coefficient may be taken as:

For a level back slope ($\beta = 0$):

$$k_a = \tan^2\!\left(45 - \frac{\phi'_f}{2}\right) = \frac{1 - \sin(\phi'_f)}{1 + \sin(\phi'_f)}$$

For a sloping back slope (β > 0):

$$K_a = \cos\ \beta \cdot \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi'_f}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi'_f}}$$

Where:

$\phi'_f$ = effective angle of internal friction of the soil (deg.)

β = angle of backfill slope to the horizontal (deg.)

2) Coulomb Theory:

Coulomb theory is based on the static equilibrium of an assumed sliding wedge of cohesionless soil and takes into account the friction that develops along the back of the retaining wall and along the failure surface in the backfill. The resultant lateral earth force acts at an angle δ measured from a line normal to the back of the wall, where δ is the angle of friction between the backfill and back of wall. Refer to *AASHTO LRFD Table 3.11.5.3-1* for values of angle δ.

The Coulomb lateral active earth pressure coefficient may be taken as:

$$k_a = \frac{\sin^2 \cdot (\theta + \phi'_f)}{\sin^2\theta \cdot \sin(\theta - \delta) \cdot \left(1 + \sqrt{\dfrac{\sin(\phi'_f + \delta)\cdot\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\cdot\sin(\beta + \theta)}}\right)^2}$$

Where:

$\phi'_f$ = effective angle of internal friction of the soil (deg.)

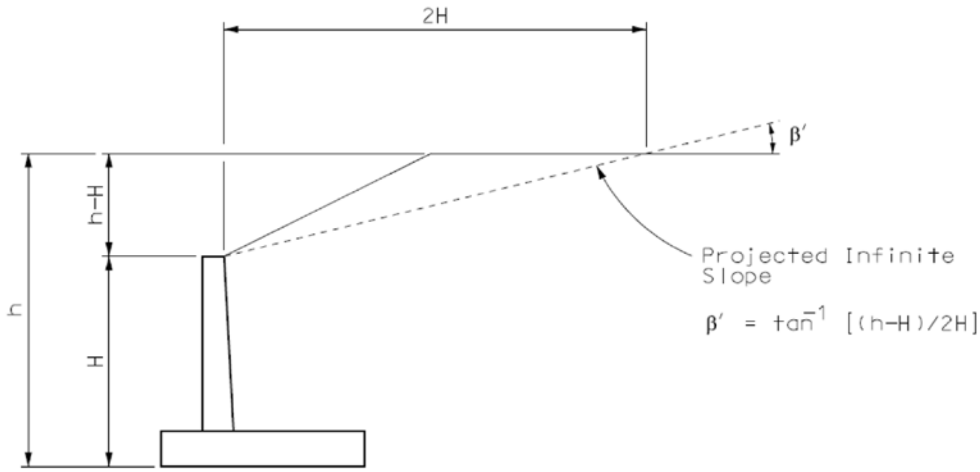
β = slope of the backfill slope to the horizontal (deg.)

θ = angle of the back of wall to the horizontal (deg.)

δ = friction angle between backfill and back of wall or virtual back of wall (deg.)

3) Adjustment for Broken Back Backfill Surface Condition:

For situations with a broken back backfill, the lateral active earth pressure may be determined assuming a projected infinite back slope of angle β′, as shown in Figure 4.3.3-1 *(AASHTO LRFD Figure 3.11.5.8.1-3*). For determining the active lateral earth pressure coefficient, $k_a$, the angle β′ is substituted for the angle β in the equations given herein.




**Broken Backfill Surface**

*Figure 4.3.3-1*

D. Cantilever Retaining Walls and Abutments (with heel projections):

Designs for cantilever retaining walls and abutments typically involve determining active lateral earth pressure on a vertical plan extending up from the heel of the wall which is referred to as the virtual back of wall. The effective wall height at the virtual back of wall, H′, is used to determine the lateral earth pressure and the weight of soil above the heel is considered as part of the wall weight (*AASHTO LRFD C3.11.5.3* and Figures 4.3.3-3 and 4.3.3-4).

*NHDOT's policy is to use Rankine Theory in determining the lateral earth pressure coefficient for all cantilever retaining walls and abutments unless otherwise approved by the Design Chief of Bureau of Bridge Design.* The current *AASHTO LRFD* code states that the use of Coulomb Theory is to be used for determining lateral earth pressure for cantilever walls with short heel projections. However, the use of the Rankine Theory in these instances will produce acceptable, conservative designs.

For many years, NHDOT has used the Equivalent-Fluid Method for determining lateral earth pressures on cantilever retaining walls and abutments. The use of Rankine Theory will produce similar results while allowing the designer to more readily account for the actual backfill slope and backfill properties when determining lateral earth pressure.

In certain situations it may be beneficial to use Coulomb Theory with cantilever retaining walls. Coulomb Theory is also used in designing gravity, semi-gravity, prefabricated modular, and non-gravity cantilevered (e.g., sheetpile) walls.

The following information is provided as a reference to illustrate the differences between Rankine and Coulomb Theories for determining active lateral earth pressure on a virtual back of wall.

1) Application of Active Lateral Earth Pressure Theories:

For a cantilever retaining wall, it may be necessary to first determine if full Rankine failure wedges can occur within the backfill on either side of the virtual back of wall or if the wall interferes with their formation. This determination is made by considering the location of the outer failure plane and its relationship to back of wall.

The outer failure plane may be located by rotating a vertical line through the heel of the footing by an angle α as shown in Figure 4.3.3-2 (*AASHTO LRFD Figure C3.11.5.3-1a)*. If the projection of the outer failure plane intersects the back of the wall, the heel projection is considered to be short; otherwise, the heel projection is considered to be long. Rankine Theory is theoretically only applicable to walls with long heel projections, whereas Coulomb Theory is applicable to walls with either short or long help projections.

