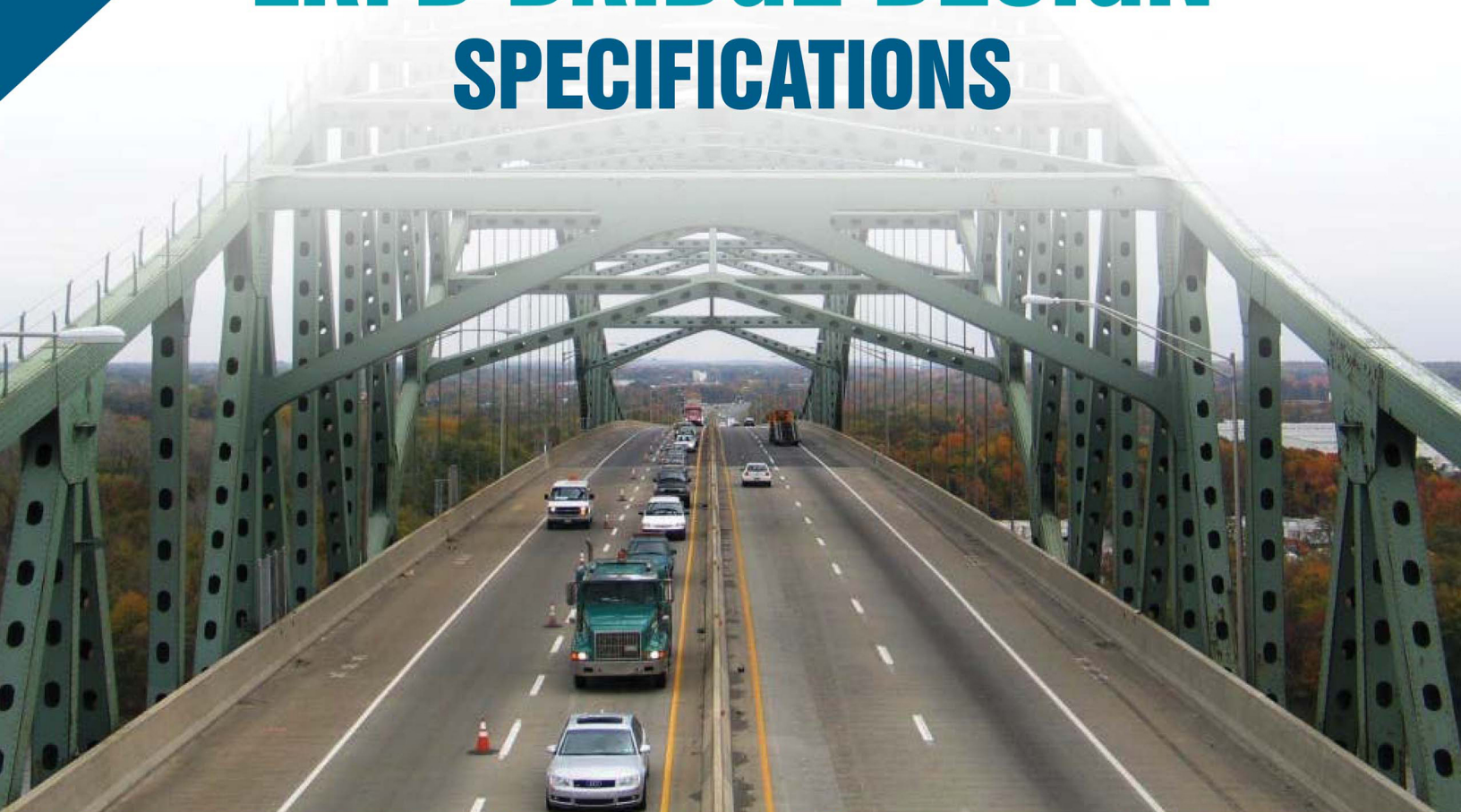


**2016**  
**Interim Revisions**

# AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS



AMERICAN ASSOCIATION  
OF STATE HIGHWAY AND  
TRANSPORTATION OFFICIALS

**AASHTO**

ISBN: 978-1-56051-637-8  
Publication Code: LRFDUS-7-I2

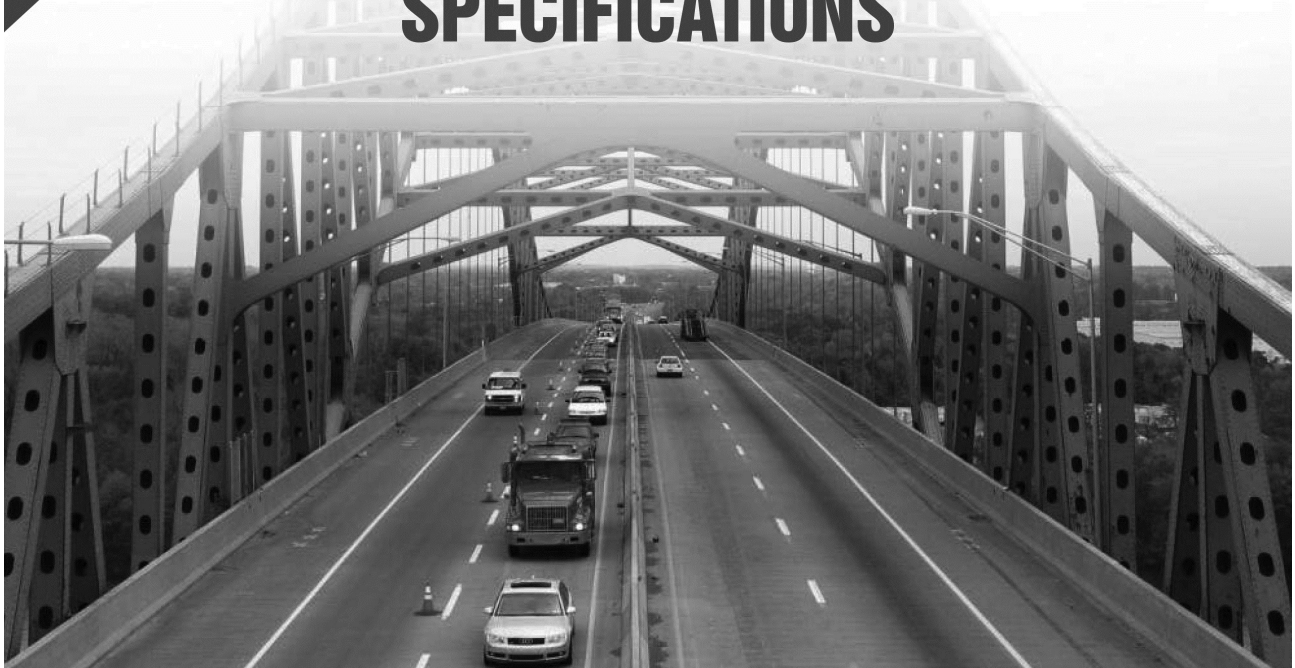
**Seventh Edition, 2014**  
**U.S. Customary Units**



**Documents**  
Document Sharing Hub

**2016**  
Interim Revisions

# AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS



AMERICAN ASSOCIATION  
OF STATE HIGHWAY AND  
TRANSPORTATION OFFICIALS

**AASHTO**

**Seventh Edition, 2014**  
**U.S. Customary Units**

ISBN: 978-1-56051-637-8  
Publication Code: LRFDUS-7-12



**Documents**  
Document Sharing Hub

© 2015 by the American Association of State Highway and Transportation Officials.  
All rights reserved. Duplication is a violation of applicable law.

Full document details: [AASHTO LRFDUS-7-2014 interim revisions 2016](#)



444 North Capitol Street, NW, Suite 249  
Washington, DC 20001  
202-624-5800 phone/202-624-5806 fax  
[www.transportation.org](http://www.transportation.org)

© 2015 by the American Association of State Highway and Transportation Officials. All rights reserved. Duplication is a violation of applicable law.

ISBN: 978-1-56051-637-8

© 2015 by the American Association of State Highway and Transportation Officials.  
All rights reserved. Duplication is a violation of applicable law.

Full document details: [AASHTO LRFDUS-7-2014 interim revisions 2016](#)

 **Documents**  
Publication Code: LRFDUS-7-12  
Document Sharing Hub

# INSTRUCTIONS AND INFORMATION

## General

AASHTO has made interim revisions to the *AASHTO LRFD Bridge Design Specifications*, Seventh Edition (2014). This packet contains the revised pages. They are not designed to replace the corresponding pages in the book but rather to be kept with the book for quick reference.

## Affected Articles

Underlined text indicates additions that were approved in 2015 by the AASHTO Highways Subcommittee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Subcommittee. A list of affected articles is included below.

All interim pages are printed on pink paper to make the changes stand out when inserted in the seventh edition binder. They also have a page header displaying the section number affected and the interim publication year. Please note that these pages may also contain nontechnical (e.g. editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

Please note that in response to user concerns, page breaks and blank pages are now being added within sections between noncontiguous articles. This makes it an option to insert the changes closer to the affected articles.

### Table I—2016 Changed or Deleted Articles

#### SECTION 2: GENERAL DESIGN AND LOCATION FEATURES

2.5.2.6.2                      2.8

#### SECTION 3: LOADS AND LOAD FACTORS

3.4.1                          3.5.1                          3.11.5.8.1  
3.4.2.1                      3.8                              3.16

#### SECTION 5: CONCRETE STRUCTURES

|           |           |             |            |             |
|-----------|-----------|-------------|------------|-------------|
| 5.1       | 5.6.3     | 5.8.6.3     | 5.11.2.1.1 | 5.13.2.5.4  |
| 5.2       | 5.7.3.4   | 5.8.6.5     | 5.11.2.1.2 | 5.13.2.5.5  |
| 5.3       | 5.8.2     | 5.8.6.6     | 5.11.2.4.1 | 5.13.3.6.3  |
| 5.4.2.1   | 5.8.2.2   | 5.9.4.1.1   | 5.11.2.4.2 | 5.14.2.3.2  |
| 5.4.2.6   | 5.8.2.5   | 5.9.4.1.2   | 5.11.2.5.1 | 5.14.2.3.3  |
| 5.4.2.8   | 5.8.3.3   | 5.9.4.2.2   | 5.11.2.5.2 | 5.14.2.4.4  |
| 5.5.1     | 5.8.3.4.3 | 5.10.4.3.1b | 5.11.2.6.2 | 5.14.5.3    |
| 5.5.4.2.1 | 5.8.4.1   | 5.10.11.4.2 | 5.11.2.6.4 | 5.15        |
| 5.5.4.2.2 | 5.8.4.3   | 5.10.11.4.3 | 5.13.2.4.2 | Appendix C5 |

#### SECTION 6: STEEL STRUCTURES

|           |             |            |             |
|-----------|-------------|------------|-------------|
| 6.3       | 6.9.4.3.1   | 6.11.5     | 6.13.2.6.5  |
| 6.4.1     | 6.10.1.8    | 6.11.8.2.2 | 6.13.2.6.6  |
| 6.6.1.2.3 | 6.10.8.2.3  | 6.11.10    | 6.13.6.1.4a |
| 6.7.4.2   | 6.10.11.1.1 | 6.12.2.2.2 | 6.17        |

SECTION 10: FOUNDATIONS

10.3                      10.4.6.5                      10.6.3.4

SECTION 11: WALLS, ABUTMENT, AND PIERS

11.5.7

SECTION 14: JOINTS AND BEARINGS

14.5.6.9.2                      14.6.3.1

SECTION 15: DESIGN OF SOUND BARRIERS

15.8.1                      15.8.2

## SECTION 2: GENERAL DESIGN AND LOCATION FEATURES

### 2.5.2.6.2—Criteria for Deflection

Revise the 3<sup>rd</sup> bullet after the 3<sup>rd</sup> paragraph as follows:

- For composite design, the stiffness of the design cross-section used for the determination of deflection and frequency should include the entire width of the roadway and the structurally continuous portions of the railings, sidewalks, and median barriers;

### C2.5.2.6.2

Add the following to Article C2.5.2.6.2 opposite the 4<sup>th</sup> paragraph of specification:

Other criteria may include recognized deflection-frequency-perception requirements such as that specified in the Canadian Highway Bridge Design Code (CSA, 2006). Application of the CSA criteria is discussed in Kulicki et al. (2014), including statistical data for live load based on WIM data, a load factor for the HL-93 live load, and a target reliability index.



