
 <b>NEW YORK</b> STATE OF OPPORTUNITY.	<b>Department of Transportation</b>	<b>ENGINEERING INSTRUCTION</b>	<b>EI</b> <b>21-004</b>
<b>Title: NYSDOT LRFD BRIDGE DESIGN SPECIFICATIONS - 2021</b>			
		Approved:  James Flynn III, PE Deputy Chief Engineer (Structures)	2-2-21 Date

**ADMINISTRATIVE INFORMATION:**

- This Engineering Instruction (EI) is effective beginning with projects submitted for the letting of September 1, 2021
- This EI supersedes EI 19-001 "NYSDOT LRFD BRIDGE DESIGN SPECIFICATIONS - 2019"
- Disposition of Issued Materials: The technical information transmitted by this EI will be incorporated into the next revision of the NYSDOT Bridge Manual

**PURPOSE:** This EI officially adopts the NYSDOT LRFD Bridge Design Specifications – 2021 for use in New York State and announces the availability of "NYSDOT LRFD Blue Pages" dated January 2021.

**TECHNICAL INFORMATION:** The *AASHTO LRFD Bridge Design Specifications - 9<sup>th</sup> Edition, 2020*, together with the "NYSDOT LRFD Blue Pages" dated January 2021 constitute the NYSDOT LRFD Bridge Design Specifications.

- The LRFD specifications will continue to be used for the design of all new and replacement bridges for NYSDOT. This includes both superstructure designs and substructure designs. This EI does not discontinue use of the NYSDOT Standard Specifications for Highway Bridges – 2003. Both specifications will continue to be used until further notice. The existing NYSDOT Standard Specifications for Highway Bridges - 2003 will be used when necessary for the repair and rehabilitation of structures. The NYSDOT Standard Specifications for Highway Bridges – 2003 consists of the *AASHTO Standard Specifications for Highway Bridges - 17<sup>th</sup> Edition* plus the "NYSDOT Blue Pages", issued by EB 02-038 and EB 03-016.
- The NYSDOT Design Permit Vehicle has been removed from the NYSDOT LRFD Blue Pages.
- Currently, NYSDOT overload permitting and bridge posting policies require that new and replacement bridges be load rated using the Load Factor Design (LFD) or Allowable Stress Design (ASD) methods. For this reason, load ratings will continue to be computed by the LFD or ASD method and shown on the contract plans. Also, load rating factors for all new, replacement, and rehabilitated bridges will be computed by the Load and Resistance Factor Rating (LRFR) method and shown on the contract plans. LRFR ratings shall be shown at the Inventory and Operating levels as rating factors of the AASHTO HL-93 live load. Once overload permitting and bridge posting policies are revised to accommodate LRFR, load ratings using the LFD and ASD methods will be discontinued.

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- The NYSDOT Bridge Manual supplements both the NYSDOT LRFD Specifications and the NYSDOT Standard Specifications for Highway Bridges. Designers are required to consult the Bridge Manual for additional policies, guidance, details and interpretations of the design specifications.
- The “NYSDOT LRFD Blue Pages” dated January 2021 do not replace any existing “NYSDOT LRFD Blue Pages” for the *AASHTO LRFD Bridge Design Specifications – 9<sup>th</sup> Edition, 2020*

### **IMPLEMENTATION:**

This Engineering Instruction (EI) is implemented immediately for all structural design projects in New York State unless immediate implementation will result in undue delay to projects currently under design as determined by the Regional Director and the Deputy Chief Engineer (Structures).

### **TRANSMITTED MATERIALS:**

The NYSDOT LRFD Blue Pages dated January 2021 are attached. They can also be found at the following web address:

<https://www.dot.ny.gov/divisions/engineering/structures/manuals>

**CONTACT:** Direct questions regarding this EI to Julianne Fuda of the Office of Structures at (518) 457-0704 or via e-mail at [Julianne.Fuda@dot.ny.gov](mailto:Julianne.Fuda@dot.ny.gov).

**2021 NYSDOT Blue Pages**

<b>Article No.</b>	<b>Proposed vs. Current</b>	<b>Comments</b>
1.1	No changes	
1.3.5	No changes	
2.5.2.6.2	New	LL deflection criteria required.
3.4.1	No changes	
3.6.1.2.1	Removed	Removed requirements for NYSDOT Design Permit Vehicle.
3.6.1.2.4a	Removed	Removed requirements for NYSDOT Design Permit Vehicle.
3.6.1.2.6a	No changes	
3.6.1.6	Modified	Removed requirements for NYSDOT Design Permit Vehicle.
3.6.5.1	Modified	Eliminated text that is now covered by the 9 <sup>th</sup> edition.
3.7.5	No changes	
C3.7.5	No changes	
3.8.1.1.2	No changes	
3.8.1.2.1	No changes	
3.9.2.1	No changes	
C3.9.2.2	No changes	
3.9.3	No changes	
C3.9.3	No changes	
3.10.1	No changes	
C3.10.2.1	No changes	
3.10.2.2	No changes	
3.10.5	No changes	
3.10.7.1	No changes	
3.10.9.1	No changes	
3.10.11.1	No changes	
C3.10.11.1	No changes	
3.10.11.2	No changes	
A3.10	No changes	
3.11.5.4	New	Criteria for passive lateral earth pressure coefficients.

3.11.5.6	New	Criteria for lateral earth pressure.
3.11.6.4	No changes	
C3.11.6.4	No changes	
3.12.2	No changes	
3.12.2.1	No changes	
3.12.2.2	No changes	
3.14.1	No changes	
3.14.2	No changes	
3.14.3	No changes	
3.14.4	No changes	
3.15.1	No changes	
4.6.2.2.1	No changes	
C4.6.2.2.1	No changes	
4.6.2.8.1	No changes	
4.7.4.1	No changes	
4.7.4.2	No changes	
4.7.4.3.1	No changes	
4.7.4.3.4b	No changes	
5.4.2.1	No changes	
C5.4.2.1	No changes	
5.6.7	No changes	
5.9.2.3.2b	Modified	Removed requirements for NYSDOT Design Permit Vehicle.
5.9.4.3.3	New	Added criteria for debonded strands.
5.10.6	No changes	
5.11.4.1.4	No changes	
5.11.4.1.5	No changes	
5.11.4.1.6	No changes	
5.11.4.2	No changes	
6.6.1.2.3	No changes	
C6.6.2.1	New	Added reference to NYS Steel Construction Manual.
6.6.2.2	New	Added reference to NYS Steel Construction Manual.
6.7.2	New	Added reference to NYSDOT Bridge Manual
6.7.3	No changes	
6.7.4.1	No changes	

C 6.7.4.1	No changes	
6.7.4.2	Removed	Removed blue page since requirements are now included in the 9 <sup>th</sup> edition.
6.7.5.3	No changes	
6.10.3	No changes	
6.10.3.1a	No changes	
6.13.2.4.1b	No changes	
6.13.2.4.1c	No changes	
6.13.2.4.1d	No changes	
6.13.2.6.1	No changes	
6.13.2.8	No changes	
6.13.3.1	No changes	
6.13.6.1.1	No changes	
6.13.6.1.2	No changes	
6.13.6.2	Modified	Added reference to the NYS Steel Construction Manual.
6.16.4.1	No changes	
9.7.1.3	No changes	
C9.7.1.3	No changes	
9.7.2	No changes	
10.5.5.2.3	No changes	
C10.5.5.2.3	No changes	
10.7.2.4	No changes	
10.7.3.6	No changes	
10.7.3.8.6f	No changes	
10.7.9	No changes	
11.5.4.1	No changes	
11.10.6.2.1a	Removed	Removed blue page since requirements are now included in the 9 <sup>th</sup> edition.
C11.10.6.2.1a	New	Added criteria for live load surcharge.
11.10.6.3.2	Removed	Removed blue page since requirements are now included in the 9 <sup>th</sup> edition.
12.6.2.1a	No changes	
12.6.4	New	Added reference for hydraulic design criteria.
12.11.2.1	No changes	
12.14.5.3	No changes	

A13.4.3.1	No changes	
14.4.2.2.1	No changes	
14.4.2.2.2	No changes	

# NEW YORK STATE DEPARTMENT OF TRANSPORTATION

## LRFD BLUE PAGES

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## **1.1 SCOPE OF THE SPECIFICATION**

Delete the sixth paragraph of Article 1.1 and replace it with the following:

Seismic design shall be in accordance with the provisions in these specifications. Seismic design in accordance with the provisions given in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* can be used with the approval of Deputy Chief Engineer (Structures).

## **1.3 DESIGN PHILOSOPHY**

### **1.3.5 Operational Importance**

Delete the second paragraph and replace it with the following:

For the strength limit state:

$\eta_I = 1.05$  for critical bridges\*  
     $= 1.00$  for essential and other bridges

For all other limit states:

$\eta_I = 1.00$

\*For definition of critical bridges, see Blue Page 3.10.5.

## **2.5 DESIGN OBJECTIVES**

### **2.5.2 Serviceability**

#### **2.5.2.6 Deformation**

##### *2.5.2.6.2 Criteria for Deflection*

Delete the first paragraph and replace it with the following:

The criteria in this Article shall be adhered to for all state-owned bridges, including the following:

Add to the end of the second paragraph:

Any violation of the criteria in this Article shall be approved by DCES.

Delete the third paragraph and replace it with the following:

The following principles shall be applied to state-owned bridges:

Delete the fourth paragraph and replace it with the following:

The following deflection limits shall be adhered to for state-owned steel, aluminum, and/or concrete vehicular bridges:

- Vehicular load on bridge, general..... Span/800
- Vehicular and pedestrian loads\*  
on bridge.....Span/1000
- Vehicular load on cantilever arms.....Span/300
- Vehicular and pedestrian loads\* on cantilever  
arms .....Span/375

\*This deflection limit shall be adhered to for all bridges with sidewalks.

## **3.4 LOAD FACTORS AND COMBINATIONS**

### **3.4.1 Load Factors and Load Combinations**

Delete the sixth bullet of the second paragraph and replace it with the following:

- EXTREME EVENT I - Load combination including earthquake. The load factor for live load  $\gamma_{EQ}$ , shall be 0.50 for bridges with Average Daily Truck Traffic (ADTT) greater than 5,000 and 0.00 for all other bridges.

## **3.4 LOAD FACTORS AND COMBINATIONS**

### **3.4.1 Load Factors and Load Combinations**

Add the following note to the end of Table 3.4.1-2:

For horizontal reinforcement design of a cantilevered wingwall rigidly attached to an integral abutment, a passive lateral earth pressure load factor of 1.5 shall be used.

## **3.4 LOAD FACTORS AND COMBINATIONS**

### **3.4.1 Load Factors and Load Combinations**

Delete the second to last paragraph of Article 3.4.1.

## **3.6 LIVE LOADS**

### **3.6.1 Gravity Loads: *LL* and *PL***

#### **3.6.1.2 Design Vehicular Live Load**

##### *3.6.1.2.6 Distribution of Wheel Loads through Earth Fills*

##### *3.6.1.2.6a General*

In the first sentence of the second and third paragraphs, add 'box or,' just before the words 'flat top three-sided'.

## **3.6 LIVE LOADS**

### **3.6.1 Gravity Loads: *LL* and *PL***

#### **3.6.1.6 Pedestrian Loads**

Delete the first paragraph of Article 3.6.1.6 and replace it with the following:

Pedestrian load shall be neglected if the sidewalk width is less than or equal to 10.0 ft., and no physical barrier exists between vehicular traffic and sidewalk.

If the pedestrian load is neglected, then the HL-93 live load shall be assumed to mount the sidewalk, and shall be applied at 1.0 ft. from the face of the bridge railing for the design of the overhang, and 2.0 ft. from the face of the bridge railing for the design of all other components.

When the sidewalk is greater than 10.0 ft., or when a physical barrier exists between vehicular traffic and the sidewalk, a pedestrian load of 0.075 ksf shall be applied to all sidewalk areas, and shall be considered simultaneously with the vehicular design live load applied 2.0 ft. from the edge of curb.

All bridges shall be checked for the live load case where the sidewalk is removed in the future.

The Load Rating shall be for the case where the HL-93 live load is applied 2.0 ft. from the edge of curb and no pedestrian load on the sidewalk.



## **3.6 LIVE LOADS**

### **3.6.5 Vehicular Collision Force: *CT***

#### **3.6.5.1 Protection of Structures**

Add the following to the end of second paragraph.

For hammerhead and multi-column piers, the preferred design choice is to redirect or absorb the collision load wherever it is feasible.

Add the following paragraph to the end of Article 3.6.5.1:

Structures crossing railroad tracks with crash walls designed in accordance with the American Railway Engineering and maintenance of Way Association (AREMA) specifications are exempt from the provisions of Article 3.6.5.1.

### **3.7 WATER LOADS: WA**

#### **3.7.5 Change in Foundations Due to Limit State for Scour**

Add the following paragraphs to the end of this article:

The following changes in foundation condition shall be considered for the service, strength, and extreme event limit states for typical foundations supported on piles and drilled shafts.

For piles and drilled shafts at all scour depths, the surrounding soil above the scour elevation provides no lateral support.

For abutments and independent wingwalls with a minimum of two rows of piles, with at least one row battered, the reduced lateral soil resistance and increased displacement caused by the scour is counteracted by the battered piles and negates the need for a scour case lateral analysis. The structural resistance of the piles for the scour case shall be determined based on applied vertical loads without lateral loads, using an unbraced length set equal to the scour depth. This applies to integral abutments with vertical piles as well, except the lateral stability comes from the abutments fixed connection with the superstructure rather than the battered piles. This does not apply to independent wingwalls with a single row of piles, even if those piles are battered.

For piers, the structural resistance of the pile or drilled shaft for the scour case shall be determined based on applied vertical and horizontal loads, using an unbraced length set equal to the scour depth.

If the calculated displacement of the pile or drilled shaft, based on group analysis, under the applied lateral loads is less than 4.0 in., no further checks are necessary.

If the pile or drilled shaft displacement is greater than 4.0 in. or when the displacement is unknown, the superstructure to substructure connection shall be designed for the applicable limit state.

In all cases, the axial geotechnical resistance of the piles and drilled shafts shall account for scour as stated in Articles 10.7.3.6 or 10.8.3.3, as applicable.

## **3.7 WATER LOADS: WA**

### **3.7.5 Change in Foundations Due to Limit State for Scour**

**C3.7.5** Delete the Commentary to Article 3.7.5 and replace it with the following:

Designing for large scour depths is impractical for pile foundations at abutments given the conservatism of the current state of practice. Based on NYSDOT's experience and design methodology, the excess resistance of the abutment battered piles sufficiently counter balances the decrease in lateral resistance and the theoretical increase in displacement that the design models predict for a scour event.

Therefore, because of the inherent conservatism in NYSDOT's current state of practice combined with NYSDOT's inspection program, a rigorous lateral analysis for scour events is unnecessary for typical pile abutment foundations. Large, critical, signature structures, or other economically significant structures exposed to unusual scour risk may warrant a rigorous lateral analysis.

## **3.8 WIND LOAD: *WL* AND *WS***

### **3.8.1 Horizontal Wind Loading**

#### **3.8.1.1 Exposure Conditions**

##### **3.8.1.1.2 Wind Speed**

Delete the second sentence in Article 3.8.1.1.2 and replace it with the following:

In the absence of more precise information, the 3-second gust wind speed for areas designated as a Special Wind Region in Figure 3.8.1.1.2-1 may be taken as 115 MPH.

**3.8 WIND LOAD:  $WL$  AND  $WS$** **3.8.1 Horizontal Wind Load****3.8.1.2 Wind Load on Structures:  $WS$** **3.8.1.2.1 General**

Delete Table 3.8.1.2.1-1 Gust Effect Factor,  $G$  and replace it with the following:

Structure Type	Gust Effect Factor, $G$
Sound Barriers Mounted on the Ground	0.85
Sound Barriers Mounted on a Structure	1.00
All other Structures	1.00

**3.9 ICE LOADS: *IC***

**3.9.2 Dynamic Ice Forces on Piers**

**3.9.2.1 Effective Ice Strength**

Delete Article 3.9.2.1 and replace it with the following:

In the absence of more precise information, a value of 24.0 ksf may be used for the effective ice crushing strength.

The height of action of the Dynamic Ice Load shall be the Ordinary High Water elevation.

### **3.9 ICE LOADS: *IC***

#### **3.9.2 Dynamic Ice Forces on Piers**

##### **3.9.2.2 Crushing and Flexing**

##### **C3.9.2.2** Delete the following definition:

" $S_f$  = freezing index, being the algebraic sum,  $\Sigma(32 - T)$ , summed from the date of freeze-up to the date of interest, in degree days."

and replace with:

$S_f$  = freezing index. Values for each county of New York are given in Fig. C3.9.2.2-1. For a bridge crossing a county line or a line separating areas with different values of  $S_f$ , the smaller value of  $S_f$  should be used.

Delete the following:

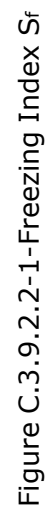
"As a guide, Neill (1981) indicates the following values for  $\alpha$ :

windy lakes without snow.....	0.8
average lake with snow .....	0.5-0.7
average river with snow.....	0.4-0.5
sheltered small river with snow.....	0.2-0.4"

and replace with:

In the absence of more precise information, the following values may be used for  $\alpha$ :

All lakes.....	0.70
All rivers.....	0.50





## **3.9 ICE LOADS: *IC***

### **3.9.3 Static Ice Loads on Piers**

Add the following at the end of the article:

When the designer determines that static ice loads should be considered in the design of a member, the Static Ice Force may be assumed to be the product of a 7.5 ksf pressure applied over a depth equal to the thickness of the ice, not to exceed 2.0 ft.

The height of action of the Static Ice Load shall be the Ordinary Water elevation.

Static Ice Loads shall not be combined with Dynamic Ice Forces.

#### **C.3.9.3** Add the following at the end of the commentary:

In the absence of more precise information, the thickness of ice can be estimated using the procedure in C3.9.2.

The maximum ice pressure developed in the static condition is known to have an upper limiting value of 7.5 ksf due to cracking and creep relaxation of ice pressures.

For an ice sheet having a thickness greater than 2.0 ft., the force per unit width does not depend on ice thickness, because the ice layer below the 2.0 ft. depth does not undergo a change in temperature.

### **3.10 EARTHQUAKE EFFECTS: *EQ***

#### **3.10.1 General**

Delete the first paragraph of Article 3.10.1 and replace it with the following:

Bridge Operational Categories are defined in Article 3.10.5. Bridges defined as "other" shall be designed to have a low probability of collapse but may suffer significant damage and disruption to service when subject to earthquake ground motions that have a 7% probability of exceedance in 75 years. Partial or complete replacement may be required. Higher levels of performance may be used with the authorization of the bridge owner.

Add the following after the last paragraph:

For bridges located in the counties of Bronx, Kings, New York, Queens, Richmond, Nassau, Rockland, and Westchester (defined as the Downstate Zone), delete Section 3.10, including all subsequent articles, and replace it with Blue Page A3.10. For seismic analysis and design procedures, all remaining sections of the LRFD Bridge Design Specifications along with the NYS Blue Pages shall be followed.

**3.10 EARTHQUAKE EFFECTS: *EQ***

**3.10.2 Seismic Hazard**

**3.10.2.1 General Procedure**

**C3.10.2.1** Delete the fourth and fifth paragraphs of the Commentary to Article 3.10.2.1.

### **3.10 EARTHQUAKE EFFECTS: *EQ***

#### **3.10.2 Seismic Hazard**

##### **3.10.2.2 Site Specific Procedure**

Delete Article 3.10.2.2 and replace it with the following:

A site-specific procedure to develop design response spectra of earthquake ground motions shall be performed when required by Article 3.10.2 and may be performed for any site. The objective of the site-specific procedure should be to generate a response spectrum for the 1,000 year return period earthquake for spectral values over the entire period range of interest. An additional response spectrum for a 2,500 year return period earthquake is required for critical bridges. The uniform-hazard acceleration response spectrum for rock below soil at the bridge site shall be based on the 2009 AASHTO spectra for the 1,000 year earthquake and shall be based on the 2009 NEHRP spectra for the 2,500 year earthquake.

Where analyses to determine site soil response effects are required by Article 3.10.3.1 for Site Class F soils, the influence of the local soil conditions shall be determined based on site-specific geotechnical investigations and dynamic site response analyses using appropriate rock spectra.

Where response spectra are determined from a site-specific study, the spectra shall not be lower than two thirds of the response spectra determined using the general procedure of Article 3.10.2.1 in the region of  $0.5T_F$  to  $2.0T_F$  of the spectrum where  $T_F$  is the bridge fundamental period.

### **3.10 EARTHQUAKE EFFECTS: EQ**

#### **3.10.5 Operational Classification**

Add the following after the last paragraph:

##### **Seismic Performance Criteria**

**Critical Bridge:** A critical bridge must provide immediate access after the lower level (functional) event and limited access after the upper level (safety) event and continue to function as a part of the lifeline, social/survival network and serve as an important link for civil defense, police, fire department and/or public health agencies to respond to a disaster situation after the event, providing a continuous route. Any bridge on a critical route shall be classified as critical if there is no readily accessible detour, and shall at a minimum be classified as essential if there is such a detour.

Any bridge that crosses a critical route, whose collapse would block the critical route, shall at a minimum be classified as essential if there is no readily accessible detour for the critical route. However, the bridge owner may classify such bridges as critical with the concurrence of the DCES.