Referring to Figure 4.3.3-2, the determination of the heel projection may be made as follows:

Long heel projection: $\alpha < \psi$

Short heel projection: $\alpha > \psi$

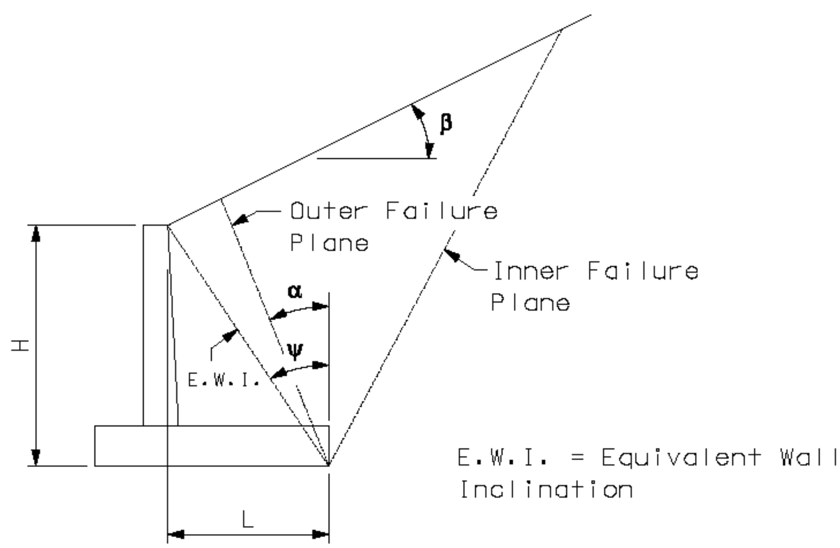
Where:

$\alpha$ = outer failure plane from vertical (deg.)
$= \frac{1}{2}\,(90 + \beta - \phi'_f - \sin^{-1}[\sin\beta / \sin\phi'_f])$ (*AASHTO LRFD Figure C3.11.5.3-1a*)

$\psi$ = equivalent wall inclination from vertical (deg.)
$= \tan^{-1}(L / H)$

$\phi'_f$ = effective angle of internal friction of the soil (deg.)

$\beta$ = angle of backfill slope to the horizontal (deg.)

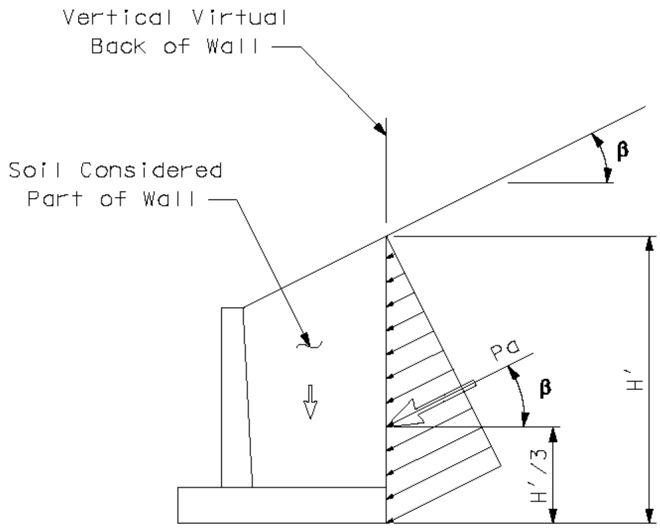



**Heel Projection and Outer Failure Plane**

*Figure 4.3.3-2*

a) Long Heel Projection:

For a concrete cantilever with a long heel projection, the Rankine or Coulomb Theory may be used to determine the active lateral earth pressure on the vertical virtual back of wall (H′). For the Rankine Theory the resultant lateral earth force (Pa) acts on the virtual back of wall at an angle parallel to the backfill as shown in Figure 4.3.3-3. For the Coulomb Theory, the resultant lateral earth force acts as described in section b).

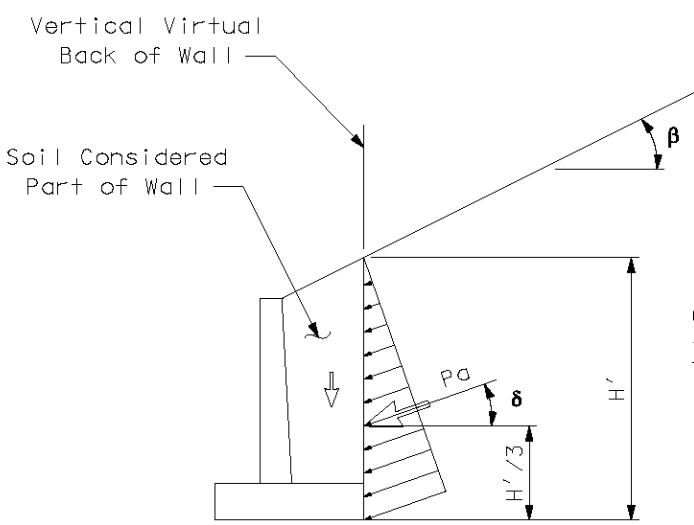



**Cantilever Wall with Long Heel Projection (α < ψ): Rankine Theory**

*Figure 4.3.3-3*

b) Short Heel Projection:

For a cantilever retaining wall with a short heel projection, the Coulomb Theory is used to determine the active lateral earth pressure on the vertical virtual back of wall (H′) as shown in Figure 4.3.3-4. The resultant lateral earth force (Pa) acts on the virtual back of wall at an angle δ which may be assumed to be within the range of 1/3 $\phi'_f$ to 2/3 $\phi'_f$.




**Cantilever Wall with Short Heel Projection (α > ψ): Coulomb Theory**

*Figure 4.3.3-4*

E. Passive Lateral Earth Pressure Coefficient ($k_p$)

Values of the passive lateral earth pressure coefficient, $k_p$, should be determined using *AASHTO LRFD Figures 3.11.5.4-1 & 2* which are based on the Log-Spiral Method. Both Rankine and Coulomb Theories assume the failure surface to be a straight plane, which is a reasonable assumption for active earth pressure; however, for passive earth pressure this assumption may give unsafe results. For passive earth pressure, most failure surfaces are curved and commonly assumed to be the arc of a logarithmic spiral.

Passive resistance at the front of gravity and semi-gravity retaining walls should typically be neglected due to the potential for erosion or future excavation of the material. Passive resistance may be considered in stability calculations for seismic loading cases, or otherwise approved by the Design Chief of the Bureau of Bridge Design.

