RHODE ISLAND
LRFD BRIDGE DESIGN MANUAL

STATE OF RHODE ISLAND
DEPARTMENT OF TRANSPORTATION

2007 EDITION

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SECTION 16 – EXISTING BRIDGE EVALUATION AND REHABILITATION

THIS SECTION IS UNDER DEVELOPMENT

SECTION 1
INTRODUCTION

1.1 SCOPE OF THE BRIDGE DESIGN MANUAL

This Manual is intended primarily for use as a reference and guide for the Rhode Island Department of Transportation (RIDOT) Engineers and Consultant Designers. It is not the intent of this Manual to replace the *American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications*, but rather to present:

- A compilation of design procedures.
- Interpretations of the *AASHTO LRFD Bridge Design Specifications*
- The standard practices and guidelines which constitute RIDOT policies.

Any exceptions to or deviations from these policies are subject to the approval of the Managing Bridge Engineer, and they must be justified based on sound engineering principles and judgment.

This Design Manual should also be used in conjunction with the Rhode Island Bridge Design Standard Details and the Design Policy Memos referenced in this Manual.

Any questions, requests for further clarifications, or suggestions for modifications and improvements regarding the material presented in this Manual may be emailed or addressed in writing to the Rhode Island Department of Transportation (Department) and directed to:

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Electronic copies of this document and the *Rhode Island Bridge Design Standard Details* are available and can be accessed at the Department's web site at http://www.dot.state.ri.us/engineering/pages/bluebookstart.htm

1.2 DESIGN SPECIFICATIONS

Unless otherwise amended or modified in this Manual, the provisions of the latest edition of the *AASHTO LRFD Bridge Design Specifications* shall apply. The specifications and publications as listed below shall also apply within the context of or as otherwise referenced in this Manual:

- *Rhode Island Department of Transportation Standard Specifications for Road and Bridge Construction*
- *AASHTO/AWS Bridge Welding Code*
- *AASHTO LRFD Bridge Construction Specifications*
- *Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals*
- *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*
- *AASHTO LRFD Movable Highway Bridge Design Specifications*

- *AASHTO Guide Specifications for Design of Pedestrian Bridges*
- *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges*
- *AASHTO Guide Specifications for Seismic Isolation Design*
- *AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges*
- *AASHTO Guide Design Specifications for Bridge Temporary Works*
- *AASHTO Construction Handbook for Bridge Temporary Works*
- *AASHTO Guide Specifications for Bridge Railings*
- *The American Railway Engineering and Maintenance of Way Association (AREMA) Manual for Railway Engineering*
- *American Institute of Timber Construction Manual*
- *AASHTO Guide Specifications for the Design of Stress-Laminated Wood Decks*
- *AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*
- *AASHTO Model Drainage Manual*

1.3 DESIGN PHILOSOPHY

Unless otherwise approved in writing by the Managing Bridge Engineer, the design and analysis of all new bridge structures and related highway structural components and connections shall be in accordance with the latest revision of the *AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications*.

Bridges and other related highway structural components and connections shall be designed for the specified limit state load combinations specified in the *AASHTO LRFD Bridge Design Specifications;* and they shall be designed with the objective of achieving overall safety, economy, ease of construction, ease of future inspection and maintenance, and aesthetics.

AASHTO 1.3.2 & 3.4

1.4 LOAD MODIFIERS

Unless otherwise approved by the Managing Bridge Engineer, the following load modifiers shall be used for the design of all new bridges.

AASHTO 1.3.2

1.4.1 Ductility

For all limit states, the load modifier for Ductility η_D, shall be taken as 1.00. Non-ductile components and connections shall not be used. Components of all new bridges, designed and detailed in accordance with the provisions of the *AASHTO LRFD Bridge Design Specifications*, shall generally be considered to exhibit adequate ductility.

1.4.2 Redundancy

In general, for the strength limit state, the load modifier for Redundancy η_R, shall be taken as 1.00. When possible, alternate load paths shall be designed for all members. Main elements and components whose failure could potentially cause collapse of the bridge shall be designated as failure-critical and the associated structural system as nonredundant. The load modifier for Redundancy η_R, for nonredundant members shall be taken as 1.05.

For all other limit states the load modifier for Redundancy η_R, shall be taken as 1.00.

1.4.3 Operational Importance

For the strength limit state, the load modifier for Operational Importance η_I shall be taken as follows:

η_I = 1.05 for bridges with Average Daily Traffic (ADT) in excess of 5,000.
η_I = 1.00 for all other bridges.

For all other limit states, the load modifier for the operational Importance η_I shall be taken as 1.00.

SECTION 2
GENERAL LOCATION AND DESIGN FEATURES

2.1 GENERAL LOCATION FEATURES

2.1.1 General

The general procedures for new bridge location and alignment evaluation are addressed in the *AASHTO LRFD Bridge Design Specifications* and should be used as a guide. These include minimum guideline requirements with respect to such factors as economics, environmental protection, construction, inspection and maintenance, highway geometrics, traffic safety, and hydraulics.

AASHTO 2.3.1 & 2.3.2

It is understood that all projects will have various constraints with respect to the above guidelines as well as other sensitive community and cultural considerations. Prior to commencement of the preliminary design, project constraints should be identified and discussed with the Department, and as the project is advanced any newly identified constraints will also need to be considered.

2.1.2 Vertical and Horizontal Clearances

2.1.2.1 Vertical and Horizontal Clearance - Highway Structures

In general the minimum design vertical clearance will be 14’-3” over all lanes and shoulders. Structures on the National Highway System and major interstate highway truck routes shall have a 16’-3” minimum vertical clearance. The above specified clearances include the provision for an additional 3 inch future wearing surface. The vertical clearance shall not be less than 13’-6” for the special cases when an exception to the above minimum clearances is warranted and approved by the Department.

AASHTO 2.3.3

Horizontal clearances for highway structures are subject to the requirements of *AASHTO Policy on Geometric Design of Highways and Streets* and the *AASHTO Roadside Guide*.

In all cases the vertical and horizontal clearance requirements must be approved by the Managing Bridge Engineer.

2.1.2.2 Vertical and Horizontal Clearance - Railroad Structures

Railroad vertical and horizontal clearance requirements shall be as stated in the *AREMA Specifications* and/or as determined by the affected railroad agencies having jurisdiction. In all cases the clearance requirements must meet the approval of the Managing Bridge Engineer.

2.1.2.3 Vertical and Horizontal Clearance - Waterway Crossings

Waterway crossing clearance requirements shall be based on the hydraulic study. For bridges over navigable waters, the clearance requirements must be coordinated with the U.S. Coast Guard and/or other agencies having jurisdiction. Additionally the *AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges* should be referenced with respect to clearances. In all cases the clearance requirements must meet the approval of the Managing Bridge Engineer.

2.1.2.4 Vertical Clearance - Pedestrian Bridges

The minimum vertical clearance under pedestrian bridges shall be 17'-3". This clearance includes the provision for an additional 3" future wearing surface.

2.2 SUBSURFACE EXPLORATION PROGRAM

Requirements for subsurface exploration programs are included in Article 10.2 of this Manual.

2.3 BRIDGE SUPERSTRUCTURE AND SUBSTRUCTURE SELECTION

2.3.1 General

The selection of the bridge superstructure and substructure types at a given site will be governed by such factors as span length; span arrangement; geometry of the roadway, railroad, river or other feature to be spanned; economics; foundation conditions; durability and design life; future maintenance and inspection; construction considerations and restrictions; traffic management and safety; environmental considerations; cultural impacts; and aesthetics. No fixed rules have been formulated for the selection of bridge types, but certain general policies have been outlined herein as well as in the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. In the selection of the structure type, sound engineering judgment shall be exercised for any exceptions to the policies specified herein. It is important that the structure type requirements be coordinated with the development of the approach vertical and horizontal geometry. The cross section of the bridge, including shoulder and sidewalk widths, shall be consistent with the highway approach section and shall meet the additional requirements of Article 2.6.3.2 of this Manual.

AASHTO 2.5

The selection of preliminary span arrangements, superstructure and substructure types, and other considerations (as described above) to assist in making a final decision, must be presented in the Bridge Type Study Report and discussed with the Managing Bridge Engineer.

The selection of the bridge system (superstructure and substructure) shall be consistent with the latest Department Policy Memo on *Context Sensitive Solutions*.

2.3.2 Structural Steel Superstructures

Typical steel superstructures as described below may consist of:

- Composite rolled beam sections
- Composite built-up welded plate girder sections
- Composite built-up welded box girders

Composite multiple rolled beam sections (with or without cover plates) may generally be used for simple spans up to about 90 feet and for continuous spans up to about 180 feet. The availability of the proposed rolled beam sections must be considered during the design stage.

Composite multiple built-up welded plate girders should be considered for span lengths in excess of about 90 feet.

Composite multiple built-up welded box girders should be considered for span lengths in excess of about 150 feet.

For span lengths in excess of about 300 feet, refer to Article 2.3.4.

The use of High Performance Steel (HPS) should only be specified upon written approval of the Chief Engineer.

2.3.3 Concrete Superstructures

Typical concrete superstructures as described below may consist of:

- Multiple prestressed butted voided slabs (with or without composite concrete overlay)
- Multiple prestressed butted voided box beams (with or without composite concrete overlay)
- Composite prestressed spread box beams
- Composite prestressed New England Bulb-T beams
- Composite prestressed New England Bulb-T beams (spliced)

Multiple prestressed butted voided slabs, with or without composite concrete overlay, should be considered for span lengths of up to about 60 feet.

Prestressed butted voided slabs without a composite concrete overlay shall not be considered for bridges with an average daily traffic (ADT) in excess of 5,000, or when the average daily truck traffic (ADTT) is in excess of 200.

Multiple prestressed butted voided box beams, with or without composite concrete overlay, should be considered for span lengths from about 60 feet to about 120 feet.

Prestressed butted voided box beams without a composite concrete overlay shall not be considered for bridges with an average daily traffic (ADT) in excess of 5,000, or average daily truck traffic (ADTT) in excess of 200.

Composite prestressed simple span New England Bulb Tee (NEBT) sections should be considered for span lengths from about 70 to about 120 feet. Spliced post-tensioned prestressed New England Bulb-Tee sections may be considered for spans up to about 200 feet.

For span lengths larger than about 150 feet where the repetition of fabricating precast sections is economical (if there are 20 or more spans), the use of segmental concrete bridges should be considered.

For long span lengths in excess of about 300 feet, refer to Article 2.3.4.

2.3.4 Other Superstructure Types

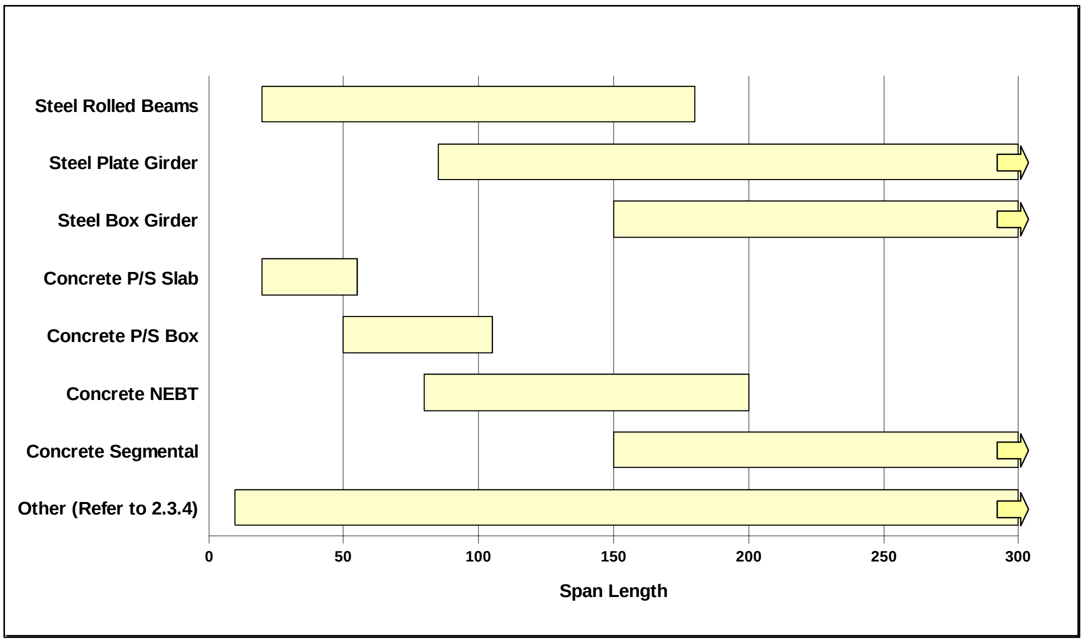
Single or multiple-cell precast concrete culverts or corrugated steel/aluminum culverts should be considered for short span waterway crossings. Precast three sided concrete structures (open bottom culverts) may be considered for span lengths up to 40 feet.

Cast-in-place slab superstructures may be considered for certain short span bridges over highways or waterways (up to about 30 feet) whose geometrical or cross-sectional limitations would preclude the use of precast prestressed superstructure members.

Arches and rigid frames (cast-in-place or precast bridges) may be considered, particularly on parkways or in other locations where aesthetics is given special consideration.

Timber bridges may be considered for short single-span bridges in low-speed rural locations with low traffic and low truck volumes (ADT less than 700 and ADTT less than 20). The timber bridge type must be approved by the Managing Bridge Engineer.

For span lengths larger than about 300 feet, the following structure types must be considered within their economical span ranges: steel plate girders, steel box girders, segmental concrete, steel trusses, cable-stayed (steel or concrete), and arch type structures (steel or concrete).



2.3.5 Substructure Types

The selection of substructure type will be influenced by the foundation support system (spread footing, pile or drilled shaft supported). The factors discussed in Article 2.3.1, should also be considered in the substructure selection process. The feasible foundation types considered as part of the geotechnical investigation should also be discussed in the Type Study Report.

The abutment types considered during the substructure selection process may include stub abutments, wall (or full height) type abutments, stub abutments used in conjunction with MSE type walls, integral abutments, and semi-integral abutments. In general, when geometric and subsurface conditions permit, the use of integral abutments is preferred. Refer to Articles 11.3.3 and 11.3.4 of this Manual for Integral and semi-integral abutment design requirements and limitations.

Pier types may include hammerhead piers, cap and column-type piers, wall-type piers, and pile trestle piers. Pier caps may be post-tensioned if determined to be economically feasible. Additionally, the use of integral post-tensioned pier caps may be appropriate when eliminating roadway joints and/or when vertical clearance requirements control.

The substructure foundations should be set deep enough to allow for future widening of the roadway below the bridge accounting for reasonable adjustments in roadway profile and cross slopes. The use of pile trestles and perched abutments is also encouraged at waterway crossings in order to eliminate costly cofferdam construction and to minimize environmental disturbances.

Raised piers constructed using “floating” cofferdams should be considered in deep water construction.

2.3.6 Prefabricated Bridge Elements and Systems

The feasibility of specifying prefabricated superstructure and substructure elements (decks; stay-in-place forms; bridge barriers; abutment and pier footings; abutment and pier stems; pier caps; and retaining wall components) must be considered when site conditions permit and when traffic disruptions during construction are of primary concern. The considerations for the use of prefabricated superstructure and substructure components must be presented and discussed in the Bridge Type Study Report.

2.4 ADDITIONAL DESIGN CONSIDERATIONS

2.4.1 Elimination of Roadway Joints

In order to eliminate the joint-related problems commonly associated with intermediate roadway deck joints, continuous span bridges should be used whenever possible. The design and construction of bridges without deck joints may also be achieved by detailing jointless deck slabs at piers. Jointless deck slabs may be used on multiple single-span new prestressed bridges or on multiple single-span existing bridges being retrofitted. In addition, roadway expansion joints at abutments and piers may be eliminated by incorporating integral abutment and/or pier design and construction (superstructure integrated with the abutment and/or pier caps). The designs incorporating integral abutments, integral pier caps, and jointless bridge decks must consider the secondary stresses caused by the response of the superstructures to thermal changes as well as by the anticipated substructure differential settlement(s).

2.4.2 Maintenance and Inspection

Bridge abutments, piers, decks, and associated structural components shall be detailed to facilitate future inspection and maintenance. Details shall be developed that will enhance the inspection, rehabilitation or replacement of the bridge roadway deck expansion joints and bearings. Provisions for maintenance and inspection of these elements shall assume that (a portion of) the facility will remain open to traffic unless a feasible detour is available. When it is impractical or unfeasible to develop details which allow for the inspection, rehabilitation or replacement of these components as stated above, the Managing Bridge Engineer shall be so notified at the preliminary design phase.

2.4.3 Bridge Beam Spacing

In determining girder spacing, consideration should be given to providing beam spacing that is consistent with 11 foot minimum travel lanes as a temporary condition during future deck repair or reconstruction work on the bridge. However, when a feasible detour is available, maximizing the girder spacing is preferred so as to achieve a more cost-effective design.

2.4.4 Provisions for Jacking

Provisions for jacking the superstructure for future maintenance and/or inspection shall be incorporated into the design and detailed on the plans. This may include (but not be limited to) designing and detailing adequately sized diaphragms, connections and related components and/or properly designing and detailing the substructure units. The details and design shall account for a practical jack height (minimum 6 inches), width, capacity, location and the number of jacks that may be required. For prestressed and cast-in-place concrete bridges, jacking from the diaphragms may not be practical, and therefore alternate methods should be investigated with appropriate provisions incorporated into the design.

Refer to Article 3.2.5 of this Manual for the temporary jacking load requirements.

Factored dead load and live load reactions for the Strength I limit state and the permissible locations and schemes for jacking the superstructure must be indicated on the drawings.

2.4.5 Fracture Critical and Fatigue Sensitive Details

Fracture critical and/or fatigue sensitive details must be identified and brought to the attention of the Managing Bridge Engineer during the development of the Type Study Report. Fracture critical and fatigue sensitive members, or any other details that may require special attention during scheduled inspections, must be clearly noted on the plans.

2.4.6 Inspection and Maintenance Manuals

For complex bridges, such as movable bridges and long span bridges, an “Inspection and Maintenance Manual” shall be prepared by the Consultant. This Manual shall address the inspection and maintenance of the complex components requiring specific instruction. The Manual shall be submitted to the Department along with the final contract documents.

2.4.7 Protective Screening of Overpass Bridges

The need for protective screens on an overpass bridge and the detailing requirements are addressed in the Rhode Island Bridge Standard Details.

2.5 DEFORMATIONS

2.5.1 General

The referenced AASHTO criteria for deflection are mandatory for all bridge types and shall not be considered optional. *AASHTO 2.5.2.6.2*

2.5.2 Vehicular Bridge Deflection Criteria

Except for orthotropic decks, precast reinforced concrete three sided structures (open bottom culverts), metal grid decks, and wood construction, the vehicular deflection due to live load plus impact shall not exceed 1/1100 of the span. Live load plus impact deflections shall be computed in accordance with the assumptions made when computing the stress in the member. The modular ratio for composite design shall be the same as that used for member stress calculations.