**2.8—REFERENCES**

Add the following references:

CSA. 2006. *Canadian Highway Bridge Design Code*, CAN/CSA-S6-06. Includes Supplement 1, Supplement 2, and Supplement 3. Canadian Standards Association International, Toronto, ON, Canada.

Kulicki, J. M., W. G. Wassef, D. R. Mertz, A. S. Nowak, N. C. Samtani, and H. Nassif. 2015. *Bridges for Service Life Beyond 100 Years: Service Limit State Design*, Report S2-R19B-RW-1. Transportation Research Board, National Research Council, Washington, DC.



## SECTION 3: LOADS AND LOAD FACTORS

### 3.4.1—Load Factors and Load Combinations

Revise the Article as follows:

The total factored force effect shall be taken as:

$$Q = \sum \eta_i \gamma_i Q_i \quad (3.4.1-1)$$

where:

- $\eta_i$  = load modifier specified in Article 1.3.2
- $Q_i$  = force effects from loads specified herein
- $\gamma_i$  = load factors specified in Tables 3.4.1-1, and 3.4.1-2, and 3.4.1-3.

Components and connections of a bridge shall satisfy Eq. 1.3.2.1-1 for the applicable combinations of factored extreme force effects as specified at each of the load combinations specified in Table 3.4.1-1 at the following limit states:

- 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 velocity exceeding 55 mph the design wind speed at the location of the bridge.
- Strength IV—Load combination relating to very high-emphasizing dead load to live-load force effects ratios in bridge superstructures.

### C3.4.1

Revise the Article as follows:

The background for the load factors specified herein, and the resistance factors specified in other Sections of these Specifications is developed in Nowak (1992).

The permit vehicle should not be assumed to be the only vehicle on the bridge unless so assured by traffic control. See Article 4.6.2.2.5 regarding other traffic on the bridge simultaneously.

Vehicles become unstable at higher wind velocities. Therefore, high winds prevent the presence of significant live load on the bridge.

Wind load provisions in earlier editions of the specifications were based on fastest-mile wind speed measurements. The current wind load provisions are based on 3-second wind gust speed with 7 percent probability of exceedance in 50 years (mean return period of 700 years).

The Strength IV load combination shown in these specifications was not fully statistically calibrated. It does not include live load; it controls over Strength I for components with dead load to live load ratio exceeding 7.0. These are typically long span bridges. The reliability indices tend to increase with the increase in the dead load to live load ratio, albeit at slow rate for bridges with high ratios.

A study was performed by Modjeski and Masters, Inc. (2013) using the same process to calibrate Strength IV as was used to statistically calibrate the Strength I load combination. Some load combinations that still emphasized dead load force effects, but produced a more

uniform reliability across the possible practical range of dead load to live load ratios, were proposed. However except for steel trusses, the relative effect on the controlling factored design loads was small and did not warrant changing the current load combination. Trusses and other structures with high  $DL/LL$  ratios can come closer to the targeted reliability of 3.5 by using the equation  $1.4DC + 1.5DW + 1.45LL$ . Trusses see the largest increase in the reliability index. However the true reliability of steel trusses, steel box girders, and concrete box girder structures may be higher than reported in this study due to:

- Not having been included in the live load distribution factor study that refined the design load for more common bridge types.
- The HL93 loading being conservative for long-span bridges as discussed in Article C3.6.1.3.

~~The standard calibration process for the strength limit state consists of trying out various combinations of load and resistance factors on a number of bridges and their components. Combinations that yield a safety index close to the target value of  $\beta = 3.5$  are retained for potential application. From these are selected constant load factors  $\gamma$  and corresponding resistance factors  $\phi$  for each type of structural component reflecting its use.~~

~~This calibration process had been carried out for a large number of bridges with spans not exceeding 200 ft. These calculations were for completed bridges. For the primary components of large bridges, the ratio of dead and live load force effects is rather high, and could result in a set of resistance factors different from those found acceptable for small and medium span bridges. It is believed to be more practical to investigate one additional load case than to require the use of two sets of resistance factors with the load factors provided in Strength Load Combination I, depending on other permanent loads present. Spot checks had been made on a few bridges with up to 600 ft spans, and it appears that Strength Load Combination IV will govern where the dead load to live load force effect ratio exceeds about 7.0. This load combination is not applicable to investigation of construction stages, substructures, earth retaining systems, and bearing design. Other load combinations adequately address substructures and bearings.~~

When applied with the load factor specified in Table 3.4.1-1 (i.e. 1.0), the 80 mph 3-second gust wind speed is approximately equivalent to the 100 mph fastest-mile wind used in earlier specifications applied with a load factor of 0.4. The latter was meant to be equivalent to a 55 mph fastest-mile wind applied with a load factor of 1.4.

Past editions of the Standard Specifications used  $\gamma_{EQ} = 0.0$ . This issue is not resolved. The possibility of partial live load, i.e.,  $\gamma_{EQ} < 1.0$ , with earthquakes should be considered. Application of Turkstra's rule for combining uncorrelated loads indicates that  $\gamma_{EQ} = 0.50$  is

- Strength V—Load combination relating to normal vehicular use of the bridge with wind of ~~55~~ 80 mph velocity.
- Extreme Event I—Load combination including earthquake. The load factor for live load  $\gamma_{EQ}$ , shall be determined on a project-specific basis.

- Extreme Event II—Load combination relating to ice load, collision by vessels and vehicles, check floods, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, *CT*. The cases of check floods shall not be combined with *BL*, *CV*, *CT*, or *IC*.

reasonable for a wide range of values of average daily truck traffic (ADTT).

The following applies to both Extreme Event I and II:

- The design objective is life safety, i.e., noncollapse of the structure. Inelastic behavior such as spalling of concrete and bending of steel members is expected. In most cases, the risk does not warrant the expense of designing for elastic behavior so long as vertical-load-carrying capacity is maintained for service-level loads.
- Prior to 2015, these Specifications used a value for  $\gamma_p$  greater than 1.0. This practice went against the intended philosophy behind the Extreme Event Limit State. A more conservative design is attained by increasing the hazard and using ductile detailing, rather than increasing  $\gamma_p$ , i.e., force effects due to permanent loads.
- The recurrence interval of extreme events is thought to exceed the design life.
- Although these limit states include water loads, *WA*, the effects due to *WA* are considerably less significant than the effects on the structure stability due to scour. Therefore, unless specific site conditions dictate otherwise, local pier scour and contraction scour depths should not be combined with *BL*, *EQ*, *CT*, *CV*, or *IC*. However, the effects due to degradation of the channel should be considered. Alternatively, one-half of the total scour may be considered in combination with *BL*, *EQ*, *CT*, *CV*, or *IC*.
- The joint probability of these events is extremely low, and, therefore, the events are specified to be applied separately. Under these extreme conditions, the structure may undergo considerable inelastic deformation by which locked-in force effects due to *TU*, *TG*, *CR*, *SH*, and *SE* are expected to be relieved.

The 0.50 live load factor signifies a low probability of the concurrence of the maximum vehicular live load (other than *CT*) and the extreme events.

Compression in prestressed concrete components and tension in prestressed bent caps are investigated using this load combination. Service III is used to investigate tensile stresses in prestressed concrete components.

When applied with the load factor specified in Table 3.4.1-1 (i.e. 1.0), the 70 mph 3-second gust wind speed is equivalent to the 100 mph fastest-mile wind used in earlier specifications applied with a load factor of 0.3. The latter was meant to be equivalent to a 55 mph fastest-mile wind applied with a load factor of 1.0.

- Service I—Load combination relating to the normal operational use of the bridge with a ~~55~~ 70 mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for the investigation of slope stability.

- Service II—Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load. For structures with unique truck loading conditions, such as access roads to ports or industrial sites which might lead to a disproportionate number of permit loads, a site-specific increase in the load factor should be considered.
- Service III—Load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders.

This load combination corresponds to the overload provision for steel structures in past editions of the AASHTO Specifications, and it is applicable only to steel structures. From the point of view of load level, this combination is approximately halfway between that used for Service I and Strength I Limit States. An evaluation of WIM data from 31 sites around the country (Kulicki et al., 2014) indicated that the probability of exceeding the load level specified in Table 3.4.1-1 for this limit state could be less than once every six months.

Prior to 2014, the longitudinal analysis relating to tension in prestressed concrete superstructures was investigated using a load factor for live load of 0.8. The live load specified in these specifications This load factor reflects, among other things, current exclusion weight limits mandated by various jurisdictions at the time of the development of the specifications in 1993. Vehicles permitted under these limits have been in service for many years prior to 1993. It was concluded at that time that, for longitudinal loading, there is no nationwide physical evidence that these vehicles have caused cracking in existing prestressed concrete components. The 0.8 load factor was applied regardless of the method used for determining the loss of prestressing. The statistical significance of the 0.80 factor on live load is that the event is expected to occur about once a year for bridges with two traffic lanes, less often for bridges with a single traffic lane.

The calibration of the service limit states for concrete components (Wassef et al., 2014) concluded that typical components designed using the Refined Estimates of Time-Dependent Losses method incorporated in the specifications in 2005, which includes the use of transformed sections and elastic gains, have a lower reliability index against flexural cracking in prestressed components than components designed using the prestress loss calculation method specified prior to 2005 based on gross sections and do not include elastic gains. For components designed using the currently-specified methods for instantaneous prestressing losses and the currently-specified Refined Estimates of Time-Dependent Losses method, an increase in the load factor for live load from 0.8 to 1.0 was required to maintain the level of reliability against cracking of prestressed concrete components inherent in the system.