F. Select Backfill and Prepared Foundation Material Properties

NHDOT uses material conforming to Item 209.20x Granular Backfill (Bridge) for backfilling all retaining structures. This material gradation is the same as Item 304.2, Gravel and is a free draining, very dense, granular material. The following properties shall be assumed for this backfill material and for the in-situ soil mass within the failure wedge, unless otherwise recommended by the Geotechnical Report:

$\phi'_f$ = effective angle of internal friction of backfill
= 34°

$\gamma_s$ = unit weight of backfill
= 120-pcf (1922-kg/$m^3$)

NHDOT uses material conforming to Item 508 Structural Fill under foundations for providing a well graded support of structures. This material consists of crushed gravel gradation as noted in the specification. The investigating foundation failure by sliding, the following properties shall be assumed for this material unless otherwise recommended by the Geotechnical Report:

$\phi_f$ = effective angle internal friction of structural fill
= 36°

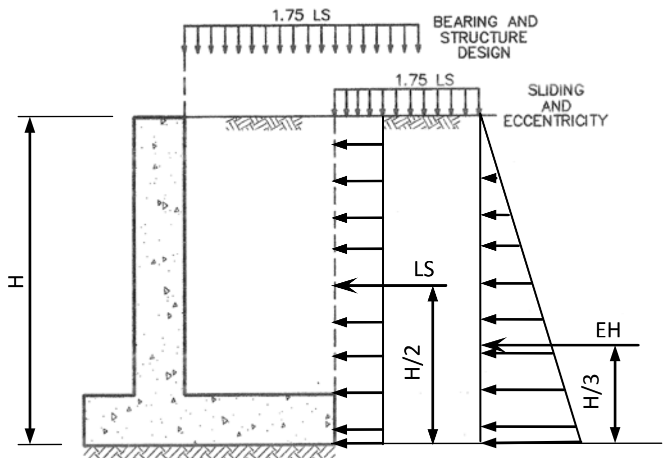
### 4.3.4 Surcharge Loads

Lateral earth pressures due to surcharge loads shall be taken into consideration and applied in accordance with *AASHTO LRFD 3.11.6*.

Surcharge due to vehicular loads shall be applied in accordance with *AASHTO LRFD 3.11.6.4*. Equivalent heights of soil for vehicular loading for various retaining wall and abutment heights are given in *AASHTO LRFD Tables 3.11.6.4-1 & 2.*

A live load surcharge shall be applied to all walls that retain soil supporting a roadway where vehicular load acts within a distance equal to one-half the wall height behind the back face of the wall, or the virtual back of wall. If the live load extends onto the heel of a cantilever retaining wall, the weight of the surcharge above the heel shall be considered in the design. Refer to *AASHTO LRFD Figure C11.5.6-3* and Figure 4.3.4-1 for application of the live load surcharge.

Live load surcharge need *not* be considered where reinforced concrete approach slabs of sufficient length are provided at bridge ends, or the portions of wingwalls adjacent to an approach slab; however, the designer shall consider the reactions on the abutment due to the axle loads on the approach slab (See Chapter 6, Substructure).




**Live Load Surcharge (LS)**
**Loading Diagram**

*Figure 4.3.4-1*

### *4.3.5 Live Loads*

A. Traffic Live Load

Live load design criteria are specified in the “Design Loads, Materials and Specifications” note on the Bridge Notes Plan Sheet. The live load design criteria are determined using the following guidelines:

- New bridges, new bridge decks, and bridge widenings with addition of substructure: HL-93
- Bridge rehabilitation of superstructures and substructures: Live Load criteria as determined by the Design Chief.
- Diversion and other temporary bridges: HL-93

The Live Load consists of the following load types:

- Design truck
- Design tandem
- Design lane
- Two design trucks
- Fatigue truck
- Pedestrian load

Using these basic load types, *AASHTO LRFD* combines and scales them to create live load combinations that apply to different limit states, as described in *AASHTO LRFD 3.6.1*. The NHDOT overload provisions (additional 25%) previously used on bridges designed by the Load Factor Design (LFD) method will not be used with the HL-93 loading.

The extreme force effect shall be taken as the larger of the loadings noted in *AASHTO LRFD 3.6.1.3*.

*AASHTO LRFD C3.6.1.3.1* states consideration shall be given to investigating the negative moment between the points of contraflexure under a uniform load on all spans and reactions at interior supports with a *dual design tandem* spaced from 26.0-ft. to 40.0-ft. apart, combined with the design lane load. The pairs of the design tandem shall be placed in adjacent spans in such position to produce maximum force effect. This loading shall be investigated for spans ≥ 150-ft.

Where appropriate, consider additional live loads including snow removal equipment on sidewalks and bridge inspection or snooper loads on bridges with large overhangs. If construction equipment or maintenance equipment can or will operate adjacent to retaining walls and abutments, a live load surcharge should be incorporated into the design.

B. Load Factor

The load factor for live load, Extreme Event I, $\gamma_{EQ,}$ shall be taken as 0.5 for all bridges.

C. Dynamic Load Allowance

The HL-93 loading is based on a static live load applied to the bridge. However, in reality, the live load is not static but is moving across the bridge. Since the roadway surface on a bridge is usually not perfectly smooth; a dynamic load is applied to the bridge and must be considered with the live load. This is referred to as dynamic load allowance (IM). The dynamic load allowance shall be applied in accordance with *AASHTO LRFD 3.6.2*.