When the vehicular live load deflection, computed based on the above assumption, exceeds the limiting deflection criteria of 1/1100 of the span, an alternate method may be used to compute live deflection. The alternate method may consider the "principles" listed in the referenced *AASHTO LRFD Bridge Design Specifications* and the vehicular load for live load deflection in accordance with Article 3.4.5 of this Manual. *AASHTO 2.5.2.6.2*

The vehicular deflection limiting criteria of 1/1100 of the span may be waived subject to the approval of the Managing Bridge Engineer, but in no case shall the limiting live load deflection criteria be less than the optional deflection limits noted in the *AASHTO LRFD Bridge Design Specifications* using the vehicular load specified in Article 3.4.5 of this Manual.

Deflection criteria for orthotropic decks, precast reinforced concrete three-sided structures (open bottom culverts), metal grid decks, and wood construction shall be in accordance with the referenced AASHTO criteria. *AASHTO 2.5.2.6.2*

2.5.3 Pedestrian Bridge Deflection Criteria

For bridges with only pedestrian and/or bicycle traffic, the deflection due to live load shall not exceed 1/500 of the span. Live load shall be computed in accordance with the assumptions made when computing the stress in the member. The modular ratio for composite design shall be the same as that used for member stress calculations.

2.5.4 Span to Depth Ratio

The referenced AASHTO span to depth ratio controls are not mandatory but may be used as a guide during the preliminary design. *AASHTO 2.5.2.6.3*

2.6 HYDROLOGY AND HYDRAULICS

2.6.1 Hydraulic Analysis

The Requirements for hydrology and hydraulic analysis and studies shall be in accordance with the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 2.6 & 12.6.4*

The main objective of the hydrology and hydraulic design is not only to address the hydraulic-related design features of a project, but also to consider the non-hydraulic factors, specifically related to the environmental impacts and to the project permitting requirements. Generally in order to minimize environmental impacts, when replacing a bridge on existing alignment (or very close proximity), it is the Department's preference to maintain the hydraulic opening of the existing bridge.

In accordance with the State of Rhode Island Department of Environmental Management's (RIDEM) Rules and Regulations Governing the Administration and Enforcement of the Freshwater Wetland Act, projects proposing a change in the drainage characteristics of freshwater wetlands must identify and describe the project components that may alter and/or affect the wetlands' ability to store, meter out, or reduce the damaging effects of flooding and flood flows. As part of the wetland functions, values and impacts evaluation, the regulations require an analysis comparing the pre- and post- project conditions due to the 2-year, 10-year, 25-year, and 100-year, 24-hour, Type III storm events.

The Designer is referred to RIDEM's Office of Water Resources website for the rules and regulations governing the enforcement of the freshwater wetland act, applications, hydrologic and hydraulic modeling guidance, and other related information and required engineering documentations.

2.6.2 Bridge Scour

The Bridge Scour design requirements shall be in conformance with the referenced Articles of *AASHTO LRFD Bridge Design Specifications* and Articles 10.3 and 3.2.2 of this Manual.

AASHTO 2.6.4.4.2 & 3.7.5

For the design of new bridge structures, the use of riprap or other scour counter-measures as a means of scour protection is not permitted. All foundations must be designed to withstand the conditions of scour for the design flood and the check flood. Bridge scour shall be considered for the service and strength limit states in accordance with Article 10.3 of this Manual and for the extreme limit state in accordance with Article 3.2.2 of this Manual. The design flood for scour considered for the service and strength limit states shall be the 100-year event. The check flood for scour considered for the extreme limit state shall be the 500-year event.

As part of the site-specific data collection, the Designer is also referred to the information available in the *Phase I Scour Screening Report* (a preliminary evaluation and ranking of the scour susceptibility of Rhode Island Bridges), prepared by Whitman & Howard dated June 16, 1993. This document is available for review at the office of the Managing Bridge Engineer.

2.6.3 Deck Drainage

2.6.3.1 General

The general requirements for deck drainage shall be in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications* unless modified herein.

AASHTO 2.6.6

In design of the drainage for bridge decks, the publication Design of Bridge Deck Drainage, Hydraulic Engineering Circular 21, (HEC 21), FHWA publication FHWA-SA-92-0100*,* may be used as a guide.

Due to the high maintenance associated with deck drainage structures, where possible, it is preferred that all deck drainage be carried off the bridge to the approach drainage structures and that the number of deck scuppers be kept to a minimum (consistent with the requirements specified herein). Scupper design requirements shall be in accordance with Article 2.6.3.3 of this Manual.

The following relationship (the "rational method") with the indicated assumptions shall be used:

$Q = CiA$ where,

Q = Peak rate runoff (ft^3/sec)
C = Runoff coefficient (0.9 for both concrete and asphalt pavements)
i = The average rainfall intensity (inches per hour). For the given frequency and duration, the time of concentrations shall be as follows:

- For the 10 year frequency flood, five (5) minute duration taken to be 5.6 inches per hour.
- For the 50 year frequency flood, five (5) minute duration taken to be 7.2 inches per hour.

A = Drainage area (acres)

Sub-pavement drains shall be provided at expansion dams and scuppers when sub-pavement ponding would otherwise tend to develop at the top of the deck slab.

All concrete deck overhangs shall be provided with a drip notch.

2.6.3.2 Cross Slopes and Profiles

The minimum cross slope for driving lanes on bridge decks shall preferably be 2%. The minimum longitudinal superstructure profile gradient shall be not less than 0.5%. Sag vertical curves with low points should be avoided in deck profiles.

2.6.3.3 Scupper Design

Bridge scuppers shall be provided when the inlet spacing analysis indicates that they are required as herein specified. Scuppers shall preferably be located to intercept the flow upgrade of the expansion joints.

The spread of the deck drainage shall not encroach on any portion of the adjacent travel lane. However, for roadways with very narrow shoulders, the spread of the deck drainage may encroach on one-half of the adjacent traffic lane when approved by the Bridge Managing Engineer.

Scuppers shall be spaced for the 10 year frequency storm with a five (5) minute duration. When sags can not be avoided, scuppers in sags shall be designed for a 50 year frequency storm with a five (5) minimum duration.

Where possible, bridge scuppers shall be detailed with “free fall” drop pipes, provided that the runoff is not discharged on roadway, sidewalks, embankment, private property, Railroad Right-of-Way or against any portion of the structure. In addition, disposal of discharge shall be in a manner consistent with all environmental permitting requirements.

Downspouts, where required, shall not be less than 8” in diameter and shall be provided with readily accessible cleanouts. Discharge shall be into storm drains when possible. However, when such accommodations are not available or practical, appropriate splash blocks shall be provided on the finished ground below the vertical downspout. Details of the scupper piping system shall be such that no water is discharged against any portion of the structure or onto natural ground where it may cause erosion. The drainage discharge should not be allowed to flow across sidewalks or roadways. Drain pipes shall also not be permitted to discharge onto the railroad right-of-way without prior written permission from the appropriate railroad agency.

All scupper drainpipes shall be pitched preferably at 8 percent, but in no case less than 2 percent. Changes in direction should be with long sweep transitions of no less than an 18 inch radius. Bends sharper than 45 degrees are not permitted.

Two-way and three-way bridge scupper details as well as down spout details are included in the Rhode Island Bridge Standard Details.

2.6.4 Project Documentation of Hydraulic Data

As applicable, as a minimum the following hydraulic information shall be provided on the contract drawings:

- Design flow drainage area
- Waterway opening and clearance (vertical and horizontal)
- Water surface elevations
- Design tidal elevations (mean high tide, mean high water, mean low water, mean low low water)
- Design velocities for the 100-year and 500-year storms events.
- Anticipated depth of scour at each substructure due to the 100-year and the 500-year storm events

2.7 BRIDGE AESTHETICS

It is the Department's intent to consider aesthetics as an integral part of the Context Sensitive Solution policy. As a general approach, the process will require establishing the project requirements during the preliminary design phase and setting specific objectives with respect to the aesthetic design. This will be accomplished through the incorporation of the Department's Context Sensitive Solutions process, which begins at the project inception and continues through out the design process. The level of aesthetic involvement will vary form project to project. Some may only consist of the use of standard architectural treatments, while other projects will require architectural design input and specific aesthetic considerations. In all cases, bridge aesthetics design will involve collaboration with various design disciplines of the Department and should be coordinated through the Bridge Section.

As a minimum, unless otherwise directed by the Department, abutments, piers and walls shall have an architectural surface treatment specified using concrete form liners. There are a wide variety of different architectural treatments (including simulated stone masonry) which may be achieved with the use of form liners. Some possible treatments are included in the Rhode Island Bridge Standard Details, and several others are available through various form liner manufacturers. The final decision on an appropriate surface treatment must be coordinated with and approved by the Bridge Section.

SECTION 3
LOADS AND LOAD FACTORS

3.1 GENERAL SCOPE

The minimum requirements for loads and forces and their application, the applicable load factors, and the load combinations used for the design of new bridges shall be in accordance with the *AASHTO LRFD Bridge Design Specifications* unless otherwise modified or clarified in this Section.

3.2 LOAD FACTORS AND COMBINATIONS

3.2.1 General

Except as modified herein, the *AASHTO LRFD Bridge Design Specifications'* minimum requirements for load and load combinations shall apply. *AASHTO 3.4*

3.2.2 Limit States

3.2.2.1 Strength II Limit State

The Strength II limit state shall not be considered. The Department has no permit or special design vehicle requirement. *AASHTO 3.4*

3.2.2.2 Extreme Event I Limit State

The Extreme Event I limit state shall include load combinations considering both the Upper-Level Event Earthquake (ULE) as well as the Lower-Level Event Earthquake (LLE). The designer shall only consider the maximum design force and displacement effects produced from each event.

Since the probability of an earthquake occurring in the presence of the maximum design scour is small, only 30% of the scour depth resulting from the check flood needs to be considered under this limit state.

The presence of scour has the effect of changing the overall bridge geometry and may increase the natural period of the bridge, thus leading to smaller forces but larger displacements. Therefore design forces and displacements shall be based on the worst case load scenarios of 30% scour and no scour.

3.2.2.3 Extreme Event II Limit State

This limit state includes ice load and vessel collision forces. Since the probability of these loads occurring in the presence of the maximum scour is small, only 50% of the maximum scour depth resulting from the check flood needs to be considered under this limit state.

3.2.3 Load Factors

Except as specified herein, the load factors for the various limit states shall be in accordance with the referenced *AASHTO LRFD Bridge Design Specifications.* *AASHTO 3.4.1*

- For open girder and multiple steel box girders systems, the load factor for temperature gradient γ_{TG} shall be taken as 0. For all other bridge types, the load factor for temperature gradient γ_{TG} shall be in accordance with the referenced *AASHTO LRFD Bridge Design Specifications.*

- The load factor for uniform temperature γ_{TU} for the design of integral abutments shall be taken as 1.00 (refer to Article 11.3.3.3.5 of this Manual).
- The load factor for live load γ_{EQ} for Extreme Event I shall be taken as 0.
- The load factor for dead load γ_p for Extreme Event I and Extreme Event II shall be taken as 1.00.
- The load factor for settlement γ_{SE} shall be taken as 1.00.

3.2.4 Construction Load Combination

Construction loads that include force effects developed during construction should be considered if warranted. This load combination must also include force effects from any project construction constraints that are likely to induce additional stresses.

The load combination and factors for construction loads shall be in accordance with the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 3.4.2*

Additionally, when cranes or other heavy construction equipment are expected to come within close proximity of abutments or retaining walls, the construction load combination must consider a surcharge load appropriate with the anticipated construction equipment (but in no case less than an equivalent soil height of 5 feet).

All construction constraints and construction load assumptions shall be specified on the contract drawings.

3.2.5 Temporary Jacking Loads

Components designed for future jacking of the superstructure (including the sizing of the jacks) in accordance with Article 2.4.4 of this Manual shall be designed for the Strength I limit state. Loading shall include both dead load and live load in anticipation that the bridge will remain open to traffic during the future jacking operations. To account for possible load redistribution during the jacking operation, the load factors for both dead load and live load shall be increased by 10% (unless a higher percentage is warranted).

3.3 PERMANENT LOADS

3.3.1 Superimposed Dead Load Distribution

The wearing surface superimposed dead load shall be distributed equally among all beams.

Unless a refined method of analysis is used (refer to Article 4.4.4 of this Manual), the sidewalk, safety walk, barrier/railing, and sidewalk live load superimposed dead loads shall be distributed 60 percent to the fascia beams and 40 percent evenly distributed among all interior beams. If the sidewalk spans over more than one beam, then 60 percent of the above superimposed dead loads shall be distributed evenly among the beams carrying the sidewalk and 40 percent among the remaining interior beams.

3.4 LIVE LOADS

3.4.1 Design Vehicular Live Load

The design vehicular live load shall be the HL-93 designated load as provided in the *AASHTO LRFD Bridge Design Specifications*.

AASHTO 3.6.1.2

3.4.2 Design Pedestrian Live Load

The design pedestrian live load shall be as specified in the referenced Article of the *AASHTO LRFD Bridge Design Specifications,* except that pedestrian bridges (with only pedestrian and/or bicycle traffic) shall also be designed for a live load truck consisting of 6,000 lbs and 24,000 lbs axles spaced 14 feet apart. The dynamic load allowance need not be applied to this specified design truck.

AASHTO 3.6.1.6

3.4.3 Multiple Presence Factors

The multiple presence factors specified in Table 1 of the *AASHTO LRFD Bridge Design Specifications* shall not be applied in conjunction with the load distribution factors determined using the approximate method specified in the *AASHTO LRFD Bridge Design Specifications*. These multiple presence factors have already been included in the approximate equations for distribution factors. In addition, the multiple presence factor of 1.2 from Table 1 should be removed for the purpose of fatigue investigations.

AASHTO 3.6.1.1.2 & AASHTO 4.6.2.2 & 4.6.2.3

3.4.4 Application of Design Vehicular Live Load

The application of vehicular live load shall be in accordance with the *AASHTO LRFD Bridge Design Specifications*. The additional investigation (for the negative moment and reaction at interior supports) for pairs of the design tandem combined with the design lane, as discussed in the commentary Article of the referenced *AASHTO LRFD Bridge Design Specifications* need not be considered.

AASHTO 3.6.1.3

3.4.5 Vehicular Load for Live Load Deflection Evaluation

The vehicular live load used in the evaluation of live load deflection as outlined in Article 2.5 of this Manual shall be taken as the larger of:

- 125% of the HL-93 design truck only.
- 33% of the HL-93 design truck taken together with the design lane load.

3.4.6 Dynamic Load Allowance

3.4.6.1 Dynamic Load Allowance for Design Vehicular Live Load

Static effects of the HL-93 design truck or design tandem shall be increased by the percentage specified in the *AASHTO LRFD Bridge Design Specifications* for Dynamic Load Allowance (IM):

AASHTO 3.6.2

3.4.6.2 Vehicular Dynamic Load Allowance Exceptions

Dynamic Load Allowance does not need to be applied to:

- The HL-93 design lane load
- Retaining walls not subject to vertical reaction from the superstructure
- Pedestrian loads

- Centrifugal force effects
- Braking force
- All foundation components entirely below ground (except as specified for integral abutments under Article 11.3.3.3.2 of this Manual).
- Elastomeric bearings

3.5 WIND LOADS

3.5.1 General

Wind loads on live load and wind loads on structure shall be determined in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications,* unless otherwise modified here in this Article. *AASHTO 3.8*

3.5.2 Design Wind Velocity

Except for highway signs, luminaries and traffic signals (refer to Section 15 of this Manual), the wind velocity at 30 feet above low ground or above the design water level (V_{30} used in the referenced equation of the *AASHTO LRFD Bridge Design Specifications*), may be obtained from the latest ASCE 7-05 wind speed figure for Exposure C Category (Surface Roughness C) or alternatively be taken as follows: *AASHTO Equation 3.8.1.1-1*

County	V_{30} (mph)
Providence, Kent	110
Washington, Newport, Bristol	120

3.5.3 Exposure Category and Design Criteria

Unless otherwise specified below, the wind pressures at various heights shall be determined in accordance with the criteria as specified for “Open Country” (described in the commentary of the referenced article of the *AASHTO LRFD Bridge Design Specifications*). The AASHTO LRFD “Open Country” exposure criteria most closely corresponds with the wind design criteria for the “Exposure Category C” as described in the latest revision of the ASCE 7-05 “*Minimum Design Loads for Buildings and other Structures*”. *AASHTO C3.8.1.1*

The AASHTO LRFD “Suburban” category shall only be used at the approval of the Managing Bridge Engineer. Should it be determined that this category is appropriate, the wind pressures shall be calculated in accordance with the criteria for “Exposure Category B” as specified in the latest revision of the above referenced ASCE 7-05 publication.

The AASHTO LRFD “City” category shall not apply for Rhode Island.

3.5.4 Loads from Superstructure

For typical steel or concrete beam (or girder) type bridges with vertical under-clearances less than 30 feet (above low ground or the design water level), in lieu of computing wind pressures for the various angles of attacks as specified in the referenced Article of the *AASHTO LRFD Bridge Design Specifications,* the following design wind pressures (P_D) may be used: *AASHTO 3.8.1.2.2*

	V_{30} (mph)	
	110	**120**
Lateral Wind Pressure (psf)	60	72
Longitudinal Wind Pressure (psf)	14	17
The above calculated values assume a base wind pressure P_B (at 100 mph) of: 50 psf lateral load (0 degree skew angle) 12 psf longitudinal load (30 degrees skew)		

The above transverse (lateral) and longitudinal design wind pressures shall be applied simultaneously.

3.5.5 Wind Pressure on Vehicle: WL

For typical steel or concrete beam (or girder) type bridges with vertical under clearances less than 30 feet (above low ground or the design water level), in lieu of computing wind components of live load for the various angles of attacks as specified in the referenced Article of the *AASHTO LRFD Bridge Design Specifications,* the designer may apply the following components of normal and parallel force to the live load: *AASHTO 3.8.1.3*

100 pounds per linear foot (normal component)
40 pounds per linear foot (parallel component)

The above loading components shall be applied simultaneously.

3.6 EARTHQUAKE EFFECTS

3.6.1 Applicability

Articles 3.10.1 through 3.10.7 of the *AASHTO LRFD Bridge Design Specifications* shall be deleted and replaced with the requirements specified in this Article 3.6.

These provisions shall apply to bridges of conventional beam or girder, box girder, and truss superstructure construction. For other types of bridges (cable-stayed, suspension, arch types and movable), appropriate design provisions shall be proposed by the designer subject to the approval of the Managing Bridge Engineer.

Seismic effects on buried structures (as defined in Section 12 of this Manual) need not be considered, except when they may be subject to unstable ground motions, such as liquefaction or large ground deformations due to very soft ground.