The designated detour for the critical route shall also be treated as a critical route.

It is expected that relatively few bridges will be classified as critical. Critical bridges would generally be limited to those on life safety routes in an urban area or on the approaches to an urban area. Critical bridges would also be located on routes to a defense facility that has limited access. Bridges on limited access highways in rural areas would generally not be classified as critical unless they are major structures. The designation of a bridge as critical is at the discretion of the Regional Director/Bridge Owner and it is to be documented in the Design Report and included in the Site Data Package.

### 3.10.5 (continued)

Critical bridges shall be analyzed for two earthquake hazard design levels: a lower level event (functional evaluation/design level) having a 7% probability of being exceeded in 75 years (1,000 year return period), and an upper level event (safety evaluation/design level) having a 3% probability of being exceeded in 75 years (2,500 year return period). These analysis requirements shall apply to critical bridges located within Seismic Zone 1 as well. A site specific analysis is required for critical bridges, except for single-span bridges located on non-liquefiable soil sites. In the case of long span bridges, the effects of spatial variation on the seismic ground motions must also be considered.

Critical bridges shall survive the upper level event (2,500 year return period) with repairable damage (see definition of damage levels). Traffic access following this event may be limited; within 48 hours for emergency/defense vehicles and within months for general traffic. After the lower level event (1,000 year return period) the bridge shall suffer no damage to primary structural elements and minimal damage to other components (see definition of damage levels). Access after this event shall be immediate to all traffic with an allowance of a few hours for inspection.

**Essential Bridge:** An essential bridge must provide at least limited access after a lower level event and serve as an important link for civil defense, police, fire department and/or public disaster situation after the event, providing a continuous route. A bridge that crosses an essential route, whose collapse would block the essential route, shall also be classified as essential if there is no readily accessible detour for the essential route. The designated detour for the essential route shall also be treated as an essential route.

Essential bridges should include those on interstate highways and others of importance as determined by the Regional Director/Bridge Owner. The designation of a bridge as essential is to be documented in the Design Report and included in the Site Data Package.

### 3.10.5 (continued)

Essential bridges shall be analyzed for a single earthquake hazard design level event having a 7% probability of being exceeded in 75 years (1,000 year return period). Essential bridges shall survive the design event with repairable damage (see definition of damage levels). Access following the seismic event may be limited: one or two lanes shall be available within 72 hours for emergency vehicles, full service within months.

**Other Bridges:** All bridges not classified as critical or essential shall be classified as Other Bridges. Other bridges shall be analyzed for a single earthquake hazard design level event having a 7% probability of being exceeded in 75 years (1,000 year return period). Other bridges may suffer significant damage (see definition of damage levels) although collapse shall not occur. The designation of a bridge as Other is to be documented in the design report and included in the Site Data Package.

#### Damage Levels – Definitions

- **Minimal Damage:** The Bridge should essentially behave elastically during the earthquake, although minor inelastic response could take place. Post-earthquake damage should be limited to narrow flexural cracking in concrete and masonry elements. There should be no permanent deformations to structural members. Only minor damage or permanent deformations to non-structural members should take place.
- **Repairable Damage:** The extent of damage should be limited so that the structure can be restored to its pre-earthquake condition without replacement of structural members. Inelastic response may occur resulting in: concrete cracking, minor cover spalling and reinforcement yielding; minor yielding of structural steel members; some damage to secondary members and non-structural components; some damage to masonry. Repair should not require complete closure of the bridge. Permanent offsets should be small and there should be no collapse.

### 3.10.5 (continued)

- **Significant Damage:** There is no collapse, but permanent offsets may occur. Extensive cracking, major spalling of concrete and reinforcement yielding, cracking of deck slab at the shear studs, may force closure for repair. Similar consequences could result from yielding or local buckling of steel members. There could be yielding of member connections, fracture of limited number of bolts/rivets, serious damage to secondary structural members and non-structural components, as well as to masonry. In sites with significant ground lateral spreading due to liquefaction, large inelastic deformations might be induced to piles. Liquefaction could also result in excessive differential settlements. Partial or complete replacement may be required in some cases.



### 3.10 EARTHQUAKE EFFECTS: *EQ*

#### 3.10.7 Response Modification Factors

##### 3.10.7.1 General

Add to Article 3.10.7.1 the following:

The response modification factors given in Table 3.10.7.1-1 shall be applied, where applicable, to the seismic design forces that result from a 1,000 year return period event. In the case of a critical bridge where a 2,500 year return period event is being evaluated, the response modification factors for an essential bridge shall be applied, as shown in the table below.

Response Modification Factors – Critical Bridges

Substructure	Operational Category	
	Critical 1000 yr.	Critical 2500 yr.
Wall-type piers—larger dimension	1.5	1.5
Reinforced concrete pile bents		
• Vertical piles only	1.5	2.0
• With batter piles	1.5	1.5
Single columns	1.5	2.0
Steel or composite steel and concrete pile bents		
• Vertical pile only	1.5	3.5
• With batter piles	1.5	2.0
Multiple column bents	1.5	3.5

### **3.10.9 Calculation of Design Forces**

#### **3.10.9.1 General**

Delete the first paragraph of the article and replace with the following:

For single-span bridges, regardless of seismic zone, there shall be no required seismic design connection force in the restrained direction between the superstructure and the substructure. Minimum support lengths at both expansion and fixed bearings shall be provided as given in Article 4.7.4.4.

Insert Article 3.10.11 and Appendix A3.10 in between Article 3.10.10 and Article 3.11

**3.10.11 Criteria for Seismic Retrofitting of Bridges  
Programmed for Rehabilitation**

**3.10.11.1 General**

For evaluating and upgrading the seismic resistance of existing highway bridges, FHWA's "Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges" (January 2006, Publication No. FHWA-HRT-06-032) is referenced. The provisions of this manual shall apply to highway bridges of conventional steel and concrete construction with spans not exceeding 500 ft. The Owner shall specify and/or approve appropriate provisions for nonconventional construction and for bridges with spans exceeding 500 ft.

**C3.10.11.1** Conventional bridges include those with slab, beam, box girder, or truss superstructures, and single or multiple-column piers, wall-type piers, or pile-bent substructures. In addition, conventional bridges are founded on shallow or piled footings or shafts. Nonconventional bridges include suspension bridges, cable-stayed bridges, arches, movable bridges, and bridges with truss towers or hollow piers for substructures.

### **3.10.11.2 Seismic Performance Criteria**

Refer to the FHWA Seismic Retrofitting Manual and the NYSDOT Bridge Manual for guidance on the appropriate extent of seismic retrofitting.

The following changes to the FHWA Seismic Retrofitting Manual shall be applied:

- FHWA's seismic retrofitting manual recommends evaluation of bridges for lower level earthquake event of 100 years and an upper level event of 1,000 years. Bridges shall be evaluated only for the upper level earthquake ground motions with a 7% probability of exceedance in 75 years corresponding to a 1,000 year return period.
- Critical and essential bridges shall be combined in one category and will be evaluated under "essential" bridges as defined in the Retrofit Manual.
- Other bridges shall be considered the same as "standard" bridges as defined in the Retrofit Manual.
- Design Response Spectrum shall be constructed as per Article 3.10.4.1 incorporating site factors as per Article 3.10.3.2.
- Minimum support length requirements are to be calculated as per Article 4.7.4.4.

### A3.10 SEISMIC DESIGN CRITERIA FOR THE DOWNSTATE ZONE

Downstate Zone: The counties of Bronx, Kings, New York, Queens, Richmond, Nassau, Rockland and Westchester as shown in Figure A3.10-1.



**Figure A3.10-1 Downstate Zone**

#### A3.10.1 General

The Seismic Design Criteria for the Downstate Zone provides criteria for the analysis, evaluation, design and retrofit of bridges in the Downstate Zone. For additional information regarding these provisions refer to the 2016 New York City Department of Transportation Seismic Design Guidelines for Bridges in Downstate Region (*NYCDOT SDGBDZ 2016*).

The design earthquake ground motion levels specified herein could result in both structural and non-structural damage. For most bridge systems designed and constructed or retrofitted according to these criteria, structural damage from the design earthquake ground motion would be repairable. It is expected that the damage from the design earthquake ground motions would not be so severe as to preclude continued function of the bridge. The actual ability to accomplish these goals depends upon a number of factors including site conditions, the structural type and configuration of the bridge, construction materials, and as-built details of construction.

The following criteria identify minimum requirements for seismic design. Each bridge presents a unique set of design challenges. The designer must determine the appropriate methods and level of refinement necessary to design and analyze each bridge on a case-by-case basis. The designer must exercise judgment in the application of these criteria.

Situations may arise that warrant detailed attention beyond what is provided in the Guidelines. It is the prerogative of each bridge owner to decide under which cases these *Guidelines* would need to be modified to suit specific circumstances.

The ***NYCDOT SDGBDZ 2016*** includes requirements related to the following:

- Bridge Classification and Performance Criteria
- Very Hard Rock Spectra and Time History Records
- Classification of a Site as a Rock or Soil Site
- Rock Classes and Rock Generic Horizontal Design Spectra
- Soil Site Characterization and Soil Generic Horizontal Design Spectra
- Vertical Motions and Generic Design Spectra for Rock and Soil Sites
- Site Liquefaction
- Site Specific Studies

### **A3.10.2 Seismic Hazard**

The seismic hazard at a bridge site shall be characterized by the acceleration response spectrum for the relevant site class. The acceleration spectrum shall be determined using either the General Procedure specified in Article A3.10.2.1 or the Site Specific Procedure specified in Article A3.10.2.2.

The Seismic Hazard for the Downstate Zone (see Figure A3.10-1) has been quantified in the form of 5% damped Uniform Hazard Spectra (UHS) for three earthquake return periods, 1,000, 1,500, and 2,500 years, corresponding to probabilities of exceedance in 75 years of 7%, 5%, and 3%, respectively. The horizontal UHS are presented in Table A3.10.2-1 and Figure A.3.10.2-1 as coefficients corresponding to spectral accelerations in terms of  $g$ , the acceleration of gravity. The vertical UHS are presented in Table A3.10.2-2 and Figure A.3.10.2-2 as coefficients corresponding to spectral accelerations in terms of  $g$ , the acceleration of gravity. The spectra in Table A3.10.2-1 and A3.10.2-2, and in Figures A3.10.2-1 and A.3.10.2-2, represent an 85<sup>th</sup> percentile (median plus one standard deviation level) of ground motions corresponding to each one of the three return periods of 1,000 years, 1,500 years, and 2,500 years. The motions are for Very Hard Rock (VHR) in NYC, typical of the eastern United States (US), with a shear wave velocity of at least 2.83 km/sec (approximately 9,000 ft/sec). This 2.83 km/sec shear wave velocity is an average of eastern US continental crust. UHS in the horizontal direction for softer rock conditions and soil site conditions are given in Article A3.10.4.

Vertical UHS for Very Hard Rock, softer rock conditions, as well as for soil site conditions, are also presented in Article A3.10.4 and shall be used according to Article A3.10.2.1.

Very Hard Rock ground motion time history records have been developed to match the spectral accelerations in Tables A3.10.2-1 and A3.10.2-2 and are available in digital form from the NYSDOT Office of Structures website for the 1,000 years, 1,500 years, and 2,500 years earthquake return periods.

( <https://www.dot.ny.gov/divisions/engineering/structures/manuals/seismic-references> )

Three sets of multiple support ground motion time-histories, for 1,000 year, 1,500 year, and 2,500 year earthquake return periods, were derived for use as inputs to seismic analyses, (as input to the bridge dynamic analysis in the time domain or as input to the soil dynamic site response analyses) for each of the three earthquake return periods (1,000 year, 1,500 year, and 2,500 year).

Providing three sets of time histories for each of the three return periods allows for accounting of uncertainties in the earthquake excitation and variations in the non-linear response of bridge components. In addition, each one of these three sets incorporates the effects of spatial variation along 21 hypothetical piers on Very Hard Rock spaced at 100 m (328 ft.), and extended over a straight line having a total length of 2 km (1.24 mile). These sets are to be used as the basis for spatial variation analyses of long-span bridges as required and described in article 4.7.4.3.4b.

These Very Hard Rock response spectra and time history records may be used either for the structural dynamic analysis of the bridge (design of the bridge) in the case of a bridge at a rock site, or as rock input to the soil in dynamic site response analyses. Whether used as input to the bridge analysis or as input to the soil site response analyses, these spectra and time histories shall be assumed to be located at the surface of an outcrop of Very Hard Rock (VHR).

Design Acceleration Response Spectra (5% damped) and associated acceleration time histories for Rock Classes A and B (see Article A3.10.3.2 for rock site classifications), shall be obtained by one of the following approaches:

- 1) Modifying the corresponding available UHS spectra and records on Very Hard Rock.
- 2) Site-specific ground response analysis if the depth and properties of softer rock over VHR are known.

The procedures for the modification of the Very Hard Rock motions are as follows:

- Design Acceleration Response Spectrum and associated acceleration time histories for Rock Class A: Multiply UHS on Very Hard Rock (see Tables A3.10.2-1 and A3.10.2-2) by a factor of 1.15; multiply the acceleration time histories on Very Hard Rock by a factor of 1.15.
- Design Acceleration Response Spectrum and associated acceleration time histories for Rock Class B: Multiply UHS on Very Hard Rock (see Tables A3.10.2-1 and A3.10.2-2) by a factor of 1.65; multiply the acceleration time histories on Very Hard Rock by a factor of 1.65.

Coefficients for horizontal spectra on Rock Classes A and B, already multiplied by 1.15 and 1.65, respectively, are included in Table A3.10.2-3 (1,000 years), Table A3.10.2-4 (1,500 years), and Table A3.10.2-5 (2,500 years). Plots of these horizontal spectra for 1,000 years, 1,500 years, and 2,500 years earthquakes are presented in Figures A.3.10.2-3, A.3.10.2-4, and A.3.10.2-5, respectively.

Coefficients for vertical spectra on Rock Classes A and B, already multiplied by 1.15 and 1.65, respectively, are included in Table A3.10.2-6 (1,000 years), Table A3.10.2-7 (1,500 years), and Table A3.10.2-8 (2,500 years). Plots of these vertical spectra for 1,000 years, 1,500 years, and 2,500 years earthquakes are presented in Figures A.3.10.2-6, A.3.10.2-7, and A.3.10.2-8, respectively.

These design spectra and time histories on Rock Class A or B, may be used either for the structural dynamic analysis of the bridge (design of the bridge) in the case of a bridge at a rock site, or as rock input to the soil in dynamic site response analyses. Whether used for design of the bridge or as input to the soil site response analyses, these spectra and time histories shall be assumed to be located at the surface of an outcrop of Rock Class A or B (see Figure A3.10.3.1-1).

Vertical design spectra are for reference only and shouldn't be used for the response spectrum analysis (general procedure in Article A3.10.4). Vertical ground motion time-histories on Rock Class A or B may be used for the structural dynamic analysis of the bridge (design of the bridge) in the case of a bridge at a rock site. Vertical motions for a bridge founded at a soil site shall be obtained using site specific procedure in Article A3.10.2.2.

#### ***A3.10.2.1 General Procedure***

For design values, the spectra in Article A3.10.4 based on Site Classification of Article A3.10.3 shall be used for both longitudinal and transverse direction of a structure.

#### ***A3.10.2.2 Site Specific Procedure***

Within the Downstate Zone, a site-specific procedure to develop design response spectra of earthquake ground motions shall be performed for the following conditions.

- When required by Article 3.10.2 of AASHTO LRFD Bridge Design Specifications.
- All bridge categories with Site Class F.
- All "Critical" bridges with Soil Sites or Rock under Soil Sites as defined in Articles A.3.10.3.2 and A.3.10.3.3.

A site-specific procedure may be performed for any bridge site at the discretion of the Bridge Owner and shall be documented in the design report.

Site specific studies shall be performed using acceleration time histories (available at NYSDOT Office of Structures website) described in Article A3.10.2 and satisfying procedures and requirements in this section. General provisions of Article 3.10.2.2 of AASHTO LRFD Bridge Design Specifications are also applicable.

The development of site-specific design response spectra, and, when needed, site-specific acceleration time-histories, shall account for the effects of the local subsurface site conditions, and, in the case of long-span bridges, also for the spatial variation of the motions (see article 4.7.4.3.4b). The influence of the local site conditions on the free field ground motions shall be based on site-specific geotechnical investigations and dynamic site response analyses.

#### ***1) Site-Specific Studies and Depth of Rock Surface, $H_r$***

Site-specific analyses shall account for the local site conditions, by incorporating parameters based on the data collected through subsurface investigations performed at the site (drilling soil borings, taking samples, laboratory testing, geophysical testing), in order to obtain the necessary information regarding local geology, establishing soil types and layering, depth to rock and dynamic properties of the soils and rock at the site. The subsurface investigation shall include in-situ seismic measurements to provide accurate seismic shear wave and compressional wave velocity values for the soil and rock.



Wave velocity measurements are required to a minimum depth of  $H_r + 20$  ft. if  $H_r \leq 100$  ft., or to a minimum depth of 100 ft. if  $H_r > 100$  ft.; where  $H_r$  is the depth of the rock surface defined in Article A3.10.3.1. In the case of  $H_r > 100$  ft., while wave velocity measurements are not required below 100 ft., they are strongly recommended if possible to a depth of  $H_r + 20$  ft., in order to reduce the uncertainty of the site-specific calculated spectra and corresponding time-histories.

For sites where the value of  $H_r$  is known and the Rock Class under the soil is also known, the horizontal and vertical ground motions at the soil surface will be determined as follows:

- *Horizontal* soil ground motions (spectra and corresponding time-histories) will be obtained from dynamic site response analyses of the soil profile, with the corresponding rock motion input placed on a rock outcrop. If needed for time-history analysis of the bridge, in addition to the time-histories calculated in the dynamic site analyses, it may also be necessary to generate time-histories to match the horizontal design spectrum on soil, as discussed in article 4.7.4.3.4b.
- *Vertical* soil ground motions (spectra) will be obtained by multiplying the corresponding final horizontal soil spectra by the appropriate period dependent V/H design ratios. These V/H ratios are listed in Tables A3.10.2.2-1, A3.10.2.2-2 and A3.10.2.2-3 for return periods of 1,000 years, 1,500 years and 2500 years, respectively, as a function of  $H_r$ , Rock Class under the soil, and Soil Site Class. These spectra can be used for the generation of vertical time-histories on soil if needed for use in time-history analysis of the bridge, see article 4.7.4.3.4b.

For bridge soil sites where the depth to rock,  $H_r > 100$  ft. and the value of  $H_r$  is unknown, the site response analyses for horizontal ground motions shall proceed by assuming two or more values of the depth to rock  $H_r$ . As a minimum, the following two values of  $H_r$  should be used: (i) the depth of the deepest measured  $V_s$ ; and (ii) a best estimate of the depth to the rock surface based on the geological and geotechnical information available to the engineer, including depths to rock in nearby sites. For these cases of estimated  $H_r$ , Rock Class B shall be assumed to exist under the soil. In addition, a conservative approach shall be taken in the selection of the horizontal design spectrum on top of the soil, including selecting an envelope of the soil spectra calculated with the various assumed  $H_r$ .

If the depth and properties of softer rock over VHR are known, the softer rock layer may be included in the site-specific analysis. In this case, the rock input shall use the VHR motions.

## 2) **Parametric Variations of $V_s$**

To account for uncertainties in the soil properties (including parameters such as  $V_s$ , nonlinear  $G/G_{\max}$  modulus reduction curve versus strain, Plasticity Index of clays, etc.), “Best Estimate” values, as well as lower range and upper range values of the soil shear wave velocities, shall be considered in the dynamic site response analyses. Therefore, three or more site response analyses must be conducted for every soil profile selected. As a minimum, the following three analyses shall be conducted: (i) analyses using the “Best Estimate”  $V_s$  for all soil layers; (ii) analyses using the “Best Estimate”  $V_s$  minus 20% for all soil layers; and (iii) analyses using the “Best Estimate”  $V_s$  plus 20% for all soil layers. The envelope spectra shall be used for design, and can be obtained by picking the maximum spectral acceleration for each period.

## 3) **Minimum Two-Thirds Rule for Selected Horizontal Design Spectrum and PGA**

If no peer review of the site-specific study is performed, final design horizontal spectrum and PGA shall not be less than two-thirds of the corresponding site “generic” spectrum and PGA. This requirement applies to both Critical and Non-Critical Bridges. For the purpose of applying the two-thirds rule, the following shall apply:

- Soil Class F due to soil liquefaction: the site shall be re-classified as Soil Class C, D or E following the procedures in Article A3.10.3.3 and ignoring liquefaction.
- Soil Class F due to reasons other than liquefaction: the generic spectra and PGA will be selected for the appropriate combination of  $H_r$ , Rock Class under the soil, and Soil Class D.

## 4) **Soil-Structure Interaction Studies**

Soil-structure interaction effects must be included whenever necessary at a Soil or Rock Site.

## 5) **Critical Bridges**

For critical bridges, site-specific soil effects, including the possible effect of soil liquefaction, shall be incorporated through dynamic site response analyses. Spatial variation effects shall also be evaluated in the case of long-span bridges. Selection of rock design spectra for bridges at rock sites, dynamic site response analyses at soil sites, and spatial variation analyses, shall comply with the pertinent sections of these specifications.