Dynamic load allowance shall be applied to the following:

- Integral abutment caps and piles
- Portion of pier above the footing
- Counterforts and caps of spilled through abutments
- Piles in a pile bent, including the portion of the piles below ground line
- Drilled shafts integral with columns

Dynamic load allowance shall *not* be applied to the following:

- Retaining wall not subject to vertical reaction from the superstructure
- Pedestrian loads
- Centrifugal force effects
- Braking force
- All foundation components entirely below ground, including abutment stem (except as specified for integral abutments)
- Frame pier
- Bearings (*AASHTO LRFD 14.4.1)*
- Timber structures
- Culverts and other buried structures with 8-ft. or greater depth of earth cover (see *AASHTO LRFD 3.6.2.2*)

D. Live Load Deflection (Straight Girder Bridges)

The loading for live load deflection criteria is defined in *AASHTO LRFD 3.6.1.3.2* as using the larger effects from the following loadings:

- (Design Truck) x (Live Load Distribution Factor) x (Dynamic Load Allowance) x (Multiple Presence Factor)

- [(25% of Design Truck) x (Dynamic Load Allowance) + (100% of Design Lane Load)] x (Live Load Distribution Factor) x (Multiple Presence Factor)

  Where: Live Load Distribution Factor = (#Lanes / #Beams)

The distribution factor for computing live load deflection is not the same as the moment distribution factor, because it is assumed that for straight bridges all beams or girders act together and have an equal deflection. However, for curved bridges, each girder must be checked individually. When investigating the maximum absolute live-load deflection, all design lanes should be loaded, and the supporting components should be assumed to deflect equally. For multi-girder bridges, this is equivalent to saying that the distribution factor for computing live-load deflection is equal to the number of lanes divided by the number of girders *(AASHTO LRFD C2.5.2.6.2)*. Use the Service I live load and include the multiple presence factor as defined in *AASHTO LRFD 3.6.1.1.2*.

Live load deflections for the Service I limit state shall satisfy the requirements of *AASHTO LRFD 2.5.2.6.2 except* as noted below:

⇒ NHDOT live load deflection limit for steel and concrete vehicular bridges:

- Vehicular loads: span/1000
- Vehicular and pedestrian loads: span/1000

E. Live Load Distribution to Substructure

For steel and prestressed concrete superstructures where the live load is transferred to the substructure through bearings, the girder reaction may be used for substructure design. Live load placement is dependent on the member under design.

Some examples of live load placement are as follows.

- For abutment design, the live load reaction is calculated using a vehicle lane reaction. The vehicle lane reaction is determined by positioning the lane loading and the HL-93 truck loading longitudinally on the structure to obtain the maximum reaction at the support location. The vehicle lane reaction should be increased for the number of design traffic lanes and distributed over the entire length of the substructure stem to obtain a load per unit length.

  Live load distribution factors are not used for abutment design, because it is too conservative to apply the maximum live load reaction for each individual girder. Each individual girder will generally not experience its maximum live load reaction simultaneously because each one is based on a different configuration of design lane locations. See Chapter 6, Section 6.4 Abutments, for further design information.

- For pier design, the maximum live load effects in the pier cap, column, and footing are based on the number of design lanes and their location. The lane loading and the HL-93 truck loading are placed within each lane to obtain the maximum transverse moment at the top of the pier; then re-located to obtain the maximum axial force of the pier.

  For pier design, possible future changes in the physical or functional clear roadway width of the bridge shall be considered. The number of design lanes is determined by the clear roadway width between curbs (*AASHTO LRFD 3.6.1.1.1*). If the bridge has a sidewalk, consideration shall be given for any future changes such as removing the sidewalk and adding a turning lane. The design of the exterior girders will not be affected since the exterior girder is the same design as the interior girder. The pier design *will* be affected because the live load placement will change if the sidewalk is removed.

For pier design, determine the unfactored reactions for the lane and truck load given on a per lane basis (do not use a distribution factor for girder reactions). The reactions should include the dynamic load allowance. A 10% reduction for interior pier reactions should be applied if the truck pair in conjunction with lane loading controls (*AASHTO LRFD 3.6.1.3.1*). Distribute the truck reactions and lane load reaction to the bearings assuming the deck is a continuous beam over the girders. See Chapter 6, Section 6.6 Piers, for further design information.

### 4.3.6 Vehicular Braking Force

The braking force (BR) shall be determined according to *AASHTO LRFD 3.6.4.*

- The dynamic load allowance factor is not applied to braking forces.
- Multiple presence factors are to be used.
- In bridges where there is a pinned connection (i.e., beam bridges on bearings) between the superstructure and substructure, the braking force can be assumed to be applied at the bearings.
  - For fixed bearings and anchored (vulcanized) elastomeric expansion bearings, the force is divided among the bearings and is applied to the substructure.
  - For bridges with all the bearings anchored (vulcanized) elastomeric expansion bearings, the force is divided among the bearings and is applied to the substructure.
  - For bridges with fixed and sliding bearings, any amount of force above the friction force of the sliding bearings must be distributed to the fixed bearings.
- In bridges where there is a moment connection between the superstructure and substructure (integral), the braking force is applied at a height of 6-ft. (1.8-m) above the roadway surface in a longitudinal direction.
- For skewed piers, the applications of braking force at 6-ft. (1.8-m) above the deck will create a moment component parallel with the center line of the pier cap. The designer may apply the parallel component by means of vertical coupler applied at the exterior girder bearings. The component normal to the pier cap shall be applied as a moment on the weak axis of the pier.
- The number of lanes used to determine the total force shall be the number of lanes likely to become one directional during the service life of the bridge.
  - For one-way bridges, apply the braking force in all lanes.
  - For two-lane, two-way bridges, apply the braking force in one direction in all lanes.
  - For two-way bridges with more than two traffic lanes, determine the traffic direction with the greatest width and apply the braking force to the number of lanes that fit within the width.

### 4.3.7 Superimposed Deformation Loads

Bridge elements change size or position due to temperature change (TU), shrinkage (SH), creep (CR), settlement, and post-tensioning (PS). These changes in geometry cause movements at expansion joints, bearings, and stresses which shall be considered in design.

Curved and skewed bridges have transverse as well as longitudinal movement due to temperature effects, shrinkage, and creep. Transverse movement may become significant for very wide bridges. Refer to *Appendix B of AASHTO LRFD/NSBA Steel Bridge Bearing Design and Detailing Guidelines* for additional information.

For skewed bridges, the longitudinal and transverse movement of the superstructure causes superimposed deformation both perpendicular and parallel to the pier/abutment. The longitudinal and transverse superimposed deformation is resolved into perpendicular and parallel force components to be applied to the pier/abutment.

Expansion joints and bearings shall be designed to accommodate movements resulting from changes in the superstructure length.

The design of the substructure shall include forces resulting from restraint of superstructure movements as well as forces resulting from the change in length of substructure elements.