3.6.2 Design Philosophy

The *AASHTO LRFD Bridge Design Specifications'* philosophy is based on the objective that exposure of structures to earthquakes of a given intensity will not cause the collapse of a structure and that the damage will be detectable and repairable. Additionally, the AASHTO LRFD seismic provisions require that bridges which are considered "essential" should as a minimum remain open to emergency vehicles immediately after the "design" earthquake (475-year return period) and that bridges which are considered "critical" should remain open to all traffic after the "design" earthquake and be usable by emergency vehicles after a "large" earthquake (2500-year return period). Based on this objective a design approach is established as a function of the bridge importance classifications ("other", "essential" or "critical"), with a more conservative approach taken for structures classified as critical or *AASHTO 3.10.3* essential. Thus the *AASHTO LRFD Bridge Design Specifications* has an implied "three-level" design performance objective.

Consistent with the above objectives and philosophy, the current Rhode Island seismic design criteria are based on a "two-level" design principle. The intent is to prevent collapse during a rare earthquake, referred to as the upper-level event (ULE), and to achieve minimal or no damage to bridges during the expected earthquake, referred to as the lower-level event (LLE). For each design earthquake (ULE and LLE) a desired performance objective is established for each bridge classification (critical or non-critical). A more comprehensive discussion of the Rhode Island seismic design philosophy and objectives is provided in commentary format included in Appendix B (Section 3) of this Manual. Appendix B also includes a flow chart which summarizes the Rhode Island seismic design and analysis process stipulated in Articles 3.6 and 4.5 of this Manual.

3.6.3 Seismic Design Approach and Considerations

All bridge components and their foundation systems must provide the means of adequately dissipating energy or must be capable of sufficiently resisting seismically induced structure component displacements. The systems must provide for uninterrupted load paths for transmitting seismically induced forces into the ground.

The primary objective in seismic design is to ensure against: foundation failure, liquefaction failure, support failure at the bearings and expansion joints within a span, shear or moment failure of the columns, and failure of the structure connecting components. Based on the bridge classification and the performance objective, it is the Department's policy to permit designs such that inelastic deformation is ductile and damage may occur provided it is at locations which can be readily inspected and repaired after the design earthquakes. Plastic hinging in superstructure components (which may impact the gravity load support capabilities of the structure) or bearing systems not capable of providing the expected seismic displacement and forces are not permitted.

Some basic seismic design concepts that will contribute to an improved seismic performance by enhancing the load distribution to the substructures are: continuity of the superstructure, symmetry in structure stiffness and geometry, and overall structure redundancy. These design concepts not only benefit the seismic performance but also the overall performance of the structure.

3.6.4 Bridge Classification and Performance Objectives

All bridges will be classified as either **critical** or **non-critical** to meet the performance objectives specified in this Article (also refer to commentary in Appendix B - Section 3, of this Manual). The Department will classify all bridges.

Critical Bridges: Critical bridges are generally those that provide a vital link and that have to remain open to emergency vehicles and for security/defense purposes immediately following an earthquake. The performance objective for critical bridges is for a structure to sustain no damage and remain in service (permit full access to normal traffic after the bridge has been inspected) immediately following the lower-level earthquake. The bridge may sustain minimal (or no) damage but allow limited access to emergency vehicles and be repairable within a very short period of time following the upper-level earthquake.

Non-Critical Bridges: Non-critical bridges are all bridges that are not classified as critical. For non-critical bridges the structure may sustain minimal or no damage but remains in service (permit full access to normal traffic after the bridge has been inspected) immediately following the lower-level earthquake. Minimal damage is defined as some minor inelastic behavior with no permanent deformation after which repairs can be made under non-emergency conditions. However, following the upper-level earthquake, the bridge may sustain significant damage but must not collapse. Non-critical bridges should allow limited traffic access to light emergency vehicles after temporary shoring is installed.

3.6.5 Seismic Analysis and Design Procedure

The bridge classifications described in Article 3.6.4, in conjunction with the site classifications as described in Article 3.6.8, form the basis for the selection of the seismic analysis, design and detailing procedures. These procedures (described in Articles 3.6.10, 3.6.11 and 3.6.12) outline the specific requirements for analysis and design reflecting the variation in seismic risk.

3.6.6 Design Earthquakes

The following earthquakes are used to be used for the seismic analysis and design of bridges in Rhode Island:

Upper-Level Earthquake (ULE): The upper-level ground motions correspond to a ground motion having approximately 3% probability of exceedance in 75 years. This earthquake has a return period of approximately 2475 years and represents the rare design earthquake.

Lower-Level Earthquake (LLE): The lower-level ground motions correspond to a ground motion having approximately 15% probability of exceedance in 75 years. This earthquake has a return period of approximately 475 years and represents the expected design earthquake. The LLE is similar to the *AASHTO LRFD Bridge Design Specification* earthquake.

3.6.7 Design Response Spectra

Unless a site-specific procedure is required, the design spectra acceleration shapes shall be as shown In Figures 3.6.7.1-1 and 3.6.7.1-2 (Appendix A, Section 3).

Site-specific special studies will be required for both critical and non-critical bridges when the site is classified as Site Class F (as defined in Table 3.6.8-1, Appendix A, Section 3) or for critical bridges for which a higher degree of confidence is desired. The need for a site specific study shall be discussed with the Managing Bridge Engineer.

3.6.8 Site Classifications

The site shall be classified as defined in Table 3.6.8-1 (Appendix A, Section 3) according to the weighted average soil shear wave velocity in the upper 100 feet of the site profile. If the soil shear wave velocity is not known, the site class may be classified according to the SPT blow count (N-value) or the undrained shear strength according to one of the following two methods:

- N Method: Weighted average of the standard penetration resistance of all soils in the upper 100 feet of the soil profile. The standard penetration resistance shall be as directly measured in the field without corrections (not to exceed 100 blows/ft for any given layer).

- S_u Method: Weighted average of N_{ch} for only the cohesionless soil layers (PI<20) in the top 100 feet and the weighted average S_u for only the cohesive soil layers (PI>20) in the top 100 feet. If the N_{ch} and S_u criteria differ, the softer soil category shall be selected.

Soils characteristics corresponding to Site Class F, as defined in Table 3.6.8-1 (Appendix A, Section 3) will require a site specific evaluation.

When the soil properties are not known in sufficient detail, Site Class D may be used. Site Class E and F shall not be assumed unless clearly supported by geotechnical data and when approved by the Department.

The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated by a geotechnical engineer or geologist/seismologist on the basis of similar competent rock with moderate fracturing and weathering. Unless measured on site for shear wave velocity, softer and more highly fractured and weathered rock shall be classified as Site Class C.

The hard rock category, Site Class A, shall be supported by shear wave velocity measurements on site or shall be estimated by a geotechnical engineer or geologist/seismologist on the basis of profiles of the same rock type in the same formation with an equal degree of weathering and fracturing.

The Rock Category sites, Site Classes A and B, shall not be used if there is more than 10 feet of soil between the rock surfaces and the bottom of the spread footing.

3.6.9 Combination of Seismic Force Effects

The combination of the seismic force effects shall be in accordance with the referenced *AASHTO LRFD Bridge Design Specifications*.

AASHTO 3.10.8

3.6.10 Seismic Analysis and Design Procedure for Single Span Bridges

3.6.10.1 Seismic Analysis

No detailed seismic analysis is required for any single span bridge. The minimum design force at the connections in the horizontal restraint direction between the superstructure and substructure shall not be less than the product of the single span factor (S_f) and the vertical reaction due to the tributary permanent load (dead loads).

Table 3.6.10-1 Single Span Factor (S_f)

Site Class	S_f	
	Critical Bridge	Non-Critical Bridge
A	0.08	0.03
B	0.11	0.04
C	0.13	0.05
D	0.17	0.06
E	0.26	0.09

3.6.10.2 Minimum Beam Seat Requirements

The minimum beam seat width criteria shall be in accordance with Article 4.5.3 of this Manual.

3.6.11 Seismic Analysis and Design Procedure for Non-Critical Bridges

All non-critical bridges other than single-span bridges shall conform to the following criteria:

3.6.11.1 Non-Critical Bridges Classified Site Class A, B, C or D

3.6.11.1.1 General: Except as otherwise specified herein, no detailed seismic analysis or design considerations (including substructure foundations, abutments, or liquefaction) are required for non-critical bridges classified as Site Class A, B, C, or D.

3.6.11.1.2 Horizontal Connection: The connections in the horizontal restraint direction between the superstructure and substructure shall be designed for a design force not less than 0.2 times the vertical reaction due to the tributary permanent load.

3.6.11.1.3 Minimum Beam Seat Requirements: The minimum seat width criteria shall be in accordance with Article 4.5.3 of this Manual.

3.6.11.1.4 Column Transverse Reinforcing Design Requirements: The transverse reinforcement requirements at the top and bottom of a column shall meet the requirements specified in the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 5.10.11.4.1(d) & (e)*

3.6.11.1.5 Concrete Pile Requirements: Concrete pile seismic requirements shall satisfy those specified in the referenced *AASHTO LRFD Bridge Design Specifications.* *AASHTO 5.13.4.6.2*

3.6.11.2 Non-Critical Bridges Classified Site Class E

Non-critical bridges classified as Site Class E shall follow the design and analysis procedure as outlined in Article *3.6.12.1,* except that the Uniform Load Method, as discussed in Article 4.5.2.1 of this Manual, may be used for the analysis of “regular” bridges.

3.6.11.3 Non-Critical Bridges Classified Site Class F

Non-critical bridges classified as Site Class F will require a site specific study. The design shall follow the design and analysis procedure as outlined in Article *3.6.12.1,* except that the Uniform Load Method, as discussed in Article 4.5.2 of this Manual, may be used for the analysis of “regular” bridges.

3.6.12 Seismic Design and Analysis Procedure – Critical Bridges

3.6.12.1 Seismic Design and Analysis Procedure

3.6.12.1.1 Analysis and Force Effects: For bridges classified as critical, other than single span bridges, the Multimode Spectral Method of analysis shall be used in accordance with Article 4.5 of this Manual. The analysis and force calculations shall be performed for both design earthquakes (the ULE and the LLE).

Except for foundation design forces, the design forces for each component shall be taken as the lesser of the following*:*

- The elastic forces modified by the response modification factors (R-Factors) of Article 3.6.13 of this Manual.
- The inelastic hinging forces as specified in the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 3.10.9.4.3*

The design forces for foundations (footings, pile caps, and piles) shall be taken as the elastic seismic forces modified by half of the response modification factors of Article 3.6.13 of this Manual (R/2 shall not be taken as less than 1.0); but need not be larger than the forces at the bottom of the columns corresponding to the column plastic hinging forces as determined in the referenced Article of the *AASHTO LRFD Bridge Design Specifications.* A discussion is provided in commentary format included in Appendix B (Section 3) of this Manual. *AASHTO 3.10.9.4.3*

3.6.12.1.2 Design: For bridges classified as critical, the Zone 3 design criteria of the *AASHTO LRFD Bridge Design Specifications* shall apply for the design of the various components (including foundations).

3.6.12.2 Alternate Design and Analysis Procedure

In accordance with Article 4.5.2.2.2 of this Manual, an inelastic static (pushover) analysis method may be used when a more reasonable prediction and hence a better understanding of the expected deformation demands on the columns and foundations is warranted. The designer must provide justification to the Managing Bridge Engineer when the use of the inelastic static analysis method procedure is warranted. The “pushover” response modification factors (R-Factors) of Article 3.6.13 of this Manual shall be used to modify the elastic response values when the pushover analysis method is used.

The more rigorous nonlinear time history analysis method in accordance with Article 4.5.2.2.3 of this Manual may also be considered when warranted. The designer must provide justification to the Managing Bridge Engineer as to whether the use of time history method procedure is warranted.

3.6.13 Response Modification Factors

Structures designed using multimode spectral method, or (when approved) the pushover, shall use the response modification factors defined in Table 3.6.13-1, 3.6.13-2, and 3.6.13-3 (Appendix A, Section 3).

To apply the response modification factors specified herein, the structural details shall satisfy the additional provisions of the referenced Articles of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 5.10.2.2, 5.10.11, & 5.13.4.6*

The response modification factor R shall be taken as 1.0 for all components when the inelastic time history method is used.

3.6.14 Seismic Geotechnical Considerations

Article 10.2.5.5 of this Manual addresses seismic geotechnical considerations including seismic induced lateral forces and wall inertia forces on abutments and retaining walls resulting from earthquake induced motions.

3.6.15 Seismic Isolation Design

Seismic design incorporating seismic isolation devises may be considered for bridges classified as critical, provided that the following requirements are followed:

- The design of the seismic isolation devices is in accordance with *AASHTO Guide Specifications for Seismic Isolation Design,* as modified in the applicable Articles (3.6.1 through 3.6.14) of this Manual. The response modification factor for all elements shall be taken as 1.0.
- The specific isolation systems and manufacturers being considered meet the approval of the Managing Bridge Engineer.
- At the preliminary and final design stage, the design of the isolators must be closely coordinated with all isolation device manufacturers approved by the Managing Bridge Engineer.
- Non-seismic loads and movements are determined and adequately accounted for in the design of the isolation systems(s).
- The larger displacement resulting from an increase in the period of vibration due to increased flexibility is adequately accommodated and detailed (such as providing adequate roadway joint movement capability or designing and detailing portion of the backwall to break away upon superstructure impact).
- Design and cost information shall be obtained for a cost evaluation of the various seismic design alternatives.

3.7 RAIL TRANSIT EARTH SURCHARGE

The live load surcharge used in the design of abutments, walls, and piers located in close proximity to railroad tracks shall conform to the requirements of the railroad agency having jurisdiction. The minimum live load surcharge shall be for the effects of Cooper E-80 loading.

3.8 FORCES DUE TO TEMPERATURE

For typical steel or concrete beam/girder type bridges (designed compositely with concrete deck slabs), the use of Procedure B as referenced in the *AASHTO LRFD Bridge Design Specifications* is preferred. *AASHTO 3.12.2*

The following minimum and maximum temperature shall apply for Procedure B:

Steel Girder Bridges $T_{Min\ Design}$ = -10 degrees
$T_{Max\ Design}$ = 105 degrees

Concrete Girder Bridges $T_{Min\ Design}$ = 0 degrees
$T_{Max\ Design}$ = 100 degrees

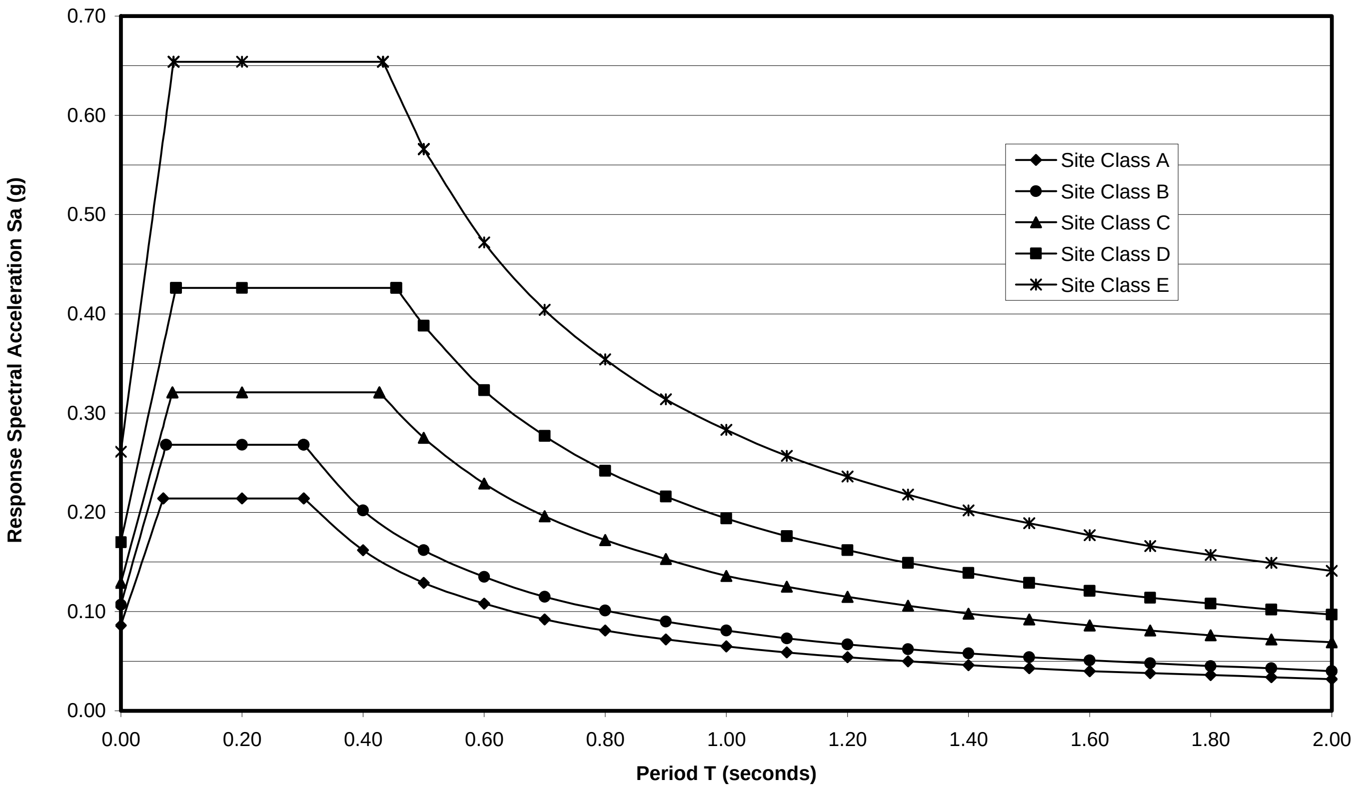
Procedure A shall apply for all other bridges. When the design thermal movements and forces are calculated using Procedure A of the referenced *AASHTO LRFD Bridge Design Specifications,* the Cold Climate temperature range shall be used. *AASHTO 3.12.2*

3.9 VESSEL COLLISION

The minimum requirements for loads and forces and their application shall be in accordance with the *AASHTO LRFD Bridge Design Specifications* except that the guidelines for establishing the geometry of the navigable channel and the bridge span layout shall be in accordance with Article 4.2 and 8.5.1, respectively, of the *AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges*. *AASHTO 3.14*