Design Acceleration Response Spectra (5% damped) for the 2,500 year earthquake on soil and, when needed, associated time-histories obtained from site-specific dynamic site response analyses, will be selected as follows:

- *Horizontal ground motions for design and liquefaction evaluation:* Site-specific dynamic site response analyses shall be conducted using the rock input in accordance with Articles A.3.10.2 and 4.7.4.3.4b. The horizontal design spectrum and design peak ground acceleration (PGA) will be selected at the soil surface from the results of these analyses. The potential for liquefaction during the 2,500 year earthquake shall be evaluated in accordance with Article A.3.10.2.2(7), assuming an earthquake of magnitude  $M = 6.25$ .

If it is determined that liquefaction will occur, the effect of both liquefied and non-liquefied configurations must be considered on the design horizontal spectrum and PGA. After selection of the final design horizontal spectrum and design horizontal PGA, if needed for time-history analyses, acceleration time-histories calculated as part of the dynamic response analyses may be used, or modified as needed, to match the final design horizontal spectrum. If this is not possible, acceleration time-histories shall be obtained that match the final design horizontal spectrum using appropriate commercially available software (see article 4.7.4.3.4b).

- *Vertical ground motions for design:* The vertical design spectrum for the 2,500 year earthquake will be obtained by multiplying the site-specific final design horizontal spectrum and PGA by the appropriate period dependent V/H ratios in Table A.3.10.2.2-3. If needed for time-history analyses of the bridge, acceleration time-histories may be obtained from matching the design vertical spectrum, using appropriate commercially available software.

The design criteria for the 1,000 year earthquake provides for minimal damage and normal service following a post-earthquake inspection. Thus, the engineer shall evaluate the response of the bridge to this event for compliance to the serviceability requirements. Only one analysis may be necessary to assess the performance requirements associated with this event. Design Acceleration Response Spectra (5% damped) for the 1,000 year earthquake for soil, and when needed, also associated time-histories obtained from site-specific dynamic site response analyses will be selected as follows:

- *Horizontal ground motions for design:* Site-specific dynamic site response analyses will be conducted using the rock input in accordance with Articles A.3.10.2 and 4.7.4.3.4b. The horizontal design spectrum and PGA will be selected at the soil surface from the results of these analyses. If needed for time-history analyses of the bridge, acceleration time-histories calculated as part of the dynamic response analyses may be used, or modified as needed, to match the final design horizontal spectrum. If this is not possible, acceleration time-histories shall be obtained that match the final design horizontal spectrum, using appropriate commercially available software.
- *Vertical ground motions for design:* The vertical design spectrum for the 1,000 year earthquake will be obtained by multiplying the site-specific final design horizontal spectrum and PGA by the appropriate period dependent V/H ratios in Article A.3.10.2.2-1. If needed for time-history analyses of the bridge, acceleration time-histories shall be obtained by matching the final design vertical spectrum using appropriate commercially available software.

Peer review of site specific spectra shall be carried out as per the requirements in Article A.3.10.2.3.

## 6) *Non-Critical Bridges (Essential and Others)*

Non-critical Bridges shall be analyzed for a single earthquake hazard level, corresponding to a 7% probability of being exceeded in 75 years (1,000 year return period). The owner has the option to use a single earthquake hazard level, corresponding to a 5% probability of being exceeded in 75 years (1,500 year return period) instead. Site-specific soil effects, including the possible effect of soil liquefaction, shall be incorporated through dynamic site response analyses, and spatial variation effects shall also be evaluated in the case of long-span bridges. Selection of rock design spectra for bridges at rock sites, as well as dynamic site response analyses at soil sites, and spatial variation analyses, shall comply with the specifications in Articles A.3.10.2 and 4.7.4.3.4b.

Design Acceleration Response Spectra (5% damped) for the design earthquake on soil and, when needed, associated time-histories obtained from site-specific dynamic site response analyses, will be selected as follows:

- *Horizontal ground motions for design and liquefaction evaluation:* Site-specific dynamic site response analyses will be conducted using the rock input in accordance with Articles A.3.10.2 and 4.7.4.3.4b. The horizontal design spectrum and design peak ground horizontal acceleration (PGA) will be selected at the soil surface from the results of these analyses. The potential for liquefaction during the design earthquake shall be evaluated in accordance with Article A.3.10.2.2(7), and assuming if necessary, an earthquake of magnitude  $M = 6$ . If it is determined that liquefaction will occur, the effect of both liquefied and non-liquefied configurations must be considered on the design horizontal spectrum and PGA (see Article A.3.10.2.2(7)). If the liquefaction evaluation indicates no liquefaction, the liquefaction evaluations shall be repeated using the procedures in Article A.3.10.3.4 using surface horizontal peak ground acceleration (PGA) equal to either the site-specific value, or two thirds of the corresponding PGA in Table A.3.10.3-1, whichever is greater. An earthquake magnitude,  $M = 6$  will be used in this liquefaction re-evaluation. If this re-evaluation indicates that liquefaction will occur, the occurrence of liquefaction will be considered in the design of the bridge, including its effects on the response spectra (see Article A.3.10.2.2(7)).

*A final design horizontal spectrum and PGA will be selected* that is at all periods equal or greater than two thirds of the corresponding generic spectrum and PGA. After selection of the horizontal design spectrum and design horizontal PGA, if needed for time-history analyses, acceleration time-histories calculated as part of the dynamic response analyses may be used, or modified as needed, to match the final design horizontal spectrum. If this is not possible, acceleration time-histories shall be obtained that match the final design horizontal spectrum, using appropriate commercially available software (Article 4.7.4.3.4b).

If the engineer determines that liquefaction will occur for a site that was originally classified as Soil Class C, the generic spectra used as a basis for the two thirds rule in the range of periods greater than 0.5 sec will be recalculated based on the appropriate combination of  $H_r$ , Rock Class under the soil, and Soil Class D. The spectrum in the range of periods below 0.5 sec, including the PGA, and computed using Soil Class C, shall not be affected by this recalculation.

- *Vertical ground motions for design:* The design vertical spectrum for the 1,000 year or 1,500 year earthquake will be obtained by multiplying the site-specific final design soil horizontal spectrum and PGA by the appropriate period dependent V/H ratios in Table A3.10.2.2-1 (for 1,000 year) and Table A3.10.2.2-2 (for 1,500 year). The final design horizontal spectrum to be used for the calculations should be that already adjusted by the two thirds rule. If needed for time-history analyses of the bridge, acceleration time-histories shall be obtained by matching the final design vertical spectrum using appropriate commercially available software.

Peer review of site specific spectra shall be carried out as per the requirements in Article A.3.10.2.3.

#### 7) **Liquefaction Evaluation and Effect on Ground Motions**

If liquefiable layers are present at a soil site for any bridge, one of the objectives of the site-specific study is to conduct the corresponding liquefaction evaluation. The liquefaction evaluation shall be performed after performing site response analyses, assuming that liquefaction does not occur. The liquefaction assessment may use either the PGA or other parameters from the site response analyses (e.g., shear stresses or strains) selected by the engineer to quantify the earthquake demand. Liquefaction assessment may be carried out following the procedures in Article A3.10.3.4 or by a more advanced approach selected by the engineer. If it is determined that liquefaction will occur at the bridge site for the given earthquake level, and if the site response analyses do not already incorporate the effect of excess pore water pressures on the soil ground motions, then additional site response analyses may have to be performed to consider the effect of liquefaction on the soil ground motions. If it is determined that liquefaction will occur, the bridge site shall be analyzed for two configurations: (i) non-liquefied configuration, where the site is analyzed assuming no pore pressure buildup and no liquefaction; and (ii) liquefied configuration, where the site is reanalyzed assuming that liquefaction occurs in the liquefiable soil layers.

The ground motion spectra shall be the envelope of the results calculated with both non-liquefied and liquefied configurations. If time-history analyses are needed, the provisions of Article 4.7.4.3b shall be followed.

#### ***A3.10.2.3 Peer Review of Site Specific Studies***

##### **1) Critical Bridges**

A peer review of a site-specific study shall be performed by an owner-approved independent engineer, who has experience in performing site specific seismic analyses. The engineer or engineering panel performing the peer review shall submit a final written report that summarizes the conclusions and recommendations.

If the owner decides to waive the requirement of a peer review for the site-specific study of a Critical Bridge, the selected 2,500 year and 1,000 year horizontal design spectra must comply with the minimum two thirds rule in Article A3.10.2.2(3) using the generic spectra for 2,500 year and 1,000 year return periods in Article A.3.10.4 as the baseline for the two-thirds requirement.

If no peer review is performed, and the liquefaction evaluation conducted as part of the site-specific study following Article A3.10.2.2(7) indicates no liquefaction, the liquefaction evaluation shall be repeated with the procedures in Article A3.10.3.4 using a surface horizontal peak ground acceleration (PGA) equal to either that calculated in the site-specific study, or two-thirds of the corresponding PGA for 2,500 years in Table A3.10.3-1, whichever is greater. An earthquake magnitude  $M = 6.25$  will be used in this liquefaction re-evaluation. If this re-evaluation indicates that liquefaction will occur, the occurrence of liquefaction shall be considered in the design of the bridge, including its effects on the response spectra (see Article A3.10.2.2(7)). The final design soil vertical spectra will be computed using the corresponding V/H ratios from the final design soil horizontal spectra after the horizontal spectra have been adjusted to comply with the two thirds rule.

If no peer review is performed, and the engineer determines that liquefaction will occur for a site that was originally classified as Soil Class C, the generic spectra used as a basis for the two thirds rule in the range of periods greater than 0.5 sec will be recalculated based on the appropriate combination of  $H_r$ , Rock Class under the soil, and Soil Class D. The spectrum in the range of periods below 0.5 sec, including the PGA, and computed using Soil Class C, shall not be affected by this recalculation.

## 2) **Non-Critical Bridges**

The owner has the option to require a peer review of a site-specific study. This review shall be performed by an owner-approved independent engineer who has experience in performing site specific seismic analyses. The engineer or engineering panel performing the peer review shall submit a final written report that summarizes the conclusions and recommendations.

If a peer review is performed, and the site-specific liquefaction evaluation as per Article A3.10.2.2(7) indicates no liquefaction, the requirement for a liquefaction re-evaluation using the procedure in Article A3.10.3.4 is waived. That is, the conclusion of no liquefaction obtained from the site-specific study may stand, provided that the engineer or engineering panel performing the review approves this in the final written report.

If a peer review is performed for a Non-Critical Bridge, requirements of Article A3.10.2.2(3) related to the two thirds rule are waived. That is, the final design soil horizontal and vertical spectra and PGA selected for the 1,000 year earthquake from the site-specific study may be lower than that required by the two-thirds rule, provided that the engineer or engineering panel performing the review approves this in the final written report.

### A3.10.3 Site Effects

Site classes specified herein shall be used in the General Procedure for Seismic Hazard (Design Response) Spectrum specified in Article A3.10.4.

A site is classified as a soil site if there is more than 10.0 ft. of soil between the bottom of spread footing or pile cap and the rock surface; otherwise it is classified as a rock site. The specific definitions for rock surface determination are shown in Article A3.10.3.1. Rock sites are classified as Rock Site Class VHR, A, or B, as described in Article A3.10.3.2. Soil sites are classified as Soil Site Class C, D, E or F, as described in Article A3.10.3.3. Article A3.10.3.4 describes the site liquefaction assessment procedure.

#### A3.10.3.1 Rock Surface Determination

The depth of the rock surface below the ground surface of the site is labeled  $H_r$  in these Guidelines (see Figure A3.10.3.1-1).

The “rock surface” is defined as the shallowest depth for which the following three conditions are concurrently satisfied:

1. The geotechnical material in the 20.0 ft. immediately below the rock surface is either rock material or cemented or very dense soil with an average shear wave velocity  $\bar{V}_{s20} > 2,500$  ft./sec (see Table A3.10.3.1-1 and Figure A3.10.3.1-1). The average shear wave velocity  $\bar{V}_{s20}$  is defined later in this Article.

In the case of cemented or very dense material, determination that the rock surface has been reached, for the purposes of these provisions, shall be based only on the actual measured  $\bar{V}_{s20} > 2,500$  ft./sec in the 20.0 ft. below the assumed rock surface. In the absence of shear wave velocity measurements, the cemented or very dense material shall be considered to be a soil layer above the rock surface. In the case of rock material, determination that the rock surface has been reached shall be based preferably on shear wave velocity measurements in the 20 ft. below the assumed rock surface. However, for competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured; otherwise, it shall be assumed that the rock surface has not yet been reached and the highly fractured and weathered rock shall be considered to be a soil layer above the rock surface.

2. The value of  $V_s$  at each individual sublayer within the 20 ft. below the rock surface is at least 2,000 ft/sec.
3. Based on the geotechnical characteristics at the site, the Engineer can reasonably assume that the  $V_s$  profile below the rock surface will not decrease below  $\bar{V}_{s20} = 2,500$  ft/sec, with  $V_s$  eventually increasing with depth until the very hard rock elevation ( $V_s > 9,000$  ft/sec) is reached.

Determining that the rock surface has been reached requires calculating  $\bar{V}_{s20}$ , which is defined as follows (see Figure A3.10.3.1-1):

$$\bar{V}_{s20} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{V_{si}}} \quad \text{where} \quad \sum_{i=1}^n d_i = 20.0 \text{ ft.}$$

The symbol  $i$  refers to any one of the sublayers within the 20.0 ft. immediately below the rock surface, with measured shear wave velocity  $V_{si}$  and thickness  $d_i$ , from 1 to  $n$  (where  $n$  is the total number of sublayers). Each  $V_{si}$  must be at least 2,000 ft./sec.

### A3.10.3.2 Rock Site & Rock Under Soil Site Classification

#### 1) **Rock Site**

For a Rock Site, the Rock Class shall be defined as follows (see Table A3.10.3.1-1):

- VHR: Very Hard Rock with measured averaged shear wave velocity,  $\bar{V}_{s20} > 9,000$  ft./sec.
- Rock Class A: Hard Rock with measured averaged shear wave velocity,  $5,000 < \bar{V}_{s20} \leq 9,000$  ft./sec.
- Rock Class B: Rock material or cemented or very dense soil with averaged shear wave velocity  $2,500 < \bar{V}_{s20} \leq 5,000$  ft./sec.

In the case of cemented or very dense material (e.g., very dense glacial till), classification as Rock Class B shall be based only on shear wave velocity measurements. In the absence of shear wave velocity measurements, the cemented or very dense material shall be considered to be a soil layer above the rock surface, as defined in Article A3.10.3.3.

In the case of rock material, classification as Rock Class B shall be based preferably on shear wave velocity measurements. However, for competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured or the highly fractured and weathered rock shall be considered a soil layer above the rock surface.

Assignment of either Rock Class A or VHR shall be based on either in-situ shear wave velocity measurements, or shear wave velocity measurements on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing.

If the measured shear wave velocities indicate Rock Class B, but Rock Class A (or VHR) is found by drilling at a depth below the rock surface not greater than 40 ft, the Engineer shall have the option to classify the site as Rock Site Class A (or VHR) instead of Rock Site Class B, for the purpose of selecting the generic spectra in Article A3.10.4 (see Figure A3.10.3.2-1). The decision to drill deeper than 20 ft. below the rock surface is optional (see Article A3.10.3.1).



## 2) **Rock Under Soil Site**

For a Soil Site with  $H_r < 100$  ft., the selection of design spectra in Article A3.10.4 and the selection of Peak Ground Acceleration (PGA) for liquefaction evaluation (see Article A3.10.2.2), require classification of the rock under the site as either Rock Class A/VHR or B. The same definition of  $\bar{V}_{s20}$  (see Figure A3.10.3.1-1), and the same ranges of  $\bar{V}_{s20}$  given for Rock Sites in Article A3.10.3.2(1), shall be used. Classification of Rock under Soil Sites is as follows (see Table A3.10.3.1-1):

- Rock Class A/VHR: Hard to Very Hard Rock with measured shear wave velocity  $\bar{V}_{s20} > 5,000$  ft./sec.
- Rock Class B: Rock material or cemented or very dense soil with shear wave velocity  $2,500 < \bar{V}_{s20} \leq 5,000$  ft./sec.

In the case of cemented or very dense material (e.g., very dense glacial till), classification as Rock Class B shall be based only on shear wave velocity measurements. In the absence of shear wave velocity measurements, the cemented or very dense material shall be considered to be a soil layer above the rock surface.

In the case of rock material, classification as Rock Class B shall be based preferably on shear wave velocity measurements. However, for competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured or the highly fractured and weathered rock shall be considered a soil layer above the rock surface.

Assignment of Rock Class A/VHR shall be based on either in-situ shear wave velocity measurements, or shear wave velocity measurements on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing.

If  $H_r \leq 100$  ft. and the measured averaged shear wave velocity  $\bar{V}_{s20}$  indicates Rock Class B directly beneath the soil, but Rock Class A/VHR is found by drilling at a depth below the rock surface not greater than 40.0 ft., the Engineer shall have the option to specify that the soil profile is on top of Rock Class A/VHR instead of Rock Class B (see Figure A3.10.3.2-1), for the purpose of selecting the generic spectral accelerations (see Article A3.10.4), and the Peak Ground Acceleration (PGA) for liquefaction evaluation (see Article A3.10.2.2). To exercise this option, drilling must continue at least 20.0 ft. below the Rock Class A/VHR surface shown in Figure A3.10.3.2-1.

### A3.10.3.3 Soil Sites Classification

#### 1) Soil Site Classes C, D, E, F

A soil site is defined as one where the distance between the rock surface and the bottom of spread footing or pile cap is greater than 10.0 ft.

Soil Site Classes shall be characterized on the basis of average soil properties. The sites of Critical Bridges shall only be classified using the measured soil shear wave velocity  $V_s$ . The preferred soil property used for the Non-Critical Bridges is also the soil shear wave velocity. Other alternative soil properties that may be used for Non-critical bridges are the Standard Penetration Test (SPT) resistance  $N$  ( $N_{ch}$  for cohesionless soils) and the undrained shear strength  $s_u$  for cohesive soils. The corresponding average soil properties  $\bar{V}_s$ ,  $\bar{N}$ ,  $\bar{N}_{ch}$ , and  $\bar{s}_u$  are specifically defined in Table A3.10.3.3-1. Definitions of these average soil properties are provided in subsection 2). The Soil Classes used to classify a Soil Site are defined as follows:

- **Soil Class C:** Very dense soil with shear wave velocity,  $1,200 < \bar{V}_s \leq 2,500$  ft./sec or with either standard blow count ( $\bar{N}$  or  $\bar{N}_{ch}$ )  $> 50$ , or undrained shear strength  $\bar{s}_u > 2,000$  psf. Soil above the rock surface which has a  $\bar{V}_s > 2,500$  ft./sec due to the presence of cemented or very dense soil layers, but does not meet the requirements of a rock site, as specified in Articles A3.10.3.1 and A3.10.3.2, shall be classified as Soil Class C.
- **Soil Class D:** Stiff soil with  $600 \leq \bar{V}_s \leq 1,200$  ft/sec or with  $15 \leq (\bar{N} \text{ or } \bar{N}_{ch}) \leq 50$  or  $1,000 \leq \bar{s}_u \leq 2,000$  psf.
- **Soil Class E:** A soil profile with  $\bar{V}_s < 600$  ft./sec, or any profile at a soil site with more than 10 feet of soft clay defined as soil with plasticity index  $PI > 20$ , water content  $w \geq 40\%$ , and  $\bar{s}_u < 500$  psf, or any profile with  $\bar{N}$  or  $\bar{N}_{ch} < 15$ , or  $\bar{s}_u < 1,000$  psf.
- **Soil Class F:** Soils requiring site-specific evaluations:
  1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils. The site of a Critical Bridge will be classified as Soil Class F if it will liquefy during the 2,500 year earthquake, as determined according to Article A3.10.2.2. The site of an Essential or Other Bridge will be classified as Soil Class F if it will liquefy during the 1,000 year earthquake (or 1,500 year earthquake, optionally), as determined according to Article A3.10.2.2.
  2. Peats and/or highly organic clays ( $H > 10.0$  ft. of peat and/or highly organic clay, where  $H$  = total thickness of soil layers with those characteristics).
  3. Very high plasticity clays ( $H > 25.0$  ft. with  $PI > 75$ ).
  4. Very thick soft/medium stiff clays ( $H > 120.0$  ft.) with  $\bar{s}_u < 1,000$  psf.

Table A3.10.3.3-1 summarizes the classification of Soil Sites.

## 2) Definitions of Soil Class Parameters

The definitions presented in this Article apply to soil profiles where the rock surface, as defined in Article A3.10.3.1, is located more than 10.0 ft. below the bottom of a spread footing or pile cap. If the depth of rock surface  $H_r \leq 100$  ft., only the properties of the soil between the ground surface and the rock surface are used. If  $H_r > 100$  ft., only the properties of the soil between the ground surface and a depth of 100 ft. are used. Therefore, definition of the Soil Class at a soil site does not require subsurface exploration deeper than 100 ft. (see Figure A3.10.3.3-1), except in cases of very thick soft/medium stiff clays (i.e. Site Class F). Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to  $n$  at the bottom, where there are a total of  $n$  distinct layers down to the depth  $H_r$  or down to a depth of 100 ft., depending on the case. The symbol  $i$  refers to any one of the layers between 1 and  $n$ .