Service I should be used for checking tension related to transverse analysis of concrete segmental girders. The principal tensile stress check is introduced in order to verify the adequacy of webs of segmental concrete girder bridges for longitudinal shear and torsion.

The principal tensile stress check is introduced in order to verify the adequacy of webs of segmental concrete girder bridges for longitudinal shear and torsion.

- Service IV—Load combination relating only to tension in prestressed concrete columns with the objective of crack control.

Wind load for Service IV load combination in earlier specifications was based on fastest-mile wind of 100 mph applied with a load factor of 0.7. The 0.70 factor on wind. This load represents an 84 mph fastest-mile wind applied with a load factor of 1.0 wind. This should This wind load was meant to result in zero tension in prestressed concrete columns for ten-year mean reoccurrence winds. The wind speed specified in Table 3.8.1.1.2-1 for Service IV limit state is a product of the following:

- The wind speed used for the Strength III load combination taken as the 3-second gust wind speed with 7 percent probability of exceedance in 50 years.
- The ratio of the 3-second gust wind speed with 7 percent probability of exceedance in 10 years and 50 years, approximately 87 percent.
- A reduction factor equal to the square root of  $1/1.4$  or 0.845. This reduction factor is meant to reduce the resulting wind pressure by  $1/1.4$ , the traditional ratio between wind pressures used for the strength limit states and service limit states for the same wind speed.

The prestressed concrete columns must still meet strength requirements as set forth in Load Combination Strength III in Article 3.4.1.

It is not recommended that thermal gradient be combined with high wind forces. Superstructure expansion forces are included.

- Fatigue I—Fatigue and fracture load combination related to infinite load-induced fatigue life.

The load factor for the Fatigue I load combination, applied to a single design truck having the axle spacing specified in Article 3.6.1.4.1, reflects load levels found to be representative of the maximum stress range of the truck population for infinite fatigue life design. The factor was chosen on the assumption that the maximum stress range in the random variable spectrum is twice the effective stress range caused by Fatigue II load combination.

- Fatigue II—Fatigue and fracture load combination related to finite load-induced fatigue life.

The load factor for the Fatigue II load combination, applied to a single design truck, reflects a load level found to be representative of the effective stress range of the truck population with respect to a small number of stress range cycles and to their cumulative effects in steel elements, components, and connections for finite fatigue life design.

The load factors for various loads comprising a design load combination shall be taken as specified in Table 3.4.1-1. All relevant subsets of the load combinations shall be investigated. For each load combination, every load that is indicated to be taken into account and that is germane to the component being designed, including all significant effects due to distortion, shall be multiplied by the appropriate load factor and multiple presence factor specified in Article 3.6.1.1.2, if applicable. The products shall be summed as specified in Eq. 1.3.2.1-1

This Article reinforces the traditional method of selecting load combinations to obtain realistic extreme effects and is intended to clarify the issue of the variability of permanent loads and their effects. As has always been the case, the Owner or Designer may determine that not all of the loads in a given load combination apply to the situation under investigation.

It is recognized herein that the actual magnitude of permanent loads may also be less than the nominal value. This becomes important where the permanent load reduces the effects of transient loads.

and multiplied by the load modifiers specified in Article 1.3.2.

The factors shall be selected to produce the total extreme factored force effect. For each load combination, both positive and negative extremes shall be investigated.

In load combinations where one force effect decreases another effect, the minimum value shall be applied to the load reducing the force effect. For permanent force effects, the load factor that produces the more critical combination shall be selected from Table 3.4.1-2. Where the permanent load increases the stability or load-carrying capacity of a component or bridge, the minimum value of the load factor for that permanent load shall also be investigated.

The larger of the two values provided for load factor of  $TU$  shall be used for deformations and the smaller values for all other effects. For simplified analysis of concrete substructures in the strength limit state, a value of 0.50 for  $\gamma_{TU}$  may be used when calculating force effects, but shall be taken in conjunction with the gross moment of inertia in the columns or piers. When a refined analysis is completed for concrete substructures in the strength limit state, a value of 1.0 for  $\gamma_{TU}$  shall be used in conjunction with a partially cracked moment of inertia determined by analysis. For concrete substructures in the strength limit state, the value of 0.50 for  $\gamma_{PS}$ ,  $\gamma_{CR}$ , and  $\gamma_{SH}$  may similarly be used when calculating force effects in nonsegmental structures, but shall be taken in conjunction with the gross moment of inertia in the columns or piers. For steel substructures, a value of 1.0 for  $\gamma_{TU}$ ,  $\gamma_{PS}$ ,  $\gamma_{CR}$ , and  $\gamma_{SH}$  shall be used.

It has been observed that permanent loads are more likely to be greater than the nominal value than to be less than this value.

The earth load factor for thermoplastic culverts is set to 1.3; however, to preserve the overall safety at the same levels as historical specifications, an earth-load-installation factor is introduced later in these Specifications as part of the implementation of NCHRP Report 631. This factor may be adjusted based on field control of construction practices.

In the application of permanent loads, force effects for each of the specified six load types should be computed separately. It is unnecessary to assume that one type of load varies by span, length, or component within a bridge. For example, when investigating uplift at a bearing in a continuous beam, it would not be appropriate to use the maximum load factor for permanent loads in spans that produce a negative reaction and the minimum load factor in spans that produce a positive reaction. Consider the investigation of uplift. Uplift, which was treated as a separate load case in past editions of the AASHTO Standard Specifications, now becomes a strength load combination. Where a permanent load produces uplift, that load would be multiplied by the maximum load factor, regardless of the span in which it is located. If another permanent load reduces the uplift, it would be multiplied by the minimum load factor, regardless of the span in which it is located. For example, at Strength I Limit State where the permanent load reaction is positive and live load can cause a negative reaction, the load combination would be  $0.9DC + 0.65DW + 1.75(LL + IM)$ . If both reactions were negative, the load combination would be  $1.25DC + 1.50DW + 1.75(LL + IM)$ . For each force effect, both extreme combinations may need to be investigated by applying either the high or the low load factor as appropriate. The algebraic sums of these products are the total force effects for which the bridge and its components should be designed.

$PS$ ,  $CR$ ,  $SH$ ,  $TU$ , and  $TG$  are superimposed deformations as defined in Article 3.12. Load factors for  $TU$ , and  $TG$  are as shown in Table 3.4.1-1. Load factors for  $PS$ ,  $CR$ , and  $SH$  are as shown in Table 3.4.1-3. For prestressed members in typical bridge types, secondary prestressing, creep and shrinkage are generally designed for in the service limit state. In concrete segmental structures,  $CR$  and  $SH$  are factored by  $\gamma_P$  for  $DC$  because analysis for time-dependent effects in segmental bridges is nonlinear. Abutments, piers, columns, and bent caps are to be considered as substructure components.

The calculation of displacements for  $TU$  utilizes a factor greater than 1.0 to avoid undersizing joints, expansion devices, and bearings.

The evaluation of overall stability of retained fills, as well as earth slopes with or without a shallow or deep foundation unit should be investigated at the service limit state based on the Service I Load Combination and an appropriate resistance factor as specified in Article 11.5.6 and Article 11.6.2.3.

For structural plate box structures complying with the provisions of Article 12.9, the live load factor for the vehicular live loads  $LL$  and  $IM$  shall be taken as 2.0.

The load factor for temperature gradient,  $\gamma_{TG}$ , should be considered on a project-specific basis. In lieu of project-specific information to the contrary,  $\gamma_{TG}$  may be taken as:

- 0.0 at the strength and extreme event limit states,
- 1.0 at the service limit state when live load is not considered, and
- 0.50 at the service limit state when live load is considered.

The load factor for settlement,  $\gamma_{SE}$ , should be considered on a project-specific basis. In lieu of project-specific information to the contrary,  $\gamma_{SE}$  may be taken as 1.0. Load combinations which include settlement shall also be applied without settlement.

For segmentally constructed bridges, the following combination shall be investigated at the service limit state:

$$DC + DW + EH + EV + ES + WA + CR + SH + TG + EL + PS$$

(3.4.1-2)

Applying these criteria for the evaluation of the sliding resistance of walls:

- The vertical earth load on the rear of a cantilevered retaining wall would be multiplied by  $\gamma_{pmin}$  (1.00) and the weight of the structure would be multiplied by  $\gamma_{pmin}$  (0.90) because these forces result in an increase in the contact stress (and shear strength) at the base of the wall and foundation.
- The horizontal earth load on a cantilevered retaining wall would be multiplied by  $\gamma_{pmax}$  (1.50) for an active earth pressure distribution because the force results in a more critical sliding force at the base of the wall.

Similarly, the values of  $\gamma_{pmax}$  for structure weight (1.25), vertical earth load (1.35) and horizontal active earth pressure (1.50) would represent the critical load combination for an evaluation of foundation bearing resistance.

Water load and friction are included in all strength load combinations at their respective nominal values.

For creep and shrinkage, the specified nominal values should be used. For friction, settlement, and water loads, both minimum and maximum values need to be investigated to produce extreme load combinations.

The load factor for temperature gradient should be determined on the basis of the:

- Type of structure, and
- Limit state being investigated.

Open girder construction and multiple steel box girders have traditionally, but perhaps not necessarily correctly, been designed without consideration of temperature gradient, i.e.,  $\gamma_{TG} = 0.0$ .