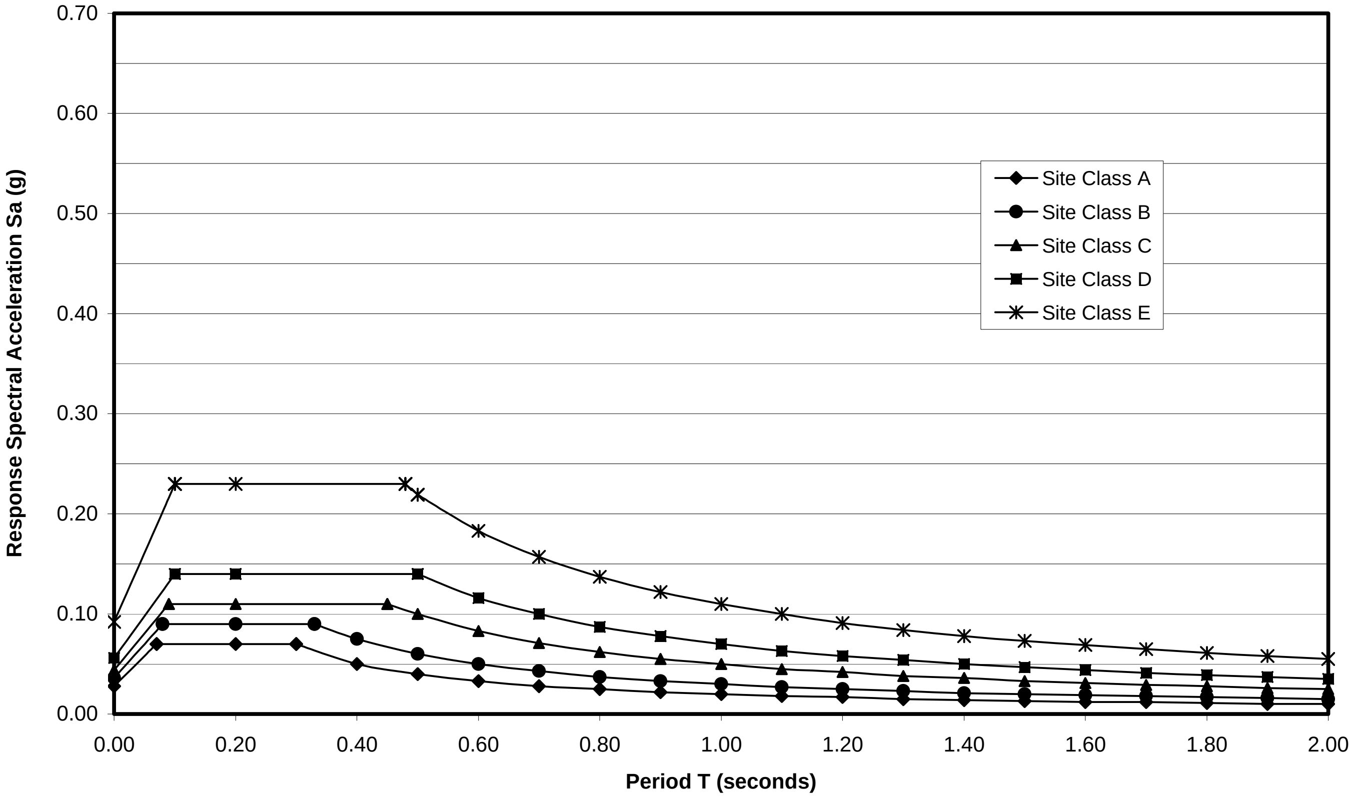
SECTION 3
APPENDIX A

Figure 3.6.7.1-1 Upper Level Earthquake (ULE - 3 % in 75 Years) Design Spectra for Site Class A, B, C, D & E, 5% Damping



TABULAR SPECTRAL VALUES FOR SITE CLASS A, B, C, D & E UPPER-LEVEL EARTHQUAKE (ULE)									
Class A		Class B		Class C		Class D		Class E	
Period	Acc	Period	Acc	Period	Acc	Period	Acc	Period	Acc
0.00	0.086	0.00	0.107	0.00	0.129	0.00	0.170	0.00	0.261
0.06	0.214	0.06	0.268	0.09	0.321	0.09	0.426	0.09	0.654
0.20	0.214	0.20	0.268	0.20	0.321	0.20	0.426	0.20	0.654
0.30	0.214	0.30	0.268	0.43	0.321	0.46	0.426	0.43	0.654
0.40	0.162	0.40	0.202	0.43	0.321	0.50	0.388	0.50	0.566
0.50	0.129	0.50	0.162	0.50	0.275	0.50	0.388	0.50	0.566
0.60	0.108	0.60	0.135	0.60	0.229	0.60	0.323	0.60	0.472
0.70	0.092	0.70	0.115	0.70	0.196	0.70	0.277	0.70	0.404
0.80	0.081	0.80	0.101	0.80	0.172	0.80	0.242	0.80	0.354
0.90	0.072	0.90	0.090	0.90	0.153	0.90	0.216	0.90	0.314
1.00	0.065	1.00	0.081	1.00	0.137	1.00	0.194	1.00	0.283
1.10	0.059	1.10	0.073	1.10	0.125	1.10	0.176	1.10	0.257
1.20	0.054	1.20	0.067	1.20	0.115	1.20	0.162	1.20	0.236
1.30	0.050	1.30	0.062	1.30	0.106	1.30	0.149	1.30	0.218
1.40	0.046	1.40	0.058	1.40	0.098	1.40	0.139	1.40	0.202
1.50	0.043	1.50	0.054	1.50	0.092	1.50	0.129	1.50	0.189
1.60	0.040	1.60	0.051	1.60	0.086	1.60	0.121	1.60	0.177
1.70	0.038	1.70	0.048	1.70	0.081	1.70	0.114	1.70	0.166
1.80	0.036	1.80	0.045	1.80	0.076	1.80	0.108	1.80	0.157
1.90	0.034	1.90	0.043	1.90	0.072	1.90	0.102	1.90	0.149
2.00	0.032	2.00	0.040	2.00	0.069	2.00	0.097	2.00	0.141

FIGURE 3.6.7.1-2 Lower Level Earthquake (LLE - 15% in 75 Years) Design Spectra for Site Class A, B, C, D & E, 5% Damping



TABULAR SPECTRAL VALUES FOR SITE CLASS A, B, C, D & E LOWER-LEVEL EARTHQUAKE (LLE)									
Class A		Class B		Class C		Class D		Class E	
Period	Sa	Period	Sa	Period	Sa	Period	Sa	Period	Sa
0.00	0.028	0.00	0.036	0.00	0.044	0.00	0.056	0.00	0.092
0.06	0.070	0.07	0.090	0.09	0.110	0.10	0.140	0.10	0.230
0.29	0.070	0.20	0.090	0.20	0.110	0.20	0.140	0.20	0.230
0.30	0.067	0.33	0.090						
0.40	0.050	0.40	0.075	0.45	0.110	0.50	0.140	0.48	0.230
0.50	0.040	0.50	0.060	0.50	0.100	0.50	0.140	0.50	0.220
0.60	0.033	0.60	0.050	0.60	0.083	0.60	0.116	0.60	0.183
0.70	0.029	0.70	0.043	0.70	0.071	0.70	0.100	0.70	0.157
0.80	0.025	0.80	0.037	0.80	0.062	0.80	0.087	0.80	0.137
0.90	0.022	0.90	0.033	0.90	0.055	0.90	0.078	0.90	0.122
1.00	0.020	1.00	0.030	1.00	0.050	1.00	0.070	1.00	0.110
1.10	0.018	1.10	0.027	1.10	0.045	1.10	0.064	1.10	0.100
1.20	0.017	1.20	0.025	1.20	0.042	1.20	0.058	1.20	0.091
1.30	0.015	1.30	0.023	1.30	0.038	1.30	0.054	1.30	0.084
1.40	0.014	1.40	0.021	1.40	0.036	1.40	0.050	1.40	0.078
1.50	0.013	1.50	0.020	1.50	0.033	1.50	0.047	1.50	0.073
1.60	0.012	1.60	0.019	1.60	0.031	1.60	0.044	1.60	0.069
1.70	0.012	1.70	0.018	1.70	0.029	1.70	0.041	1.70	0.065
1.80	0.011	1.80	0.017	1.80	0.028	1.80	0.039	1.80	0.061
1.90	0.010	1.90	0.016	1.90	0.026	1.90	0.037	1.90	0.058
2.00	0.010	2.00	0.015	2.00	0.025	2.00	0.035	2.00	0.055

TABLE 3.6.8-1 Site Class Classification

Site Class	Soil Shear Wave Velocity ν_s	Standard Penetration Resistance N or N_{ch}	Undrained Shear Strengths S_u
A	>5000 ft/sec (>1500 m/sec)	Not Applicable	Not Applicable
B	>2500 to 5000 ft/sec (>760 to 1500 m/sec)	Not Applicable	Not Applicable
C	>1200 to 2500 ft/sec (>360 to 760 m/sec)	>50 blows/ft	>2,000 psf (>100 kPa)
D	>600 to 1200 ft/sec (>180 to 360 m/sec)	15 to 50 blows/ft	1,000 to 2,000 psf (50 to 100 kPa)
E	< 600 ft/sec (<180 m/sec)	<15 blows per ft	<1,000 psf (<50 kPa)
	or Any profile with more than 10 feet of soft clay (PI>20, w=40 percent, and S_u<500psf)[1]		
F	Conduct Site Specific Evaluation if one of the following applies: 1. Peat and/or highly organic clays (H>10 feet of peat and/or highly organic clay where H=thickness of soil). 2. High Plasticity clays, H>25 feet (PI>75) 3. Very thick soft/medium stiff clays, H>120 feet		

(1) Plasticity index (PI) and moisture content (w) shall be determined in accordance with the latest ASTM D4318 and ASTM D2216, respectively.

TABLE 3.6.13-1 Base Response Modification Factors (R) for Substructures Upper Level Earthquake (ULE)

Substructure Element	Bridge Classification				
	Non-Critical (Site Class E only)	Critical			
		Multimode		Pushover	
		T≤0.5	T≥1	T≤0.5	T≥1
Wall Piers – Larger Dimensions	2	1	1.2	1.5	1.5
Columns – Single and Multiple	4	2	3	2.5	3.5
Pile Bents with Vertical Piles and Drilled Shafts					
Above Ground	4	2	3	2.5	3.5
Below Ground	1	1	1	1	1
Pile Bents with Batter Piles					
Above Ground	1	1	1.5	1.5	2
Below Ground	1	1	1	1	1
Seismically Isolated Bridges	1	1	1	1	1
Steel Braced Frame - Ductile Components	3	1.5	2	1.5	2
Steel Brace Frame - Nominally Ductile Components	1.5	1	1	1	1

Notes

(1) "T" is the period of vibration of the bridge and may be applied separately in the longitudinal and transverse directions provided the bridge has no significant skew or curvature. Otherwise the period with the most significant mass participation may be used to determine the response modification factor in both directions.

(2) Between the values of 0.5 ≤T≤1 the response modification factors may be linearly interpolated.

(3) The design forces for foundations (including footings, pile caps, and piles) shall be determined in accordance with Article 3.6.12.1.1 of this Manual.

TABLE 3.6.13-2 Base Response Modification Factors (R) for Substructures Lower Level Earthquake (LLE)

Substructure Element	Bridge Classification	
	Non-Critical (Site Class E only)	Critical (Multimode)
All Elements	1.5	1

TABLE 3.6.13-3 Base Response Modification Factors (R) for Connections

Connection	Bridge Classification	
	Non-Critical (Site Class E only)	Critical (Multimode)
Superstructure to Abutment	0.8	0.8
Expansion Joint within a span of the superstructure	0.8	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0	1.0
Columns or piers to foundations	1.0	1.0

SECTION 3
APPENDIX B

Commentary
Article 3.6.2 - Design Philosophy

The *AASHTO LRFD Bridge Design Specifications* philosophy is based on the objective that exposure of structures to earthquakes of given intensity will not cause the collapse of a structure and that the damage will be detectable and repairable. Additionally, the AASHTO LRFD seismic specifications also require that bridges which are considered "essential" (that is, that they provide a vital link to critical facilities) should as a minimum remain open to emergency vehicles for security/defense purposes immediately after the "design" earthquake (475-year return period). Bridges which are considered "critical" should remain open to all traffic after the "design" earthquake and be usable by emergency vehicles after a "large" earthquake (2500 year return period). Based on this objective the AASHTO LRFD provides design criteria as a function of the bridge importance classifications ("other", "essential" or "critical"), with a more conservative approach taken for structures classified as critical or essential. Thus the *AASHTO LRFD Bridge Design Specifications* has an implied "three-level" design performance objective.

In addition, the AASHTO LRFD design approach is a "force-based" assessment approach. In a force-based approach, the design earthquake demands are determined by reducing the ultimate elastic response spectra forces by a reduction factor (response modifications factor). The reduction factors are selected based on an acceptable risk and the components expected ductile behavior. This approach varies significantly with the more current "displacement-based" approach, which compares the elastic displacement demand to the inelastic displacement capacity while insuring a minimum inelastic capacity at plastic hinging locations. The displacement-based approach has been adapted by some state agencies[1] and is also being assessed by the AASHTO T-3 Committee (as part of the NCHRP 20-07/Task 193 Study).

Until the "displacement-based' approach is fully adopted into *AASHTO LRFD Bridge Design Specifications*, it will be the Department's policy to retain the force-based assessment approach.

Consistent with the above objectives and philosophy, the current Rhode Island seismic design criteria are based on a force-based "two-level" design principle. Thus the objective is that during the expected earthquake (the lower-level event) a critical structure (that is, a structure that provides a vital link to critical facilities) must sustain no damage and provide full access to all traffic. A non-critical bridge may sustain minimal or no damage but must be repairable during normal traffic after the expected earthquake. During the rare earthquake (the upper-level event), a non-critical structure may sustain significant damage but must not collapse. A critical bridge may sustain minimal or no damage, but it must provide access to emergency vehicles and be repairable within a very short time period after the rare earthquake. Additionally, where damage is permitted, for all structures the damage must be detectable and repairable. This is also consistent with the latest proposed guidelines presented in *NCHRP 12-49 Comprehensive Specifications for the Seismic Design of Bridges* (NCHRP 12-49). The results of the NCHRP 12-49 Report have been reformatted into stand-alone guidelines entitled MCEER/ATC *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges* published by the ATC/MCEER Joint Venture and are available through the Multidisciplinary Center for Earthquake Engineering Research Center website (MCEER).

Probabilistic ground motions, developed for the upper-level earthquake ground motion (referred to as the Maximum Credible Earthquake in NCHRP 12-49 report) by the U.S. Geological Survey (USGS), were calculated for a probability of exceedance of 2% in 50 years. These are nearly identical to ground motion values of 3% in 75 years, corresponding to a 2475 year ground motion return period. Likewise ground motions developed for the lower-level earthquake were calculated for a probability of exceedance of 10% in 50 years. These are nearly identical to ground motion values of 15% in 75 years, corresponding to a 475 year ground motion return period.

[1] Refer to the California Department of Transportation or the South Carolina Department of Transportation latest seismic design criteria.

The table below summarizes the performance objectives as a function of the bridge classification and the design earthquakes.

			Performance Objectives	
Bridge Classification	**Design Earthquake**	**Probability of Exceedance**	**Damage**	**Description**
Critical	Upper-Level Earthquake (ULE)	3% in 75 years	Minimal or none	Allow limited access to emergency vehicles; be repairable within a very short period of time
	Lower-Level Earthquake (LLE)	15% in 75 years	None	Full access to all traffic
Non-Critical	Upper-Level Earthquake (ULE)	3% in 75 years	Significant	No Collapse
	Lower-Level Earthquake (LLE)	15% in 75 years	Minimal or none	Repairable during normal Traffic

In addition to the two-level design approach, the seismic provisions specified in Article 3.6 of this Manual incorporate the new soil site classes proposed in the *NCHRP 12-49* Report*.* The site classes are classified according to the average shear wave velocity, Standard Penetration Test (SPT) blow count (N-value), or undrained shear strength in the upper 100 feet of the site.

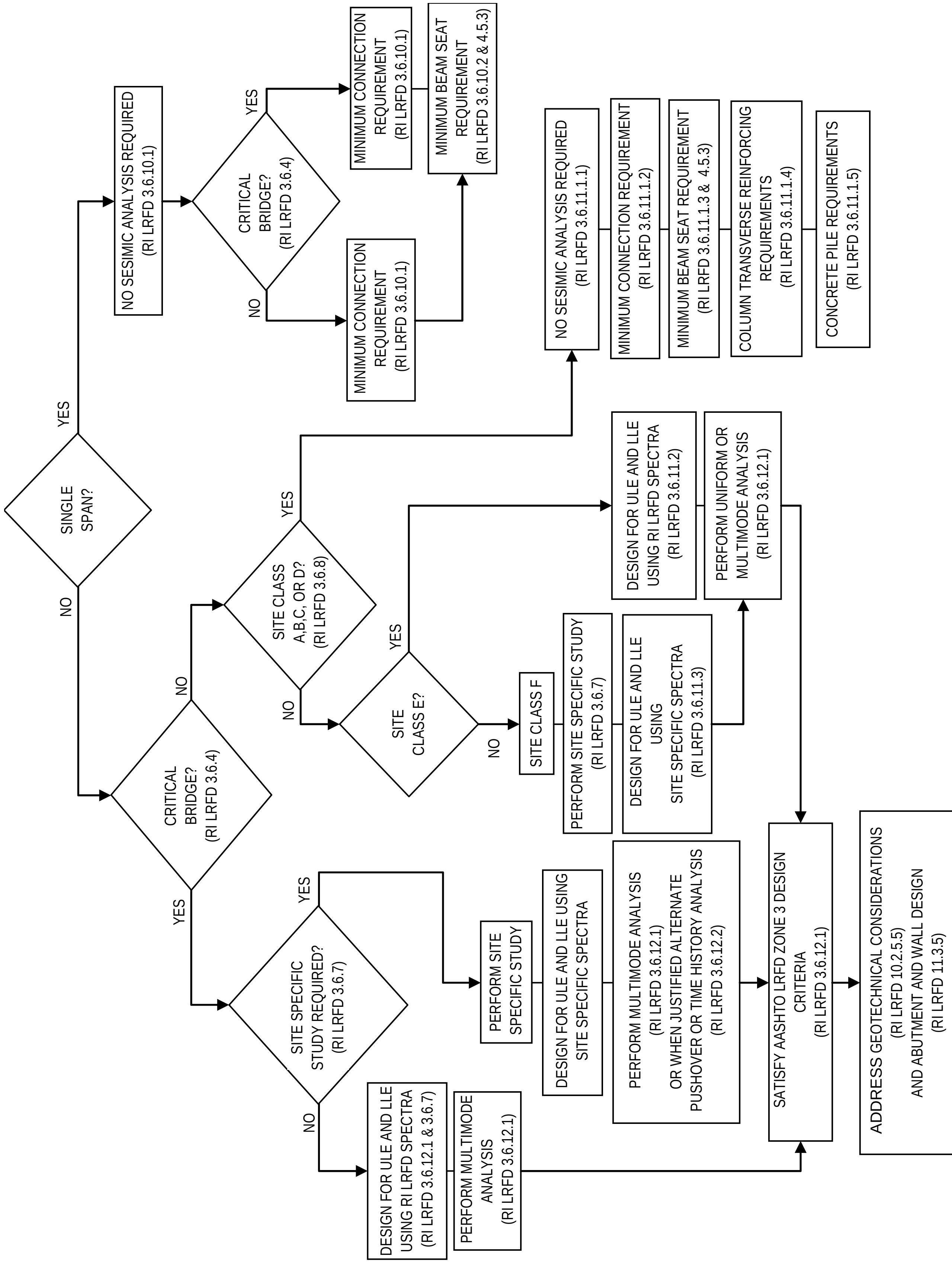
These provisions also incorporate the use of spectral acceleration. This differs from the *AASHTO LRFD Bridge Design Specifications* in that the peak ground acceleration (PGA) or the acceleration coefficient is no longer used. The design spectrum for both earthquakes have been constructed from the latest national ground motion maps based on probabilistic national ground motion mapping conducted by the USGS. The construction of the response spectra follows the “two-point” method procedure outlined in the NCHRP 12-49 Report with a response spectral acceleration at the short period (0.2 sec) and at the 1 second period. These new spectral shapes have removed the arbitrary conservatism that currently exists in the *AASHTO LRFD Bridge Design Specifications* with respect to the long period portion of the response spectrum. The long period portion of these spectral shapes decays at a faster rate of $1/T$ as compared to the $1/T^{2/3}$ of the *AASHTO LRFD Bridge Design Specifications*.