The average shear velocity  $\bar{V}_s$  is calculated as

$$\bar{V}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{V_{si}}}$$

where  $V_{si}$  is the shear wave velocity of a layer in ft/sec,  $d_i$  is the thickness of any layer between 0.0 ft. and 100 ft. and  $\sum_{i=1}^n d_i$  shall not be greater than 100 ft. If the depth to rock  $H_r$  is less than 100 ft., then  $\sum_{i=1}^n d_i = H_r$ .

The average standard penetration resistance  $\bar{N}$  is calculated as

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}}$$

where  $N_i$  is the Standard Penetration Resistance determined in accordance with ASTM D1586, as directly measured in the field without corrections, and shall not be taken greater than 100 blows/ft. In the equation above,  $N_i$  and  $d_i$  are for cohesionless and cohesive soil layers (i.e. rock shall not be included) within a total depth not exceeding 100 ft. If the depth to rock  $H_r$  is less than 100 ft., then  $\sum_{i=1}^n d_i = H_r$ .

For cohesionless soils,  $\bar{N}_{ch}$  is calculated as:

$$\bar{N}_{ch} = \frac{\sum_{i=1}^m d_i}{\sum_{i=1}^m \frac{d_i}{N_i}}$$

where  $\sum_{i=1}^m d_i = d_s$  is the total thickness of cohesionless soil layers. Use  $d_i$  and  $N_i$  only for cohesionless soils within a total depth not exceeding 100 ft. for layers from 1 to  $m$ .

Average undrained shear strength  $\bar{s}_u$  is calculated as:

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^k \frac{d_i}{s_{ui}}}$$

where  $s_{ui}$  is the undrained shear strength in psf, and shall be determined in accordance to ASTM D2166 or ASTM D2850. It shall not exceed 5,000 psf.  $d_c = \sum_{i=1}^k d_i$  is the total thickness of cohesive soil top layers within a total depth not exceeding 100 ft., from 1 to  $k$ , and  $d_s + d_c \leq 100$  ft.

Plasticity Index (PI) is determined in accordance to ASTM D4318.

Moisture content (w), in percent, is determined in accordance to ASTM D2216.

### 3) **Steps for Classifying Soil Site as Class C, D, E or F**

The steps described below are to be taken after establishing that the site is a Soil Site rather than a Rock Site (see Articles A3.10.3.1 and A3.10.3.2). The definitions of  $\bar{V}_s$ ,  $\bar{N}$ ,  $\bar{N}_{ch}$  and  $\bar{s}_u$ , used below, are given in the previous subsection.

Step 1: Check in Part 1 of Article A3.10.3.3 for the four categories of Soil Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Soil Class F.

Step 2: Check for the existence of a total thickness of soft clay  $> 10$  ft. where a soft clay layer is defined by:  $\bar{s}_u < 500$  psf,  $w \geq 40\%$ , and  $PI > 20$ . If these criteria are satisfied, classify the Site as Class E.

Step 3: Categorize the site using one of the following three methods with  $\bar{V}_s$ ,  $\bar{N}$ , or  $\bar{N}_{ch}$  and  $\bar{s}_u$  computed in all cases as specified in subsection A3.10.3.3(2):

- i.  $\bar{V}_s$  for the soil layers, computed from the ground surface down to a depth of  $H_r$ , with a maximum depth of 100 ft. if  $H_r > 100$  ft. ( $\bar{V}_s$  method). Classify the site as Soil Class C, even if  $\bar{V}_s > 2,500$  ft/sec, when this is due to the presence of high shear wave velocity cemented or very dense intermediate soil sub-layers above the rock surface (see Article A3.10.3.1).
- ii.  $\bar{N}$  for the soil layers computed from the ground surface down to a depth of  $H_r$ , with a maximum depth of 100 ft. if  $H_r > 100$  ft. ( $\bar{N}$  method).
- iii.  $\bar{N}_{ch}$  for cohesionless soil layers ( $PI < 20$ ), and  $\bar{s}_u$  for cohesive soil layers ( $PI > 20$ ), with both  $\bar{N}_{ch}$  and  $\bar{s}_u$  computed from the ground surface down to a depth of  $H_r$  with a maximum depth of 100 ft. if  $H_r > 100$  ft. ( $\bar{s}_u$  method).

$V_s$  data are required for site classification of Critical Bridges. For Non-critical bridges, where reliable  $V_s$  data (i.e. based on field measured  $V_s$  data) are available for the site,  $\bar{V}_s$  shall be used to classify the site. If  $\bar{V}_s$ ,  $\bar{N}$ ,  $\bar{N}_{ch}$  and  $\bar{s}_u$  criteria listed above are used and two different Soil Classes are obtained for the site, both Soil Classes shall be used in Article A3.10.4 to select the soil horizontal and vertical spectra and the Peak Ground Acceleration (PGA) for liquefaction evaluation (see Article A3.10.3.4). In such a case, the envelope of both curves in the horizontal and vertical directions shall be used for design. The envelope spectra shall be used for design, and can be obtained by picking the maximum spectral acceleration for each period. For example, when selecting the spectral acceleration values for a Non-Critical bridge, if a site of  $H_r > 100$  ft. can be classified either as Soil Class C or Soil Class D, then the Soil Class C spectral accelerations shall be used for periods lower than  $T = 0.5$  sec, and the Soil Class D spectral accelerations shall be used for periods higher than  $T = 0.5$  sec.

#### **A3.10.3.4 Site Liquefaction**

Liquefaction design requirements of AASHTO LRFD Article 10.5.4.2 shall be used in conjunction with “Downstate Zone” provisions below. The provisions below shall govern over any requirements of AASHTO LRFD Article 10.5.4.2. Specifically, the evaluations of liquefaction potential of these Guidelines shall be performed irrespective of the Seismic Performance Zone assigned to the bridge.

Soil liquefaction assessment shall be required for those “Downstate Zone” bridges where the geotechnical investigation indicates the presence of potentially liquefiable saturated soils and following the specific requirements for Critical and Non-Critical bridges. This assessment shall include:

1. Possible occurrence of liquefaction.
2. Effect of liquefaction on the dynamic ground motions and spectra used for design.
3. Effects of liquefaction-induced soil deformations and decreases in the stiffness and strength of the soil on the performance and capacity of the foundations and structure of the bridge.

For items 1 and 2, the evaluation of liquefaction potential at bridge sites, as a minimum, shall follow the procedures of the updated Seed and Idriss “Simplified Procedure” summarized in the paper by Youd, T.L. et al. (2001), “Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils,” *Journal of Geotechnical & Geo-environmental Engineering*, ASCE, 127(10): 817-833, but the liquefaction susceptibility of soils with plastic fines in Youd et al. (2001) shall not be used. It should be replaced by either the one in Bray and Sancio (2006), “Assessment of the liquefaction susceptibility of fine-grained soils,” *Journal of Geotechnical and Geoenvironmental engineering*, ASCE, 132(9): 1165–1177, or the one in Boulanger and Idriss (2006), “Liquefaction susceptibility criteria for silts and clays,” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 132(11): 1413-1426.

It is recommended that Item 3 should follow, as much as possible, the recommendations contained in the Liquefaction Study Report MCEER/ATC-49-1 (prepared under NCHRP Project 12-49, 2003), with such assessment including, as needed, soil-foundation and soil-structure interaction analyses, in order to evaluate the effects of liquefaction and large ground deformations on the performance of the bridge. Item 3 may also be addressed using a more updated state of the practice as approved by the Bridge Owner. The methodology adopted for item 3 shall be documented in the design report.

The following provides specific requirements for the evaluation of liquefaction potential for Critical and Non-Critical Bridges.

**Critical Bridges:** a site-specific study to select the design spectra is required, regardless of Soil Class. Liquefaction assessment shall be performed including items 1, 2, and 3 mentioned above.

**Non-Critical Bridges:** The Soil Class, depth to the rock surface, and Rock Class under the soil, shall be determined in accordance with Articles A3.10.3.1 and A3.10.3.2. When determining the Soil Class, the potential for liquefaction should be initially ignored. If the site is classified as Soil Class F for reasons other than liquefaction (see Article A3.10.3.3), then generic spectra may not be used for design, and a site-specific study needs to be conducted with liquefaction potential being evaluated as part of the site response analyses.

For Non-Critical Bridges (Essential and Others) that do not include a site-specific study, an evaluation of liquefaction potential shall be performed for the 1,000 year earthquake using the generic PGA values listed in Table A3.10.3-1 and an earthquake magnitude  $M = 6.0$ . The owner may instead require an evaluation of liquefaction potential for the 1,500 year earthquake, using the generic PGA values listed in Table A3.10.3-1 and an earthquake magnitude  $M = 6.0$ .

Possible outcomes of this liquefaction evaluation may be:

- No liquefaction will occur during the design earthquake. In this case, the effects of liquefaction do not have to be considered in the design.
- Liquefaction will occur during the design earthquake. In this case, the site classification shall be switched to Soil Class F and a site-specific study shall be conducted. The potential for liquefaction shall be reevaluated using the site specific results. If liquefaction is estimated to occur in the site-specific study, site response analyses taking into account the effect of liquefaction must be performed according to Articles A3.10.2.2(6) and A3.10.2.2(7)

- If the owner decides to use a site-specific study to define the design ground motions for a Non-Critical Bridge, instead of using the generic design spectra from Article A3.10.4, then the liquefaction evaluation shall be performed according to Articles A3.10.2.2(6) and A3.10.2.2(7)

#### **A3.10.4 General Procedure - Seismic Hazard (Design Response) Spectrum**

This section provides guidelines on the selection of 5% damped generic horizontal and vertical response spectra for rock and soil sites. If needed, vertical response quantities shall be calculated using acceleration time-histories in 4.7.4.3.4b. Vertical response spectral shall only be used for reference purposes (i.e., comparing with spectra of generated ground motions) or for generating spectrum compatible ground motion time-histories.

##### **1) Generic Horizontal Design Spectra for Rock Site Classes VHR, A and B**

Non-Critical Bridges (Essential or Others) founded on a Rock Site, at a minimum, shall be analyzed using the 1,000 year return period (7% probability of being exceeded in 75 years) rock generic 5% damped spectra presented in Table A3.10.2-3 (Figure A3.10.2-3). The owner has the option to require the analysis of these bridges using the 1,500 year return period (5% probability of being exceeded in 75 years) rock generic 5% damped spectra presented in Table A3.10.2-4 (Figure A3.10.2-4) instead.

Critical Bridges founded on a Rock Site shall be analyzed using the following spectra:

- 2,500 year return period (3% probability of being exceeded in 75 years) rock generic 5% damped spectra presented in Table A3.10.2-5 (Figure A3.10.2-5).
- 1,000 year return period (7% probability of being exceeded in 75 years) rock generic 5% damped spectra presented in Table A3.10.2-3 (Figure A3.10.2-3).
- Optionally, the owner may decide to use a site-specific study (see Article A3.10.2.2) including Soil-Structure Interaction effects for a Critical Bridge at a Rock Site, using the rock input motions (see Article A3.10.2). In that case, the seismic design will be based on the results of the site-specific study. Site-specific ground response analysis for rock sites can be carried out if the depth and properties of softer rock over VHR are known. The site-specific procedure described in Article A3.10.2.2 can be used for this purpose.

##### **2) Generic Horizontal Design Spectra for Soil Site Class C, D, and E**

The soil generic horizontal design spectra are presented in Table A3.10.4-1 and Figures A3.10.4-1 to A3.10.4-3 for 1,000 year return period, in Table A3.10.4-2 and Figures A3.10.4-4 to A3.10.4-6 for 1,500 year return period, and in Table A3.10.4-3 and Figures A3.10.4-7 to A3.10.4-9 for 2,500 year return period as a function of three parameters:

- The depth to the rock surface  $H_r$ , as defined in Article A3.10.3.1
- The Rock Class (A/VHR, B or Deep Rock of Any Type) immediately underneath the rock surface, as defined in Article A3.10.3.2
- The Soil Site Class (C, D or E), as defined in Article A3.10.3.3

Non-Critical Bridges (Essential or Others) founded on a Soil Site Class C, D or E, at a minimum, shall be analyzed using the 1,000 year return period (7% probability of being exceeded in 75 years) soil generic 5% damped horizontal spectra presented in Table A3.10.4-1 and Figures A3.10.4-1 to A3.10.4-3. The owner has the option to require the analysis of these bridges using the 1,500 year return period (5% probability of being exceeded in 75 years) soil generic 5% damped horizontal spectra presented in Table A3.10.4-2 and Figures A3.10.4-4 to A3.10.4-6, instead.

If insufficient data are available to classify a soil profile under a non-critical bridge, Soil Site Class D shall be assumed. For periods less than 0.5 seconds, spectral accelerations for “Soil Class D on top of Rock Class B,  $H_r \leq 100$  ft. shall be used. For periods larger than 0.5 seconds, spectral accelerations for “Soil Class D on top of Deep Rock of Any Type” shall be used.

3) **Generic Vertical Design Spectra for Rock Site Classes VHR, A and B**

When necessary, Non-Critical Bridges (Essential or Others) founded on a Rock Site, at a minimum, shall be analyzed using the 1,000 year return period (7% probability of being exceeded in 75 years) rock vertical generic 5% damped spectra presented in Table A3.10.2-6. The owner has the option to require the analysis of these bridges using the 1,500 year return period (5% probability of being exceeded in 75 years) rock vertical generic 5% damped spectra in Table A3.10.2-7, instead.

Unless the bridge owner decides that a site specific and/or soil-structure interaction study is justified, Critical Bridges founded on a Rock Site, shall be analyzed using the following spectra when necessary:

- 2,500 year return period (3% probability of being exceeded in 75 years) rock vertical generic 5% damped spectra presented in Table A3.10.2-8.
- 1,000 year return period (7% probability of being exceeded in 75 years) rock vertical generic 5% damped spectra presented in Table A3.10.2-6.

Optionally, the engineer may decide to conduct a site-specific study, see 1) above, including Soil-Structure Interaction effects for a Critical Bridge at a Rock Site, using the rock input motions listed see Article A3.10.2. In that case, the seismic design will be based on the results of the site-specific study following the specifications of see Article A3.10.2.2.

4) **Generic Vertical Design Spectra for Soil Site Class C, D, and E**

The soil generic vertical design spectra for Non-critical and Critical Bridges are presented in Tables A3.10.4-4 to A3.10.4-6 as a function of three parameters:

- The depth to the rock surface ( $H_r$ ), as defined in A3.10.3.1.
- The Rock Class (A/VHR or B) immediately underneath the rock surface, as defined in A3.10.3.2.
- The Soil Site Class (C, D or E), as defined in A3.10.3.3.



When necessary, non-critical Bridges (Essential or Others) founded on a Soil Site Class C, D or E, at a minimum, shall be analyzed using the 1,000 year return period (7% probability of being exceeded in 75 years) soil vertical generic 5% damped spectra presented in Table A3.10.4-4 (Figures A3.10.4-10 to A3.10.4-12). The owner also has the option to require the analysis of these bridges using the 1,500 year return period (5% probability of being exceeded in 75 years) soil vertical generic 5% damped spectra presented in A3.10.4-5 (Figures A3.10.4-13 to A3.10.4-15), instead.

A non-critical bridge founded on a Soil Site with a known depth to the rock surface,  $H_r \leq 100$  ft., shall be analyzed using the corresponding spectrum for  $H_r \leq 100$  ft., Soil Site Class C, D or E, on top of Rock Class A/VHR or Rock Class B in Table A3.10.4-4 (Figures A3.10.4-10 or A3.10.4-11) for 1,000 year return period or in Table A3.10.4-5 (Figures A3.10.4-13 or A3.10.4-14) for 1,500 year return period.

A non-critical bridge founded on a Soil Site with depth to the rock surface,  $H_r > 100$  ft., whether  $H_r$  is known or unknown, shall be analyzed using the corresponding spectrum for Soil Site Class C, D or E, labeled “Soil on Top of Deep Rock of Any Type” in Table A3.10.4-4 (Figure A3.10.4-12) for 1,000 year return period or in Table A3.10.4-5 (Figure A3.10.4-15) for 1,500 year return period.

If insufficient data are available to classify a soil profile under a non-critical bridge, Soil Site Class D shall be assumed. For periods less than 0.5 seconds, spectral accelerations for Soil Class D,  $H_r \leq 100$  ft. and soil on top of Rock Class B shall be used. For periods larger than 0.5 seconds, spectral accelerations for Soil Class D on top of Deep Rock of Any Type shall be used.

Critical Bridges at Soil Sites shall be analyzed based on the results of a site-specific study following the procedures specified in A3.10.2.2. The generic vertical spectra in Table A3.10.4-5 (Figures A3.10.4-16 to A3.10.4-18) are used as references during the site-specific studies.

## **A3.10.5 Operational Categories and Seismic Performance Criteria within Downstate Zone**

### ***A3.10.5.1 Performance Criteria and Seismic Hazard***

Bridges in the Downstate Zone shall be designed to meet the performance criteria outlined in Table A3.10.5-1. This Table summarizes the relationship between bridge importance and performance requirements. In all cases, collapse is not permitted. More rigorous analysis as recommended herein and satisfying the minimum requirements of Article 4.7.4.3.1 shall be performed unless otherwise required by the Bridge Owner. Seismic Performance criteria given herein address the safety and functional performance of a bridge during and after an earthquake. They are defined in terms of the bridge’s post seismic service level and the extent of damage. Bridges shall be classified by the agency having jurisdiction, as “critical”, “essential” or “other” meeting the following requirements:

**Critical Bridge:** A Critical Bridge must provide immediate access after the lower level (functional) event and limited access after the upper level (safety) event and continue to function as a part of the lifeline, social/survival network and serve as an important link for civil defense, police, fire department and/or public health agencies to respond to a disaster situation after the event, providing a continuous route. Any bridge on a critical route shall be classified as critical if there is no readily accessible detour, and shall at a minimum be classified as essential if there is such a detour.

Any bridge that crosses a critical route whose collapse would block the critical route shall at a minimum be classified as essential if there is no readily accessible detour for the critical route. However, the bridge owner may classify such bridges as critical with the concurrence of the DCES.

The designated detour for the critical route shall also be treated as a critical route.

It is expected that relatively few bridges will be classified as critical. Critical bridges would generally be limited to those on life safety routes in an urban area or on the approaches to an urban area. Critical Bridges would also be located on routes to a defense facility that has limited access. Bridges on limited access highways in rural areas would generally not be classified as critical unless they are major structures. The designation of a bridge as critical is at the discretion of the Regional Director/Bridge Owner and it is to be documented in the Design Report and included in the Site Data Package.

Critical bridges shall be analyzed for two earthquake hazard design levels: a lower level event (functional evaluation/design level) having a 7% probability of being exceeded in 75 years (1,000 year return period), and an upper level event (safety evaluation/design level) having a 3% probability of being exceeded in 75 years (2,500 year return period).

A multimode spectral analysis or time history analysis must be used to establish vulnerability for both events. The seismic analysis for the upper level event must be confirmed by either a multimodal spectral analysis augmented by non-linear static (pushover) analysis, or by nonlinear time history analysis. In the case of long-span bridges, the effects of spatial variation on the seismic ground motions must also be considered in accordance to the requirements of Article 4.7.4.3.4b.

For Critical bridges at Soil Sites (see Article A3.10.3), site-specific soil effects and, if necessary, soil-structure interaction must be considered. The required site-specific soils study shall comply with Article A3.10.2.2.

For Critical bridges at Rock Sites (see Article A3.10.3), the generic spectra specified in section (1) of Article A3.10.4 may be used for design; alternatively, the engineers shall have the option of conducting a site-specific study including Soil-Structure Interaction effects, using the rock input motions according to Article A3.10.2.2.

Critical Bridges shall survive the upper level event (2,500 year return period) with repairable damage (see definition of damage levels in Article A3.10.5.2). Traffic access following this event may be limited; within 48 hours for emergency/defense vehicles and within months for general traffic. After the lower level event (1,000 year return period), the bridge shall suffer no damage to primary structural elements and minimal damage to other components (see definition of damage levels in Article A3.10.5.2). Access after this event shall be immediate to all traffic with an allowance of a few hours for inspection.

**Essential Bridge:** An Essential Bridge must provide at least limited access after the lower level event and serve as an important link for civil defense, police, fire department and/or public disaster situation after the event, providing a continuous route. A bridge that crosses an essential route whose collapse would block the essential route shall also be classified as essential, if there is no readily accessible detour for the essential route. The designated detour for the essential route shall also be treated as an essential route.

Essential bridges should include those on interstate highways and others of importance as designated by the Regional Director/Bridge Owner. The designation of a bridge as critical is to be documented in the design report and included in the Site Data Package.

Essential bridges shall survive the design event with repairable damage (see definition of damage levels in Article A3.10.5.2). Access following the seismic event may be limited: one or two lanes shall be available within 72 hours for emergency vehicles, and full service within months.

**Other Bridges:** Other bridges are those not classified as Critical or Essential. Bridges classified as Other may suffer significant damage (see definition of damage levels in Article A3.10.5.2) although collapse shall not occur. Extended closures are acceptable.

**Non-Critical Bridges:** Essential and Other bridges (also labeled together Non-critical Bridges), at a minimum, shall be analyzed for a single earthquake hazard design level event having a 7% probability of being exceeded in 75 years (1,000 year return period). The owner has the option to use a single earthquake hazard design level event having a 5% probability of being exceeded in 75 years (1,500 year return period) instead.