A. Thermal Effects (TU)

Temperature changes in the superstructure cause it to expand and contract along its longitudinal and transverse axes. These dimensional changes induce forces in the substructure units based upon the fixity of the bearings, as well as the location and number of substructure units.

Substructures supporting fixed bearings and elastomeric bearings vulcanized to a masonry plate and anchored shall be designed for the horizontal force induced by thermal effects (TU).

Substructures supporting elastomeric bearings sitting on concrete shall be designed for the horizontal force induced by the maximum anticipated shear deformation in the elastomeric bearing due to thermal movement. The shear deformation force shall be compared against the friction force between the elastomeric pad and the concrete. The elastomeric bearing shall be designed so the friction force is greater than the shear deformation force to prevent the bearing from moving. Otherwise, the bearing shall be vulcanized to a masonry plate and anchorage provided. See Chapter 7, Section 7.5 Bearings, for calculation of shear deformation and friction force in an elastomeric bearing.

Variation in the superstructure temperature produces elongation and shortening. Therefore, thermal movement range is calculated using the maximum and minimum design superstructure temperatures during the structure's lifetime. The following are the maximum and minimum temperatures for design in New Hampshire:

Concrete Girder Superstructures: 0° F to $^{+}80°$ F ($\Delta T = 80°$ F)

Steel Girder Superstructures: $^{-}20°$ F to $^{+}105°$ F ($\Delta T = 125°$ F)

Concrete Substructures (pier caps, columns, etc.): $T_{Rise} = 35°$ F, $T_{Fall} = 45°$ F

Total thermal movement range is then calculated as:

$M_t = \alpha \cdot L_{trib} \cdot \Delta T$ (Concrete or Steel Superstructures)

$M_t = \alpha \cdot L_{trib} \cdot (T_{Rise} \text{ or } T_{Fall})$ (Concrete Substructures designed for temperature change)

where:

$L_{trib}$ = tributary length of the structure subject to thermal variation. The expansion length for superstructures is measured along the centerline of the bridge; for piers, the expansion length is measured along the center line of bearings or pier cap.

$\alpha$ = coefficient of thermal expansion; 0.000006 in./in./°F for concrete and 0.0000065 in./in./°F for steel *(AASHTO LRFD 5.4.2.2 and 6.4.1)*

$\Delta T$ = difference between the maximum design temperature and the minimum design temperature, as noted above.

$T_{Rise}$, $T_{Fall}$ = Change in temperature from time of substructure construction.

B. Shrinkage Effects (SH)

The calculation of shrinkage as a function of time requires that ambient relative humidity, volume-to-surface ratios, curing methods, and concrete strength be taken into consideration in accordance with *AASHTO LRFD 5.4.2.3*.

The sequence of construction should be considered when accounting for this effect. If expansion joints are installed in block outs after placement of the bridge deck they must accommodate only the shrinkage that occurs after the time of installation. For most situations the concrete shrinkage strain may be estimated to be 0.0002 for normal weight concrete in an unrestrained condition in lieu of more refined calculations described in *AASHTO LRFD 5.4.2.3*.

The design displacement due to shrinkage corrected for restraint conditions imposed by various superstructure types is calculated as follows *(based on WSDOT procedure)*:

This value must be corrected for restraint conditions imposed by various superstructure types.

$\Delta_{shrink} = \varepsilon_{sh} \cdot \mu \cdot L_{trib}$

where:

$L_{trib}$ = tributary length of the structure subject to shrinkage

$\varepsilon_{sh}$ = shrinkage strain at a given time; estimated as 0.0002 in lieu of more refined calculations *(AASHTO LRFD 5.4.2.3.1)*

$\mu$ = restraint factor accounting for the restraining effect imposed by superstructure elements installed before the concrete bridge deck is placed.

= 0.0 for steel girders
(Shrinkage need not be considered for steel superstructures. Although the concrete deck will shrink after placement, the typical shear stud attachment of the deck to the steel superstructure will cause the shrinkage to dissipate in small cracks throughout the expansion length rather than accumulating at the end.)

= 0.5 for precast prestressed concrete girders

= 0.8 for concrete box girders and T-beams

= 1.0 for concrete flat slabs

= 1.0 for calculation of change in length of concrete substructure elements (e.g.; pier caps, columns, etc.)

C. Load Factors ($\gamma_{TU}$, $\gamma_{SH}$, $\gamma_{CR}$ and $\gamma_{PS}$)

1) For Sizing Expansion Joints and Bearings:

At all service and strength limit states $\gamma_{TU}$ = 1.2

- When calculating the movement (horizontal displacement) due to temperature change to avoid under sizing expansion joints and bearings (*AASHTO LRFD Table 3.4.1-1*).
- The load factor is <u>not</u> applied to asphaltic plug joints because the asphaltic plug joint does not require sizing. (See Chapter 7, Section 7.4.3 for sizing expansion joints.)

2) For Determining Forces:

At all service limit states $\gamma_{TU}$, $\gamma_{SH}$, $\gamma_{CR}$ and $\gamma_{PS} = 1.0$

At all strength and extreme event limit states $\gamma_{SH}$, $\gamma_{CR}$ and $\gamma_{PS} = \gamma_P$ (as given in *AASHTO LRFD Table 3.4.1-3.)*

At all strength limit states $\gamma_{TU}$ may be taken as follows:

- $\gamma_{TU} = 0.5$ for concrete substructures using $I_g$ (e.g., tall flexible piers and columns)
- $\gamma_{TU} = 1.0$ for concrete substructures using $I_{effective}$ (e.g., short stiff abutments)
- $\gamma_{TU} = 1.0$ for steel substructures (e.g., integral bridge or stub abutment on steel piles)

For guidance on structural material behavior as it relates to modeling cracking of concrete refer to *AASHTO LRFD 4.5.2.2 and 4.7.1.3.*

### 4.3.8 Friction Forces

Forces transmitted through bearings with sliding surfaces or elastomeric bearing on concrete should be considered as friction forces (FR). Structural elements supporting bearings with sliding surfaces or elastomeric bearing on concrete should be designed to resist the force transmitted before sliding occurs.

At the strength, extreme event, and service limit states the load factor shall be 1.00 for forces applied to the substructure *(AASHTO LRFD Table 3.4.1-1).*

It is recommended that the maximum coefficient of sliding friction assumed in design be given in the contract documents to ensure the bearings are manufactured to perform as designed.