Commentary
Article 3.6.12.1 - Seismic Design and Analysis Procedure

Seismic design philosophy permits damage (inelastic deformation or plastic hinging) in substructure components provided the damage is located such that it can be readily inspected and repaired. Thus depending on the bridge classification and the site seismicity, current *AASHTO LRFD Bridge Design Specifications* practice is to design the foundation components for loads which are larger than the modified seismic design forces developed in the substructure component to which the foundation is attached. For instance, foundations for a structure determined to be Seismic Performance Zone 2 are designed for force equivalent to twice the modified seismic forces of the substructure component to which the foundation is attached and a structure assigned Seismic Performance Zone 3 or 4 is designed for the lesser of the seismic elastic forces or the plastic hinging forces. This procedure is also consistent with the latest proposed guidelines presented in *NCHRP 12-49 Comprehensive Specifications for the Seismic Design of Bridges* (NCHRP 12-49), except that only the capacity design procedure are used to design foundations. In the capacity design procedure the foundation components (unless designed using the elastic seismic loads) are designed for the maximum force effects developed from plastic hinging of the component attached to the foundation.

Though the capacity design procedure is a rational approach, the Department has determined that for critical structures this leads to an overly conservative sized foundation when compared to foundations designed in accordance with the current AASHTO criteria. This is due to the fact that the Rhode Island upper level earthquake is an earthquake corresponding to a 2500 year return period as compared to the current AASHTO earthquake which corresponds to a 475 year return period earthquake.

Thus foundation design forces for critical structure are limited to forces equivalent to the seismic elastic forces modified by half of the response modification factor of the component to which the foundation is attached (R/2 is not to be taken less than 1).



RHODE ISLAND SEISMIC DESIGN PROCESS FLOW CHART

SECTION 4
STRUCTURAL ANALYSIS AND EVALUATION

4.1 GENERAL SCOPE

In general bridge structures shall be analyzed elastically. The inelastic analysis method may be considered for the Extreme Event Limit States but only with the approval of the Managing Bridge Engineer. When justified, a nonlinear elastic analysis method may be used for extremely flexible bridges or for Extreme Event Limit States.

A more detailed discussion of the overall scope of structural analysis and evaluation is presented in the commentary of the referenced Article of the *AASHTO LRFD Bridge Design Specifications.*

AASHTO 4.1

4.2 ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS

4.2.1 General

The use of structural analysis methods other than the classical force displacement method (such as finite element methods; the grillage analogy method; or other methods specified in the *AASHTO LRFD Bridge Design Specifications)* shall meet the approval of the Managing Bridge Engineer and must be identified during the scope and fee proposal preparation phase of a project.

AASHTO 4.4

4.2.2 Use of Computer Programs

4.2.2.1 Commercially Available Software

The use of commercially available software as a tool to assist in the preparation of engineering analysis and design calculations is permitted provided that:

- All computer work is conducted under the direct supervision of an experienced engineer familiar with the specific computer application as well as the specifics of the design and analysis parameters.
- All work performed is checked completely.
- Either the primary design or the design check is performed by an experienced engineer.
- The Designer assumes responsibility for the integrity of the software application.
- The Designer assumes responsibility for ensuring that all files regardless of format are pre-screened to ensure they are free from viruses and all similar harmful effects. Damage or loss of data caused by such infected files is the sole responsibility of the Designer.

4.2.2.2 Consultant Developed Software Applications

Engineering applications utilizing spreadsheet type or equation-solving software developed internally to assist in the analysis and design are permitted provided that:

- All computer work is conducted under the direct supervision of an experienced engineer familiar with the specific computer application as well as the specifics of the design and analysis parameters.

- The program application is presented very clearly, is self-explanatory, and is easy to follow.
- The application is verified independently.
- Either the primary design or the design check is performed by an experienced engineer.
- The Designer assumes full responsibility for the integrity of the design.

4.3 MATHEMATICAL MODELING

A detailed discussion of mathematically modeling has been presented in the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. The following are additional considerations and requirements:

AASHTO 4.5

- Except as permitted for bridge rating calculations, continuous barriers or parapets shall not be considered in the structural modeling and analysis of a bridge.
- The need for the sophisticated mathematical modeling of the soil and foundations is a function of the sensitivity of the structure to foundation flexibility and movements, and it should be evaluated on a case by case basis. The response characteristics of the soil/foundation should be considered when the structural behavior is particularly sensitive to the soil or foundation boundary conditions.
- The soil and foundation response must be considered in the model for integral abutment bridges.

4.4 STATIC ANALYSIS

4.4.1 Horizontally Curved Girders

The force effect (moments, shears, reactions and loads to secondary members) of horizontally curved girders shall be determined utilizing a refined method of analysis as described in *AASHTO LRFD Bridge Design Specifications*. These include grillage analogy and finite element methods.

AASHTO 4.6.3

4.4.2 Bridges with Large Skew Angles

Bridges with large skew angles where differential transverse deflection takes place between adjacent beams or girders will generate forces that can not be predicted when the single beam analysis method is used. To account for these forces and deflections, a grid type or finite element analysis method should be performed when the bridge skew angle exceeds approximately 40 degrees. Main beams or girders, diaphragms, and connections should be designed to account for these effects. In addition, the bearing selection shall be in accordance with Article 14.4.2 of this Manual.

4.4.3 Live Load Distribution Factor

Live load distribution factors (for bridges where any variable falls outside the “Range of Applicability” as provided in the various Live Load Distribution Factor tables of AASHTO LRFD) must be discussed with the Department. The more refined analysis method required by the *AASHTO LRFD Bridge Design Specification* may not be necessary. Studies conducted by other state agencies have demonstrated that the Live Load Distribution factors may be valid outside of the “Range of Applicability” established by AASHTO.

AASHTO 4.6.2.2 & 4.6.2.3

4.4.4 Refined Method of Analysis

Other than for curved girders and bridges with large skew angles, the use of Refined Analysis methods are permitted only with the approval of the Managing Bridge Engineer.

4.4.5 Redistribution of Negative Moment in Continuous Beam Bridges

If warranted due to design considerations, the need for redistribution of negative moment in continuous beam bridges should be discussed with the Managing Bridge Engineer. If justified, the simplified redistribution procedure of the referenced Articles of the *AASHTO LRFD Bridge Design Specifications* may be used.

AASHTO 4.6.4 & 4.6.4.3

4.5 DYNAMIC ANALYSIS FOR EARTHQUAKE LOADS

4.5.1 Applicability

Except as referenced herein, Article 4.7.4 of the *AASHTO LRFD Bridge Design Specifications* shall be deleted in its entirety and replaced with the requirements specified in Article 4.5 of this Manual.

4.5.2 Analysis Method

4.5.2.1 Non-Critical Bridges Classified as Site Class E

For non-critical bridges (Site Class E classification) with a “regular” geometry, the Uniform Load Method outlined in the referenced *AASHTO LRFD Bridge Design Specifications* may be used. Bridges shall be considered “regular” if they have less than seven spans, no abrupt or unusual changes in weight, stiffness or geometry, and no large changes in these parameters from span to span.

AASHTO 4.7.4.3.2c

Bridges, which are not regular, shall be analyzed using the multimode analysis method described in Article 4.5.2.2.1 of this Manual.

4.5.2.2 Critical Bridges

4.5.2.2.1 Multimode Spectral Method

For critical bridges, the multimode spectral analysis method shall be used. As a minimum this consists of a linear dynamic analysis using a three-dimensional model representing the structure.

A discussion of practical guidelines for modeling bridges (which includes the number and selection of nodal locations, mass distribution, material and section properties, and foundation modeling for spread footings and pile supported foundations) is beyond the scope of this Manual. These can be found in several other widely available publications. However, the following minimum modeling guidelines shall apply:

- The number of modes to be combined in a dynamic analysis is mainly influenced by the number of degrees of freedom used to define structure geometry. As a guide, the investigation of seismic response for most highway bridges should include the effects of a minimum number of mode shapes to capture at least 90% mass participation in the longitudinal and transverse directions. A minimum of three elements per column and four elements per span shall be used in the model.

- When loads are transferred through the abutments, the abutment stiffness should be considered in the dynamic analysis of the structure. Careful attention should be given, not only to the abutment modeling but also to the detailing of the abutment components. Abutments which contribute to the overall stiffness of the structure usually attract a larger portion of the seismic forces during an earthquake and are therefore more susceptible to damage. Examples where the abutment stiffness may contribute directly to the overall behavior of the structure are: monolithically designed abutments, seat-type abutments with fixed bearings, and integral abutments. In addition, seat-type abutments with expansion bearing and joints which can not accommodate the maximum seismic event displacement will also contribute to the dynamic behavior of the structure, once the longitudinal and transverse movement capacities of the bearings or expansion joints are exceeded.

  For deep foundations the use of either an “equivalent base spring” model or an “equivalent cantilever” model is acceptable.

- Due to the uncertain nature of soil behavior when subjected to applied loads, the determination of the overall foundation flexibility (abutment/pier and soil interaction) is inherently subjective. Therefore the analysis for solutions to this type of a problem may only be quantified within a reasonable degree or accuracy. It is, therefore, sound engineering practice to determine an upper and lower bound for the structural response, which takes into consideration the possible variations in the foundation flexibility. This is particularly significant when determining elastic structure displacements.

- The member forces and displacements may be estimated by combining the respective response quantities (moment, force, displacement) from the individual modes by the Complete Quadratic Combination (CQC) method.

4.5.2.2.2 Inelastic Static Analysis (Pushover) Method

This method requires an elastic (with cracked section properties) response spectrum analysis for the governing design spectra (either the ULE or the LLE) and a P-delta design check. The analysis shall be performed for preliminary flexural design of plastic hinging in columns and to determine the displacement of the structure. The displacement capacities shall be verified using a two-dimensional inelastic static (pushover) analysis in the principal structure direction. The response modification factors (R-Factors) of Article 3.6.13 of this Manual shall be used to modify the elastic response values.

This analysis method provides a more reasonable prediction and hence a better understanding of the expected deformation demands on the columns and foundations. However, it also requires a more sophisticated design. Therefore the use of this method should be considered only when it is determined that a more accurate assessment of the expected deformations in the critical elements is needed and with the approval of the Managing Bridge Engineer. The designer shall provide justification to the Managing Bridge Engineer when the use of the pushover method is warranted.

4.5.2.2.3 Nonlinear Time History Method

For certain critical bridges with a complex geometric configuration the use of the more rigorous time history analysis method in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications* may be considered. The use of this method should be considered only when warranted and approved by the Managing Bridge Engineer. The designer must provide justification to the Managing Bridge Engineer when the use of the time history method procedure is warranted.

AASHTO 4.7.4.3.4

4.5.3 Minimum Beam Seat Requirements

The bridge seats at expansion bearings without restrainers, shock transmission units (STU) or dampers shall accommodate the greater of the maximum calculated seismic displacement or the minimum support length as measured by the following equation:

All bridges:

$$N = (8+0.02L+0.08H)(1+0.000125S^2)$$

The above variables are as identified in the referenced Article of the *AASHTO LRFD Bridge Design Specifications.*

AASHTO 4.7.4.4

Alternatively an adequately designed restrainer may be provided. Longitudinal or transverse restrainers must be designed for the anticipated loads in compliance with the criteria specified in Article 3.6 of this Manual.

SECTION 5
CONCRETE STRUCTURES

5.1 GENERAL SCOPE

The minimum requirements for the design of concrete structures shall be in accordance with the *AASHTO LRFD Bridge Design Specifications* unless otherwise modified or further clarified in this Section.

5.2 MATERIALS PROPERTIES

5.2.1 Concrete

The classes of concrete corresponding to particular structural components shall be as shown in the latest revision of Table 1 included in Section 601 of the Rhode Island Standard Specifications for Road and Bridge Construction.

Lightweight concrete for structural components shall only be used with the approval of the Managing Bridge Engineer.

Unless otherwise specified herein, the minimum compressive strength for each class of concrete shall be as listed in the latest revision of Table 2 included in Section 601 of the Rhode Island Standard Specifications for Road and Bridge Construction. Concrete strengths in excess of those indicated may be proposed subject to the approval of the Managing Bridge Engineer. The use of specified concrete strength in excess of 10,000 psi is not permitted.

The concrete used for precast prestressed pre-tensioned (or post-tensioned) construction shall have a specified minimum concrete compressive strength of 5,000 psi. Higher strengths of concrete (up to 7,000 psi) is encouraged and may be specified. Concrete strength in excess of 7,000 psi may be specified subject to the approval of the Managing Bridge Engineer.

5.2.2 Reinforcing Steel

The reinforcing steel used for both cast-in-place and precast construction shall be AASHTO designation M 31 (ASTM A 615), Grade 60.

5.2.3 Prestressing Steel

Strands shall meet the requirements of AASHTO designation M 203 or M 275, Grade 270 (ASTM A 416 & ASTM A722 respectively). Prestressing strand shall be low relaxation strands. Stress-relieved strands shall not be specified.

Except for prestressed concrete stay-in-place forms, $\frac{1}{2}$ inch or 0.6 inch nominal diameter strands shall be specified for prestressed concrete components. The strand diameter for prestressed concrete stay-in-place forms shall be $\frac{3}{8}$ inch.

The use of epoxy coated strands is not permitted.

5.3 LIMIT STATES

The resistance factors shall for the various limit states shall be in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 5.5*

For the strength limit state, the resistance factor for concrete piles shall be the resistance factor used for the "compression-controlled" section in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specification*s. *AASHTO 5.5.4*

The resistance factor for the Extreme Limit states shall be taken as 1.0.

5.4 DESIGN CONSIDERATIONS

At the strength and extreme event limit states, the use of the strut-and-tie model should be considered to determine the internal forces near the supports; the points of application of concentrated loads; and for the design of deep footings and pile caps, all in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications.* *AASHTO 5.6.3*

5.5 DESIGN FOR FLEXURAL AND AXIAL EFFECTS

5.5.1 Control of Cracking by Distribution of Reinforcement

The provisions for distribution of steel reinforcement to control cracking shall be in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications.* Except for footings, Class 2 exposure condition shall be assumed for all reinforced members. Class 1 exposure condition may be assumed for footings below grade. *AASHTO 5.7.3.4*

For concrete components exposed to aggressive exposure conditions or to a corrosive environment, lower exposure factors γ_e may be considered in conjunction with additional protection measures such as the use of high performance concrete.

5.5.2 Skin Reinforcement

The minimum skin reinforcement shall be determined in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications,* but in no case shall it be less than #4 at 12 inches. *AASHTO 5.7.3.4*

5.5.3 Deflection of Prestressed Concrete

The magnitude of long term cambers (deflections) and the camber at time of erection for precast prestressed members is somewhat complex as it is influenced by the strength of the concrete after the release of prestress, as well as other factors such as the effects of prestress and the loss of prestress over time. It is therefore accepted practice to estimate the camber of a member after a period of time by multiplying the camber due to prestress at the time of release (initial camber) by a "multiplier". The PCI Design Handbook provides suggested multipliers which may be used as a guide in estimating long term cambers. These camber multipliers for non-composite construction are as follows:

- The camber at time of erection may be taken to be 1.80 times the camber at transfer.
- Long term camber may be taken to be 2.45 times the camber at transfer.

5.6 PRESTRESSING AND PARTIAL PRESTRESSING

5.6.1 Specified Concrete Strengths

The concrete strength at transfer shall be calculated and indicated on the plans. Concrete strength at transfer shall be based on stress calculations but limited to a maximum compressive strength of approximately 4,000 to 4,200 psi (for 6,000 psi compressive strength concrete). Higher transfer strength for concrete strengths in excess of 6,000 psi may be achievable and may be specified provided that the producer's daily casting cycle is not interrupted. The designer shall coordinate with local producers to establish a practical concrete strength at transfer.

Higher release strengths may be permitted when it is absolutely necessary subject to the approval of the Managing Bridge Engineer. Justification must be provided to validate the added cost.

5.6.2 Stress Limits for Concrete

Except as specified in Article 5.10.1 of this Manual, the limits for compression and tensile stress shall be in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications,* except as modified as follows:

AASHTO 5.9.4

Temporary Tensile Stress Limits in Prestressed Concrete at Service Limits State before Losses, Fully Prestressed Components (other than Segmentally Constructed Bridges)

In areas other than the precompressed tensile zone and without bonded reinforcement	No Tension
In areas with bonded reinforcement (reinforcing or prestressing steel) sufficient to resist the tensile force in the concrete computed, assuming an uncracked section where the reinforcement is proportioned using a stress of 0.5 f_y (*not* to exceed 30 ksi).	$0.12 \sqrt{f'_{ci}}$ (ksi)
Handling stresses in prestressed piles	$0.079 \sqrt{f'_{ci}}$ (ksi)

Tensile Stress Limits in Prestressed Concrete at Service Limits State After Losses, Fully Prestressed Components (other than Segmentally Constructed Bridges)

For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosive conditions*****	$0.0948 \sqrt{f'_c}$ (ksi)
For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions*****	No Tension

***** Corrosive condition is defined as bridges subject to de-icing salt spray or salt-laden coastal spray.

Higher tensile stresses may be allowed when it is absolutely necessary to utilize a shallower section for vertical clearance purposes, when additional savings may be realized by eliminating a girder line, or when performing load rating calculations. When higher tensile stresses are permitted (subject to the approval of the Managing Bridge Engineer) the tensile stresses must be limited to the values indicated in the *AASHTO LRFD Bridge Design Specifications*.

5.6.3 Debonding of Strands

The use of debonded strands in the design of pretensioned concrete members is permitted in accordance with the provisions of the referenced Article of the *AASHTO LRFD Bridge Design Specifications.* The only method permitted for debonding shall be by the use of plastic sheathing taped to the strands. Other methods such as greasing, chemical retarders and taping will not be allowed.