The seismic ground motions specified in these guidelines are defined in terms of generic design spectra described in Article A3.10.4. For a Rock Site, the generic design spectra are specified in section (1) of Article A3.10.4. For a Soil Site, the generic design spectra are specified in section (2) of Article A3.10.4. If the Essential and Other bridge is on a Soil Site classified as Soil Class C, D, or E (see Article A3.10.3.3), the Engineer shall have the option of conducting a site specific study accounting for the local subsurface conditions, to produce site-specific acceleration ground surface spectra and corresponding time histories, for use in the design instead of the generic spectra. A site-specific study accounting for the local subsurface site conditions shall be required for a Soil Site classified as Soil Class F (see Article A3.10.3.3). For an Essential bridge at a Rock Site, the Engineer shall have the option of conducting a site-specific study including the effects of Soil-Structure Interaction. Any required or optional site-specific study at a Soil or Rock Site shall comply with Article A3.10.2.2. In the case of long-span bridges, the effects of spatial variation on the seismic ground motions must be considered in accordance with Articles A3.10.2 and A4.7.3b.

#### ***A3.10.5.2 Damage Levels – Definitions***

Bridge component detailing or retrofit shall be such that the damage caused by an earthquake would be in a controlled pattern in order to allow for a desired post event service level.

- **Minimal Damage:** The Bridge should essentially behave elastically during the earthquake, although minor inelastic response could take place. Post-earthquake damage should be limited to narrow flexural cracking in concrete and masonry elements. There should be no permanent deformations to structural members. Only minor damage or permanent deformations to non-structural members should take place.

- **Repairable Damage:** The extent of damage should be limited so that the structure can be restored to its pre-earthquake condition without replacement of structural members. Inelastic response may occur resulting in: concrete cracking, minor cover spalling and reinforcement yielding; minor yielding of structural steel members; some damage to secondary members and non-structural components; some damage to masonry. Repair should not require complete closure of the bridge. Permanent offsets should be small and there should be no collapse.
- **Significant Damage:** There should be no collapse, but permanent offsets may occur. Extensive cracking, major spalling of concrete and reinforcement yielding, cracking of deck slab at the shear studs, may force closure for repair. Similar consequences could result from yielding or local buckling of steel members. There could be yielding of member connections, fracture of limited number of bolts/rivets, serious damage to secondary structural members and non-structural components, as well as to masonry. In sites with significant ground lateral spreading due to liquefaction, large inelastic deformations might be induced to piles. Liquefaction could also result in excessive differential settlements. Partial or complete replacement may be required in some cases.

#### **A3.10.6 Seismic Performance Zones**

Each bridge shall be assigned to one of the three Seismic Performance Zones (2, 3 or 4), determined by the One Second Period Spectral Acceleration,  $S_{D1}$ , of the horizontal design spectrum, in accordance with Table A3.10.6-1 below (Seismic Performance Zone 1 is not applicable to Downstate Zone).

**Table A3.10.6-1 Downstate Seismic Performance Zones**

Acceleration Coefficient, $S_{D1}$	Seismic Performance Zone
	Seismic Performance Zone 1 Not applicable to NYC
$S_{D1} \leq 0.30g$	2
$0.30g < S_{D1} \leq 0.50g$	3
$0.50g < S_{D1}$	4

The One Second Period Acceleration  $S_{D1}$  shall be obtained as the spectral acceleration at the period  $T = 1.0$  second. The horizontal design spectra of Critical Bridges shall be obtained from site-specific study, except for all rock sites (see Article A3.10.3.2) where site specific study is optional. The horizontal design spectra of Non-Critical Bridges may use the generic ones specified in Article A3.10.4 depending on the site class.

#### **A3.10.7 Seismic Rehabilitation of Bridges**

As in the case of any rehabilitation project, judgment should be exercised in assessing options and costs of seismic retrofit measures, and to incorporate into the rehabilitation plans those retrofit measures deemed warranted for eliminating or mitigating such seismic vulnerabilities. Seismic rehabilitation in the downstate region shall be based on applicable provisions of NYSDOT LRFD Bridge Design Specifications.

**Table A3.10.2-1**  
**Very Hard Rock-VHR - Horizontal Uniform Hazard Spectra for the Downstate Zone**  
**Spectral Acceleration (g)**

Period (sec)	1,000 year	1,500 year	2,500 year
0.01	9.98E-02	1.40E-01	2.02E-01
0.04	2.33E-01	3.36E-01	5.27E-01
0.1	1.87E-01	2.60E-01	3.64E-01
0.2	1.16E-01	1.58E-01	2.31E-01
0.5	4.47E-02	6.21E-02	9.15E-02
1	2.16E-02	2.98E-02	4.35E-02
2	1.13E-02	1.55E-02	2.24E-02
4	4.13E-03	5.90E-03	9.05E-03
5	2.94E-03	4.27E-03	6.92E-03
8	1.37E-03	2.01E-03	3.23E-03
10	1.05E-03	1.54E-03	2.49E-03

**Table A3.10.2-2**  
**Very Hard Rock-VHR - Vertical Uniform Hazard Spectra for the Downstate Zone.**  
**Spectral Acceleration (g)**

Period (sec)	1,000 year	1,500 year	2,500 year
0.01	7.76E-02	1.09E-01	1.58E-01
0.04	1.75E-01	2.52E-01	3.95E-01
0.1	1.25E-01	1.74E-01	2.44E-01
0.2	7.77E-02	1.06E-01	1.55E-01
0.5	3.00E-02	4.16E-02	6.13E-02
1	1.45E-02	2.00E-02	2.91E-02
2	7.60E-03	1.04E-02	1.50E-02
4	2.76E-03	3.95E-03	6.06E-03
5	1.97E-03	2.86E-03	4.64E-03
8	9.21E-04	1.35E-03	2.16E-03
10	6.99E-04	1.03E-03	1.67E-03

**Table A3.10.2-3**  
**1,000 year Return Period – Downstate Zone – Horizontal Design Spectra**  
**Spectral Acceleration (g)**

Period (sec)	Very Hard Rock-VHR	Rock Class A	Rock Class B
0.01	9.98E-02	1.15E-01	1.65E-01
0.04	2.33E-01	2.68E-01	3.84E-01
0.1	1.87E-01	2.15E-01	3.09E-01
0.2	1.16E-01	1.33E-01	1.91E-01
0.5	4.47E-02	5.15E-02	7.35E-02
1	2.16E-02	2.49E-02	3.57E-02
2	1.13E-02	1.30E-02	1.87E-02
4	4.13E-03	4.75E-03	6.82E-03
5	2.94E-03	3.38E-03	4.86E-03
8	1.37E-03	1.58E-03	2.26E-03
10	1.05E-03	1.20E-03	1.72E-03

**Table A3.10.2-4**  
**1,500 year Return Period – Downstate Zone – Horizontal Design Spectra**  
**Spectral Acceleration (g)**

<b>Period (sec)</b>	<b>Very Hard Rock-VHR</b>	<b>Rock Class A</b>	<b>Rock Class B</b>
0.01	1.40E-01	1.61E-01	2.31E-01
0.04	3.36E-01	3.86E-01	5.54E-01
0.1	2.60E-01	2.99E-01	4.29E-01
0.2	1.58E-01	1.82E-01	2.61E-01
0.5	6.21E-02	7.14E-02	1.02E-01
1	2.98E-02	3.43E-02	4.92E-02
2	1.55E-02	1.78E-02	2.56E-02
4	5.90E-03	6.79E-03	9.74E-03
5	4.27E-03	4.91E-03	7.05E-03
8	2.01E-03	2.31E-03	3.32E-03
10	1.54E-03	1.77E-03	2.54E-03



**Table A3.10.2-5**  
**2,500 year Return Period – Downstate Zone – Horizontal Design Spectra**  
**Spectral Acceleration (g)**

Period (sec)	Very Hard Rock-VHR	Rock Class A	Rock Class B
0.01	2.02E-01	2.32E-01	3.33E-01
0.04	5.27E-01	6.06E-01	8.70E-01
0.1	3.64E-01	4.19E-01	6.01E-01
0.2	2.31E-01	2.66E-01	3.81E-01
0.5	9.15E-02	1.05E-01	1.51E-01
1	4.35E-02	5.00E-02	7.18E-02
2	2.24E-02	2.58E-02	3.70E-02
4	9.05E-03	1.04E-02	1.49E-02
5	6.92E-03	7.96E-03	1.14E-02
8	3.23E-03	3.71E-03	5.33E-03
10	2.49E-03	2.86E-03	4.11E-03

**Table A3.10.2-6**  
**1,000 year Return Period – Downstate Zone – Vertical Design Spectra**  
**Spectral Acceleration (g)**

<b>Period (sec)</b>	<b>Very Hard Rock-VHR</b>	<b>Rock Class A</b>	<b>Rock Class B</b>
0.01	7.76E-02	8.90E-02	1.28E-01
0.04	1.75E-01	2.01E-01	2.89E-01
0.1	1.25E-01	1.44E-01	2.06E-01
0.2	7.77E-02	8.94E-02	1.28E-01
0.5	3.00E-02	3.45E-02	4.95E-02
1	1.45E-02	1.67E-02	2.40E-02
2	7.60E-03	8.76E-03	1.26E-02
4	2.76E-03	3.18E-03	4.56E-03
5	1.97E-03	2.27E-03	3.25E-03
8	9.21E-04	1.06E-03	1.52E-03
10	6.99E-04	8.00E-04	1.15E-03

**Table A3.10.2-7**  
**1,500 year Return Period – Downstate Zone – Vertical Design Spectra**  
**Spectral Acceleration (g)**

<b>Period (sec)</b>	<b>Very Hard Rock-VHR</b>	<b>Rock Class A</b>	<b>Rock Class B</b>
0.01	1.09E-01	1.25E-01	1.80E-01
0.04	2.52E-01	2.90E-01	4.16E-01
0.1	1.74E-01	2.00E-01	2.87E-01
0.2	1.06E-01	1.22E-01	1.75E-01
0.5	4.16E-02	4.78E-02	6.86E-02
1	2.00E-02	2.30E-02	3.30E-02
2	1.04E-02	1.20E-02	1.72E-02
4	3.95E-03	4.54E-03	6.52E-03
5	2.86E-03	3.29E-03	4.72E-03
8	1.35E-03	1.55E-03	2.23E-03
10	1.03E-03	1.18E-03	1.70E-03

**Table A3.10.2-8**  
**2,500 year Return Period – Downstate Zone – Vertical Design Spectra**  
**Spectral Acceleration (g)**

<b>Period (sec)</b>	<b>Very Hard Rock-VHR</b>	<b>Rock Class A</b>	<b>Rock Class B</b>
0.01	1.58E-01	1.82E-01	2.61E-01
0.04	3.95E-01	4.54E-01	6.52E-01
0.1	2.44E-01	2.81E-01	4.03E-01
0.2	1.55E-01	1.78E-01	2.56E-01
0.5	6.13E-02	7.05E-02	1.01E-01
1	2.91E-02	3.35E-02	4.80E-02
2	1.50E-02	1.73E-02	2.48E-02
4	6.06E-03	6.97E-03	1.00E-02
5	4.64E-03	5.34E-03	7.66E-03
8	2.16E-03	2.48E-03	3.56E-03
10	1.67E-03	1.92E-03	2.76E-03

**Table A3.10.2.2-1**  
**1,000 yr. Return Period - Downstate Zone Soil Sites**  
**V/H Design Ratio for Site-Specific Studies**

<b><i>Soil on Top of Rock Class A /VHR - Hr&lt;100 ft.</i></b>			
<b><i>Period(sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.65	0.65	0.90
0.02	0.80	0.80	1.10
0.04	0.80	0.80	1.10
0.10	0.60	0.60	0.70
0.20	0.50	0.50	0.50
0.50	0.50	0.50	0.50
1.00	0.50	0.50	0.50
2.00	0.50	0.50	0.50
4.00	0.50	0.50	0.50
<b><i>Soil on Top of Rock Class B - Hr&lt;100 ft.</i></b>			
<b><i>Period(sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.65	0.65	0.90
0.02	1.05	1.05	1.30
0.04	1.05	1.05	1.30
0.10	0.70	0.70	0.80
0.20	0.50	0.50	0.50
0.50	0.50	0.50	0.50
1.00	0.50	0.50	0.50
2.00	0.50	0.50	0.50
4.00	0.50	0.50	0.50
<b><i>Soil on Top of Deep Rock of Any Type - Hr&gt;100 ft.</i></b>			
<b><i>Period(sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.73	0.80	1.20
0.02	1.15	1.30	1.75
0.04	1.15	1.30	1.75
0.10	0.75	0.80	1.00
0.20	0.60	0.60	0.75
0.50	0.50	0.50	0.50
1.00	0.50	0.50	0.50
2.00	0.50	0.50	0.50
4.00	0.50	0.50	0.50

**Table A3.10.2.2-2**  
**1,500 yr. Return Period - Downstate Zone Soil Sites**  
**V/H Design Ratio for Site-Specific Studies**

<b><i>Soil on Top of Rock Class A / VHR - Hr&lt;100 ft</i></b>			
<b><i>Period (sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.65	0.65	0.90
0.02	0.80	0.80	1.10
0.04	0.80	0.80	1.10
0.10	0.60	0.60	0.70
0.20	0.50	0.50	0.50
0.50	0.50	0.50	0.50
1.00	0.50	0.50	0.50
2.00	0.50	0.50	0.50
4.00	0.50	0.50	0.50
<b><i>Soil on Top of Rock Class B - Hr&lt;100 ft</i></b>			
<b><i>Period (sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.65	0.65	0.90
0.02	1.05	1.05	1.30
0.04	1.05	1.05	1.30
0.10	0.70	0.70	0.80
0.20	0.50	0.50	0.50
0.50	0.50	0.50	0.50
1.00	0.50	0.50	0.50
2.00	0.50	0.50	0.50
4.00	0.50	0.50	0.50
<b><i>Soil on Top of Deep Rock of Any Type - Hr&gt;100 ft</i></b>			
<b><i>Period (sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.80	0.80	1.20
0.02	1.30	1.30	1.75
0.04	1.30	1.30	1.75
0.10	0.80	0.80	1.00
0.20	0.60	0.60	0.75
0.50	0.50	0.50	0.50
1.00	0.50	0.50	0.50
2.00	0.50	0.50	0.50
4.00	0.50	0.50	0.50

**Table A3.10.2.2-3**  
**2,500 yr. Return Period - Downstate Zone Soil Sites**  
**V/H Design Ratio for Site-Specific Studies**

<b>Soil on Top of Rock Class A / VHR - Hr&lt;100 ft</b>			
<b>Period (sec)</b>	<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
<b>PGA</b>	0.65	0.65	0.90
0.02	0.80	0.80	1.10
0.04	0.80	0.80	1.10
0.10	0.60	0.60	0.70
0.20	0.50	0.50	0.50
0.50	0.50	0.50	0.50
1.00	0.50	0.50	0.50
2.00	0.50	0.50	0.50
4.00	0.50	0.50	0.50
<b>Soil on Top of Rock Class B - Hr&lt;100 ft</b>			
<b>Period (sec)</b>	<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
<b>PGA</b>	0.65	0.65	0.90
0.02	1.05	1.05	1.30
0.04	1.05	1.05	1.30
0.10	0.70	0.70	0.80
0.20	0.50	0.50	0.50
0.50	0.50	0.50	0.50
1.00	0.50	0.50	0.50
2.00	0.50	0.50	0.50
4.00	0.50	0.50	0.50
<b>Soil on Top of Deep Rock of Any Type - Hr&gt;100 ft</b>			
<b>Period (sec)</b>	<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
<b>PGA</b>	0.80	0.80	1.20
0.02	1.30	1.30	1.85
0.04	1.30	1.30	1.85
0.10	0.80	0.80	1.00
0.20	0.60	0.60	0.75
0.50	0.50	0.50	0.50
1.00	0.50	0.50	0.50
2.00	0.50	0.50	0.50
4.00	0.50	0.50	0.50

### A3.10.3.1-1: Rock Classification

#### Rock Sites

<b>Rock Class</b>	<b>Name</b>	<b>Average Shear-Wave Velocity</b> $\bar{V}_{s20}$ ft./sec.	<b>Remarks</b>
<b>VHR</b>	Very Hard Rock	$\bar{V}_{s20} > 9,000$	Very Hard Rock shall be established only by measured $V_s$ .
<b>A</b>	Hard Rock	$5,000 < \bar{V}_{s20} \leq 9,000$	Rock Class A shall be established only by measured $V_s$ .
<b>B</b>	Rock (or Cemented or Very Dense Soil)	$2,500 < \bar{V}_{s20} \leq 5,000$	Assignment of Rock Class B for cemented or very dense soil shall be based on shear wave velocity measurements. Rock Class B may be assigned for moderately fractured and weathered rock.

#### Rock Under Soil Sites

<b>Rock Class</b>	<b>Name</b>	<b>Average Shear-Wave Velocity</b> $\bar{V}_{s20}$ ft./sec.	<b>Remarks</b>
<b>A / VHR</b>	Hard Rock	$\bar{V}_{s20} > 5,000$	Rock Class A / VHR shall be established only by measured $V_s$ .
<b>B</b>	Rock (or Cemented or Very Dense Soil)	$2,500 < \bar{V}_{s20} \leq 5,000$	Assignment of Rock Class B for cemented or very dense soil shall be based on shear wave velocity measurements. Rock Class B may be assigned for moderately fractured and weathered rock.

Notes:

- 1) The Rock Class shall be established through properly substantiated geotechnical data.
- 2) For the definition of  $\bar{V}_{s20}$  see Article A3.10.3.1.



**Table A3.10.3.3-1: Soil Classification at Soil Sites**

Soil Class	Soil General Description	Average Shear-Wave Velocity $\bar{V}_s$ ft./sec.	Average Undrained Shear Strength $\bar{s}_u$ psf	Average Penetration Resistance $\bar{N}$ , $\bar{N}_{ch}$ blows/ft.
<b>C</b>	Very Dense Soil	$1,200 < \bar{V}_s \leq 2,500^*$	$\bar{s}_u > 2,000$	$(\bar{N} \text{ or } \bar{N}_{ch}) > 50$
<b>D</b>	Stiff Soil	$600 \leq \bar{V}_s \leq 1,200$	$1,000 \leq \bar{s}_u \leq 2,000$	$15 \leq (\bar{N} \text{ or } \bar{N}_{ch}) \leq 50$
<b>E</b>	Non Special-Investigation Soft Soil	$\bar{V}_s < 600$	$\bar{s}_u < 1,000$	$(\bar{N} \text{ or } \bar{N}_{ch}) < 15$
		Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity Index $PI > 20$ 2. Moisture Content $w \geq 40\%$ 3. Undrained Shear Strength $\bar{s}_u < 500$		
<b>F</b>	Special-Investigation Soft Soil	Require Site-Specific Investigation/Analyses. Include any profile containing soils with one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils (see article A3.10.3.4), quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ( $H > 10.0$ ft., where H = total thickness of peat and/or highly organic clay soil layers). 3. Very high plasticity clays ( $H > 25$ ft. with plasticity index $PI > 75$ ). 4. Very thick soft/medium stiff clays ( $H > 120$ ft.) with $\bar{s}_u < 1,000$ psf.		

## Notes:

- 1) The Soil Class shall be established through properly substantiated geotechnical data.
- 2) Soil Classification applies to depths down to the rock surface (as defined in Article A3.10.3.1) to a maximum of 100 ft. of soil.  
Parameters measured or estimated below the rock surface shall not be included in the soil classifications.

\* Soil above the rock surface which happens to have a  $\bar{V}_s > 2,500$  ft./sec due to the presence of high shear wave velocity cemented or very dense soil layers, but does not meet the requirements of a rock site, as specified in Article A3.10.3.1, shall also be classified as Soil Class C.