Include friction forces where design loads would increase, but neglect friction forces where design loads would decrease.

A friction-acting bearing can only transmit the longitudinal force that causes the bearing to move. Any amount of force above the friction force must be distributed to the fixed bearings.

For skewed bridges, the longitudinal and transverse movement of the superstructure causes friction forces both perpendicular to the pier or abutment and parallel with the pier or abutment. The longitudinal and transverse friction force is resolved into perpendicular and parallel force components to be applied to the pier/abutment.

The force transmitted to supporting elements before sliding or walking occurs is calculated as follows:

$FR = \mu\,(DC_1 + DC_2 + DW)$

where:

FR = friction force

$DC_1$, $DC_2$, DW = total dead load reaction

$\mu$ = coefficient of friction between bearing sliding surfaces (*AASHTO LRFD Table 14.7.2.5-1* for PTFE surface)

⇒ NHDOT sliding bearings:

*Choose one {
- o PTFE (Teflon) surface: unfilled sheet slides across a smooth stainless steel plate (typical NHDOT bearing).
- o PTFE (Teflon) surface: filled, dimpled-lubricated
}
- o Determine pressure on PTFE surface from max. vertical design load of bearing ÷ area of PTFE surface
- o Use the lowest temperature used for the bearing design
- o Intermediate values may be determined by interpolation
- o See *AASHTO LRFD 14.7.2.5*

⇒ NHDOT elastomeric bearing on concrete:

- o A friction coefficient of 0.20 may be assumed for elastomeric pads that are in contact with clean concrete or steel surfaces (*AASHTO LRFD C14.8.3.1).*

* Note: If a lower coefficient of friction is needed for the design, then the filled, dimpled-lubricated can be specified and the thickness of the PTFE increased for the machining of the dimples as noted in *AASHTO LRFD 14.7.2.1*. The dimpled condition is rarely used, requires longer fabrication time, and is expensive.

### 4.3.9 Ice Forces

New Hampshire bridges subject to ice conditions shall be designed for ice loads (IC) in accordance with *AASHTO LRFD 3.9* and the following information.

For all ice loads, investigate each site for existing conditions. The local resources should be contacted for information regarding the ice conditions. The Cold Regions Research and Engineering Laboratory (CRREL) Ice Engineering Group has an Ice Jam Database that contains historical information from across the US. The Ice Jam Database is located at: http://rsgisias.crrel.usace.army.mil/apex/f?p=273:1.

New Hampshire Rivers known to have severe ice conditions are the Connecticut River, Pemigewasset River, Baker River, Contoocook River, and the Israel River. However, the designer shall investigate each specific project site for ice conditions. NH lake crossings also require investigation of ice conditions.

Additional ice analysis information can be found from the *Army Corps of Engineers, "Ice Engineering Manual", EM-1110-2-1612, 30 October 2002* located at: http://www.publications.usace.army.mil/USACEPublications/EngineerManuals.aspx?udt_43544_param_page=6, and *"Method to Estimate River Ice Thickness Based on Meteorological Data", ERDC/CRREL TN-04-03, June 2004* located at: http://acwc.sdp.sirsi.net/client/search/asset/1001540.

Extreme Event Ice loads need not be considered concurrently with Service or Extreme Event scour conditions. Forces from floating ice and expanding ice, IC, do not act on a pier at the same time. Consider each force separately when applying these design loads.

For pier shafts skewed to stream flow the total force on the pier will be determined based on projected pier width *(AASHTO LRFD 3.9.2.4.2).* The projected width will increase the ice load and, if the load seems excessive, the designer should investigate a circular pier shaft or other pier alternatives.

Static loading should be used when it is anticipated that ice may occur between two substructure units while having open water in an adjacent span. Static ice loads should be applied separately and not combined with dynamic ice loads. It is not necessary to design for ice uplift or ice jams except in very special circumstances.

A. Dynamic Ice Load

If no data is available, use the following data as the minimum design criteria:

- Ice pressure (effective ice crushing strength) = 24 ksf or as noted in *AASHTO LRFD 3.9.2.1*
- Ice thickness = 1.5 ft.

  (River bridges that have severe ice conditions are special cases that need to be investigated as recommended by the Design Chief.)
- Point of application = $Q_{50}$ Elevation

  (River bridges that have severe ice conditions are special cases that need to be investigated as recommended by the Design Chief.)
- The designer may assume the friction angle, $\theta_f$ of the transverse force equation in *AASHTO LRFD 3.9.2.4.1-1* to be 0 degrees unless more precise information is available. This value is conservative. If the loads become unrealistic use $\theta_f = 6°$ *(Wyoming DOT Bridge Design Manual)*

B. Static Ice Load

If no data is available, use the following data as the minimum design criteria:

- For bridges over lakes, static ice pressure shall be considered. *(AASHTO LRFD 3.9.3)*
- Design Load = 5 k/ft on pier side
- Ice thickness = Obtained from local resources or historic data.
- Point of application = Mean Water Elevation

### *4.3.10 Water Loads*

The force of flowing water (WA) acting on piers located in streams shall be determined according to *AASHTO LRFD 3.7*. This stream flow force acts in both the longitudinal and lateral directions (if water is flowing at an angle to the longitudinal axis of the pier). Computed longitudinal pressure shall be based on the drag coefficient for piers *(AASHTO LRFD Table 3.7.3.1-1)*. Computed lateral pressure shall be based on the drag coefficient for the actual pier skew angle with respect to stream flow *(AASHTO LRFD 3.7.3.2)*.

The Hydraulic Report will state the stream velocity and elevation for the design flood event (100-year) and the normal high water, which should be used to determine loads at the strength and service limit states. See Chapter 2 Section 2.7, Bridge Hydraulic Study for additional waterway opening information.

A. Scour

Scour is not a load by itself but because it changes the supporting conditions and ultimately the force effects, substructure force effects must consider fully scoured conditions.

All foundations must be designed to withstand the conditions of scour for the design flood and the “check flood”. The design flood for scour considered for the service and strength limit states shall be the 100-year event. The "check flood" for scour water and scour elevations considered for the extreme event limit state shall be the lesser of the overtopping and 500-year flood.