AASHTO 5.11.4.3

5.7 DETAILS OF REINFORCEMENT

5.7.1 Minimum Spacing of Prestressing Tendons

The minimum distance between pretensioning strands for both ½ inch and 0.6 inch diameter strands shall be 2 inches.

5.7.2 Shrinkage and Temperature Reinforcement

The minimum shrinkage and temperature reinforcement shall be determined in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications,* but in no case shall be less than #4 at 12 inches.

AASHTO 5.10.8

5.8 DEVELOPMENT AND SPLICE LENGTH

Resistance factors shall not be applied to the development and splice lengths of reinforcement.

5.9 DURABILITY

5.9.1 Concrete Cover

Unless otherwise specified in Article 9.6.2 of this Manual, the clear cover for prestressing and reinforcing steel shall be in accordance with the referenced Article and Table of the *AASHTO LRFD Bridge Design Specifications.*

AASHTO 5.12.3 & Table 5.12.3-1

5.9.2 Protective Measures

5*.9.2.1 General*

The policy below is intended to be used as a guide, and it will apply to most bridges. Special cases will necessitate that the designer examine the specific conditions and provide protection to the substructure and superstructure elements accordingly.

5*.9.2.2 Use of Epoxy Coated Reinforcement*

Except for footings cast below grade, all reinforcing steel shall be epoxy-coated reinforcement. The use of epoxy-coated reinforcement for footings cast below grade shall be optional.

5*.9.2.3 Deck Waterproofing Membrane*

All new bridge decks, including prestressed box beams and slabs, shall be provided with an approved waterproofing membrane system prior to placement of the bituminous wearing surface.

5.9.2.4 Concrete Surface Treatment

The entire surface of beam seats, faces of backwalls (except for prestressed butted boxed and slabs where the backwall is cast against the boxes or slabs), and pier caps and columns shall be provided with a Concrete Surface Treatment – Protective Coating in accordance with the latest revision of the Rhode Island Standard Specifications for Road and Bridge Construction. The color of the protective coating shall be coordinated with the Department and be specified on the contract drawings.

5.10 PROVISIONS FOR STRUCTURE TYPES

5.10.1 Simple Span Precast Concrete Girders Made Continuous

Except as modified in this Article, the requirements for the design of simple span precast concrete girders made continuous shall be in accordance with the provisions of the referenced Article of the *AASHTO LRFD Bridge Design Specifications.* *AASHTO 5.14.1.4*

Multi-span bridges (consisting of simple-span precast concrete girders with concrete decks and with continuity diaphragms cast between the ends of the girders at interior supports), must be designed as continuous for loads placed on the bridge after the continuity diaphragms and concrete deck is cured (superimposed dead loads and live loads).

In order to satisfy a fully effective connection between the precast girders and the continuity diaphragm, the requirements specified herein shall apply.

- The minimum ages of the girder prior to the placement of the continuity diaphragms and deck shall be 90 days. This requirement shall clearly be specified in the contract documents (plans and contract special provisions).

- Restraint moments as specified herein shall be accounted for. Restraint moments are developed as a result of time-dependent effects such as creep, shrinkage and temperature variation after continuity is established. Provided that the minimum girder age requirement (as specified above) is satisfied, the positive restraint moments caused by girder creep and shrinkage and deck slab shrinkage may be taken to be zero and no computation of restraint moments shall be required. However, a positive moment connection shall be provided with factored resistance, ϕM_n, not less than 1.2 M_{cr}, as specified in the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 5.14.1.4.9*

- Simple span precast concrete girders made continuous shall be designed to satisfy the service limit state stress limits of the referenced provisions of the *AASHTO LRFD Bridge Design Specifications.* Tensile stress at the service limits state (after losses) developed at the top of the girders of interior supports shall satisfy the tensile limits of the referenced table of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 5.14.1.4.6* *Table 5.9.4.2.2-1*

- The reinforcement in the cast-in-place composite deck slab shall be proportioned to resist the negative design moments at the strength limit state in accordance with the requirements of the referenced provisions of the *AASHTO LRFD Bridge Design Specifications.* *AASHTO 5.14.1.4.8*

SECTION 6
STEEL STRUCTURES

6.1 GENERAL SCOPE

The minimum requirements for the design of steel structures shall be in accordance with the *AASHTO LRFD Bridge Design Specifications* unless otherwise modified or further clarified in this Section.

6.2 MATERIALS PROPERTIES

6.2.1 Steel Components

All steel material and components such as structural steel, bolts, nuts, washers, shear connectors, weld metal, stainless steel, and cables shall meet the AASHTO material property requirements unless otherwise specified herein or in the Rhode Island Standard Specifications for Road and Bridge Construction.

6.2.2 Structural Steel

Structural steel shall conform to the AASHTO material property requirements of the following steel designations.

AASHTO Designation M 270 Grade 36
AASHTO Designation M 270 Grade 50
AASHTO Designation M 270 Grade 50W

The use of High Performance Steel (HPS) should only be specified upon written approval of the Chief Engineer.

The use of weathering steel (AASHTO M 270 Grade 50W) shall be approved by the Managing Bridge Engineer and shall be specified within the limitation indicated here in this Article.

Unless both a macro-environmental and a micro-environmental study is conducted by a corrosion expert, the use of unpainted weathering steel should only be specified when the bridge is located in a suitable environment and when the structural details are properly designed, all with the understanding that it will not provide a maintenance-free superstructure.

Unpainted weathering steel shall not be used in the following environment or locations:

- In marine environments where salt-laden air from the ocean can be deposited on the bridges. As a minimum, weathering steel should not be used for bridges located within five (5) miles of the coastline.

- In areas close to industrial facilities capable of producing emissions of contaminated material.

- In areas where continuous moisture or prolonged wetting of the steel is possible (such as in foggy environments with high relative humidity or when structures are adjacent to high-banked rivers or streams).

- On low-clearance structures (when the vertical clearance to the roadway below is less than 24 feet) where the structure is exposed to salt-laden traffic spray.

Bridges constructed using unpainted weathering steel shall take into consideration the following restrictions:

- The number of expansion joints shall be minimized. When an expansion joint is used, field splicing of the expansion joint sealing element shall not be permitted under any circumstance.
- All structural steel within five feet of expansion joints or within a length equal to two beam or girder depths (whichever is greater) shall be painted. The color of the paint shall closely match the color of the weathering steel.
- Drip plates as detailed on the Rhode Island Bridge Design Standard Details shall be provided.
- The number of scuppers shall be minimized.
- Only mechanical fasteners suitable for use with unpainted weathering steel shall be specified.

For additional information related to the above guidelines for the use and proper structural detailing of weathering steel, reference is made to FHWA 1989 Technical Advisory on Uncoated Weathering Steel in Structures.

6.3 LIMIT STATES

The resistance factors for the various limit states shall be in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 6.5*

6.4 FATIGUE AND FRACTURE CONSIDERATIONS

6.4.1 Fatigue

Components and details identified as fatigue resistant Detail Category D, E or E' shall not be used for new bridges. The use of such details, when unavoidable, must be approved by the Managing Bridge Engineer. *AASHTO 6.6*

Refer to Article 2.4.5 of this Manual for additional requirements regarding fatigue sensitive details.

6.4.2 Fracture

The temperature zone designation for Charpy V-Notch requirements shall be taken as Temperature Zone 2. *AASHTO 6.6.2*

The Charpy V-notch requirement shall be considered mandatory for all bridge components, except for:

- Intermediate transverse web stiffeners not serving as connection plates,
- Bearings, sole plates and masonry plates,

- Expansion dams other than Modular Bridge Joint Systems, and

- Drainage material.

Refer to Article 2.4.5 of this Manual for additional requirements regarding fracture critical members (FCM).

6.5 GENERAL DIMENSIONS AND DETAIL REQUIREMENTS

6.5.1 General

Where applicable, the detailing of structural components shall be in accordance with the Rhode Island Bridge Design Standard Details.

Several publications developed by the AASHTO/NSBA Steel Bridge Collaboration outlining the best practices for the design, fabrication, and erection of steel bridges is available to be viewed or downloaded from the AASHTO website. These publications are:

- *Guidelines for Design for Constructability* (Publication G 12.1)
- *Design Drawing Presentation Guidelines* (Publication G 1.2)
- *Steel Bridge Fabrication Guide Specification* (Publication S 2.1)
- *Steel Bridge Fabrication QC/QA Guide Specification* (Publication S 4.1)
- *Shop Detail Drawing Review/Approval Guidelines* (Publication G 1.1)
- Shop Detail Presentation Guidelines (Publication G1.3)
- *Guide Specification for Coating Systems with Inorganic Zinc-Rich Primer* (Publication S 8.1 - 2002)
- *Steel Bridge Bearing Design and Detailing Guidelines (G9.1)*

Unless otherwise stipulated in this Section and in the pertinent structural steel details included in the Rhode Island Bridge Design Standard Details, these publications should be used where applicable on all Rhode Island bridge projects.

6.5.2 Camber

Unless modified here in this Article, the general requirements for camber shall be in accordance with the provision of the referenced Article of the *AASHTO LRFD Bridge Design Specifications,* taking into consideration the impacts of composite action, staged construction, horizontally curved geometry and heavy skew geometry where any are applicable. *AASHTO 6.7.2*

Structural steel beams and girders shall be cambered for profile and for full dead load plus superimposed dead load deflections. Rolled beams with computed deflections less than ½ inch shall be detailed with natural camber up.

A camber diagram, to include the effect of vertical curves, shall be shown on the contract plans. For simple spans the mid-ordinate will usually be considered sufficient, but for continuous spans the ordinates at tenth points and at all critical points, such as points of contraflexure and splice points, must be shown.

6.5.3 Minimum Thickness and Width of Steel

The minimum tension or compression flange thickness shall be ¾ inch. The minimum tension flange width shall be 12 inches. The minimum compression flange width shall be 12 inches but not less than L/85 (where L is the length of the girder shipping piece in inches). *AASHTO C6.10.3.4*

The minimum web thickness shall be $^{7}/_{16}$ inch.

The minimum transverse or bearing stiffener thickness shall be $^{7}/_{16}$ inch.

The minimum diaphragm connection plate thickness and width shall be ½ inch and 5 inches respectively.

6.5.4 Diaphragms and Cross-Frames

In general, for bridges with skew angles of 20° or less, the diaphragms or cross-frames shall be placed parallel to the centerline of bearing and shall not be staggered. For bridges with skew angles greater than 20°, the diaphragms or cross-frames shall be placed perpendicular to the beam or girder. Refer to the Rhode Island Bridge Design Standard Details.

Diaphragms which carry utility loads (such as, water pipes, gas pipes, telephone, and electric conduits) shall be specifically designed to carry these loads.

Diaphragms and cross-frames (including their connecting parts) which act as primary load carrying members, such as in curved girder and heavily skewed bridges, should be designed for all applicable limit states for the actual calculated loads. The appropriate assumed contact surface treatment for the faying surfaces (Class A, B or C surface condition) shall be clearly noted on the contract documents. *AASHTO 6.13.2.8*

6.5.5 Economy

The publication entitled *Guidelines for Design for Constructability* (Publication G 12.1 – 2003 referenced in Article 6.5.1 of this Manual) provides guidelines which may be used to achieve an economical design. In addition to the guidelines discussed in this publication, the following are some general suggested practices to be followed:

- Built-up girders should, in general, be designed with a reasonable number of transverse stiffeners, instead of a thicker web plate with no stiffeners or a thinner web plate with numerous transverse stiffeners. The vertical intermediate stiffeners shall be placed on one side of the fascia beam only and shall not be exposed to view.

- In general, longitudinal stiffeners should not be considered, except at the haunched sections of plate girders with span lengths exceeding 300 feet or at the bottom flange of box girders when economically justified.

- Haunched girders should not be considered for most conventional bridges less than 400 feet in length and may only be used when economically justified.

- Reduction in flange area along the length of the plate girder may be accomplished by either reducing the thickness or the width of the flanges or by a combination of both; but in general, the thickness reduction is favored. Plate thickness shall not be reduced by more than 50 percent at any splice, and it preferably shall not be reduced by more than 33 percent. The cost of welding and the additional cost of non-destructive examination shall be considered in determining the locations for the variations in thickness of the flanges. As a general guide, a flange splice is justified when more than an average of 800 pounds of flange material is saved (also refer to *Guidelines for Design for Constructability*, Publication G 12.1 - 2003). To reduce the cost of the bridge bearings, consideration shall be given to narrowing the width of the bottom flanges at the piers and abutments.

- Maximizing the flange width to flange thickness ratio, within the limitations specified in the *AASHTO LRFD Bridge Design Specifications*, will provide for a more cost effective design and (for horizontally curved girders) will minimize the effects of lateral flange bending.

6.5.6 Heat-Curved Rolled Beams and Welded Plate Girders

The heat curving of rolled beams and welded plate girders (to obtain a horizontal curvature) shall be in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 6.7.7*

Additional camber shall be considered to compensate for possible loss of camber due to heat curving of girders and shall be computed in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 6.7.7.3*

6.6 I-SECTION FLEXURAL MEMBERS

6.6.1 Composite Sections

Composite design shall be used for all I-section bridges with span lengths in excess of 40 feet. Composite design need not be considered for bridges with span lengths less than 40 feet.

In continuous spans, both the positive and negative moment areas shall be designed to act compositely with the concrete deck in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 6.10.1.1*

Beam and girder haunches shall be detailed with a minimum 2 inch haunch at any point along the composite member. A 2 inch nominal haunch shall be used for composite member design computations.

6.6.2 Hybrid Sections

When economically feasible, hybrid sections (consisting of a web with minimum yield strength lower than that of one or both flanges) should be considered. *AASHTO 6.10.1.3*

6.6.3 Constructability

All the provisions of the referenced Article of the *AASHTO LRFD Bridge Design Specifications* must be considered in the design. *AASHTO 6.10.3*

Instability during construction (as a result of the overturning tendency of horizontally curved girder bridges as well as the low frictional characteristics of certain types of bearings) may present potentially hazardous situations during construction. The Consultant must therefore provide a statement on the contract drawings advising the Contractor to take all necessary precautions to insure stability, and thereby safety, during steel erection. A similar precautionary statement should also appear on the shop drawings.

6.6.4 Shear Connectors

Shear connectors shall, in general, be ¾ inch diameter studs, with a minimum of two across the width of the beam or girder. However, larger diameters may be specified if required by design. Shear connectors shall be placed parallel to the transverse deck reinforcement. Their minimum heights shall be 5 inches, and they shall extend a minimum of 2 inches above the bottom of the slab. Studs shall be spaced to allow for field welding but shall not be less than six stud diameters. *AASHTO 6.10.10*

6.6.5 Cover Plates

The use of cover plates on rolled beams are permitted, provided that the provisions of the referenced Article of the *AASHTO LRFD Bridge Design Specifications* and the requirements specified here in this Article are satisfied. *AASHTO 6.10.12*

Rolled beams with cover plates maybe used for simple spans up to approximately 90 feet in length and for continuous spans up to about 110 feet in length. An economic comparison of beams with and without cover plates, taking into consideration the fatigue requirements, should be undertaken for each design situation.

The thickness of cover plates shall not exceed 1½ times the thickness of the beam flange.

The cover plate end termination shall be detailed with a bolted slip-critical connection in accordance with the provisions of the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. This detail meets the requirements of Detail Category B. The sequence of installation specified in the referenced Article of the *AASHTO LRFD Bridge Design Specifications* shall be included on the Contract Drawings. *AASHTO 6.10.12.2.3 & Table 6.6.1.2.3-1 (Illustrative Example 22)*

6.7 BOX-SECTION FLEXURAL MEMBERS

6.7.1 Composite Sections

In continuous spans, both the positive and negative moment areas shall be designed to act compositely with the concrete deck in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 6.11.1*

Beam and girder haunches shall be detailed with a minimum 2-inch haunch at any point along the composite member. A 2-inch nominal haunch shall be used for composite member design computations.

6.7.2 Hybrid Sections

When economically feasible, hybrid sections (consisting of a web with minimum yield strength lower than that of one or both flanges) should be considered.

6.7.3 Web Proportions

The minimum web depth of the box girders shall be 6 feet to allow for future interior inspection of the box girders. Exceptions to this requirement must be approved by the Managing Bridge Engineer.

6.7.4 Constructability

All the provisions of the referenced Article of the *AASHTO LRFD Bridge Design Specifications* must be considered in the design. *AASHTO 6.11.3*

Instability during construction (as a result of the overturning tendency of horizontally curved box girder bridges as well as the low frictional characteristics of certain types of bearings) may present potentially hazardous situations during construction. The Consultant must therefore provide a statement on the contract drawings advising the Contractor to take all necessary precautions to insure stability, and thereby safety, during steel erection. A similar precautionary statement should also appear on the shop drawings.

6.7.5 Shear Connectors

The provisions of Article 6.6.4 of this Manual and the referenced Article of the AASHTO LRFD Bridge Design Specifications shall apply. *AASHTO 6.11.6.4*

6.8 CONNECTIONS AND SPLICES

6.8.1 Bolted Connections

Unless otherwise permitted by the Managing Bridge Engineer, all bolted connections shall be slip-critical connections and shall be designed in accordance with the provisions of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 6.13*

6.8.2 Splices

In continuous spans, field splices shall preferably be located as close as possible to the point of dead load contraflexure. When confirmed through the constructability study, girders up to 120 feet in length may be detailed to be shipped in one piece. Girders between 120 feet and 150 feet, shall be designed with an optional field splice which shall be shown on the contract drawings. Girders over 150 feet shall be designed with a mandatory field splice which shall be shown on the plans. A note should be provided on the contract drawings stating that (for girders less than 120 feet), a field splice may be provided when approved by the Engineer at no additional cost to the State.

SECTION 7
ALUMINUM STRUCTURES

7.1 SCOPE

Section 7 of the *AASHTO LRFD Bridge Design Specifications* covers the design of aluminum components used in bridge construction. Except as noted below, no exception is taken with regard to that Section:

NOTE:

Unless specifically specified in the latest edition of the Rhode Island Department of Transportation Standard Specifications for Road and Bridge Construction, the Rhode Island Standard Details, or the Rhode Island Bridge Design Standards, the use of aluminum as a bridge or structural material is not permitted.

SECTION 8
WOOD STRUCTURES

8.1 SCOPE

Section 8 of the *AASHTO LRFD Bridge Design Specifications* covers the specific design requirements for structural components made of sawn lumber products, stressed wood, glue laminated timber, timber piles and related mechanical connections. No exception is taken with regard to that Section.

SECTION 9
DECKS AND DECK SYSTEMS

9.1 GENERAL SCOPE

The minimum requirements for loading, analysis, design and detailing of bridge decks and deck systems are addressed in Sections 3, 4 and 9 of the *AASHTO LRFD Bridge Design Specifications*.