Table 3.4.1-1—Load Combinations and Load Factors

| Load<br>Combination<br>Limit State | DC<br>DD<br>DW<br>EH<br>EV<br>ES<br>EL<br>PS<br>CR<br>SH | LL<br>IM<br>CE<br>BR<br>PL<br>LS | WA   | WS                 | WL   | FR   | TU        | TG            | SE            | Use One of These at a Time |      |      |      |      |
|------------------------------------|----------------------------------------------------------|----------------------------------|------|--------------------|------|------|-----------|---------------|---------------|----------------------------|------|------|------|------|
|                                    |                                                          |                                  |      |                    |      |      |           |               |               | EQ                         | BL   | IC   | CT   | CV   |
| Strength I<br>(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 | $\frac{1.0}{1.40}$ | —    | 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 | $\frac{1.0}{0.40}$ | 1.00 | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | —                          | —    | —    | —    | —    |
| Extreme<br>Event I                 | 1.0                                                      | $\gamma_{EQ}$                    | 1.00 | —                  | —    | 1.00 | —         | —             | —             | 1.00                       | —    | —    | —    | —    |
| Extreme<br>Event II                | $\gamma_p$                                               | 0.50                             | 1.00 | —                  | —    | 1.00 | —         | —             | —             | —                          | 1.00 | 1.00 | 1.00 | 1.00 |
| Service I                          | 1.00                                                     | 1.00                             | 1.00 | $\frac{1.0}{0.30}$ | 1.00 | 1.00 | 1.00/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | —                          | —    | —    | —    | —    |
| Service II                         | 1.00                                                     | 1.30                             | 1.00 | —                  | —    | 1.00 | 1.00/1.20 | —             | —             | —                          | —    | —    | —    | —    |
| Service III                        | 1.00                                                     | $\gamma_{LL}$                    | 1.00 | —                  | —    | 1.00 | 1.00/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | —                          | —    | —    | —    | —    |
| Service IV                         | 1.00                                                     | —                                | 1.00 | $\frac{1.0}{0.70}$ | —    | 1.00 | 1.00/1.20 | —             | 1.00          | —                          | —    | —    | —    | —    |
| Fatigue I—<br>LL, IM & CE<br>only  | —                                                        | 1.50                             | —    | —                  | —    | —    | —         | —             | —             | —                          | —    | —    | —    | —    |
| Fatigue II—<br>LL, IM & CE<br>only | —                                                        | 0.75                             | —    | —                  | —    | —    | —         | —             | —             | —                          | —    | —    | —    | —    |

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

| Type of Load, Foundation Type, and<br>Method Used to Calculate Downdrag                            |                                                                   | Load Factor |         |
|----------------------------------------------------------------------------------------------------|-------------------------------------------------------------------|-------------|---------|
|                                                                                                    |                                                                   | Maximum     | Minimum |
| <i>DC</i> : Component and Attachments                                                              |                                                                   | 1.25        | 0.90    |
| <i>DC</i> : Strength IV only                                                                       |                                                                   | 1.50        | 0.90    |
| <i>DD</i> : Downdrag                                                                               | Piles, $\alpha$ Tomlinson Method                                  | 1.4         | 0.25    |
|                                                                                                    | Piles, $\lambda$ Method                                           | 1.05        | 0.30    |
|                                                                                                    | Drilled shafts, O'Neill and Reese <del>(1999)</del> (2010) Method | 1.25        | 0.35    |
| <i>DW</i> : Wearing Surfaces and Utilities                                                         |                                                                   | 1.50        | 0.65    |
| <i>EH</i> : Horizontal Earth Pressure                                                              |                                                                   |             |         |
| • Active                                                                                           |                                                                   | 1.50        | 0.90    |
| • At-Rest                                                                                          |                                                                   | 1.35        | 0.90    |
| • <i>AEP</i> for anchored walls                                                                    |                                                                   | 1.35        | N/A     |
| <i>EL</i> : Locked-in Construction Stresses                                                        |                                                                   | 1.00        | 1.00    |
| <i>EV</i> : Vertical Earth Pressure                                                                |                                                                   |             |         |
| • Overall Stability                                                                                |                                                                   | 1.00        | N/A     |
| • Retaining Walls and Abutments                                                                    |                                                                   | 1.35        | 1.00    |
| • Rigid Buried Structure                                                                           |                                                                   | 1.30        | 0.90    |
| • Rigid Frames                                                                                     |                                                                   | 1.35        | 0.90    |
| • Flexible Buried Structures                                                                       |                                                                   |             |         |
| ○ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and<br>Fiberglass Culverts |                                                                   | 1.5         | 0.9     |
| ○ Thermoplastic Culverts                                                                           |                                                                   | 1.3         | 0.9     |
| ○ All others                                                                                       |                                                                   | 1.95        | 0.9     |
| <i>ES</i> : Earth Surcharge                                                                        |                                                                   | 1.50        | 0.75    |

Add new Table 3.4.1-4 as follows:

**Table 3.4.1-4—Load Factors for Live Load for Service III Load Combination,  $\gamma_{LL}$** 

| <u>Component</u>                                                                                                                                                                              | <u><math>\gamma_{LL}</math></u> |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------|
| <u>Prestressed concrete components designed using the refined estimates of time-dependent losses as specified in Article 5.9.5.4 in conjunction with taking advantage of the elastic gain</u> | <u>1.0</u>                      |
| <u>All other prestressed concrete components</u>                                                                                                                                              | <u>0.8</u>                      |



**3.4.2.1—Evaluation at the Strength Limit State**

Revise the Article as follows:

All appropriate strength limit state load combinations in Table 3.4.1-1, modified as specified herein, shall be investigated.

When investigating Strength Load Combinations I and III for maximum force effects during construction, load factors for the weight of the structure and appurtenances, *DC* and *DW*, shall not be less than 1.25.

Unless otherwise specified by the Owner, construction loads including dynamic effects (if applicable) shall be added in Strength Load Combination I with a load factor not less than 1.5 when investigating for maximum force effects.

~~Unless otherwise specified by the Owner, the load factor for wind during construction in Strength Load Combination III shall not be less than 1.25 when investigating for maximum force effects.~~ be as specified by the Owner. Any applicable construction loads shall be included with a load factor not less than 1.25.

Unless otherwise specified by the Owner, primary steel superstructure components shall be investigated for maximum force effects during construction for an additional load combination consisting of the applicable *DC* loads and any construction loads that are applied to the fully erected steelwork. For this additional load combination, the load factor for *DC* and construction loads including dynamic effects (if applicable) shall not be less than 1.4.



**3.5.1—Dead Loads:  $DC$ ,  $DW$ , and  $EV$** 

Revise the 6<sup>th</sup> row of Table 3.5.1-1 as follows:

| Material |                                               | Unit Weight<br>(pcf)                   |
|----------|-----------------------------------------------|----------------------------------------|
| Concrete | Lightweight                                   | <del>0.110</del> <u>0.110 to 0.135</u> |
|          | <del>Sand Lightweight</del>                   | <del>0.120</del>                       |
|          | Normal Weight with $f'_c \leq 5.0$ ksi        | 0.145                                  |
|          | Normal Weight with $5.0 < f'_c \leq 15.0$ ksi | $0.140 + 0.001 f'_c$                   |



Replace Article 3.8 with the following:

### **3.8—WIND LOAD: WL AND WS**

#### **3.8.1—Horizontal Wind Load**

##### **3.8.1.1—Exposure Conditions**

###### **3.8.1.1.1—General**

Wind pressure shall be assumed to be uniformly distributed on the area exposed to the wind. The exposed area shall be the sum of areas of all components, including floor system, railing, and sound barriers, as seen in elevation taken perpendicular to the wind direction. The wind load shall be the product of the wind pressure and exposed area. The wind shall be assumed horizontal, except as otherwise specified in Article 3.8.2, and can come from any horizontal direction. Areas that do not contribute to the extreme force effect under consideration may be neglected in the analysis.

For typical bridges, wind loads on the substructure from the superstructure may be determined for the wind in the direction transverse to the bridge in elevation then adjusted for various angles of attack using the provisions of Article 3.8.1.2.3a.

###### **3.8.1.1.2—Wind Speed**

The design 3-second gust wind speed,  $V$ , used in the determination of design wind loads on bridges shall be determined from Figure 3.8.1.1.2-1. For areas designated as a special wind region in Figure 3.8.1.1.2-1, the owner shall approve the 3-second gust wind speed.

The wind speed shall be increased where records, experience, or site-specific wind studies indicate that wind speeds higher than those reflected in Figure 3.8.1.1.2-1, based upon 7 percent probability of exceedance in 50 years, are possible at the bridge location.

Unless a site-specific wind study is performed, wind speeds used for different load combinations shall be taken from Table 3.8.1.1.2-1.

###### **C3.8.1.1.1**

For most bridges, the same wind pressure will be used for all components of the structure and the wind load is applied as a uniformly distributed load on the entire exposed area of the structure. However, some situations may warrant using different wind pressures on different components. The most common cases are for long bridges or when the substructure is unusually tall which may warrant using different structure heights in determining the wind pressure on different portions of the superstructure or substructure.

###### **C3.8.1.1.2**

Previous editions of these Specifications were based on fastest-mile wind speed. Since that criteria was based on distance, the effect was to average the wind speed over different lengths of time. The fastest-mile wind speed is no longer utilized by modern wind codes and the provisions herein are based on 3-second gust wind speed which means that the wind speed is averaged over 3 seconds.

Figure 3.8.1.1.2-1 shows the reference 3-second gust wind speed, at an elevation of 33 ft, for wind exposure Category C, as defined in Article 3.8.1.1.5, with a mean recurrence interval (MRI) of 700 years. The figure is taken from ASCE 7-10 (2010).

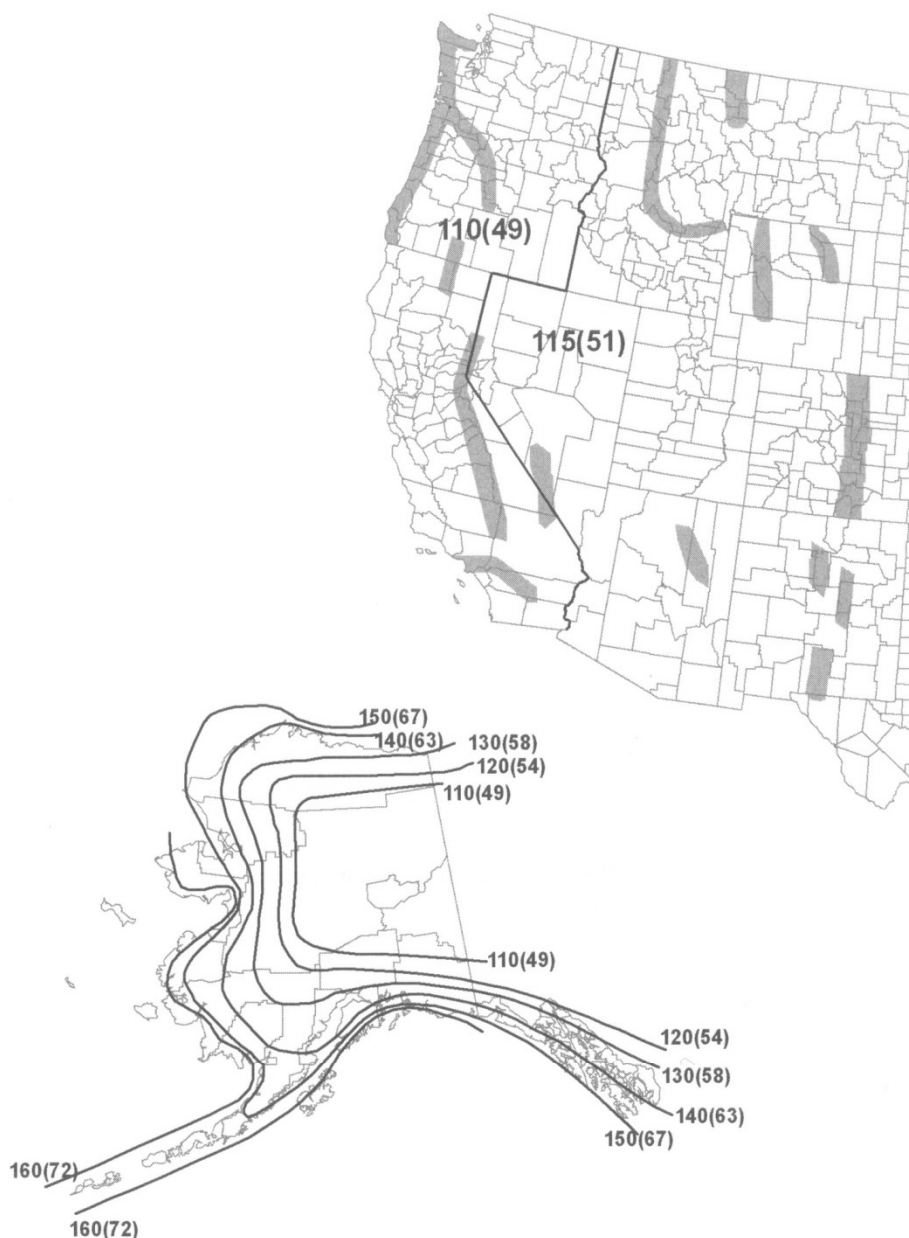
The wind pressure determined using the design 3-second gust wind speed with a load factor of 1.0 is approximately equivalent to the wind pressure determined using fastest-mile wind speed with a MRI of 100 years and a load factor of 1.4. This is the reason the load factor for wind load on the structure,  $WS$ , in the Strength III load combination has been reduced from the 1.4 used in earlier editions of the specifications to the 1.0 currently shown. The change in the load factor was instituted when the design wind speed was changed from the fastest-mile wind used earlier to the design 3-second gust wind speeds shown in Figure 3.8.1.1.2-1.