Bridge scour shall be considered for the service, strength and extreme limit state in accordance with Chapter 2, Section 2.7.7, Stability Analysis and Countermeasures.

B. Buoyancy

Buoyancy, a component of water load WA, is specified in *AASHTO LRFD 3.7.2* and is taken as the sum of the vertical components of buoyancy acting on all submerged components.

- The footings of piers in the floodplain are to be designed for uplift due to buoyancy.
- Full hydrostatic pressure based on the water depth measured from the bottom of the footing is assumed to act on the bottom of the footing.
- The upward buoyant force equals the volume of concrete below the water surface times the unit weight of water.
- The effect of buoyancy on column design is usually ignored.
- Use high water elevation when analyzing the pier for overturning.
- Use low water elevation to determine the maximum vertical load on the footing. The submerged weight of the soil above the footing is used for calculating the vertical load on the footing.

### *4.3.11 Wind Loads*

Wind loads on live load (WL) and wind loads on structures (WS) shall be determined in accordance with *AASHTO LRFD 3.8,* unless otherwise modified in this manual. Wind loads on sign structures, luminaires and traffic signals shall be in accordance with *AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*," and Chapter 10, Non-Bridge Structures.

A. Wind Velocity

The design 3-second gust wind speed, V, shall be the wind speed at the project location as interpolated from *AASHTO LRFD* Figure 3.8.1.1.2-1 (ASCE 7-10, 2010, MRI 700 years). For the Special Wind Region (i.e. regions along the NH-VT border and Franconia Notch) as shown in *AASHTO LRFD* Figure 3.8.1.1.2-1, the maximum-recorded wind speed in this area shall be used if it is greater than 115-mph (185-kph), else use 115-mph (185-kph). See Chapter 10, Appendix 10.2-A1 and weather stations in the special wind region for recorded wind speeds.

B. Wind Load to Superstructure

The design wind pressures, $P_z$, shall be applied to the superstructure elements as specified in *AASHTO LRFD 3.8.1.2.*

- Longitudinal components of wind on superstructure should be distributed to the fixed substructure(s).
- Longitudinal and transverse (lateral) wind pressures shall be applied simultaneously at the centroid of the exposed area.

- Transverse stiffness of the superstructure may be considered, as necessary, to properly distribute loads to the substructure provided that the superstructure is capable of sustaining such loads.
- For determination of wind loads applied to steel or aluminum railing or chain link fence, designers may assume an equivalent solid area equal to 30% of the gross rail or fence surface area.

C. Wind Load to Substructure

Wind forces shall be applied to the substructure units in accordance with the loadings specified in *AASHTO LRFD 3.8.1.2.3*.

- Vertical wind pressure, per *AASHTO LRFD 3.8.2*, shall be included in the design where appropriate.
- Wind loads shall be distributed to the piers and abutments in accordance with the laws of statics.
- Transverse wind loads can be applied directly to the piers assuming the superstructure to act as a rigid beam.
- For large structures a more appropriate result might be obtained by considering the superstructure to act as a flexible beam on elastic supports.

D. Wind Load on Live Load

Both transverse and longitudinal wind forces (with respect to the superstructure) on vehicles (WL) shall be determined as required in *AASHTO LRFD 3.8.1.3*.

- The force effects of wind on live load shall be considered for the Strength V and the Service I load combinations.
- The wind on vehicular live loads is applied to the bearings in the same manner as the wind load from the superstructure. That is, the total transverse and longitudinal load is equally distributed to each bearing and applied at the top of the bearing.
- The transverse load acting six feet above the roadway applies a moment to the pier cap. This moment induces vertical reactions at the bearings.

E. Vertical Wind Load

As specified in *AASHTO LRFD 3.8.2* an overturning vertical wind force, WS, shall be applied to the underside of the superstructure for limit states that include wind but not wind on live load and only when the direction of wind is perpendicular to the longitudinal axis of the bridge.

- This lineal force shall be applied at the windward ¼ point of the deck, which causes the largest upward force at the windward fascia girder.
- For typical bridges the pressure is applied only in the Strength III and Service IV load combination when the wind skew angle with respect to the superstructure is zero degrees.
- The designer shall check the structure both with and without the vertical wind load.

F. Wind Load on Soundwalls

Wind forces shall be applied to soundwalls in accordance with *AASHTO LRFD Section 15, Design of Sound Barriers* and Chapter 10, Section 10.7 of this Manual.

### 4.3.12 Construction Loads

Construction loads that include force effects developed during construction should be considered if warranted. This load combination must also include force effects from any project construction constraints that are likely to induce additional stresses. If the project is a bridge rehabilitation, the existing bridge components shall be investigated for any force effects due to construction loads.

The load combination and factors for construction loads shall be in accordance with *AASHTO LRFD 3.4* and *AASHTO Guide Design Specifications for Bridge Temporary Works*.

When cranes or other heavy construction equipment are expected to come within close proximity of abutments or retaining walls, the construction load combination must consider a surcharge load appropriate with the anticipated construction equipment. All construction constraints and construction load assumptions shall be specified on the contract drawings.

Components designed for future jacking of the superstructure (including the sizing of the jacks) shall be designed for the Strength I limit state. Loading shall include both dead load and live load in anticipation that the bridge will remain open to traffic during the future jacking operations.

In the absence of more accurate information, some possible construction loads listed in Table 4.2.3-2 can be assumed for investigation of the strength and service limit states during construction in accordance with *AASHTO LRFD 3.4* and *AASHTO Guide Design Specifications for Bridge Temporary Works*.

| Possible Construction Loads | |
|---|---|
| **Load Description** | **Unit Weight** |
| Screed Machine LL | 7000 lb (applied over 10-ft. longitudinal length at center of exterior girders for bridge widths up to 32-ft.; 3500-lb over each exterior girder) |
| Construction LL | 20 lb/ft$^2$ (applied over a 20-ft. longitudinal length extended the entire width of the bridge) |
| Walkway LL | 75 lb/ft (applied over a 20-ft. longitudinal length outside of bridge coping) |
| Temporary Formwork DL | 15 lb/ft$^2$ |
| Temporary Walkway DL | 15 lb/ft$^2$ (applied over 2-ft. wide platform on outside of coping) |

**Possible Construction Loads**

*Table 4.2.3-2*

### 4.3.13 Earthquake Loads

Seismic design of new bridges and bridge widenings shall conform to *AASHTO LRFD Section 3.10 and AASHTO Guide Specifications for LRFD Seismic Bridge Design.*

For nonconventional bridges, bridges that are deemed critical or essential, or bridges that fall outside the scope of the Guide Specifications for any other reasons, project specific design requirements shall be developed and submitted to the Design Chief for approval.