9.2 GENERAL POLICIES AND REQUIREMENTS

The requirements for concrete compressive strength, the use of corrosion inhibiting admixtures or other admixtures, the use of epoxy-coated reinforcing, and the use of a bituminous wearing surface versus an exposed concrete deck or an exposed concrete wearing surface shall all be in accordance with the Department's latest policies. The Designer shall coordinate with the RIDOT Project Engineer regarding the above.

9.3 GENERAL DESIGN REQUIREMENTS

9.3.1 Deck Continuity

Consistent with the philosophy that jointless and continuous bridge decks improve the durability, weather resistance, and future maintenance of bridges, all bridge decks and deck supporting components must be made continuous whenever possible. Additionally, where feasible, all bridge decks should be detailed and designed compositely with their supporting components.

9.3.2 Concrete Appurtenances

AASHTO 9.4.3

Unless otherwise specified by the Managing Bridge Engineer, vehicular parapets and barriers shall be made structurally continuous. Crack control joints, consisting of V notch type joints, should be provided in bridge parapets and barriers at a spacing not to exceed 30 feet. All longitudinal reinforcing shall be continuous through the crack control joints. Open expansion joints in bridge parapets and barriers shall be provided only at the roadway expansion joints.

9.4 LIMIT STATES

AASHTO 9.5.1

The structural contributions of concrete appurtenances (curbs, parapets, barriers, and dividers) to the deck shall not be considered for any limit states.

When permitted by the Managing Bridge Engineer, the structural contribution of parapets and barriers to the deck may be considered for bridge rating calculations.

9.5 ANALYSIS

AASHTO 9.6.1 & Appendix A4 in Section 4

In general the approximate elastic method of analysis referenced in the *AASHTO LRFD Bridge Design Specifications* shall be used for the design of concrete slabs. When required (refer to Article 9.6.1 of this Manual) the table provided in the Appendix of the referenced *AASHTO LRFD Bridge Design Specifications* may be used in determining the design moments, provided that the proposed deck design is within the limitations and assumptions listed in the Appendix.

The refined method of analysis referenced in the *AASHTO LRFD Bridge Design Specifications* shall be used only when approved by the Managing Bridge Engineer.

The empirical method of analysis will be considered by the Managing Bridge Engineer on a case-by-case basis. The Designer may propose the use of the empirical design method provided that the conditions set forth in the *AASHTO LRFD Bridge Design Specifications* are satisfied. Documentation clearly outlining that all the design conditions are satisfied must be submitted to the Managing Bridge Engineer for review and approval. *AASHTO 9.7.2*

9.6 CONCRETE DECK SLABS

9.6.1 Deck Design and Details

Deck designs and details (reinforcing size and spacing) for a range of beam spacing are included in the *Rhode Island Bridge Design Standard Details*. These designs assume that the proposed deck design is within the limitations and assumptions listed in the Appendix of the referenced *AASHTO LRFD Bridge Design Specifications*. For cases where an analysis is required, the analysis shall be in accordance with Article 9.5 of this Manual. *AASHTO Appendix A4 (Section 4)*

9.6.2 Minimum Deck Thickness and Covers

The minimum deck thickness shall be as follows:

- Decks with wearing surface $7\frac{1}{2}$”
- Exposed Decks (including 1 inch sacrificial surface) $8\frac{1}{2}$”
- Concrete (Deck) overlays for butted box beams & voided slabs 5”

For bridges in a designated coastal environment or subject to de-icing salt spray, the above minimum deck thicknesses for decks with wearing surface and exposed decks shall increase by ½” to accommodate the increased clearance to bottom reinforcing noted below.

The minimum clear cover to the top and bottom reinforcing shall be as specified below:

	Top	Bottom
• Decks with wearing surface	2”	1”
• Exposed Decks (including 1 inch sacrificial surface)	3”	1”
• Concrete (Deck) overlays for butted box beams & voided slabs	$2\frac{1}{2}$”	-

The minimum clear cover to the bottom steel shall be increased to $1\frac{1}{2}$” for bridges in a designated coastal environment or subject to de-icing salt spray as determined on a case-by-case basis by the Managing Bridge Engineer.

The deck slab reinforcing cover shall have a tolerance of (+$\frac{1}{4}$”, -0”) for top bars and (+$\frac{1}{8}$”,-0”) for bottom bars, and this shall be considered in the design stresses.

9.6.3 Skewed Decks

When the skew of the deck does not exceed 30 degrees, the primary reinforcement may be placed in the direction of the skew. For skew angles exceeding 30 degrees, *AASHTO 9.7.1.3* the primary reinforcement shall be placed perpendicular to the main supporting components.

9.6.4 Cantilever Slab Design

The maximum deck slab cantilevers on steel or concrete beams or girders shall not exceed:

- The depth of the outside fascia beam or girder,
- One half the spacing between the beams or girders,
- Six feet

Any exceptions to the above criteria must be approved by the Managing Bridge Engineer.

The deck cantilever shall be designed for railing/barrier impact loads in accordance with the provisions of the referenced Appendix of the *AASHTO LRFD Bridge Design Specifications*. Alternatively, the deck reinforcing used in the cantilever from successful crash tests may be specified, provided the actual overhang length does not exceed the overhang used for the crash tests.

AASHTO Section 13 (Appendix A)

9.6.5 Reinforcement

All deck reinforcing shall be epoxy coated. Other reinforcing protection such as galvanized or stainless steel reinforcing may be considered on case-by-case basis.

Deck reinforcing size shall preferably not exceed #6 reinforcing.

9.6.6 Minimum Negative Flexural Concrete Deck Reinforcement

Wherever the longitudinal tensile stresses in the concrete deck (due to either factored construction loads or Load Combination Service II) exceed ϕf_r, the total cross sectional area of the longitudinal reinforcement shall meet the requirements provided in the referenced Article of *AASHTO LRFD Bridge Design Specifications.*

AASHTO 6.10.1.7

9.6.7 Longitudinal and Transverse Deck Joints

The need for longitudinal construction joints should be assessed based on the deck finishing machine requirements, but generally a longitudinal open joint is required for decks in excess of 90 to 100 feet wide. The longitudinal construction joints should be located at the centerline of the bridge median.

The use of transverse construction joints should be avoided. If they can not be avoided, their use must be approved by the Managing Bridge Engineer.

9.6.8 Deck Pouring Sequence

Deck pouring sequence for continuous spans should be evaluated taking into consideration deck stresses, uplift, and the total volume of concrete. The sequence of placing deck concrete on continuous spans (multiple concrete pours if required) shall be shown on the contract drawings. Large concrete pours may require retarding admixture and should be discussed with the Bridge Project Engineer. Appropriate notes pertaining to the use of admixtures should be included on the drawings and/or in the Special Provisions.

9.6.9 Stay-in-Place Forms

9.6.9.1 General

The use of permanent steel or concrete stay-in-place forms must be considered for railroad crossings, bridges over heavy traffic, heavily congested utility bays, or when extremely high underclearances may pose a safety hazard to construction workers. For all other construction projects, the use of removable versus stay-in-place forms shall be coordinated with the Department.

9.6.9.2 Concrete Stay-in-Place Forms

AASHTO 9.7.4.3

It is the Department's policy to permit only precast concrete stay-in-place forms which are prestressed. The design and detailing requirements of concrete stay-in-place forms shall be in accordance with the referenced Article of *AASHTO LRFD Bridge Design Specifications* except as modified as follows:

- The thickness of the concrete stay-in-place forms (panels) shall not be less than $3\frac{1}{2}$" or exceed 55% of the total deck thickness.
- The minimum thickness of the cast-in-place portion of the slab shall be $4\frac{1}{2}$".
- The minimum 28 day concrete compressive strength shall be 6,000 psi.
- The tension in the precompressed tensile zone under full service conditions (after all losses) shall preferably not exceed $3\sqrt{f'_c}$. A higher value up to $6\sqrt{f'_c}$ may be permitted at the approval of the Managing Bridge Engineer.
- The compression in the panel at release shall not exceed 25% of the compressive strength at release.
- Prestressing strands shall be $\frac{3}{8}$" diameter, located mid-depth of the panel.
- The strands shall extend a minimum of 4" outside the panel ends.
- The top surfaces of the panels shall be broom roughened to an amplitude of approximately 0.06 inches.
- Concrete stay-in-place forms are required to be grouted in place, and the grout shall be cured prior to the placement of the cast-in-place deck. The grout bed shall extend for the full width of the girder flange such that the area between the "grout dams" is completely filled. The top of the grout bed shall be 1 inch clear below the strand extensions.
- The temporary supports for the stay-in-place forms shall consist of continuous, high density expanded polystyrene strips with a minimum compressive strength of 55 psi. An approved adhesive shall be used to affix the grout dam to the girder and the stay-in-place forms. If leveling screws are specified, they shall be completely removed after the grouting operation and prior to the deck placement. When leveling screws are used, temporary bracing between the ends of the stay-in-place forms shall be specified to prevent transverse movement of the forms and loss of bearing on the leveling screws.
- In determining the minimum haunch height, consideration should be given to an allowance for cross slope as well as beam camber tolerances. For steel girders or beams, the haunch depth should accommodate the thickness of the splice plates and bolts.

The Designer is also referred to the *Rhode Island Bridge Design Standard Details* for concrete stay-in-place form details.

9.6.9.3 Steel Stay-in-Place Forms

The details of the steel stay-in-place forms shall be in accordance with the *Rhode Island Bridge Design Standard Details*.

9.6.10 Full Depth Precast Concrete Deck Slabs

Full Depth concrete deck slabs are permitted only with the approval of the Managing Bridge Engineer. It is the Department's policy to permit only precast full depth decks which are prestressed.

AASHTO 9.7.5

9.7 METAL DECKS

The use of corrugated metal decks is not permitted.

AASHTO 9.8.5

SECTION 10
FOUNDATIONS

10.1 GENERAL SCOPE

The minimum requirements for the design of spread footings, driven piles, and drilled shaft foundations shall be in accordance with the *AASHTO LRFD Bridge Design Specifications* unless otherwise modified or further clarified in this Section.

10.2 SOIL AND ROCK PROPERTIES

10.2.1 Subsurface Exploration and Site Investigation

In general, it is not advisable to establish strict guidelines for instituting a subsurface exploration and testing program. Site conditions will determine the extent of and therefore the number, depth, spacing, and the sampling and testing requirements for a given project. However, it is essential that a comprehensive subsurface exploration and testing program be conducted prior to establishing geotechnical design parameters and requirements. Except as modified in this Article, the minimum requirements of the *AASHTO LRFD Bridge Design Specifications* shall apply. Additionally, the Consultant Designer shall refer to the Report entitled *Guidelines for Geotechnical Site Investigations in Rhode Island, Final Report*, dated March 30, 2005. This publication provides guidelines for planning and conducting geotechnical site investigations in Rhode Island.

AASHTO 10.4

Subsurface explorations must be planned and conducted to provide information for both the design as well as construction of foundations. The extent of subsurface explorations and site investigations must be based on both the project requirements and the variability in the subsurface conditions affecting the foundation design and construction. These shall consist of: design and construction requirements, identification of limiting settlements, areas of concerns with variability of geology, hydrologic concerns, construction phasing and constraints which will necessitate additional geotechnical information, and environmental requirements. A subsurface exploration and laboratory testing program should provide sufficient data to determine the type and depth of the foundation, safe bearing resistance of the soil, predicted settlements, type and bearing resistance capacity of piles or drilled shafts, potential for slope instability, ground water elevation, and all other pertinent soil related parameters necessary to evaluate soil properties for the foundation analysis of the various structural elements on a project.

Project constraints may warrant the acquisition of subsurface data for use during the bidding and construction stage(s) in order to reduce bidding uncertainties, such as for the purpose of establishing the limit and nature of the materials to be excavated (such as ledge, unsuitable material, or remnants of any suspected existing buried structures or foundation) or for the purpose of establishing artesian conditions.

Some projects may require an Environmental (hazardous waste) subsurface exploration program. Should such an investigation be required, it shall be coordinated and performed as part of the subsurface exploration program. The staff of boring contractors bidding for hazardous waste contracts shall be properly trained and certified for hazardous material work.

Seismic design considerations shall also be taken into account when establishing the subsurface exploration and testing program.

10.2.2 Laboratory and In-situ Testing

Except as modified in this Article, the minimum requirements of the *AASHTO LRFD Bridge Design Specifications* shall apply. Additionally, the Designer shall refer to the Report entitled *Guidelines for Geotechnical Site Investigations in Rhode Island, Final Report,* dated March 30, 2005. This publication provides guidelines for planning and conducting geotechnical laboratory and in-situ testing in Rhode Island.

AASHTO 10.4

10.2.3 Subsurface Exploration and Site Investigation Plan Submission

The proposed subsurface exploration and testing program must be submitted to the Managing Bridge Engineer for review. The program shall include a general location or vicinity map plan showing the location of the properly numbered proposed borings, test pits, and observation wells. The proposed laboratory testing requirements, as well as a detailed cost estimate and description of the work to be accomplished, shall also be included.

In general, the Consultant shall prepare a soil-investigation contract in order to solicit bids from boring contractors. Such contractors shall be approved in writing by the Managing Bridge Engineer, and at least two bids for each soil investigation contract shall be secured.

Contracts exceeding an estimated cost of $100,000 will be advertised through the Department's Contract and Specifications Section. Exceptions to this limit must be approved, in writing, by the Department.

10.2.4 Recording Information on Boring Logs

The information required and the manner of securing and recording the information from the borings; shall be in accordance with the established standards of the Department. The information shall be plotted on standard size sheets and included in the preliminary submission of plans and the Geotechnical Report.

The information given on the plans shall as a minimum include: Actual logs of the borings, existing ground elevations; blows on the casing; type, general description and depth at top and bottom of each soil stratum encountered; location and depths of samples taken; size of sampler; number of blows on sampler; elevation of ground water; obstructions encountered; a legend showing the type of equipment and any other data that may prove valuable in assessing subsurface conditions. Chapter 3 of the publication entitled *Guidelines for Geotechnical Site Investigations in Rhode Island, Final Report*, dated March 30, 2005, provides some general guidelines and a sample typical boring log.

10.2.5 Geotechnical Report

10.2.5.1 General

The disclosure of subsurface information requires the preparation of a Geotechnical Report consisting of two parts: the Geotechnical Data Report (GDR) and the Geotechnical Interpretive Report (GIR). These reports provide the means by which the factual project site conditions are presented (GDR), as well as a formal design and construction recommendations with respect to all aspects of a project (GIR).

Project geotechnical data must be made available and disclosed to the bidders by inclusion in the contract documents as follows:

- A copy of the approved Geotechnical Data Reports (GDR) must be incorporated into the contract documents.
- Pertinent soil and rock samples should be made available to all bidders at a designated time and location during the bidding phase.
- The Geotechnical Interpretive Report should be made available for review to all bidders at a designated time and location during the bidding phase. Additional copies should be available if deemed necessary.

10.2.5.2 The Geotechnical Data Report

The GDR should include only the factual information included in the GIR. This report format consists of an introduction (purpose, scope, and report limitations); background information, geologic setting; a description of all field investigations as well as the procedure and methods used in the investigation; and a description of the testing program and of the procedure and methods. Copies of the boring location plan, boring logs and laboratory test results should also be included.

10.2.5.3 The Geotechnical Interpretive Report

The purpose of a Geotechnical Interpretive Report is to provide specific engineering design and construction recommendations which are essential for the design of the various elements of a project. The Geotechnical Interpretive Report should contain an interpretation of the subsurface conditions, soil profiles, and materials which may be encountered; design considerations such as type of foundation support (soil bearing or deep foundation bearing resistance with their related design considerations), slope stability, and settlement; a discussion of conditions that may be encountered during construction with recommendations; and any other pertinent design and construction aspects. Recommendations for any special notes that may be required to be indicated on the contract drawings or any specific contractual provisions to address specific geotechnical conditions should also be included in the Geotechnical Report.

Numerous publications are available through the Federal Highway Administration publication website and other sources which provide a more detailed discussion on the preparation of Geotechnical Interpretive Reports. One such publication is the publication entitled *Soils and Foundations Workshop Manual* published by the FHWA (NHI-00-045). Reference is also made to the FHWA publication entitled Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications , FHWA ED-88-053, revised February 2003.

10.2.5.4 Differing Site Condition Disclaimers

The disclosure of subsurface information to bidders has been a controversial topic on past Department projects. Though there is no comprehensive list of practices which can be summarized in order to help to avoid or minimize contract claims, there are several common pitfalls to avoid. Reference is made to a memorandum entitled Engineering Notebook Issuance GT-15 Geotechnical Differing Site Conditions, prepared by FHWA dated May 2, 1996. "The purpose of this document is to provide guidelines on the practical application of a "Differing Site Condition" (DSC) contract clause, as related to subsurface conditions, and to address the variable nature of soil and rock materials when used as a foundation or construction material. This guideline should be of benefit to Geotechnical, Design and Construction personnel. Recommendations are provided on disclosure and presentation of subsurface information to bidders.

The objective of these recommendations is, in part, to decrease bidding discrepancies on subsurface items, address unexpected subsurface problems early, and provide a basis for equitable resolution of contractor claims based on differing subsurface conditions.

10.2.5.5 Seismic Geotechnical Considerations

When, in accordance with Article 3.6 of this Manual, it has been determined that a seismic analysis is required, the structural engineer and the geotechnical engineer shall coordinate to determine the data required for the ground stability evaluation as well as for the parameters which may be necessary for the adequate seismic modeling of the structure. The soils parameters that may be required are:

- Friction angle
- Unit weight
- Young's Modulus
- Shear Modulus
- Poisson's Ratio
- Shear strength
- Liquefaction strength
- Permeability
- Coefficient of compressibility
- Relative density
- Foundation Flexibility Parameters (spring coefficients)

In addition to establishing the required soil design parameters necessary for the foundation modeling, the following supplemental design considerations should be evaluated during the geotechnical study phase:

The Potential for Liquefaction: The potential for liquefaction of saturated granular foundations soils should be evaluated by the geotechnical engineer. Saturated granular soils having low standard penetration blow counts are generally considered to be suspect for liquefaction. A factor of safety of 1.0 (between the available liquefaction strength and the earthquake-induced dynamic stress) may be used when evaluating liquefaction. In addition to cohesionless soils, there is data suggesting that certain low plasticity clayey soils may also be vulnerable to strength loss and therefore should be investigated. In general, spread footings should be avoided when there is a strong likelihood of liquefaction.

Slope Stability: A significant amount of displacement of slopes at abutments or in areas of earth fills (as a result of slope instability) will result in consequential damage to the abutments or the earth retaining structures. The possibility for earthquake induced slope instability at a given site should therefore be evaluated. A resistance factor of 1.0 shall be used for seismic earth stability calculations.

Dynamic (earthquake-induced) Settlement: At any given site the potential for dynamic settlement or volume reduction of cohesionless soil is of concern. Two forms of dynamic settlement (earthquake-induced) may occur. The potential exists for (1) overall site settlement as well as (2) localized settlement which may cause differential settlement, the effects of which should also be considered.