**Table 3.8.1.1.2-1—Design 3-Second Gust Wind Speed for Different Load Combinations,  $V$**

| <u>Load Combination</u> | <u>3-Second Gust Wind Speed (mph), <math>V</math></u>          |
|-------------------------|----------------------------------------------------------------|
| <u>Strength III</u>     | <u>Wind speed taken from Figure 3.8.1.1.2-1.</u>               |
| <u>Strength V</u>       | <u>80</u>                                                      |
| <u>Service I</u>        | <u>70</u>                                                      |
| <u>Service IV</u>       | <u>0.75 of the speed used for the Strength III limit state</u> |

The basis for the wind speeds specified for Strength V, Service I, and Service IV limit states is presented in Article C3.4.1.

For buildings and other structures, local building officials typically determine the 3-second gust wind speed to be used in the special wind regions under their jurisdiction. Bridge Owners will need to develop their own policy for the 3-second gust wind speed to be used in these regions.



## Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

**Figure 3.8.1.1.2-1—Design Wind Speed,  $V$ , in mph (m/s)**

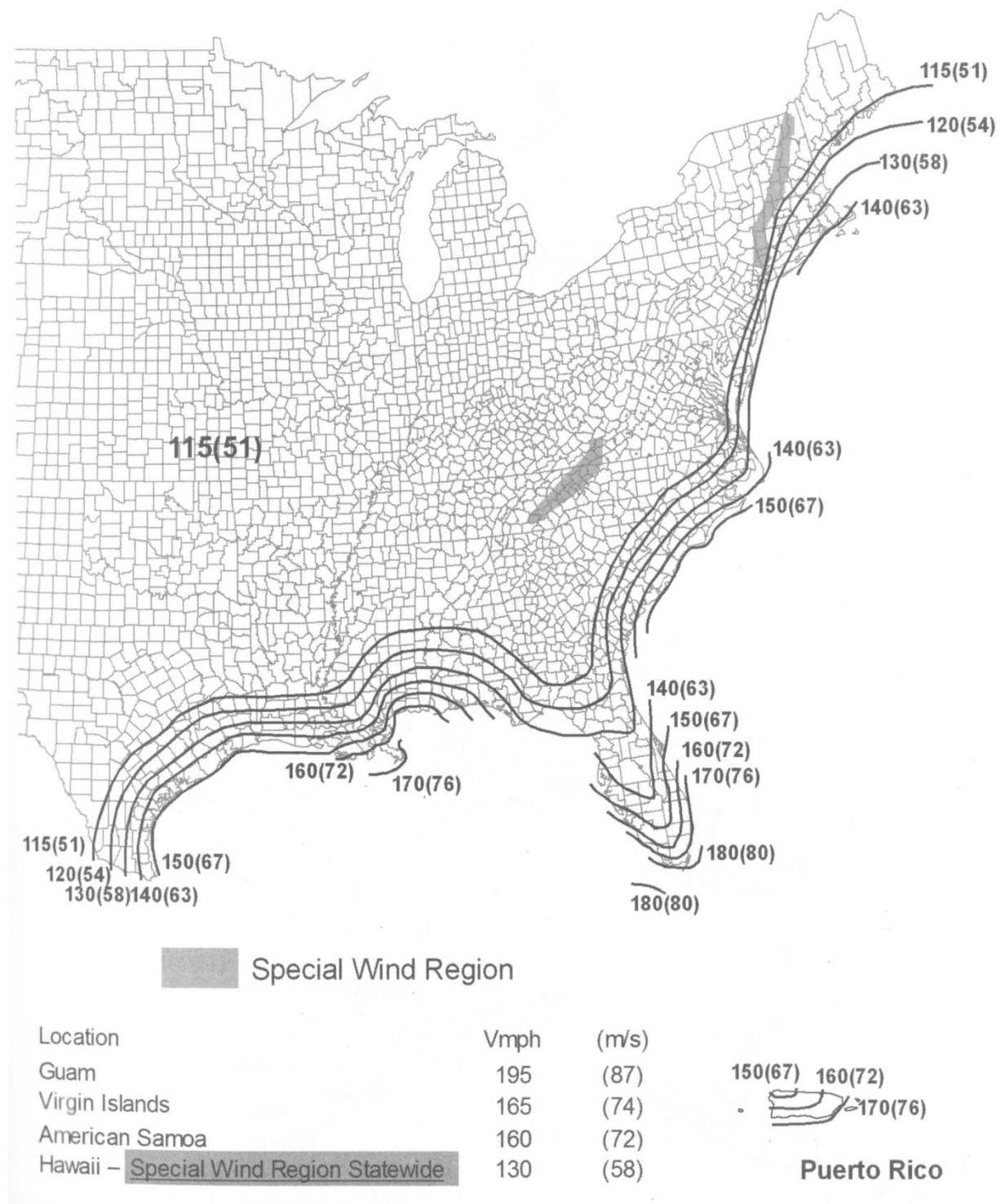


Figure 3.8.1.1.2-1 (cont'd.)—Design Wind Speed,  $V$ , in mph (m/s)

3.8.1.1.3—Wind Direction for Determining Wind Exposure Category

For each wind direction to be investigated, the wind exposure category of the bridge shall be determined for the two upwind sectors extending 45 degrees to either side of wind direction. The wind exposure category in the two sectors shall be determined in accordance with Articles 3.8.1.1.4 and 3.8.1.1.5. The wind exposure category which results in higher wind loads shall be used in determining the wind load for wind blowing from the direction being investigated.

For typical bridges, the wind exposure category as specified in Article 3.8.1.1.5 shall be perpendicular to the bridge. For long span bridges and for high level bridges, different wind exposure categories shall be investigated to determine the most critical wind direction.

C3.8.1.1.3

For a given bridge, the ground surface roughness category may be different for different wind directions based on the obstructions that exist along the direction of the wind. However, for typical bridges, the difference in wind pressure will not be significant and determining the wind exposure category perpendicular to the bridge is sufficient.

Determining various wind exposure categories may be warranted for long spans and high level bridges. In such cases, for each wind direction to be investigated, the wind exposure category of the bridge may be determined for the two upwind sectors extending 45 degrees to either side of the wind direction. The wind exposure category in the two sectors should be determined in accordance with Articles 3.8.1.1.4 and 3.8.1.1.5. The wind exposure category which results in higher wind pressure should be used in determining the wind load for wind blowing from the direction being investigated. The purpose of examining the two 45-degree sectors is to determine the maximum wind pressure associated with the wind direction being investigated.

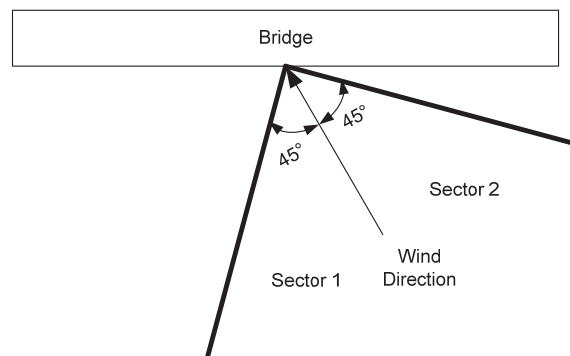


**Figure C3.8.1.1.3-1—Wind Sectors for Wind from Any Direction**

3.8.1.1.4—Ground Surface Roughness Categories

A ground surface roughness within each of the 45 degree sectors defined in Article 3.8.1.1.3 shall be determined as follows:

- Ground Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger;
- Ground Surface Roughness C: Open terrain with scattered obstructions having heights generally less than 33 ft, including flat open country and grasslands; and

C3.8.1.1.4

The ground surface roughness categories are used in determining the wind exposure category of the structure as defined in Article 3.8.1.1.5.

The ground surface roughness categories shown herein match those in ASCE 7-10 (2010). Some of the earlier editions of ASCE 7 included a separate category for urban exposure. However, for buildings, particularly tall ones, it is thought that there is not enough urban surface roughness, even in large urban areas, to develop true urban exposure conditions. Hence, the urban exposure category was eliminated from ASCE 7. However, for a low bridge in an urban area, an urban exposure could occur in nature. Using Ground Surface Roughness Category B for both urban and suburban areas results in slightly conservative wind loads for structures in urban areas.

- Ground Surface Roughness D: Flat, unobstructed areas and water surfaces; this category includes smooth mud flats, salt flats, and unbroken ice.

#### 3.8.1.1.5—Wind Exposure Categories

#### C3.8.1.1.5

The exposure category of the structure shall be determined as follows:

- Wind Exposure Category B: Wind Exposure Category B shall apply where the Ground Surface Roughness Category B, as defined in Article 3.8.1.1.4, prevails in the upwind direction for a distance greater than 1,500 ft for structures with a mean height of less than or equal to 33 ft, and for a distance greater than 2,600 ft or 20 times the height of the structure, whichever is greater, for structures with a mean height greater than 33 ft.
- Wind Exposure Category C: Wind Exposure Category C shall apply for all cases where Wind Exposure Categories B or D do not apply.
- Wind Exposure Category D: Wind Exposure Category D shall apply where the Ground Surface Roughness Category D, as defined in Article 3.8.1.1.4, prevails in the upwind direction for a distance greater than 5,000 ft or 20 times the height of the structure, whichever is greater. Wind Exposure Category D shall also apply where the structure is within a distance of 600 ft or 20 times the height of the structure, whichever is greater, from a Ground Surface Roughness Category D condition, even if Ground Surface Roughness Category B or C exist immediately upwind of the structure.