See Chapter 5, Seismic Design and Retrofit for additional information.

### 4.3.14 Vehicular Collision Force

*AASHTO LRFD 3.6.5* contains provisions for vehicular collision forces (CT) on structures that cross over roadways that routinely carry trucks and have design speeds of 50-mph (80-kph) or higher. The following shall be considered when determining if a vehicular collision force shall be applied to unprotected piers, sign structures, and other structures.

- *AASHTO LRFD 3.6.5* states "abutments and piers located within a distance of 30.0-ft. to the edge of roadway shall be investigated for collision." For clarification, NHDOT Bridge Design defines "edge of roadway" to be "edge of travel lane".
- Substructure Type:
  - Vehicular collision force is not required for abutments or retaining walls (NHDOT Bridge Design policy) due to the soil behind the walls, which will absorb the force.
  - Vehicular collision force is not required for MSE walls supporting shallow stub abutments or MSE retaining walls (NHDOT Bridge Design policy). The vehicular impact would be limited to the panel facing where some loss of adjacent backfill would occur. The tension reinforcement directly above the voids at the impact location should provide enough "bridging" support until the wall is repaired.
  - Only unprotected piers, sign structures, and other structures as directed by the Design Chief shall be considered for a vehicular collision force.
- Route classification, ADTT (average daily truck traffic in one direction), ADT (average daily traffic) and roadway design speed:
  - The pier may require a vehicular collision force only if it is located on a roadway that routinely carries trucks and has a design speed of 50-mph (80-kph) or higher.
- Geometry:
  - Vehicular collision force is not required for pier located a distance greater than 30-feet (9.14-m) from the edge of the travel lane. For new structures, consider planned widenings or future realignments of lower roadways when establishing limits of setback distances and clear zones or horizontal clearance limits.
- Design Exemption/Impact History:
  - The designer shall consult with the Design Chief regarding the possibility of a design exemption, as described in *AASHTO LRFD C3.6.5.1*, which determines the estimated annual frequency a pier may be hit by a heavy vehicle. This exemption may only be used on redundant piers (3 or more columns) or wall piers (NHDOT Bridge Design policy).

A. New Structures

- Locate piers a distance greater than 30-feet (9.1- m) from the edge of the travel lane. Clear zones for roadways at many locations are equal to or greater than 30-feet (9.14-m). Designing for a vehicular collision force is not required for piers located outside 30-feet (9.14 m) from the edge of the travel lane. Consider planned widenings or future realignments of lower roadways when establishing horizontal clearance limits.
- If a pier needs to be located within 30-feet (9.14-m) of the edge of the travel lane, the designer shall consult with the Design Chief and investigate an exemption as described in *AASHTO LRFD C3.6.5.1*.
- If a pier is within 30-feet (9.14-m) of the edge of the travel lane and does not qualify for an exemption, either the pier shall be designed for the 600-kip (2670-kN) vehicular collision force (CT) *or* it shall be protected with an embankment or barrier that meets *AASHTO LRFD 3.6.5.1*. See Chapter 6, Section 6.6.5, for vehicular collision pier protection. When considering the options, the designer shall include aesthetics, maintenance, and cost as they apply to the bridge pier.
- If the barrier protection option is chosen, no further analysis of the collision force acting on the pier is required. The vehicle protection barrier will redirect and absorb the collision force as described and shown in Chapter 6, Section 6.6.5 Vehicular Collision Pier Protection.
- Providing structural resistance in the pier may be a better and more economical option than providing an embankment or barrier, except where a median barrier will be provided as part of the highway design. The bridge designer shall work with the roadway design engineer to determine which alternative is preferred.
- The primary design objective for extreme event load cases is preventing the loss of a span. When considering the 600-kip (2670-kN) vehicular collision force in design, the load and resistance factors shall all be set to 1.0. Plastic deformation of crashwalls, pier columns, etc., is permitted, subject to the requirement that loss of span shall be prevented.
- The vehicular collision force is a point load acting on the pier. The lateral vehicular collision force will transfer to the foundation, but resistance is provided by passive soil pressure, friction, and pile structural capacity. In addition, movement beyond what is reasonable for service loadings is allowed for an extreme event situation where survival of the bridge is the goal. Vehicle collision force transferred to the foundation will be a project specific analysis.
- Generally, a new reinforced concrete wall pier can be designed to resist the vehicular collision force.

B. Existing Structures and Widening of Existing Structures.

- If the existing pier is located within 30-feet (9.14-m) of the edge of the travel lane, the designer shall consult with the Design Chief and investigate an exemption as described in *AASHTO LRFD C3.6.5.1*.
- If the existing pier is within 30-feet (9.14-m) of the edge of the travel lane and does not qualify for an exemption, either the existing pier shall be analyzed to show it is capable of resisting the 600-kip (2670 kN) vehicular collision force (CT) *or* it shall be protected with an embankment or barrier that meets *AASHTO LRFD 3.6.5.1*. See Chapter 6, Section 6.6.5 for vehicular collision pier protection. When considering the options, the designer shall include aesthetics, maintenance, and cost as they apply to the bridge pier.

- The existing pier can be evaluated for its capacity and connection to the foundation and beam elements to determine its resistance. Factors that affect the pier resistance include, but are not limited to:
  - The continuity of the superstructure.
  - The continuity of the superstructure to the substructure and any frame action which might aid in the distribution of force effects.
  - The condition of the bearing devices and ability to resist translations and rotations.
  - The degree of redundancy of the superstructure.
  - The continuity of the substructure to the foundation system.
  - The amount of confinement reinforcement within the column and potential ultimate reserve capacity beyond the design capacity.

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