**Table A3.10.3-1: Horizontal Median PGA for Use in  
Liquefaction Evaluation**

**1,000 yr, Return Period - Downstate Zone -  
PGA (g)**

<b>Soil on Top of Rock Class A /VHR - Hr&lt;100 ft.</b>		
<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
0.185	0.185	0.169
<b>Soil on Top of Rock Class B - Hr&lt;100 ft.</b>		
<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
0.184	0.184	0.180
<b>Soil on Top of Deep Rock of Any type - Hr&gt;100 ft.</b>		
<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
0.170	0.170	0.128

**1,500 yr, Return Period - Downstate Zone -  
PGA (g)**

<b>Soil on Top of Rock Class A /VHR - Hr&lt;100 ft.</b>		
<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
0.25	0.25	0.22
<b>Soil on Top of Rock Class B - Hr&lt;100 ft.</b>		
<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
0.25	0.25	0.24
<b>Soil on Top of Deep Rock of Any type - Hr&gt;100 ft.</b>		
<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
0.22	0.22	0.16

**2,500 yr. Return Period - Downstate Zone - PGA (g)**

<b>Soil on Top of Rock Class A /VHR - Hr&lt;100 ft.</b>		
<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
0.35	0.35	0.30
<b>Soil on Top of Rock Class B - Hr&lt;100 ft.</b>		
<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
0.36	0.36	0.33
<b>Soil on Top of Deep Rock of Any type - Hr&gt;100 ft.</b>		
<b>Soil Class C</b>	<b>Soil Class D</b>	<b>Soil Class E</b>
0.29	0.29	0.21

**Table A3.10.3-2**  
**Seismic Hazard and Time Histories for Rock**  
**to be used in Site-Specific Studies of Downstate Zone**

Importance Category	Seismic Hazard Return Period (Probability of Exceedance)	Site-Specific Study Mandatory or Optional	Horizontal Rock Spectra and Time Histories
<b>Critical Bridges</b>	2,500-year (3% in 75 years.)	Mandatory unless waived by the owner	<p><u>Very Hard Rock (VHR):</u> Spectra and Time Histories described in Tables A3.10.2-1 and A.3.10.2-2</p> <p><u>Rock Class A:</u> Multiply Very Hard Rock Spectra and Time Histories by a Factor 1.15 (Tables A3.10.2-3 to A3.10.2-8)</p> <p><u>Rock Class B:</u> Multiply Very Hard Rock Spectra and Time Histories by a Factor 1.65 (Tables A3.10.2-3 to A3.10.2-8)</p>
	1,000-year (7% in 75 years.)	Mandatory unless waived by the owner	
<b>Essential and Other Bridges</b>	1,000-year (7% in 75 yrs)  or	Mandatory for Soil Site Class F and for Long-Span Bridges (Spatial Variation)	
	1,500-year (5% in 75 yrs) (optional)	Optional for Other Rock and Soil Site Classes	

**Table A3.10.4-1**  
**1,000 yr. Return Period – Downstate Zone –Horizontal Design Spectra (g)**

Soil on Top of Rock Class A /VHR - Hr<100 ft.				
Period(sec)	Soil Class C	Soil Class D	Soil Class E	Notes
PGA	0.31	0.31	0.27	<b><i>Ts = period at which the spectral acceleration value start decreasing</i></b>  <i>Ts=0.20 seconds for Class C or D</i> <i>Ts=0.30 seconds for Class E</i>
0.02	0.70	0.70	0.53	
0.10	0.70	0.70	0.53	
Ts	0.70	0.70	0.53	
0.50	0.17	0.17	0.17	
1.00	0.05	0.10	0.10	
2.00	0.03	0.03	0.03	
4.00	0.01	0.01	0.01	
Soil on Top of Rock Class B - Hr<100 ft.				
Period(sec)	Soil Class C	Soil Class D	Soil Class E	Notes
PGA	0.31	0.31	0.29	<i>Ts=0.20 seconds for Class C or D</i> <i>Ts=0.27 seconds for Class E</i>
0.02	0.70	0.70	0.60	
0.10	0.70	0.70	0.60	
Ts	0.70	0.70	0.60	
0.50	0.26	0.26	0.26	
1.00	0.07	0.14	0.14	
2.00	0.03	0.04	0.04	
4.00	0.02	0.02	0.02	
Soil on Top of Deep Rock of Any Type - Hr>100 ft.				
Period(sec)	Soil Class C	Soil Class D	Soil Class E	Notes
PGA	0.27	0.27	0.19	<i>Ts=0.20 seconds for Class C</i> <i>Ts=0.25 seconds for Class D</i> <i>Ts=0.37 seconds for Class E</i>
0.02	0.53	0.50	0.40	
0.10	0.53	0.50	0.40	
Ts	0.53	0.50	0.40	
0.50	0.27	0.27	0.27	
1.00	0.12	0.18	0.18	
2.00	0.07	0.11	0.11	
4.00	0.05	0.04	0.04	

**Table A3.10.4-2**  
**1,500 yr. Return Period – NYC Soil Sites –Horizontal Design Spectra (g)**

Soil on Top of Rock Class A / VHR - Hr<100 ft.				
Period (sec)	Soil Class C	Soil Class D	Soil Class E	Notes
PGA	0.42	0.42	0.34	Ts= period at which the spectral acceleration values start decreasing  Ts= 0.20 seconds for Class C or D Ts= 0.30 seconds for Class E
0.02	0.95	0.95	0.70	
0.10	0.95	0.95	0.70	
Ts	0.95	0.95	0.70	
0.50	0.23	0.23	0.23	
1.00	0.06	0.13	0.13	
2.00	0.04	0.04	0.04	
4.00	0.01	0.01	0.01	
Soil on Top of Rock Class B - Hr<100 ft.				
Period (sec)	Soil Class C	Soil Class D	Soil Class E	Notes
PGA	0.42	0.42	0.38	Ts= 0.20 seconds for Class C or D Ts= 0.27 seconds for Class E
0.02	0.95	0.95	0.82	
0.10	0.95	0.95	0.82	
Ts	0.95	0.95	0.82	
0.50	0.35	0.35	0.35	
1.00	0.08	0.19	0.19	
2.00	0.04	0.06	0.06	
4.00	0.02	0.02	0.02	
Soil on Top of Deep Rock of Any Type - Hr>100 ft.				
Period (sec)	Soil Class C	Soil Class D	Soil Class E	Notes
PGA	0.34	0.34	0.24	Ts= 0.20 seconds for Class C Ts= 0.25 seconds for Class D Ts= 0.37 seconds for Class E
0.02	0.72	0.66	0.51	
0.10	0.72	0.66	0.51	
Ts	0.72	0.66	0.51	
0.50	0.35	0.35	0.35	
1.00	0.16	0.24	0.24	
2.00	0.09	0.15	0.15	
4.00	0.06	0.06	0.06	

**Table A3.10.4-3**  
**2,500 yr. Return Period – Downstate Zone –Horizontal Design Spectra (g)**

Soil on Top of Rock Class A / VHR - Hr<100 ft.				
Period (sec)	Soil Class C	Soil Class D	Soil Class E	Notes
PGA	0.58	0.58	0.46	Ts= period at which the spectral acceleration values start decreasing  Ts= 0.20 seconds for Class C or D Ts= 0.31 seconds for Class E
0.02	1.34	1.34	0.97	
0.10	1.34	1.34	0.97	
Ts	1.34	1.34	0.97	
0.50	0.33	0.33	0.33	
1.00	0.08	0.18	0.18	
2.00	0.04	0.06	0.06	
4.00	0.02	0.02	0.02	
Soil on Top of Rock Class B - Hr<100 ft.				
Period (sec)	Soil Class C	Soil Class D	Soil Class E	Notes
PGA	0.58	0.58	0.51	Ts= 0.20 seconds for Class C or D Ts= 0.27 seconds for Class E
0.02	1.34	1.34	1.15	
0.10	1.34	1.34	1.15	
Ts	1.34	1.34	1.15	
0.50	0.48	0.48	0.48	
1.00	0.11	0.27	0.27	
2.00	0.06	0.10	0.10	
4.00	0.02	0.03	0.03	
Soil on Top of Deep Rock of Any Type - Hr>100 ft.				
Period (sec)	Soil Class C	Soil Class D	Soil Class E	Notes
PGA	0.46	0.46	0.31	Ts= 0.25 seconds for Class C Ts= 0.26 seconds for Class D Ts= 0.39 seconds for Class E
0.02	0.98	0.89	0.67	
0.10	0.98	0.89	0.67	
Ts	0.98	0.89	0.67	
0.50	0.49	0.49	0.49	
1.00	0.24	0.34	0.34	
2.00	0.14	0.23	0.23	
4.00	0.06	0.09	0.09	

**Table A3.10.4-4**  
**1,000 yr Return Period – Downstate Zone Soil Sites**  
**Vertical Design Spectra (g)**

<i>Soil on Top of Rock Class A /VHR - Hr&lt;100 ft.</i>			
<i>Period(sec)</i>	<i>Soil Class C</i>	<i>Soil Class D</i>	<i>Soil Class E</i>
<b>PGA</b>	0.24	0.24	0.24
0.02	0.58	0.58	0.58
0.04	0.58	0.58	0.58
0.10	0.43	0.43	0.43
0.20	0.28	0.28	0.28
0.50	0.12	0.12	0.12
1.00	0.03	0.04	0.04
2.00	0.02	0.02	0.02
4.00	0.01	0.01	0.01
<i>Soil on Top of Rock Class B - Hr&lt;100 ft.</i>			
<i>Period(sec)</i>	<i>Soil Class C</i>	<i>Soil Class D</i>	<i>Soil Class E</i>
<b>PGA</b>	0.29	0.30	0.30
0.02	0.76	0.79	0.79
0.04	0.76	0.79	0.79
0.10	0.56	0.59	0.59
0.20	0.36	0.36	0.36
0.50	0.15	0.15	0.15
1.00	0.05	0.07	0.07
2.00	0.02	0.02	0.02
4.00	0.01	0.01	0.01
<i>Soil on Top of Deep Rock of Any Type - Hr&gt;100 ft.</i>			
<i>Period(sec)</i>	<i>Soil Class C</i>	<i>Soil Class D</i>	<i>Soil Class E</i>
<b>PGA</b>	0.30	0.31	0.31
0.02	0.74	0.79	0.79
0.04	0.74	0.79	0.79
0.10	0.56	0.60	0.60
0.20	0.35	0.35	0.35
0.50	0.15	0.15	0.15
1.00	0.08	0.10	0.10
2.00	0.04	0.06	0.06
4.00	0.02	0.02	0.02

**Table A3.10.4-5**  
**1,500 yr. Return Period – Downstate Zone Soil Sites**  
**Vertical Design Spectra (g)**

<b><i>Soil on Top of Rock Class A / VHR - Hr&lt;100 ft</i></b>			
<b><i>Period (sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.30	0.30	0.30
0.02	0.75	0.75	0.75
0.04	0.75	0.75	0.75
0.10	0.55	0.55	0.55
0.20	0.35	0.35	0.35
0.50	0.15	0.15	0.15
1.00	0.04	0.06	0.06
2.00	0.02	0.02	0.02
4.00	0.01	0.01	0.01
<b><i>Soil on Top of Rock Class B - Hr&lt;100 ft</i></b>			
<b><i>Period (sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.40	0.40	0.40
0.02	1.05	1.05	1.05
0.04	1.05	1.05	1.05
0.10	0.75	0.75	0.75
0.20	0.45	0.45	0.45
0.50	0.20	0.20	0.20
1.00	0.06	0.10	0.10
2.00	0.02	0.03	0.03
4.00	0.01	0.01	0.01
<b><i>Soil on Top of Deep Rock of Any Type - Hr&gt;100 ft</i></b>			
<b><i>Period (sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.40	0.40	0.40
0.02	1.00	1.00	1.00
0.04	1.00	1.00	1.00
0.10	0.75	0.75	0.75
0.20	0.45	0.45	0.45
0.50	0.21	0.21	0.21
1.00	0.10	0.13	0.13
2.00	0.05	0.08	0.08
4.00	0.03	0.03	0.03



**Table A3.10.4-6**  
**2,500 yr Return Period – Downstate Zone Soil Sites**  
**Vertical Design Spectra (g)**

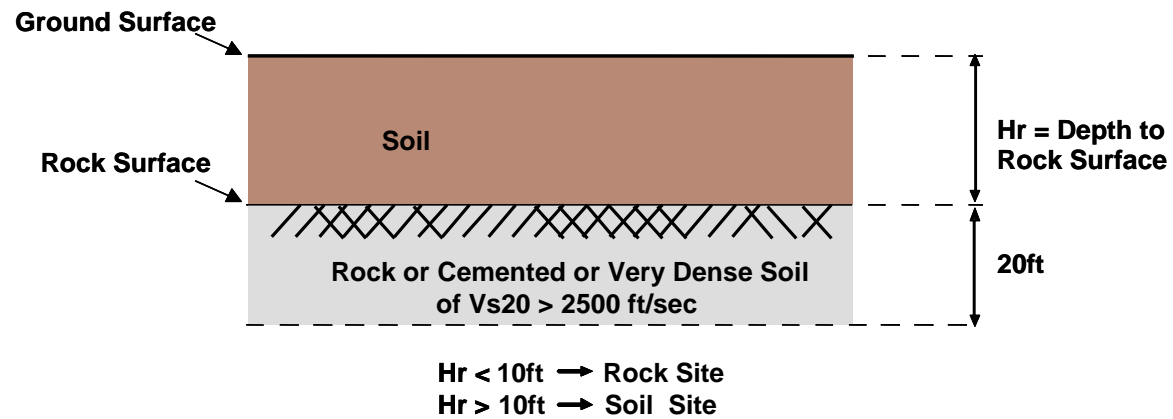
<b><i>Soil on Top of Rock Class A / VHR - Hr&lt;100 ft</i></b>			
<b><i>Period (sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.40	0.40	0.40
0.02	1.05	1.05	1.05
0.04	1.05	1.05	1.05
0.10	0.75	0.75	0.75
0.20	0.45	0.45	0.45
0.50	0.18	0.18	0.18
1.00	0.06	0.10	0.10
2.00	0.02	0.03	0.03
4.00	0.01	0.01	0.01
<b><i>Soil on Top of Rock Class B - Hr&lt;100 ft</i></b>			
<b><i>Period (sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.60	0.60	0.60
0.02	1.45	1.45	1.45
0.04	1.45	1.45	1.45
0.10	1.00	1.00	1.00
0.20	0.60	0.60	0.60
0.50	0.25	0.25	0.25
1.00	0.08	0.15	0.15
2.00	0.03	0.06	0.06
4.00	0.01	0.02	0.02
<b><i>Soil on Top of Deep Rock of Any Type - Hr&gt;100 ft</i></b>			
<b><i>Period (sec)</i></b>	<b><i>Soil Class C</i></b>	<b><i>Soil Class D</i></b>	<b><i>Soil Class E</i></b>
<b><i>PGA</i></b>	0.55	0.55	0.55
0.02	1.30	1.30	1.30
0.04	1.30	1.30	1.30
0.10	0.90	0.90	0.90
0.20	0.60	0.60	0.60
0.50	0.35	0.35	0.35
1.00	0.14	0.20	0.20
2.00	0.07	0.12	0.12
4.00	0.03	0.05	0.05

**Table A3.10.5-1**

## Performance Criteria and Seismic Hazard Level for Design and Evaluation of Bridges

Importance Category	Seismic Hazard Level	Return Period	Event	Performance Criteria
Critical Bridges	Upper Hazard Level-Safety	2,500 years	3% in 75 years Probability of Exceedance	No collapse, <i>repairable</i> damage, <i>limited</i> access for emergency traffic within 48 hours, full service within month(s).
	Lower Hazard Level-Functional	1,000 years	7% in 75 years Probability of Exceedance	No collapse, <i>no</i> damage to primary structural elements, <i>minimal</i> repairable damage to other components, full access to normal traffic available <i>immediately</i> (allow a few hours for inspection).
Essential Bridges	Single Hazard Level-Safety	1,000 years or 1,500 years (optional)	7% in 75 years Probability of Exceedance or 5% in 75 years Probability of Exceedance (optional)	No collapse, <i>repairable</i> damage, limited access for emergency vehicles, one or two lanes available within 72 hours, full service within month(s).
Other Bridges	Single Hazard Level-Safety		No collapse, <i>significant</i> damage in visible and controlled areas. Traffic interruption acceptable.	
Critical Bridges	A bridge that must provide <i>immediate access</i> after the lower level (functional) event and <i>at least limited access</i> after the upper level (safety) event and continue to function as part of the lifeline, social/survival and serve as important link for civil defense, police, fire department or/and public health agencies to respond to a disaster situation after the event, providing a continuous route. Any bridge on a <i>critical</i> route shall be classified as <i>critical</i> if there is no readily accessible detour, and shall at a minimum be classified as <i>essential</i> if there is such a detour. Any bridge that crosses a <i>critical</i> route, whose collapse would block the <i>critical</i> route, shall at a minimum be classified as <i>essential</i> if there is no readily accessible detour for the <i>critical</i> route. The designated detour for the <i>critical</i> route shall also be treated as a <i>critical</i> route.			
Essential Bridges	A bridge that must provide <i>at least limited access</i> after the <i>single-level safety event</i> and serve as important link for civil defense, police, fire department or/and public health agencies to respond to a disaster situation after the event, providing a continuous route. A bridge that crosses an essential route whose collapse would block the essential route shall also be classified as essential, if there is no readily accessible detour for the essential route. The designated detour for the essential route shall also be treated as an essential route.			
Other Bridges	A bridge not qualifying as <i>critical</i> or <i>essential</i> .			

a) Site Definition



b)  $V_{s20}$  Definition

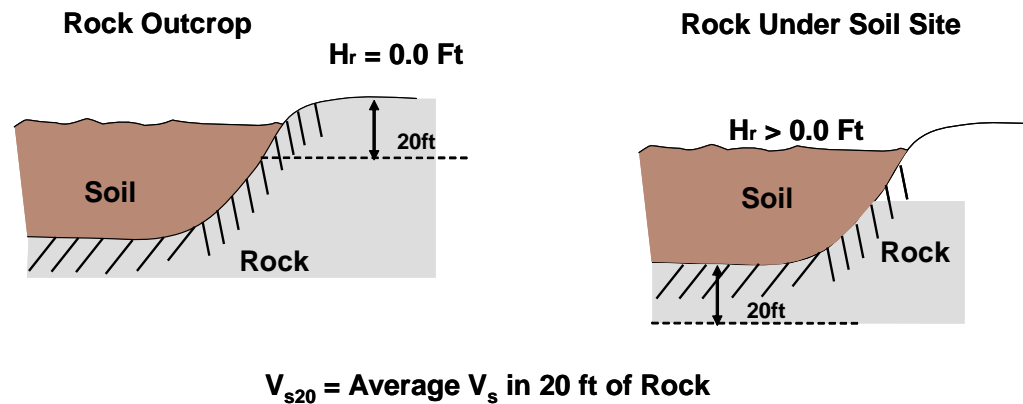
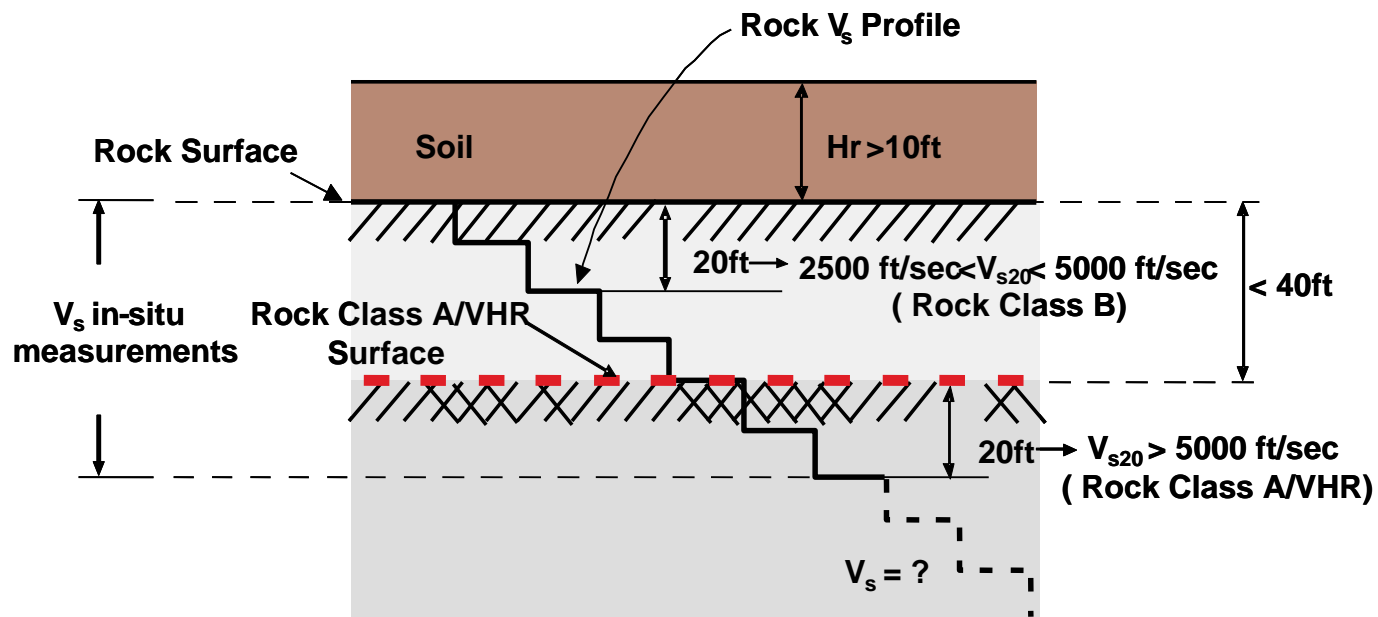


Figure A3.10.3.1-1. Downstate Zone Guidelines Rock and Soil Sites



**Figure A3.10.3.2-1. Downstate Zone Guidelines Option for Soil Site on Rock Class B Followed Within 40 ft. by Rock Class A/VHR**

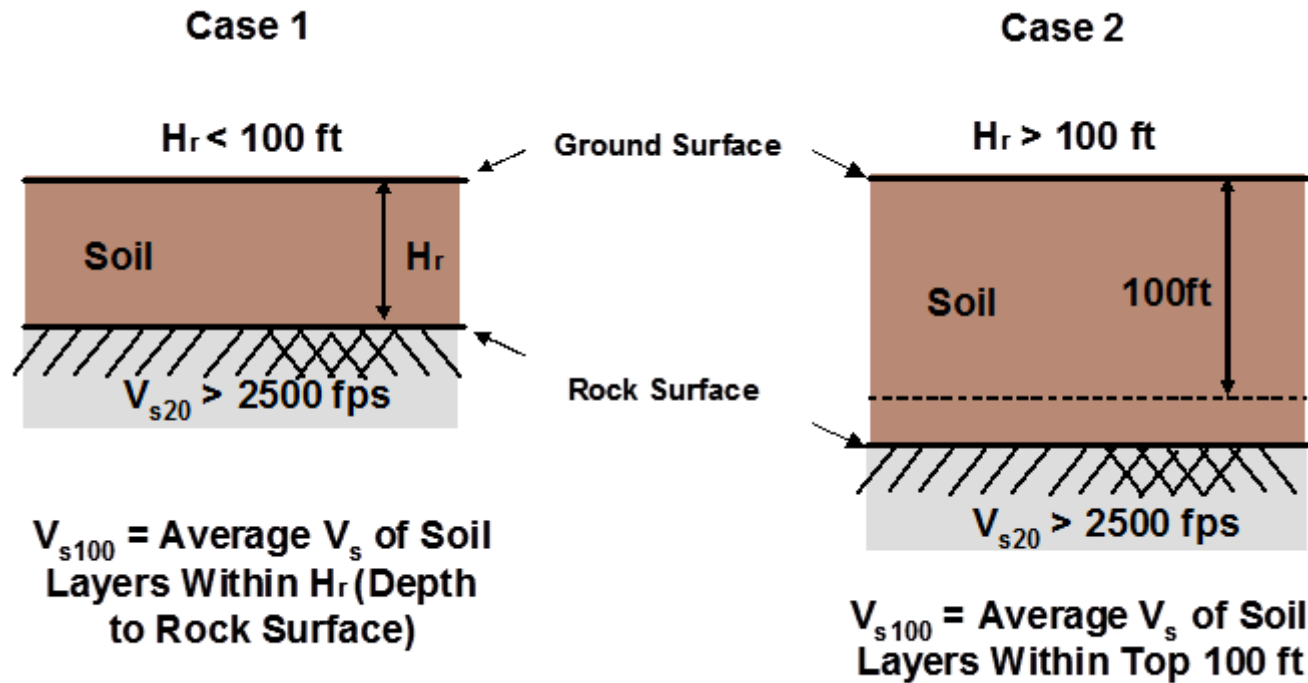


Figure A3.10.3.3-1. Downstate Zone Guidelines Soil Classification

**3.11 EARTH PRESSURE: *EH*, *ES*, *LS*, AND *DD***

**3.11.5 Earth Pressure: *EH***

**3.11.5.4 Passive Lateral Earth Pressure Coefficient,  $k_p$**

Add the following to the end of the first paragraph:

NYSDOT currently uses Rankine or Coulomb theories without wall friction for determination of passive earth pressure coefficients. In addition to Figures 3.11.5.4-1 and 3.11.5.4-2, these theories are also acceptable.