Where Ground Surface Roughness Category D prevails in the upwind direction—except when Ground Surface Roughness Category B or C exist for a relatively short distance immediately upwind from the structure—the effect of the presence of Ground Surface Roughness Category B or C may not be significant. Ground Surface Roughness Category D is conservatively specified for these situations.

### 3.8.1.2—Wind Load on Structures: *WS*

#### 3.8.1.2.1—General

#### C3.8.1.2.1

The wind pressure shall be determined as:

The basis for the development of wind load provisions exists in Wassef (2014).

$$P_z = 2.56 \times 10^{-6} V^2 K_z G C_D \quad (3.8.1.2.1-1)$$

where:

$P_z$  = design wind pressure (ksf)

$V$  = design 3-second gust wind speed specified in Table 3.8.1.1.2-1 (mph)

$K_z$  = pressure exposure and elevation coefficient to be taken equal to  $K_z (B)$ ,  $K_z (C)$ , or  $K_z (D)$  determined using Eqs. 3.8.1.2.1-2, 3.8.1.2.1-3, or 3.8.1.2.1-4, respectively, for Strength III and Service IV load combinations and to be taken as 1.0 for other load combinations

For structure heights less than 33 ft, the proximity to the ground surface causes turbulence for which the effect on wind pressure cannot be accurately determined. Therefore, no reduction in the value of  $K_z$  is shown in Table C3.8.1.2.1-1 for structure heights less than 33 ft.

$G$  = gust effect factor determined using a structure-specific study or as specified in Table 3.8.1.2.1-1 for Strength III and Service IV load combinations and 1.0 for other load combinations

$C_D$  = drag coefficient determined using a structure-specific study or as specified in Table 3.8.1.2.1-2

Strength V and Service I load combinations are based on constant wind speeds that are not functions of the bridge type, bridge height, or the wind exposure category at the location of the bridge. Therefore, the pressure exposure and elevation coefficient,  $K_z$ , is taken as 1.0 for these load combinations.

Unlike ASCE 7-10 (2010), which is based on power law wind profiles, these Specifications have always been based on logarithmic wind profiles. Therefore, logarithmic wind profiles were assumed in the development of Eqs. 3.8.1.2.1-2, 3.8.1.2.1-3, and 3.8.1.2.1-4.

The value of  $K_z$  at different elevations for different wind exposure categories are shown in Table C3.8.1.2.1-1.

The gust effect factor,  $G$ , is a function of the size and dynamic characteristics of the structure including bridge natural frequency and damping. The values specified in Table 3.8.1.2.1-1 are average values for sound barriers and typical bridge structures. For long-span arches, and cable-stayed and suspension bridges, the use of wind tunnel testing to determine a project-specific gust effect factor is warranted.

The 0.85 gust effect factor specified for sound barriers in Table 3.8.1.2.1-1 is consistent with the gust effect factor in ASCE 7-10 (2010) for walls and implies that wind gusts are not likely to engulf the entire barrier. However, the loaded area required to produce the maximum wind load on a sound barrier panel and the panel's vertical supports, if used, is relatively small. A higher gust effect factor may be justifiable because wind gusts may engulf the entire panel.

**Table C3.8.1.2.1-1—Pressure Exposure and Elevation Coefficients,  $K_z$**

| Structure Height, $Z$ (ft) | Wind Exposure Category B | Wind Exposure Category C | Wind Exposure Category D |
|----------------------------|--------------------------|--------------------------|--------------------------|
| <33                        | 0.71                     | 1.00                     | 1.15                     |
| 40                         | 0.75                     | 1.05                     | 1.20                     |
| 50                         | 0.81                     | 1.10                     | 1.25                     |
| 60                         | 0.85                     | 1.14                     | 1.29                     |
| 70                         | 0.89                     | 1.18                     | 1.32                     |
| 80                         | 0.92                     | 1.21                     | 1.35                     |
| 90                         | 0.95                     | 1.24                     | 1.38                     |
| 100                        | 0.98                     | 1.27                     | 1.41                     |
| 120                        | 1.03                     | 1.32                     | 1.45                     |
| 140                        | 1.07                     | 1.36                     | 1.49                     |
| 160                        | 1.11                     | 1.40                     | 1.52                     |
| 180                        | 1.15                     | 1.43                     | 1.55                     |
| 200                        | 1.18                     | 1.46                     | 1.58                     |
| 250                        | 1.24                     | 1.52                     | 1.63                     |
| 300                        | 1.30                     | 1.57                     | 1.68                     |

When the wind speed,  $K_z$ , and  $G$  specified for Strength V and Service I load combinations are substituted in Eq. 3.8.1.2.1-1, the resulting wind pressure on bridge structures,  $P_z$ , becomes a multiple of the drag coefficient,  $C_D$ , for the structure being considered. The wind pressure in these cases may be calculated using Table C3.8.1.2.1-2.

**Table C3.8.1.2.1-2—Wind Pressure on the Bridge Structures for Strength V and Service I Load Combinations**

| Load Combination | Wind Pressure on the Structure, $P_z$ , for the Specified Wind Speed (ksf) |
|------------------|----------------------------------------------------------------------------|
| Strength V       | $0.0163 C_D$                                                               |
| Service I        | $0.0125 C_D$                                                               |

The pressure exposure and elevation coefficient,  $K_z$ , for Strength III and Service IV load combinations shall be determined as follows:

$$K_z(B) = \frac{\left[ 2.5 \ln \left( \frac{Z}{0.9834} \right) + 6.87 \right]^2}{345.6} \quad (3.8.1.2.1-2)$$

$$K_z(C) = \frac{\left[ 2.5 \ln \left( \frac{Z}{0.0984} \right) + 7.35 \right]^2}{478.4} \quad (3.8.1.2.1-3)$$

$$K_z(D) = \frac{\left[ 2.5 \ln \left( \frac{Z}{0.0164} \right) + 7.65 \right]^2}{616.1} \quad (3.8.1.2.1-4)$$

where:

$K_z(B)$ ,  $K_z(C)$ , and  $K_z(D)$  are  $K_z$  for wind exposure Category B, C, and D, respectively.

The structure height,  $Z$ , used in determining the pressure exposure and elevation coefficient,  $K_z$ , shall be taken as:

- For bridge superstructures: The average height of the top of the superstructure above the surrounding ground or water surface.
- For bridge substructures not extending above the elevation of the superstructure: Unless otherwise approved by the Owner, the height used in determining the wind pressure on the superstructure.

In the case of a long multi-span bridge with large variation in the ground surface elevation under the bridge, such as a bridge crossing a valley, the structure height,  $Z$ , may be varied from a span to span. For each span, the structure height,  $Z$ , may be taken as the largest value in the span.

Determining the wind pressure on substructures not extending above the elevation of the superstructure using the structure height used to determine the wind pressure on the superstructure results in slightly conservative values for most substructures. For

- For bridge substructures extending above the elevation of the superstructure: Unless otherwise approved by the Owner, the height of the top of the substructure.
- For ground-mounted sound barriers: The height of the top of the sound barrier above the lower surrounding ground surface.
- For structure- or traffic-barrier-mounted sound barriers: The height of the top of the sound barrier above the low ground or water surface surrounding the support structure.

In no case shall the structure height,  $Z$ , used in calculating  $K_z$  be taken less than 33 ft.

**Table 3.8.1.2.1-1—Gust Effect Factor,  $G$**

| Structure Type       | Gust Effect Factor, $G$ |
|----------------------|-------------------------|
| Sound Barriers       | 0.85                    |
| All other structures | 1.00                    |

**Table 3.8.1.2.1-2—Drag Coefficient,  $C_d$**

| Component                                      |                    | Drag Coefficient, $C_d$ |         |
|------------------------------------------------|--------------------|-------------------------|---------|
|                                                |                    | Windward                | Leeward |
| I-Girder and Box-Girder Bridge Superstructures |                    | 1.3                     | N/A     |
| Trusses, Columns, and Arches                   | Sharp-Edged Member | 2.0                     | 1.0     |
|                                                | Round Member       | 1.0                     | 0.5     |
| Bridge Substructure                            |                    | 1.6                     | N/A     |
| Sound Barriers                                 |                    | 1.2                     | N/A     |

extremely tall substructures, using a different height, including varying the height used for different segments of the substructure, may be allowed with the approval of the Owner.

Substructures extending above the elevation of the superstructure are typically associated with cable-stayed bridges and suspension bridges. Wind loads on such structures are typically determined using a structure-specific wind tunnel test.

Where the sound barrier is constructed directly atop an embankment, the height of the sound barrier should be measured from the lower ground surface surrounding the embankment.

### 3.8.1.2.2—Loads on the Superstructure

In the general case of wind analysis, the wind load shall be determined as specified in Article 3.8.1.1 and the wind direction shall be varied. The wind loads shall be taken as the algebraic transverse and longitudinal components of the wind load. The wind direction for design shall be that which produces the maximum force effect in the component under investigation. The transverse and longitudinal components of the wind load shall be applied simultaneously.

### C3.8.1.2.2

For superstructure components, the wind load on different members should be calculated separately and used in designing the members themselves. For trusses, the wind loads from different members and from the flooring system are transferred to the top and bottom planes of wind bracing and are used in designing the wind bracing system, including the end portals and cross-frames.

The term “columns” in Table 3.8.1.2.1-2 refers to columns in superstructures such as spandrel columns in arches.

### 3.8.1.2.3—Loads on the Substructure

#### 3.8.1.2.3a—Loads from the Superstructure

The transverse and longitudinal wind load components transmitted by the superstructure to the substructure for various angles of wind directions may be taken as the product of the skew coefficients specified in Table 3.8.1.2.3a-1, the wind pressure calculated using Eq. 3.8.1.2.1-1, and the depth of the bridge. The depth of the bridge shall be as seen in elevation perpendicular to the longitudinal axis of the bridge.

Both components of the wind loads shall be applied as line loads at the mid-depth of the superstructure. In plan, the longitudinal components of wind loads shall be applied as line loads along the longitudinal axis of the superstructure.

The skew angle shall be taken as measured from the perpendicular to the longitudinal axis of the bridge in plan.

The wind direction for design shall be that which produces the maximum force effect in the substructure. The transverse and longitudinal wind load components on the superstructure shall be applied simultaneously.

#### C3.8.1.2.3a

The Seventh Edition of *AASHTO Standard Specifications for Highway Bridges* was the first edition to incorporate the wind loads per skew angle of wind (Vincent, 1953). Alternatively, the wind load on the exposed area may be determined using an algebraic summation of transverse and longitudinal components of wind load. However, a wind directionality factor and drag coefficient other than the one specified in Article 3.8.1.2.1 should be considered.

For girder bridges, the wind pressure may be taken as one line load whose intensity is equal to the product of the wind pressure, skew coefficients, and the depth of the superstructure including the depth of the girders, deck, floor system, railing, and sound barriers, as seen in elevation perpendicular to the longitudinal axis of the bridge. For trusses, columns, and arches, the wind load is the sum of the wind loads on the windward and leeward areas.

The purpose of applying the line load along the longitudinal axis of the bridge in plan is to avoid introducing a moment in the horizontal plane of the superstructure.

**Table 3.8.1.2.3a-1—Skew Coefficients for Various Skew Angles of Attack**

| <u>Skew Angle<br/>(degree)</u> | <u>Trusses, Columns, and Arches</u>    |                                          | <u>Girders</u>                         |                                          |
|--------------------------------|----------------------------------------|------------------------------------------|----------------------------------------|------------------------------------------|
|                                | <u>Transverse Skew<br/>Coefficient</u> | <u>Longitudinal<br/>Skew Coefficient</u> | <u>Transverse Skew<br/>Coefficient</u> | <u>Longitudinal<br/>Skew Coefficient</u> |
| 0                              | 1.000                                  | 0.000                                    | 1.000                                  | 0.000                                    |
| 15                             | 0.933                                  | 0.160                                    | 0.880                                  | 0.120                                    |
| 30                             | 0.867                                  | 0.373                                    | 0.820                                  | 0.240                                    |
| 45                             | 0.627                                  | 0.547                                    | 0.660                                  | 0.320                                    |
| 60                             | 0.320                                  | 0.667                                    | 0.340                                  | 0.380                                    |

For usual girder and slab bridges having an individual span length of not more than 150 ft and a maximum height of 33 ft above low ground or water level, the following wind load components may be used:

- Transverse: 100 percent of the wind load calculated based on wind direction perpendicular to the longitudinal axis of the bridge.
- Longitudinal: 25 percent of the transverse load.

Both forces shall be applied simultaneously.

#### 3.8.1.2.3b—Loads Applied Directly to the Substructure

The transverse and longitudinal forces to be applied directly to the substructure shall be calculated using the wind pressure determined using

#### C3.8.1.2.3b

When combining the wind forces applied directly to the substructure with the wind forces transmitted to the substructure from the superstructure, all wind

Eq. 3.8.1.2.1-1. For wind directions taken skewed to the substructure, the wind pressure shall be resolved into components perpendicular to the end and front elevations of the substructure. The component perpendicular to the end elevation shall act on the exposed substructure area as seen in end elevation, and the component perpendicular to the front elevation shall act on the exposed substructure area as seen in front elevation. The two substructure wind force components shall be applied simultaneously with the wind loads from the superstructure.

#### 3.8.1.2.4—Wind Loads on Sound Barriers

The wind pressure on ground-mounted or structure-mounted sound barriers shall be determined using Eq. 3.8.1.2.1-1 and assuming the wind direction perpendicular to the plane of the sound barrier.

The sound barrier panels shall be designed assuming the wind pressure is applied as a uniform load to the entire area of the panels.

The vertical support elements (if used), the foundations, and the connection of the panel or the vertical support elements to the foundations or the supporting structure shall be designed for a line load equal in value to the wind pressure multiplied by the sound barrier height. The line load shall be applied at a distance equal to 0.55 times the sound barrier height measured from the bottom of the sound barrier. For determining the location of the line load, the height of the sound barrier shall be taken as the distance from the top of the sound barrier to:

- The ground surface immediately adjacent to the sound barrier for ground-mounted sound barriers
- The elevation of the sound barrier connection to the supporting structure for structure-mounted sound barriers.

Where the sound barrier is mounted on top of a traffic railing or a retaining wall extending above ground, the magnitude and location of the wind loads transmitted to the base of the supporting traffic railing or retaining wall shall be determined as specified above, assuming that the height of the exposed area is the sum of the height of the sound barrier plus the height of the supporting railing or retaining wall.

The height of the supporting railing or retaining wall to be considered in determining the magnitude and location of the wind load shall be that measured from the top surface of the ground, bridge deck, or roadway pavement to the top of the supporting railing or retaining wall.

#### 3.8.1.3—Wind Load on Live Load: $WL$

Wind load on live load shall be represented by an interruptible, moving force of 0.10 klf acting

forces should correspond to wind blowing from the same direction.

#### C3.8.1.2.4

The wind pressure is applied as a constant pressure over the entire area of the sound barrier. In reality, the wind speed and, consequently, the wind pressure, increase with the increase in height above the surrounding ground surface. Applying the wind load as a line load at a location above mid-height of the sound barrier better reflects the effect of the uneven pressure distribution.

Where the ground surface elevation is not the same in the front and in the back of a ground-mounted sound barrier, the wind forces will need to be determined for each direction as a separate case of loading. The design of all components must satisfy the demand from both cases.

#### C3.8.1.3

Historically, the 0.10 klf wind load has been used to determine the wind load on live loads. The value

transverse to, and 6.0 ft above, the roadway and shall be transmitted to the structure.

For various angles of wind direction, the transverse and longitudinal components of the wind load on live load may be taken as specified in Table 3.8.1.3-1 with the skew angle measured from the perpendicular to the longitudinal axis of the bridge in plan.

The wind direction for design shall be that which produces the extreme force effect on the component under investigation. The transverse and longitudinal wind load components on the live load shall be applied simultaneously.

**Table 3.8.1.3-1—Wind Load Components on Live Load**

| Skew Angle<br>(degrees) | Transverse<br>Component<br>(klf) | Longitudinal<br>Component<br>(klf) |
|-------------------------|----------------------------------|------------------------------------|
| 0                       | 0.100                            | 0.000                              |
| 15                      | 0.088                            | 0.012                              |
| 30                      | 0.082                            | 0.024                              |
| 45                      | 0.066                            | 0.032                              |
| 60                      | 0.034                            | 0.038                              |

For the usual girder and slab bridges having an individual span length of not more than 150 ft and a maximum height of 33 ft above low ground or water level, the following wind load components on live load may be used:

- 0.10 klf, transverse
- 0.04 klf, longitudinal

Both forces shall be applied simultaneously.

### 3.8.2—Vertical Wind Load

The effect of forces tending to overturn structures, unless otherwise determined in Article 3.8.3, shall be calculated as a vertical upward wind load equal to:

- 0.020 ksf for Strength III load combination and
- 0.010 ksf for Service IV load combination

times the width of the deck, including parapets and sidewalks, shall be applied as a line load. This force shall be applied only when the direction of horizontal wind is taken to be perpendicular to the longitudinal axis of the bridge. This line load shall be applied at the windward quarter-point of the deck width in conjunction with the horizontal wind loads specified in Article 3.8.1.

The vertical wind load shall not be applied for load combinations other than Strength III and Service IV.

was based on a long row of randomly sequenced passenger cars, commercial vans, and trucks exposed to the maximum wind speed that vehicles can safely travel. The wind load on live load specified herein has not been changed from its value in earlier editions of these Specifications.

This horizontal live load should be applied only to the tributary areas producing a force effect of the same kind, similar to the design lane load.

### C3.8.2

The intent of this Article is to account for the effect resulting from interruption of the horizontal flow of air by the superstructure. This load is to be applied even to discontinuous bridge decks, such as grid decks. This load may govern where overturning of the bridge is investigated.

For flexible bridges, such as cable-stayed and suspension bridges, the vertical force should be checked as an upward or downward force, whichever may control the design.

**3.8.3—Wind-Induced Bridge Motions****3.8.3.1—General**

The provisions of this Article shall apply to bridges in service and to bridges during construction after the deck and all features affecting their aeroelastic behavior are installed.

Force effects of wind-induced vibrations shall be taken into account in the design of bridges and structural components apt to be wind-sensitive. For the purpose of this Article, the following bridges shall be deemed to be wind-sensitive:

- All bridges with a span-to-depth ratio, and structural components thereof with a length-to-width ratio, exceeding 30.0,
- All cable-supported bridges, and
- All bridges with fundamental vertical or translational periods greater than 1 second.

The potential of wind-induced vibrations of cables, due to any causative mechanism, shall also be considered.

**3.8.3.2—Wind-Induced Motions**

Wind-induced vibrations due to buffeting, vortex excitation, galloping, flutter, and static divergence of wind-sensitive bridges and wind-sensitive components shall be considered where applicable.

**C3.8.3.1**

Because of the complexity of analyses often necessary for an in-depth evaluation of structural wind-induced vibrations, this Article is intentionally kept to a simple statement. Many bridges, decks, or individual structural components have been shown to be insensitive to wind-induced vibrations if the specified ratios are under 30.0, a somewhat arbitrary value helpful only in identifying likely wind-sensitive cases.

The most common cable vibrations are due to vortex shedding, rain/wind-induced vibrations, galloping due to the inclination of the cable to the wind, wake galloping, galloping due to aerodynamically unsymmetrical cross-section, excitation from the cable anchorage motion, and buffeting from wind turbulence.

Flexible bridges, such as cable-supported or very long spans of any type, may require special studies based on wind tunnel information. In general, appropriate wind tunnel tests involve simulation of the wind environment local to the bridge site. Details of wind tunnel testing are part of the existing wind tunnel state of the art and are beyond the scope of this commentary.

**C3.8.3.2**

Excitation due to vortex shedding is the escape of wind-induced vortices behind the member, which tend to excite the component at its fundamental natural frequency in harmonic motion. It is important to keep stresses due to vortex-induced oscillations below the “infinite life” fatigue stress. Methods exist for estimating such stress amplitudes but they are outside the scope of this commentary.

Tubular components can be protected against vortex-induced oscillation by adding bracing, strakes, or tuned mass dampers, or by attaching horizontal flat plates parallel to the tube axis above and/or below the central third of their span. Such aerodynamic damper plates should lie about one-third tube diameter above or below the tube to allow free passage of wind. The width of the plates may be the diameter of the tube.

Galloping is a high-amplitude oscillation associated with ice-laden cables or long, flexible members that do not have round cross-sections. Cable

### **3.8.3.3—Control of Dynamic Responses**

For wind-sensitive bridges, peak vertical wind-induced accelerations of the superstructure due to vortex shedding or buffeting should be less than 5 percent of the acceleration of gravity,  $g$ , for steady wind speeds less than or equal to 30 mph, and should be less than 10 percent of the acceleration of gravity,  $g$ , for steady wind speeds greater than 30 mph and less than or equal to 50 mph. Wind-sensitive bridges and wind-sensitive structural components thereof, including cables, shall be designed to be free of fatigue damage due to vortex-induced or galloping oscillations. Bridges shall be designed to be free of divergence, galloping, and catastrophic flutter up to a steady, 10-minute averaged wind speed numerically equal to 0.85 times the design wind speed applicable at the completed bridge at the superstructure elevation.

For the purpose of determining the 10-minute averaged wind speed, the design wind speed applicable for the completed bridge at the superstructure elevation shall be taken equal to  $V(K_z)^{1/2}$ , for which  $V$  and  $K_z$  are as defined in Article 3.8.1.2.1.