### **3.11 EARTH PRESSURE: *EH*, *ES*, *LS*, AND *DD***

#### **3.11.5 Earth Pressure: *EH***

##### **3.11.5.6 Lateral Earth Pressures for Nongravity Cantilevered Walls**

Replace the first paragraph in this article with the following:

For permanent walls, the simplified lateral earth pressure distributions shown in Figure 3.11.5.6-1 through 3.11.5.6-3 may be used. If walls will support or are supported by cohesive soils for temporary applications, total stress methods of analysis and undrained strength parameters are not allowed, but procedures outlined in NYSDOT Geotechnical Design Procedure 11 (GDP-11) are acceptable alternatives for temporary wall design.

Delete the two bullets after the first paragraph.

**3.11 EARTH PRESSURE: *EH, ES, LS, AND DD***

**3.11.6 Surcharge Loads: *ES AND LS***

**3.11.6.4 Live Load Surcharge (*LS*)**

Add the following at the end of this Article:

All retaining walls shall be designed for a live load surcharge with a minimum equivalent soil height of 2.0 ft., regardless of their height or the distance to traffic.

**C3.11.6.4** Add the following at the end of the commentary:

The minimum live load surcharge requirement for retaining walls is to account for loads experienced during construction.



**3.12        FORCE EFFECTS DUE TO SUPERIMPOSED  
              DEFORMATIONS: *TU, TG, SH, CR, SE***

**3.12.2      Uniform Temperature**

Delete Article 3.12.2 and replace it with the following:

The design thermal movement associated with uniform temperature change shall be calculated using Procedure B for concrete deck bridges having concrete or steel girders and Procedure A for all other bridge types.

**3.12        FORCE EFFECTS DUE TO SUPERIMPOSED  
              DEFORMATIONS: *TU, TG, SH, CR, SE***

**3.12.2      Uniform Temperature**

**3.12.2.1    Temperature Range for Procedure A**

Add the following after Table 3.12.2.1-1:

Areas of New York State designated by NYSDOT as Region 10 and Region 11 shall be considered to have a Moderate Climate. All other areas of the State shall be considered to have a Cold Climate.

**3.12.2.2    Temperature Range for Procedure B**

Add the following after Article 3.12.2.2:

For pile design of integral abutments, the setting temperature shall be the average of the maximum and minimum design temperatures.

### **3.14 VESSEL COLLISION: CV**

#### **3.14.1 General**

Replace the third paragraph of Article 3.14.1 with the following:

For typical and essential bridges, the minimum design impact load for substructure design shall be determined using an empty hopper barge drifting at a velocity equal to the yearly mean current for the waterway location.

For critical bridges, the minimum design impact load for substructure design shall be determined using an empty hopper barge drifting at a velocity equal to the design flood event (100 year flood event).

The design barge shall be a single 35.0 ft. x 195 ft. barge, with an empty displacement of 200 ton, unless approved otherwise by the Owner.

For critical bridges, the minimum design impact load (drifting empty barge) shall be combined with one-half of the predicted long-term scour plus one-half of the predicted short-term scour. The flow rate, water level, and short-term scour depth are those associated with the design flood for bridge scour (100-year flood event).

### **3.14 VESSEL COLLISION: CV**

#### **3.14.2 Owner's Responsibility**

Replace Article 3.14.2 with the following:

The Owner shall establish and/or approve the bridge operational classification, the vessel traffic density in the waterway, and the design velocity of vessels for the bridge.

Critical bridges shall behave elastically during the application of the vessel collision force (CV) as per the Extreme Event II load combination. Post event damage should be limited to narrow flexural cracking in concrete and masonry elements. There shall be no permanent deformation to steel structural members.

Essential and typical bridges shall not collapse but may have inelastic behavior during application of the vessel collision force (CV) as per the Extreme Event II load combination. Permanent deformation shall be small and repairable. Acceptable damage includes concrete cracking, cover spalling and reinforcement yielding; minor yielding of structural steel members, damage to secondary members and nonstructural components; and some damage to masonry. Repair shall not require complete closure of the bridge.

The collapse of protection systems is acceptable from the vessel collision force (CV) Extreme Event II load combination provided that:

- If the ultimate strength of the protection system is less than the impact load caused by the minimum design impact load (drifting empty barge), any damage to the protected component caused by the residual impact load shall meet the limits described above.
- If the ultimate strength of the protection system is less than the impact load caused by a vessel used for design vessel analysis, the residual impact load shall be included in the annual frequency of collapse calculation.
- Certain protective systems could transfer load to the protected element regardless of damage to the system. If protective systems of these types are used, those loads shall similarly be accounted for in design.

### **3.14 VESSEL COLLISION: CV**

#### **3.14.3 Operational Classification**

Add the following paragraph to 3.14.3:

For the purpose of Article 3.14, an operational bridge classification, either "Critical", "Essential", or "Typical" (alternatively stated as "Other") shall be determined by the Regional Director/Owner.

#### **3.14.4 Design Vessel**

Add the following paragraph to Article 3.14.4:

Design Vessel Load occurs under typical waterway conditions. The flow rate and water level shall be taken as yearly mean conditions. For critical bridges, the impact loads shall be combined with the effects of one-half of the long-term scour and no short term scour.

### **3.15 BLAST LOADING**

#### **3.15.1 Introduction**

Add the following paragraph to the end of Article 3.15.1:

Consult Deputy Chief Engineer (Structures) for determining the application of this Article.

## **4.6 STATIC ANALYSIS**

### **4.6.2 Approximate Methods of Analysis**

#### **4.6.2.2 Beam–Slab Bridges**

##### *4.6.2.2.1 Application*

Add the following before the next to last paragraph:

Interior beams of multi-beam bridges shall not have less resistance than an exterior beam except possibly in the cases of:

- Horizontally curved girders
- Exterior girders that support other girders
- Exterior girders that have other girders framed into them
- Special loading conditions on the exterior girder where a refined analysis is necessary

##### *C4.6.2.2.1 Add the following after the last paragraph:*

Without this provision, it is possible to design interior beams with less resistance than exterior beams when using these specifications for bridges with normal dimensions and cross sections. This provision prevents the undesirable condition of interior beams having less load carrying capacity than exterior beams.

Having all beams in a bridge cross section with relatively equal stiffness helps assure that predicted deflections occur during construction. Bridge cross sections consisting of beams of unequal stiffness require refined analyses to reliably predict such deflections.

## **4.6            STATIC ANALYSIS**

### **4.6.2        Approximate Methods of Analysis**

#### **4.6.2.8     Seismic Lateral Load Distribution**

##### *4.6.2.8.1   Applicability*

Delete the existing paragraph and replace it with the following:

These provisions shall apply to diaphragms, cross-frames, and lateral bracing, which are part of the seismic lateral force resisting system in common slab-on-girder bridges in Seismic Zones 2, 3, and 4. These provisions shall also apply to critical bridges in Seismic Zone 1. The provisions of Article 3.10.9.2 shall apply to all non-critical bridges in Seismic Zone 1.



## **4.7 DYNAMIC ANALYSIS**

### **4.7.4 Analysis for Earthquake Loads**

#### **4.7.4.1 General**

Delete the third paragraph and replace it with the following:

Non-critical bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their geometry. However, the minimum requirements, as specified in Articles 4.7.4.4 and 3.10.9, shall apply.

## 4.7 DYNAMIC ANALYSIS

### 4.7.4 Analysis for Earthquake Loads

#### 4.7.4.2 Single-Span Bridges

Delete the first paragraph and replace it with the following:

Seismic analysis is not required for single-span bridges, except for critical bridges on a liquefiable soil site.

#### 4.7.4.3 Multispan Bridges

##### 4.7.4.3.1 Selection of Method

Modify Table 4.7.4.3.1-1, as shown below, to require a multimode elastic method as a minimum analysis for critical bridges in Seismic Zone 1.

**Table 4.7.4.3.1-1 – Minimum Analysis Requirements for Seismic Effects**

Seismic Zone	Single-Span Bridges	Multispan Bridges					
		Other bridges		Essential bridges		Critical bridges	
		regular	irregular	regular	irregular	regular	irregular
1	No seismic analysis required†	*	*	*	*	MM	MM
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	TH
4		SM/UL	MM	MM	MM	TH	TH

†Except for bridges on a liquefiable soil site

*This Blue Page applies only to bridges located in the Downstate Zone, as defined on Blue Page A3.10.*

## **4.7 DYNAMIC ANALYSIS**

### **4.7.4 Analysis for Earthquake Loads**

#### **4.7.4.3 Multispan Bridges**

##### *4.7.4.3.4 Time-History Method*

##### *4.7.4.3.4b Acceleration Time Histories*

Delete Article 4.7.4.3.4b and replace it with the following:

When conducting Time History Analyses of a bridge, at least three acceleration time histories, compatible with the horizontal and vertical design spectra selected shall be used for each of the three orthogonal components of the design seismic motions (two horizontals and one vertical). All three orthogonal components shall be input simultaneously. The selection of these input acceleration time histories and actual number of analyses to be conducted shall be such that the response of the bridge accounts conservatively for:

- The effect of uncertainty in the earthquake excitation, which may be especially significant for the non-linear response of bridge components. At least three site-response analyses shall be conducted; using three different input excitations (see Blue Page A3.10.2).
- The effect of uncertainty in the soil properties (which requires at least three site-response analyses,  $v_s$  best estimate  $\pm 20\%$  (see Blue Page A3.10.2.2).
- When applicable, the effect of unknown depth to the rock surface,  $H_r$ , which requires at least two site-response analyses (see Blue Page A3.10.3.1).
- When applicable, the effects of liquefaction as described below.
- When applicable, the effects of spatial variation as described below.

#### 4.7.4.3.4b (continued)

All effects mentioned above shall be treated according to Article A3.10. If needed for time-history analysis of the bridge, in addition to the horizontal time histories calculated in the dynamic site-response analyses, it may also be necessary to generate acceleration time histories by matching the selected design horizontal spectrum on soil. This might result in particular from the requirement to comply with the two thirds rule, or due to the need to envelope response spectra corresponding to non-liquefied and liquefied soil configurations. In such cases, horizontal acceleration time histories may be obtained from matching the final design soil horizontal spectrum, using appropriate commercially available software as approved by the Owner. If needed for time-history analyses of the bridge, acceleration time histories may be obtained from matching the design soil vertical spectrum, using appropriate commercially available software as approved by the Owner.

The vertical design spectrum for the design earthquake will be obtained by multiplying the site-specific final design soil horizontal spectrum and PGA by the appropriate period dependent V/H ratios tabulated in tables A3.10.2.2-1, A3.10.2.2-2 and A3.10.2.2-3.

If it is determined that liquefaction occurs (see Blue Page A3.10.2.2), the bridge site shall be analyzed for two configurations: (i) *non-liquefied configuration*, where the site is analyzed assuming no pore pressure buildup and no liquefaction; and (ii) *liquefied configuration*, where the site is reanalyzed assuming that liquefaction occurs in the liquefiable soil layers.

#### 4.7.4.3.4b (continued)

The ground motions, spectra and corresponding time histories, calculated with both non-liquefied and liquefied configurations, may be considered conservatively when developing the design ground motions, by enveloping the spectra calculated with both configurations. This simpler conservative option shall be considered carefully as it may have significant consequences on the design of the bridge and shall be approved by the Owner. If needed for time-history analysis of the bridge, in addition to the time histories calculated in the site response analyses, it may also be necessary to generate time histories by matching the final design horizontal spectrum on soil as applicable in this Article.

When establishing the rock motions for a long-span bridge, be it critical, essential or other, the spatial variation of ground motions along the length of the bridge shall be considered. Three sets of time history records for each return period are available in digital form from NYSDOT Office of Structures website (see Blue Page A3.10.2). Each set incorporates the effects of spatial variation of the seismic waves travelling through the rock medium, along 21 hypothetical piers/stations on Very Hard Rock spaced at 328 ft. (100 m), and extended over a straight line having a total length of 1.24 mi (2 km).

<https://www.dot.ny.gov/divisions/engineering/structures/manuals/seismic-references>

The engineer shall establish the ground motions for any long-span bridge length (up to 1.24 miles/2 km), by selecting an appropriate subset of piers/stations time histories whose locations match those of the bridge foundations. It is recommended to include pier/station #11 (the reference one) in the subset of ground motions selected.

In site-specific analyses with non-uniform site foundation conditions, the local rock and soil conditions should be incorporated pier by pier by appropriate modification of the Very Hard Rock horizontal and vertical time histories, as per section A3.10

## **5.4 MATERIAL PROPERTIES**

### **5.4.2 Normal Weight and Lightweight Concrete**

#### **5.4.2.1 Compressive Strength**

Delete the second and third paragraphs of Article 5.4.2.1 and replace with the following:

Normal weight concrete with a design compressive strength above 10.0 ksi shall only be used when approved by the DCES.

The design compressive strength for prestressed concrete shall not be less than 4.0 ksi

The design compressive strength for cast-in-place decks shall not be less than 3.0 ksi.

Concrete with a design compressive strength less than 2.4 ksi shall not be used in structural applications.

Delete the fifth and sixth paragraphs in Article 5.4.2.1.

#### **C5.4.2.1** Delete Article C5.4.2.1 and replace it with the following:

For concrete mix characteristics, refer to Section 501, Portland Cement Concrete – General, of the NYSDOT Standard Specifications.

## **5.6 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS – B REGIONS**

### **5.6.7 Control of Cracking by Distribution of Reinforcement**

Delete the first paragraph of Article 5.6.7 and replace it with the following:

Except for footings and deck slabs designed in accordance with Article 9.7.2, the provisions specified herein shall apply to the reinforcement of all concrete components in which tension in the cross-section exceeds 80% of the modulus of rupture, specified in Article 5.4.2.6, at applicable service limit state load combination specified in Table 3.4.1-1.

## **5.9            PRESTRESSING**

### **5.9.2        Stress Limitations**

#### **5.9.2.3     Stress Limits for Concrete**

##### **5.9.2.3.2   For Stresses at Service Limit State after Losses**

###### *5.9.2.3.2b Tensile Stresses*

Add the following directly below Table 5.9.2.3.2b-1:

Due to the extensive use of deicing salts, all bridges in New York State shall be considered to be subjected to severe corrosive conditions.



## **5.9            PRESTRESSING**

### **5.9.4        Details for Pretensioning**

#### *5.9.4.3      Development of Pretensioning Strand*

##### *5.9.4.3.3    Debonded Strands*

Delete bullet point A and replace with the following:

The number of debonded strands per row shall not exceed 40% of the strands provided in that row.

Delete bullet point B and replace with the following:

A maximum of four (4) prestressing strand debond terminations are permitted at any given location along the length of the unit.

Delete bullet point C and replace with the following:

A minimum difference of 2'-0" is required between debond termination sections.

## **5.10 REINFORCEMENT**

### **5.10.6 Shrinkage and Temperature Reinforcement**

Add to Article 5.10.6 the following:

Non-exposed faces of walls, abutment stems and footings are exempt from Article 5.10.6 except minimum reinforcement as shown on NYSDOT Bridge Detail Sheets shall be provided.

## **5.11 SEISMIC DESIGN AND DETAILS**

### **5.11.4 Seismic Zones 3 and 4**

#### *5.11.4.1 Column Requirements*

##### *5.11.4.1.4 Transverse Reinforcement for Confinement at Plastic Hinges*

After Equation 5.11.4.1.4-3, replace the definitions of 's' and  $h_c$  with the following:

s = vertical spacing of hoops, not exceeding 6.0 in. for Seismic Zones 1 and 2, and not exceeding 4.0 in. for Seismic Zones 3 and 4.

$h_c$  = core dimension of tied column in the direction under consideration, measured to the outside of the hoop (in.). For solid piers in Seismic Zone 1, a reduced effective width may be used for  $h_c$ , according to the 'reduced effective area' method outlined in Article 5.6.4.2.

## **5.11 SEISMIC DESIGN AND DETAILS**

### **5.11.4 Seismic Zones 3 and 4**

#### *5.11.4.1 Column Requirements*

##### *5.11.4.1.5 Spacing of Transverse Reinforcement for Confinement*

Replace the last bullet with the following:

- Spaced not to exceed the lesser of the following:
  - one-quarter of the minimum member dimension
  - 6.0 in. center-to-center, for Seismic Zones 1 and 2
  - 4.0 in. center-to-center, for Seismic Zones 3 and 4

##### *5.11.4.1.6 Splices*

Delete the 2<sup>nd</sup> paragraph of Article 5.11.4.1.6 and replace with the following:

Lap splices in longitudinal reinforcement shall not be used in the plastic hinge zones of columns.

Delete the 3<sup>rd</sup> paragraph of Article 5.11.4.1.6 and replace with the following:

For Seismic Zones 3 and 4 only, the spacing of the transverse reinforcement over the length of the splice shall not exceed the lesser of the following:

- one-quarter of the minimum member dimension
- 4.0 in.

## **5.11 SEISMIC DESIGN AND DETAILS**

### **5.11.4 Seismic Zones 3 and 4**

#### *5.11.4.2 Requirements for Wall-Type Piers*

Add the following after the last paragraph:

For Wall-Type Piers in Seismic Zones 1 and 2, in lieu of the above minimum requirements for the horizontal and vertical reinforcement ratio, the more critical of the following shall be used:

- a) For the horizontal and vertical reinforcement, the requirements of Shrinkage and Temperature Reinforcement as per Article 5.10.6.
- b) Design of vertical reinforcement with reduced effective area approach as per Article 5.6.4.2.
- c) Horizontal (Transverse) reinforcement as per Articles 5.10.4 and 5.11.4.1.4.

## **6.6 FATIGUE AND FRACTURE CONSIDERATIONS**

### **6.6.1 Fatigue**

#### **6.6.1.2 Load-Induced Fatigue**

##### *6.6.1.2.3 Detail Categories*

Delete the second sentence of the first paragraph and replace it with the following:

Where connector holes are depicted in Table 6.6.1.2.3-1, their fabrication shall conform to the provisions of NYSDOT Steel Construction Manual.

## **6.6 FATIGUE AND FRACTURE CONSIDERATIONS**

### **6.6.2 Fracture**

#### **6.6.2.1 Member or Component Designations and Charpy V-Notch Testing Requirements**

- C6.6.2.1** Replace "AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*" reference in the first paragraph of the commentary to "New York State Steel Construction Manual".

## **6.6 FATIGUE AND FRACTURE CONSIDERATIONS**

### **6.6.2 Fracture**

#### **6.6.2.2 Fracture-Critical Members**

Replace all references to "AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*" in this article and commentary to reference the "New York State Steel Construction Manual".



## **6.7 GENERAL DIMENSION AND DETAIL REQUIREMENTS**

### **6.7.2 Dead Load Camber and Detailing of Structural Components**

Delete the sixth paragraph and replace it with the following:

The contract documents shall state the fit condition for I-girder bridges in accordance with Section 8.1.3.2 of the NYSDOT Bridge Manual, including the appropriate standard notes provided in Section 17.3 of the NYSDOT Bridge Manual.