### **3.8.4—Site-Specific and Structure-Specific Studies**

The requirements of Article 3.8.3 may be satisfied using:

- A site-specific analysis of historical wind data in nonhurricane areas and a site-specific numerical simulation of potential hurricane wind speeds may be used to determine design wind criteria
- Representative wind tunnel tests using approved procedures may be utilized to determine wind loads and to evaluate aeroelastic stability

stays, having circular sections, will gallop when the wind is inclined to the axis of the cable, and when their circumferences are deformed by ice, dropping water, or accumulated debris.

Flexible bridge decks, as in very long spans and some pedestrian bridges, may be prone to wind-induced flutter, a wind-excited oscillation of destructive amplitudes, or, on some occasions, divergence, an irreversible twist under high wind. Analysis methods, including wind tunnel studies leading to adjustments of the deck form, are available for prevention of both flutter and divergence.

### **C3.8.3.3**

Cables in stayed-girder bridges have been successfully stabilized against excessive dynamic responses by attaching mechanical dampers to the bridge at deck level or by cross-tying multiple cable stays.

The 5 percent and 10 percent of the acceleration of gravity,  $g$ , for winds below 30 mph and winds between 30 mph and 50 mph, respectively, are typical limits for the motion criteria for pedestrian comfort. They were used successfully in the past in the design of vehicular flexible bridge systems such as cable-stayed and suspension bridges. For higher wind speeds, strength considerations, not motion considerations, govern the design.

The specified 10-minute averaged wind speed, numerically equal to  $0.85 V(K_z)^{1/2}$ , is that with an approximate mean recurrence interval of 10,000 years. Since catastrophic flutter vibrations take some time to develop, a 10-minute averaged wind speed is used to evaluate the stability of the bridge.

### **C3.8.4**

Wind tunnel testing of bridges and other civil engineering structures is a highly developed technology that may be used to study the wind response characteristics of a structural model or to verify the results of analysis (Simiu, 1976).

## 3.11.5.8.1—General

Replace Figures 3.11.5.8.1-1, 3.11.5.8.1-2, and 3.11.5.8.1-3 with the following:

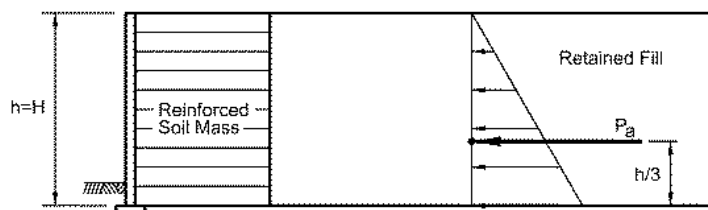


Figure 3.11.5.8.1-1—Earth Pressure Distribution for MSE Wall with Level Backfill Surface

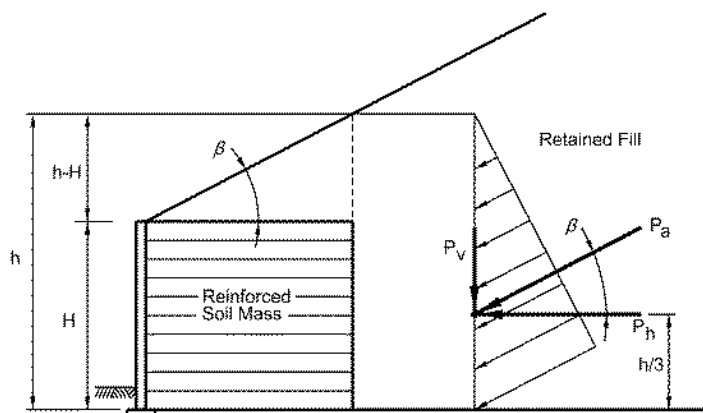


Figure 3.11.5.8.1-2—Earth Pressure for MSE Wall with Sloping Backfill Surface

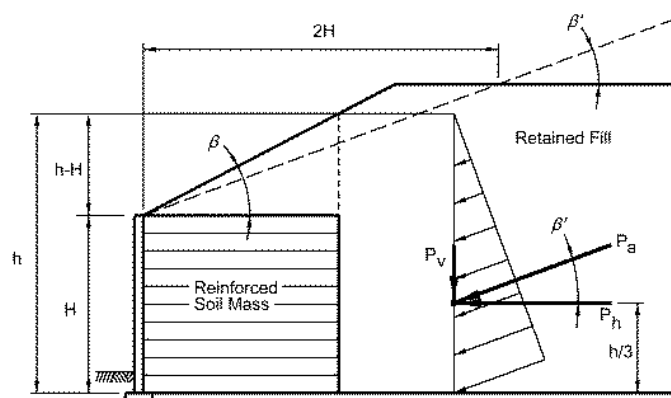


Figure 3.11.5.8.1-3—Earth Pressure Distribution for MSE Wall with Broken Back Backfill Surface



### 3.16—REFERENCES

Add and revise the following references:

ASCE. 2010. *Minimum Design Loads for Building and Other Structures*, ASCE 7-10. American Society of Civil Engineers, Reston, VA.

Kulicki, J. M., W. G. Wassef, D. R. Mertz, A. S. Nowak, N. C. Samtani, and H. Nassif. 2015. *Bridges for Service Life beyond 100 Years: Service Limit State Design*, Report S2-R19B-RW-1. Transportation Research Board, National Research Council, Washington, DC.

Nowak, A. S., M. Lutomirska, and F. I. Sheikh Ibrahim. 2010. “The Development of Live Load for Long Span Bridges,” *Bridge Structures*, IOS Press, Amsterdam, Vol. 6, 2010, pp. 4–7 73–79.

Vincent, G. 1953. *Investigation of Wind Forces on Highway Bridges*. Highway Research Board, Special Report 10, Washington, DC.

Wassef, W. G., J. M. Kulicki, H. A. Nassif, D. R. Mertz, and A. S. Nowak. 2014. *Calibration of LRFD Concrete Bridge Design Specifications for Serviceability*, Web-Only Document 201, Transportation Research Board, National Research Council, Washington, DC.

Wassef, W. and J. Raggett. 2014. *Updating the AASHTO LRFD Wind Load Provisions*, Final Report for NCHRP Project 20-7, Task 325, Transportation Research Board, National Research Council, Washington, DC.



## SECTION 5: CONCRETE STRUCTURES

### 5.1—SCOPE

Remove Article 5.1 and replace it with the following:

The provisions in this Section apply to the design of bridge and retaining wall components constructed of normal-weight or lightweight concrete and reinforced with steel bars, welded wire reinforcement, and/or prestressing strands, bars, or wires. The provisions are based on design concrete compressive strengths varying from 2.4 ksi to 10.0 ksi, except where higher strengths are allowed for normal weight concrete.

The provisions of this Section characterize regions of concrete structures by their behavior as B- (beam or Bernoulli) Regions or D- (disturbed or discontinuity) Regions, as defined in Article 5.2. The characterization of regions into B-Regions and D-Regions is discussed in Article 5.5.1.

The provisions of this Section combine and unify the requirements for reinforced and prestressed concrete. A brief outline for the design of some routine concrete components is contained in Appendix A.



## 5.2—DEFINITIONS

Add the following definitions:

*Beam or Bernoulli Region (B-Region)*—Regions of concrete members in which Bernoulli's hypothesis of straight-line strain profiles, linear for bending and uniform for shear, applies. (See Article 5.5.1.2 for more detail.)

*Disturbed or Discontinuity Region (D-Region)*—Regions of concrete members encompassing abrupt changes in geometry or concentrated forces in which strain profiles more complex than straight lines exist. (See Article 5.5.1.2 for more detail.)

Revise the following definitions:

*Lightweight Concrete*—Concrete containing lightweight aggregate conforming to AASHTO M 195 and having an equilibrium density ~~air dry unit weight~~ not exceeding ~~0.120~~ 0.135 kcf, as determined by ASTM C567. ~~Lightweight concrete without natural sand is termed "all lightweight concrete" and lightweight concrete in which all of the fine aggregate consists of normal weight sand is termed "sand lightweight concrete."~~

*Normal-Weight Concrete*—Concrete having ~~a weight between~~ an equilibrium density greater than 0.135 kcf and a density not exceeding 0.155 kcf.



**5.3—NOTATION**

Revise and add the following to Article 5.3:

- $a$  = ~~shear span (in.);~~ depth of equivalent rectangular stress block (in.); the anchor plate width (in.); the lateral dimension of the anchorage device measured parallel to the larger dimension of the cross-section (in.) (C5.6.3.1) (5.7.2.2) (5.10.9.3.6) (5.10.9.6.1)  
 $f_{ct}$  = average splitting tensile strength of lightweight aggregate concrete (ksi) ~~(5.8.2.2)~~ (5.4.2.8)  
 $h_a$  = length of the back face (in.) (5.6.3.2)  
 $\ell_a$  = effective length of a CTT node (in.) (5.6.3.2)  
 $\ell_b$  = length of the bearing face (in.) (5.6.3.2)  
 $v$  = concrete efficiency factor (5.6.3.5.3.a)  
 $\alpha$  = ~~fraction defining the bearing face length of a portion of a nodal region;~~ angle of inclination of transverse reinforcement to longitudinal axis (degrees); total angular change of prestressing steel path from jacking end to a point under investigation (rad.); the angle of inclination of a tendon force with respect to the centerline of the member (degrees) (5.6.3.2) (5.8.3.3) (5.9.5.2.2b) (5.10.9.6.3)  
 $\theta_s$  = angle between ~~compression~~ strut and longitudinal axis of the member ~~in a shear truss model of a beam~~ (degrees) ~~(5.6.3.3.2)~~ (5.6.3.2)  
 ~~$\lambda$  = parameter used to determine friction coefficient  $\mu$  (5.8.4.2)~~  
 $\lambda$  = concrete density modification factor (5.4.2.8)  
 ~~$\lambda_{lw}$  = lightweight concrete factor (5.11.2.1.1)~~



**5.4.2.1—Compressive Strength**

Revise the 4<sup>th</sup> paragraph as follows:

For lightweight structural concrete, ~~air-dry unit weight, strength and any other~~ properties required for the application shall be specified in the contract documents.



**5.4.2.6—Modulus of Rupture**

Revise the 1<sup>st</sup> paragraph and bullet lists as follows:

Unless determined by physical tests, the modulus of rupture,  $f_r$ , for lightweight concrete with specified compressive strengths up to 10.0 ksi and normal weight concrete with specified ~~concrete~~ compressive strengths up to 15.0 ksi may be taken as:

• ~~For normal weight concrete:~~

- Except as specified below:  ~~$0.24\sqrt{f'_c}$~~   $0.24\lambda\sqrt{f'_c}$
- When used to calculate the cracking moment of a member in Article 5.8.3.4.3:  ~~$0.20\sqrt{f'_c}$~~   
 $0.20\lambda\sqrt{f'_c}$

$\lambda$  = concrete density modification factor as specified in Article 5.4.2.8

• ~~For lightweight concrete:~~

- ~~For sand lightweight concrete:  $0.20\sqrt{f'_c}$~~
- ~~For all lightweight concrete:  $0.17\sqrt{f'_c}$~~