## **6.7 GENERAL DIMENSION AND DETAIL REQUIREMENTS**

### **6.7.3 Minimum Thickness of Steel**

Delete the first and second paragraphs and replace it with the following:

Structural steel, including bracing, cross-frames, and all types of gusset and splice plates, shall not be less than 0.375 in. in thickness. Excluded from this requirement are webs of rolled shapes, closed ribs in orthotropic decks, fillers, and railings.

The web thickness of rolled beams or channels shall not be less than 0.25 inches. For orthotropic decks, the deck plate thickness shall not be less than 0.625 inches or 4% of the larger spacing of the ribs, and the thickness of closed ribs shall not be less than 0.25 inches.

### **6.7.4 Diaphragms and Cross-Frames**

#### **6.7.4.1 General**

Delete the first paragraph of Article 6.7.4.1 and replace it with the following:

Diaphragms or cross-frames shall be placed at the end of the structure, and intermittently along the span at intervals not to exceed 25 ft. Intermittent diaphragms shall also be placed, such that, at least one diaphragm line passes through an interior support. The need for diaphragms or cross-frames shall be investigated for all stages of assumed construction procedures and the final condition.

## **6.7 GENERAL DIMENSION AND DETAIL REQUIREMENTS**

### **6.7.4 Diaphragms and Cross-Frames**

#### **6.7.4.1 General**

**C6.7.4.1** Delete the first paragraph in C6.7.4.1 and replace it with the following:

The requirement for maximum diaphragm spacing has been retained to ensure adequate bracing during construction and redundancy in the final condition.

## **6.7 GENERAL DIMENSION AND DETAIL REQUIREMENTS**

### **6.7.5 Lateral Bracing**

#### **6.7.5.3 Tub Section Members**

Delete the 1<sup>st</sup> and 3<sup>rd</sup> paragraphs of Article 6.7.5.3 and replace them with the following:

Open top box girders (also referred to as tub girders or trough girders), where the composite concrete deck completes the box girder enclosure, shall include a full-length lateral bracing system in the form of a horizontal truss between the top flanges of the box girder. The lateral bracing in the horizontal truss shall be proportioned to resist torsional shear and lateral deformations caused by all anticipated loads on the non-composite, open box section, including construction loads and load eccentricities encountered during placement of the concrete deck.

For straight girders, an equivalent top lateral bracing system or a partial-length top lateral bracing system may be considered adequate if it is demonstrated by a full analysis to be effective, submitted to and approved by the Deputy Chief Engineer (Structures).

## **6.10 I-SECTION FLEXURAL MEMBERS**

### **6.10.3 Constructability**

Add a new Article 6.10.3.1a

#### **6.10.3.1a Stability During Erection**

The designer shall check all girders for stability during erection. To make this check, the designer shall specify and design splice locations when girders need to be erected in multiple segments. The maximum shipping length of steel girder segments is ordinarily limited to 140 ft. The maximum shipping width of steel girder segments is ordinarily limited to 16 ft., however any width greater than 12 ft. will require an escort. Shipping widths instead of lengths may control the location of splices for steel curved girders. Further guidance on splice locations and shipping lengths can be found in Section 8 of NYSDOT Bridge Manual.

Girder segments shall be checked for all conditions where they are simply supported. The fully assembled girder shall also be checked for stability for its full length under dead load only. This condition will exist when the first fully assembled girder is erected in one piece without the use of any falsework and before any bracing is in place.

If the girder segment or fully assembled girder meets the provisions of Article 6.10.6.2.3 for web slenderness, the check shall be made according to Article A6.3.3 Lateral Torsional Buckling Resistance. If it does not meet these provisions, the check will be made according to Article 6.10.8.2.3.

In making the stability check, the load factor for the weight of the girder shall be taken as 1.25 in accordance with Article 3.4.2, Load Factors for Construction Loads.

If the girder segment or fully assembled girder does not meet the stability check, the designer shall either:

- a. Increase the girder size to meet the stability check.
- OR
- b. Place Steel Erection Note #A1 on the plans

### **6.10.3.1a (continued)**

This choice shall be based on an economic analysis comparing the cost of providing additional steel versus the cost of providing additional bracing, falsework, or holding cranes. Site conditions will need to be investigated to determine the feasibility of various erection methods.

#### **Steel Erection Notes**

A1. The Contractor shall provide for the stability of structural steel during all phases of erection and construction, as provided in Subsection 204 of the New York State Steel Construction Manual (SCM). The girders on this bridge shall be stabilized during erection by use of falsework, temporary bracing, compression flange stiffening trusses, choosing alternate picking points, or by use of a holding crane until a sufficient number of girders have been erected and cross frames installed. The methods used by the contractor shall be documented on the erection drawings with all supporting stability calculations submitted and stamped by a licensed New York State Professional Engineer and submitted to the DCES in accordance with the SCM.

If the girder segments and fully assembled girders meet the stability check, then Steel Erection Note #A2 shall be placed on the plans.

A2. The contractor shall provide for the stability of structural steel during all phases of erection and construction, as provided in Subsection 204 of the New York State Steel Construction Manual (SCM). The methods used by the contractor shall be documented on the erection drawings with all supporting stability calculations submitted and stamped by a licensed New York State Professional Engineer and submitted to the DCES in accordance with the SCM.

## **6.13 CONNECTIONS AND SPLICES**

### **6.13.2 Bolted Connections**

#### **6.13.2.4 Holes**

##### *6.13.2.4.1 Type*

##### *6.13.2.4.1b Oversize Holes*

Delete Article 6.13.2.4.1b and replace it with the following:

Oversize holes may be used on diaphragms (primary and secondary members) and all other secondary members. When oversize holes are used they shall be in accordance with the "New York State Steel Construction Manual."

##### *6.13.2.4.1c Short-Slotted Holes*

Delete Article 6.13.2.4.1c

##### *6.13.2.4.1d Long-Slotted Holes*

Delete Article 6.13.2.4.1d

## **6.13 CONNECTIONS AND SPLICES**

### **6.13.2 Bolted Connections**

#### **6.13.2.6 Spacing of Bolts**

##### *6.13.2.6.1 Minimum Spacing and Clear Distance*

Delete the first sentence of Article 6.13.2.6.1 and replace it with the following:

The minimum spacing between centers of bolts shall be as specified in the "New York State Steel Construction Manual."



## **6.13 CONNECTIONS AND SPLICES**

### **6.13.2 Bolted Connections**

#### **6.13.2.8 Slip Resistance**

Add the following immediately after Table 6.13.2.8-3:

All slip-critical connections shall be designed for Class A surface conditions unless otherwise approved by the Deputy Chief Engineer (Structures).

## **6.13 CONNECTIONS AND SPLICES**

### **6.13.3 Welded Connections**

#### **6.13.3.1 General**

Delete Article 6.13.3.1 and its Commentary C6.13.3.1 and replace it with the following:

Steel base metal to be welded, weld metal, welding design details, and fabrication shall conform to the provisions of the "New York State Steel Construction Manual."

## **6.13 CONNECTIONS AND SPLICES**

### **6.13.6 Splices**

#### **6.13.6.1 Bolted Splices**

##### *6.13.6.1.1 Tension Members*

Delete the second sentence in Article 6.13.6.1.1 and replace it with the following:

The following design rules shall be followed:

- There shall be no section changes at a splice location unless approved by the DCES. Where a section changes at the splice, the smaller of the two connected sections shall be used in the design.
- The minimum splice plate thickness shall be 0.375 in. for webs, and 0.50 in. for flanges.
- The minimum area of the splice plates shall be equal to the section area of each component spliced.

## **6.13 CONNECTIONS AND SPLICES**

### **6.13.6 Splices**

#### **6.13.6.1 Bolted Splices**

##### *6.13.6.1.2 Compression Members*

Delete the second sentence in Article 6.13.6.1.2 and replace it with the following:

The following design rules shall be followed:

- There shall be no section changes at a splice location unless approved by the DCES. Where a section changes at the splice, the smaller of the two connected sections shall be used in the design.
- The minimum splice plate thickness shall be 0.375 in. for webs, and 0.50 in. for flanges.
- The minimum area of the splice plates shall be equal to the section area of each component spliced.

## **6.13 CONNECTIONS AND SPLICES**

### **6.13.6 Splices**

#### **6.13.6.2 Welded Splices**

Delete the first paragraph and replace it with the following:

Welded splice design and details shall conform to the requirements of the "New York State Steel Construction Manual" and the following provisions specified herein.

At the end of the second paragraph, replace the word "should" with "shall".

## **6.16 PROVISIONS FOR SEISMIC DESIGN**

### **6.16.4 Design Requirements for Seismic Zones 2, 3, or 4**

#### **6.16.4.1 General**

Delete the first paragraph and replace it with the following:

Components of slab-on-steel girder bridges located in Seismic Zones 3 or 4, defined as specified in Article 3.10.6, and all critical bridges, shall be designed using one of the two types of response strategies specified in this article. Single span bridges are exempt from this requirement.

**9.7 CONCRETE DECK SLABS**

**9.7.1 General**

**9.7.1.3 Skewed Decks**

Delete Article 9.7.1.3 and replace it with the following:

If the skew angle of the deck does not exceed  $30^\circ$ , the primary reinforcement may be placed in the direction of the skew; otherwise, it shall be placed perpendicular to the main supporting components.

**C9.7.1.3** Delete the second sentence in the commentary for Article 9.7.1.3 and replace it with the following:

The  $30^\circ$  limit could affect the area of steel by as much as 15%.

## **9.7 CONCRETE DECK SLABS**

### **9.7.2 Empirical Design**

Delete Article 9.7.2 along with its associated commentaries and refer to the subsection entitled “Isotropic Decks” in Section 5 of the NYSDOT Bridge Manual.



## 10.5 LIMIT STATES AND RESISTANCE FACTORS

### 10.5.5 Resistance Factors

#### 10.5.5.2 Strength Limit States

##### 10.5.5.2.3 Driven Piles

Replace Table 10.5.5.2.3-1 with the following Table.

**Table 10.5.5.2.3-1 - Resistance Factors for Driven Piles**

Condition/Resistance Determination Method		Resistance Factor
Nominal Bearing Resistance of Single Pile – in Axial Compression – Dynamic Analysis and Static Load Test Methods ( $\phi_{dyn}$ )	Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.80
	Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing	0.75
	Driving criteria established by dynamic testing* conducted on 100% of production piles	0.75
	Driving criteria established by dynamic testing,* quality control by dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.65
	Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance	0.50
	Wave equation analysis for CIP piles in cohesionless soil, without pile dynamic measurements or load test but with field confirmation of hammer performance, at end of drive or begin of re-drive conditions (NYSDOT GEB, 2012)	0.60
	FHWA-modified Gates dynamic pile formula (End of Drive condition only)	0.40
	Engineering News (as defined in Article 10.7.3.8.5) dynamic pile formula (End of Drive condition only)	0.10

\* Dynamic testing requires signal matching, and best estimates of nominal resistance are made from a restrike. Dynamic tests are calibrated to the static load test, when available.

## 10.5.5.2.3 (continued)

**Table 10.5.5.2.3-1 - Resistance Factors for Driven Piles (continued)**

Condition/Resistance Determination Method		Resistance Factor
Nominal Bearing Resistance of Single Pile in Axial Compression – Static Analysis Methods, $\phi_{stat}$	Side Resistance and End Bearing: Clay and Mixed Soils	
	$\alpha$ – method (Tomlinson, 1987; Skempton, 1951)	0.35
	$\beta$ – method (Esrig & Kirby, 1979; Skempton, 1951)	0.25
	$\lambda$ – method (Vijayvergiya & Focht, 1972; Skempton, 1951)	0.40
	Side Resistance and End Bearing: Sand Soils	
	Nordlund/Thurman Method (Hannigan et al., 2005)	0.45
	Nordlund Method – CIP piles (NYSDOT GEB, 2012)	0.60
	SPT – method (Meyerhof)	0.30
	CPT – method (Schmertmann)	0.50
	End bearing in rock (Canadian Geotech. Society, 1985)	0.45
Block Failure, $\phi_{b1}$	Clay	0.60
Uplift Resistance Of Single Piles $\phi_{up}$	Nordlund Method	0.35
	$\alpha$ – method	0.25
	$\beta$ – method	0.20
	$\lambda$ – method	0.30
	SPT – method	0.25
	CPT – method	0.40
	Static load test	0.60
	Dynamic test with signal matching	0.50
Group Uplift Resistance, $\phi_{ug}$	All soils	0.50
Lateral Geotechnical Resistance of Single Pile or Pile Group	All soils and rock	1.00
Structural Limit State	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2.1
	Timber piles	See the provisions of Article 8.5.2.2 and 8.5.2.3
Pile Drivability Analysis, $\phi_{da}$	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2.1
	Timber piles	See the provisions of Article 8.5.2.2
	In all three Articles identified above, use $\phi$ identified as “resistance during pile driving”	

## **10.5            LIMIT STATES AND RESISTANCE FACTORS**

### **10.5.5        Resistance Factors**

#### **10.5.5.2     Strength Limit States**

##### *10.5.5.2.3   Driven Piles*

C10.5.5.2.3 Add the following at the end of the commentary:

NYSDOT Geotechnical Engineering Bureau (GEB) conducted a study to analyze dynamic pile load test data with signal matching (CAPWAP) acquired on cast-in-place (CIP) piles driven in primarily cohesionless soils at NYSDOT projects from 1990-2010. This data was compared to pile resistance estimates using the Nordlund static analysis method and the Wave Equation Analysis of Pile Driving (WEAP) dynamic method. As a result of the study, resistance factors were revised for both analysis methods.

## 10.7 DRIVEN PILES

### 10.7.2 Service Limit State Design

#### 10.7.2.4 Horizontal Pile Foundation Movement

Replace Table 10.7.2.4-1 with the following:

**Table 10.7.2.4-1, Pile P-Multipliers,  $P_m$ , for Multiple Row Shading (Rollins et al., 2006)<sup>1</sup>.**

Pile CTC spacing (in the direction of loading)	P-Multipliers, $P_m$		
	Row 1	Row 2	Row 3 and higher
2.5 B	0.738	0.476	0.300
3.0 B	0.786	0.571	0.409
3.5 B	0.826	0.651	0.502
4.0 B	0.860	0.721	0.582
4.5 B	0.891	0.782	0.652
5.0 B	0.918	0.837	0.716
5.5 B	0.943	0.886	0.773
6.0 B	0.966	0.932	0.825
6.5 B	0.987	0.973	0.873
7.0 B	1.000	1.000	0.918
7.5 B	1.000	1.000	0.959
8.0 B	1.000	1.000	0.998
8.5 B	1.000	1.000	1.000

Replace the seventh paragraph of Article 10.7.2.4 with the following:

Loading direction and spacing shall be taken as defined in Figure 10.7.2.4-1. If the loading direction is perpendicular to the row (bottom detail in the figure), a group reduction factor of less than 1.0 should only be used if the pile spacing is 7B or less, i.e., a  $P_m$  of 0.786 for a spacing of 3B, as shown in Figure 10.7.2.4-1.

In the bottom detail of Figure 10.7.2.4-1, replace the spacing designated "5B or less" with "7B or less".

<sup>1</sup> Rollins, Kyle M, Kimball G. Olsen, Derek H. Jensen, Brian H. Garrett, Ryan J. Olsen, and Jeffery J. Egbert. 2006, "Pile Spacing Effects on Lateral Group Pile Behavior: Analysis." *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, American Society of Civil Engineers, Reston, VA, Vol. 132, No. 10, pp. 1272-1283

## **10.7       DRIVEN PILES**

### **10.7.3     Strength Limit State Design**

#### **10.7.3.6   Scour**

Add the following to the end of the first paragraph:

Except for critical bridges, piles for any abutment with the front row of piles battered and piles for integral abutments can be considered to meet the nominal lateral resistance requirement as per Article 3.7.5.

## **10.7 DRIVEN PILES**

### **10.7.3 Strength Limit State Design**

#### **10.7.3.8 Determination of Nominal Axial Pile Resistance in Compression**

##### *10.7.3.8.6 Static Analysis*

##### *10.7.3.8.6f Nordlund/Thurman Method in Cohesionless Soils*

Replace equation 10.7.3.8.6f-2 with the following:

$$q_p = \alpha_t N'_q \sigma'_v$$

## **10.7       DRIVEN PILES**

### **10.7.9     Probe Piles**

Delete the first sentence of the first paragraph of Article 10.7.9 and replace it with the following:

Probe or Test piles should be included where piles that are difficult or expensive to splice, such as precast concrete piles and timber piles, are used.

## **11.5        LIMIT STATES AND RESISTANCE FACTORS**

### **11.5.4     Extreme Event Limit State**

#### **11.5.4.1   General Requirements**

Add the following sentences at the end of the Article.

Abutments for single span non-critical bridges need not be investigated for Extreme Event I limit state. Also, refer to Articles 3.10.9.1 and C3.10.9.1.



**11.10                    MECHANICALLY STABILIZED EARTH WALLS**

**11.10.6                Safety Against Structural Failure (Internal Stability)**

**11.10.6.2            Loading (Internal Stability)**

*11.10.6.2.1        Maximum Reinforcement Loads*

*11.10.6.2.1a      Special Loading Conditions*

*C11.10.6.2.1a    Replace the second paragraph with the following:*

Note that  $T_{\max}$ , the tensile load in the soil reinforcement, is calculated twice for internal stability design as follows: (1) for checking reinforcement and connection rupture, determine  $T_{\max}$  with live load surcharge included in the calculation of  $\sigma_v$ ; (2) for checking pullout, determine  $T_{\max}$  with live load surcharge excluded from the calculation of  $\sigma_v$ , unless live load surcharge is anticipated within the active zone and controls the design.

## **12.6 GENERAL DESIGN FEATURES**

### **12.6.2 Service Limit State**

#### **12.6.2.1 Tolerable Movement**

Add a new Article 12.6.2.1a

##### **12.6.2.1a Tolerable Movement of a Stem Wall Supporting a Precast Reinforced Concrete Three-Sided Structure**

A stem wall may be used as a pedestal for the leg of a precast reinforced concrete three-sided structure in situations when the leg of the precast unit cannot be fabricated long enough for the design requirements. Stem walls may be placed on rock, piles, or soil depending on the design requirements.

Once the precast reinforced concrete three-sided structure is placed on the stem wall, the tolerable subsequent lateral movement at the top of the stem wall due to flexure of the stem wall shall be limited to  $\frac{1}{16}$  inch.

## **12.6 GENERAL DESIGN FEATURES**

### **12.6.4 Hydraulic Design**

Delete this article and replace it with the following:

Design criteria, as specified in Article 2.6 and "Hydraulic Design of Highway Culverts," FHWA (Third Edition, April 2012), for hydraulic design considerations shall apply.

## **12.11 REINFORCED CONCRETE CAST-IN-PLACE AND PRECAST BOX CULVERTS AND REINFORCED CAST-IN-PLACE ARCHES**

### **12.11.2 Loads and Live Load Distribution**

#### **12.11.2.1 General**

Add the following at the end of the first paragraph:

For wheel loads on box culverts skewed  $15^\circ$  or less the effect of skew may be neglected by using the provisions as given for culverts with main reinforcement parallel to traffic. For box culverts with skews greater than  $15^\circ$ , the effect of skew shall be considered in the analysis using the provisions as given for culverts with main reinforcement perpendicular to traffic.

Traffic traveling perpendicular to the span can have two or more trucks on the same design strip at the same time. This along with multiple presence shall be accounted for in design. For the Strength II limit state only one lane loaded with multiple presence shall be considered.

## **12.14      PRECAST REINFORCED CONCRETE THREE-SIDED STRUCTURES**

### **12.14.5    Design**

#### **12.14.5.3   Distribution of Concentrated Loads in Skewed Culverts**

Delete the last sentence of the first paragraph and replace it with the following sentences:

For culvert elements with skews greater than  $15^\circ$ , the effect of skew shall be considered in the analysis using the provisions as given for culverts with main reinforcement perpendicular to traffic.

Traffic traveling perpendicular to the span can have two or more trucks on the same design strip at the same time. This along with multiple presence shall be accounted for in design. For the Strength II limit state only one lane loaded with multiple presence shall be considered.

## **A13.4 DECK OVERHANG DESIGN**

### **A13.4.3 Decks Supporting Post-and Beam Railings**

#### **A13.4.3.1 Overhang Design**

In Figure A13.4.3.1-1 – Effective Length of Cantilever for Carrying Concentrated Post Loads, Transverse or Vertical, replace the dimension equation " $2X + W_b = b$ " with " $b$ ".

## **14.4 MOVEMENTS AND LOADS**

### **14.4.2 Design Requirements**

#### **14.4.2.2 High Load Multirotational (HLMR) Bearings**

##### *14.4.2.2.1 Pot Bearings and Curved Sliding Surface Bearings*

Delete the first and third bullets after the first paragraph and replace them with the following:

- The rotations due to applicable factored loads. (Rotation due to dead loads need not be considered and rotation due to roadway grade may be effectively eliminated with the use of a tapered sole plate);
- An allowance for uncertainties which shall be taken as 0.005 rad.

##### *14.4.2.2.2 Disc Bearings*

Delete the first and second bullet after the first paragraph and replace it with the following:

- The rotations due to applicable factored loads. (Rotation due to dead loads need not be considered and rotation due to roadway grade may be effectively eliminated with the use of a tapered sole plate);
- An allowance for uncertainties which shall be taken as 0.005 rad.