Acceleration-Augmented Lateral Earth Pressure: When required under Article 11.3.5 of this Manual, abutments and retaining walls shall be designed for seismic-induced lateral forces and wall inertia forces as a result of earthquake-induced motions. The pseudo-static Mononobe-Okabe method analysis shall be used for computing the lateral active soil pressures during seismic loading.

AASHTO 11.6.5 & Appendix A (Section 11)

10.3 LIMIT STATES AND RESISTANCE FACTORS

10.3.1 Service Limit States

In addition to settlement, horizontal and rotational movements, and overall stability, the foundation design at service limit state shall also include scour at the design flood event. Scour considerations shall only apply to the ability of the foundation to meet the specified deflection criteria as specified herein.

AASHTO 10.5.2

The overall stability investigation shall be in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*.

AASHTO 11.6.2.3

Total tolerable movements shall be accessed on a project to project basis and must consider the following:

- Structure type and function
- Consequences of the movements and tolerances to differential movements
- Structure detailing (such as roadway joints and bearings)
- Economy
- Rideability

The tolerable movements must be established at the preliminary phase of the project by the Consultant and must be approved by the Department.

Spread footing bearing resistance (estimated using the AASHTO LRFD presumptive values) shall be used for preliminary foundation sizing only.

10.3.2 Strength Limit States

The strength limit states shall consider the structural and geotechnical resistance of the foundation components as well as the loss of lateral and vertical support due to scour at the design flood event. Structural and geotechnical resistance shall include axial, lateral, and flexural resistance.

For spread footings, the strength limit states shall also consider bearing resistance, overturning and sliding.

For deep foundations (driven piles and drilled shafts), the strength limit states shall also consider single and group axial compression resistance, single and group uplift resistance, and single and group lateral resistance.

10.3.3 Extreme Limit States

The foundations shall be designed for extreme load states as defined in Section 3 of this Manual.

10.3.4 Resistance Factors

10.3.4.1 General

Unless otherwise specified in this Article, the Resistance factors shall be established in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 10.5.5*

10.3.4.2 Resistance Factors for Deep Foundations

The resistance factors provide in the *AASHTO LRFD Bridge Design Specifications* are based on a partial adoption of the recommendation made in the NCHRP Report 507 "*Load and Resistance Factor Design (LRFD) for Deep Foundations* (prepared as part of the research conducted under NCHRP Project 24-17). The Department's experience has been that for specific subsurface conditions, the pile capacities will be underestimated when based on static analysis and resistance factors specified in the *AASHTO LRFD Bridge Design Specifications.* It is therefore the Departments policy to investigate the approach presented in the *AASHTO LRFD Bridge Design Specifications* as well as in the NCHRP Report 507 when evaluating resistance factors and performing static capacities analysis for deep foundations. Any discrepancies between the two approaches shall be discussed with the Department and appropriate recommendations made by the geotechnical engineer.

The proposed resistance factor for large diameter piles (larger than 24" diameter) must meet the approval of the Department. *AASHTO C10.5.5.2.3*

10.4 CONSTRUCTABILITY

The design of all foundation types must consider the effects of the anticipated method of construction, including the construction sequencing. Such considerations shall consist of, but not be limited to, the need for shoring, the use of cofferdams, dewatering, excavation stability, downdrag considerations for driven piles, and the need for permanent or temporary casing for drilled shafts.

10.5 SPREAD FOOTINGS

10.5.1 Footing Depth and Thickness

The minimum spread footing embedment depth (below finished grade) shall be the greater of the depths as determined from: *AASHTO 10.6.1.2*

- The geotechnical assessment of the underlying soil,
- The depth below the maximum computed scour depth, or
- Four feet (the maximum anticipated depth of frost potential).

The minimum thickness of spread footings shall be 1'-6".

10.5.2 Tolerable Movements

Tolerable movements shall be established based on Article 10.3.1 of this Manual.

10.5.3 Bearing Resistance

The nominal bearing resistance for the strength limit states and the extreme limit states shall be as determined based on the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 10.6.3*

10.5.4 Failure by Sliding

Except for the extreme limit states, the effects of passive soil resistance in front of the footing shall not be included as part of the shear resistance required for resisting sliding. The effects of passive soil resistance may be included for the extreme limit states. *AASHTO 10.6.3.4*

10.6 DRIVEN PILES

10.6.1 General

10.6.1.1 Driven Pile Types

The type of pile(s) to be used depends upon such factors as subsurface conditions, pile availability, required length, driving resistance characteristics of the soil, load to be supported, and relative economies. The pile types typically used in Rhode Island are cast-in-place small diameter (10 to 14 inch) concrete filled steel pipe pile (open-end or closed-end), precast prestressed concrete piles and steel H-piles. On past projects mandrel driven or metal concrete filled shell piles have also been considered. The above pile types are typically suitable in the range of 60 to 200 tons. For larger projects, higher capacity driven piles consisting of large diameter (four to six feet) pipe piles may be considered at the approval of the Department.

In general, piles shall be equipped with appropriate tip protection, specifically when pile penetration through obstructions, cobbles, boulders or debris fill is anticipated.

Timber piles are permitted only for temporary construction. Timber piles are typically suitable within a capacity range of 15 to 40 tons and a length range of 20 to 50 feet.

10.6.1.2 Footing Depth and Thickness

In general, the thickness of pile supported footings shall not be less than 2'-6". Piles must be positively anchored into the footing and shall extend a minimum of 12 inches into the footings.

The bottom layer of footing reinforcement may be placed above the top of the piles to avoid interference with the piles.

10.6.1.3 Minimum Pile Spacing and Edge Distances

To allow for pile driving tolerances, the minimum spacing between piles shall be the greater of 3'-6" or 2.5 times the pile diameter (or width) plus a six inch driving tolerance. The preferred maximum pile spacing for smaller capacity piles shall be 10 feet.

The minimum distance from the nearest edge of the footing to the face of any pile shall be 1'-3". For piles in cofferdams, due consideration shall be given to providing sufficient distance between the face of the sheeting and the pile in order to clear the cofferdam bracing system.

10.6.1.4 Battered Piles

When battered piles are considered to resist lateral horizontal forces, they shall preferably be inclined 1 on 4 or less from the vertical, and in no case shall they be battered more than 1 on 3. Battered piles should be avoided when negative skin friction (downdrag) loads are anticipated. However if they are required when downdrag is anticipated, the pile batter shall not exceed 1 on 6 from the vertical.

10.6.1.5 Pile Design Requirements

Piles may be designed as friction, end bearing, or a combination of friction and end bearing. The design pile capacity shall be the lesser of

- The capacity of the surrounding soil which will support the pile, or
- The structural capacity of the pile type chosen.

Pile design requirements shall address the design considerations outlined in the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. In all cases the footing shall be adequately reinforced to distribute the load between the piles.

AASHTO 10.7.1.5

10.6.1.6 Scour

The pile foundation shall be designed so that, after the scour event, the required strength resistances are satisfied.

10.6.2 Service Limit States

The tolerable movements for settlements and lateral deformation for the service limit states shall be established based on Article 10.3.1 of this Manual.

10.6.3 Strength Limit States

10.6.3.1 Axial Pile Foundation Resistance

Nominal axial pile resistance shall be in accordance with Article 10.3.4 of this Manual and the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. The method used to verify the resistance shall be based on a combination of static analysis, load test, dynamic test, wave equation and dynamic formula. The procedure used to determine the nominal pile resistance must be approved by the Department.

AASHTO 10.7.3.8

The method used to establish and verify the nominal resistance of large diameter driven piles (in excess of 24 inch diameter) must meet the approval of the Department.

In general, the use of the Engineering News Record (ENR) formula in determining pile axial resistance is not permitted.

10.6.3.2 Pile Foundation Resistance to Horizontal Forces

The design value of resistance to lateral forces assigned to a pile depends upon the resistance of the surrounding soil and the stiffness of the pile. When determining lateral load capacity for piles, arbitrary values (available from several sources) may not be used. The use of an analytical method is required. Two such analytical approaches are Brom's method and Reese's method.

On larger projects, the need for performing lateral pile load test(s) during the design stage should be considered.

10.6.4 Corrosion and Deterioration

The effects of corrosion and deterioration from environmental conditions outlined in the referenced Article of the *AASHTO LRFD Bridge Design Specifications* shall be taken into consideration. The protective measures for prevention versus the possible design measures taken to address deterioration and corrosion must be evaluated and discussed with the Department. Prevention or protective measures may include deduction of surface area, application of protective coatings or concrete encasement (for steel piles), and the use of high quality concrete, corrosion prevention additives or increased reinforcing clearances (for concrete piles).

AASHTO 10.7.5

10.6.5 Pile Drivability Analysis

For all projects, the Geotechnical Engineer must verify the suitability of the proposed pile types, the ultimate capacity, the desired depth, and the stresses during pile installation by the use of a Wave Equation Analysis (WEAP). Typical pile hammer sizes may be obtained from local pile driving contractor(s).

AASHTO 10.7.8

10.6.6 Pile Load Tests

On larger projects pile load test(s) are generally required for each pile capacity and type specified. The purpose of the pile load test(s) are to determine or verify the available pile capacity, the anticipated driving stresses, the pile drivability, and the final pile driving criteria. When load test(s) are required, the number specified shall be determined by the variation in subsurface conditions, the pile capacities and type used, and the availability of pile driving records within the project area. On larger projects, the load test(s) shall be performed during design. However the need for additional pile load test(s) during construction must be considered and discussed with the Department.

Pile load tests may be waived on smaller projects when there is a limited number of piles and when it is determined that it would be more cost effective to utilize lower resistance factors in accordance with the established criteria of the referenced Article in the *AASHTO LRFD Bridge Design Specifications* and Article 10.3.4 of this Manual.

AASHTO 10.5.5.2.3

On larger projects, the need to perform pile load tests during the design stage must be considered.

The method used for testing piles for axial compressive load shall be the "Quick Load Test Method" as outlined in the ASTM Standard Specification D 1143. The procedure used in determining the pile axial resistance from the test data shall be as specified in the referenced Article of the *AASHTO LRFD Bridge Design Specification*.

AASHTO 10.7.3.8.2

10.7 DRILLED SHAFTS

10.7.1 General

10.7.1.1 Scope

Drilled shafts should be considered when high vertical or lateral loads are to be resisted or when only small deformations can be tolerated. They may also be used as direct support columns as used in pier bents.

10.7.1.2 Minimum Drilled Shaft Diameter, Spacing, Edge Distance and Embedment

The minimum drilled shaft diameter shall be 3 feet.

The minimum center to center spacing of drilled shafts shall preferably be 4 shaft diameters but in no case less than 2.5 shaft diameters. For spacing less than 4 shaft diameters, group interaction effects between shafts must be taken into consideration. *AASHTO 10.8.3.6*

Drilled shafts shall extend a minimum depth into the footing to develop the required structural resistance, but in no case less than 6 inches. Drilled shafts shall be positively anchored into the footing.

The minimum distance from the nearest edge of the footing (or pier cap) to the face of any drilled shaft shall be 1 feet 3 inches.

10.7.1.3 Battered Drilled Shafts

The use of battered drilled shafts is not permitted. Larger diameter shafts or an increase in the number of shafts shall be considered when an increase in lateral resistance is needed.

10.7.1.4 Drilled Shaft Design Requirements

Drilled shafts shall be designed to provide adequate axial and structural resistance consistent with the anticipated construction method. The drilled shaft design must also satisfy the tolerable vertical and lateral displacement criteria. *AASHTO 10.8.1.5*

10.7.1.5 Scour

The drilled shaft foundation shall be designed so that after the scour event the required strength resistances are satisfied.

10.7.2 Service Limit States

The tolerable movements for settlements and lateral deformation for the service limit states shall be established based on Article 10.3.1 of this Manual.

10.7.3 Strength Limit States

The methods used for estimating nominal axial drilled shaft resistance shall be in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. The use of methods not specifically addressed in the *AASHTO LRFD Bridge Design Specifications* must meet the approval of the Department. *AASHTO 10.8.3.5*

10.8 PROJECT DOCUMENTATION OF GEOTECHNICAL DATA

As applicable, at a minimum the following geotechnical information must be shown on the contract drawings:

- For foundations bearing on earth, the nominal and the actual maximum design pressures for each substructure unit.

- For foundations supported on piles or drilled shafts, the nominal axial load and the actual maximum pile or drilled shaft design load for each substructure unit.

- The estimated pile take-up elevations or the estimated drilled shaft tip elevations.

- The resistance factors for the various limit states.

- Other information pertaining to the design of the foundations such as short term and long term settlements.

SECTION 11
ABUTMENTS, PIERS, AND WALLS

11.1 GENERAL SCOPE

The minimum requirements for the design of abutments, piers and walls shall be in accordance with the *AASHTO LRFD Bridge Design Specifications* unless otherwise modified or further clarified in this Section.

11.2 LIMIT STATES AND RESISTANCE FACTORS

11.2.1 General

The design of abutments, piers and walls shall satisfy the service limit state, strength limit state, and extreme limit state criteria.

11.2.2 Service Limit States

The provisions for accessing the acceptable total tolerable settlements and the horizontal and rotational movements discussed in Article 10.3.1 of this Manual shall apply to the investigation of abutments, piers, and walls. The tolerable movements must also consider the consequences of damage to adjacent structures and utilities as well as perceptions of unsightly deformations.

11.2.3 Resistance Factors

Resistance factors shall be in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications* and Article 10.3.4 of this Manual. *AASHTO 11.5.6*

11.3 ABUTMENTS AND CONVENTIONAL RETAINING WALLS

11.3.1 Abutment and Wall Selection

11.3.1.1 Abutment Selection

Integral and semi-integral abutments are the preferred abutment types when the limitations specified in Articles 11.3.3.2 and 11.3.4.2 of this Manual are met. All projects must therefore consider the use of integral or semi-integral bridge abutments as part of the bridge type or design alternative studies.

When integral abutments are not suitable, other abutment types described below may be considered.

- In general, for heights up to 30 feet, abutments shall be of the cantilever type. For greater heights, a reinforced concrete counterfort type should also be considered.

- For smaller heights rigid gravity abutments (abutment constructed of minimally reinforced concrete) or semi-gravity abutments (abutments constructed of reinforced concrete) should be considered.

- Stub abutments should be considered when an abutment is to be located at or near the top of the approach fill.

11.3.1.2 Wall Selection

The use of rigid gravity walls (walls constructed of stone masonry and/or minimally reinforced concrete) and semi-gravity walls (walls constructed of reinforced concrete) shall be limited to heights not exceeding 12 feet (measured from footing invert to top of wall). For heights in excess of 12 feet, cantilever, counterfort or buttress type walls (constructed of reinforced concrete with a separate footing pour) shall be used.

In addition to conventional gravity, cantilever or counterfort walls, the economics and the feasibility of other types of walls consisting of anchored walls, mechanically stabilized earth walls and prefabricated modular walls must be considered during the preliminary phase of a project. The suitability and use restrictions of these wall types are discussed in the referenced *AASHTO LRFD Bridge Design Specifications*.

AASHTO 11.9.1 11.10.1 & 11.11.1

11.3.2 Loadings

11.3.2.1 General

Loading criteria shall be in accordance with the referenced Articles of the *AASHTO LRFD Bridge Design Specifications.*

AASHTO 11.6.1.2 & 3.11

11.3.2.2 Earth Pressure

When an outward tilting of a wall or abutment is restricted such that active lateral earth pressures do not develop, the lateral earth pressures shall be computed using the at-rest condition. Walls or abutments which can move away from the soil mass shall be designed for active or passive pressure depending on the magnitude of movement. The relationship between soil backfill and wall/abutment tilting to mobilize minimum active pressure or maximum passive pressure is given in the referenced AASHTO LRFD Bridge Design Specifications.

AASHTO Table C3.11.1-1

Except for the extreme limit states, the effects of passive soil resistance in front of the footing shall not be included as part of the shear resistance required for resisting sliding. The effects of passive soil resistance may be included for the extreme limit states only.

11.3.3 Integral Abutments

11.3.3.1 Definition and Detailing

Integral abutments are abutments which are supported on single row of flexible H-piles and which are rigidly connected to the superstructure. Details of integral abutments are shown on the applicable drawings included with the Rhode Island Bridge Design Standard Details.

11.3.3.2 Limitations on Integral Abutments

The following limitations shall apply to the use of integral abutments:

- To minimize the adverse rotational impacts upon the backwalls and wingwalls, the bridge skew angle shall be limited to 30 degrees.
- The total bridge length shall be limited to 350 feet for steel bridges and 600 feet for concrete bridges.
- The maximum grade between abutments shall be 5%.

- The horizontal bridge alignment shall be straight. The criteria used to establish when a bridge is considered "straight" (that is when the effects of curvature may be ignored) are defined in the referenced Articles of the *AASHTO LRFD Bridge Design Specifications.* *AASHTO 4.6.1.2.4b & 4.6.1.2.4c*
- Only steel beams and concrete (I and box) beams shall be used with integral abutments.
- Girder depths shall be limited to six feet.
- Sufficient pile flexibility must be achievable to allow for the anticipated movements.
- For multiple span bridges, the span arrangement and interior bearing fixity selection shall be such that movements at both abutments are approximately equal.
- The ratios of span lengths shall ensure that no net uplift occurs at the abutments at all limit states.

Any deviation from the above guidelines must be approved by the Managing Bridge Engineer.

11.3.3.3 Analysis and Design Requirements

11.3.3.3.1 Loading: Integral abutments, including the supporting piles, shall be designed to resist all dead loads and live loads as well as all horizontal loads and movements. Except as modified here in this Article, the minimum requirements for loads and their application, the applicable load factors, and applicable load combinations shall be in accordance with the requirements of the *AASHTO LRFD Bridge Design Specifications* and as modified in Section 3 of this Manual.

The design of the integral abutment shall consider the combined load effects at various stages of bridge construction.

11.3.3.3.2 Dynamic Allowance: Dynamic load allowance shall be considered in the design of integral abutments, including the design of the top four feet of the piles.

11.3.3.3.3 Lateral Earth Pressure: The intensity of lateral earth pressure is a function of the type of soil and amount of anticipated backfill movement relative to the wall height. Thus the lateral earth pressure distribution is dependent on the soil and pile interaction and is some value between the at-rest earth pressure and the full passive earth pressure. The approximate values of relative movements required to reach full passive pressure are provided in the referenced *AASHTO LRFD Bridge Design Specifications*. However, several other sources provide guidance with respect to the soil pressures developed when these movements are not realized and when the soil pressure is some value between the at rest pressure and the full passive pressure. One such reference is the US Department of the Navy *Design Manual – Foundations and Earth Structures, NAVFAC DM-7.* *AASHTO Table C3.11.1-1*

11.3.3.3.4 Thermal Movements: The thermal movements shall be estimated in accordance with Procedure B of the referenced *AASHTO LRFD Bridge Design Specifications.* The minimum and maximum design temperatures are as specified in Article 3.8 of this Manual. *AASHTO 3.12.2*

11.3.3.3.5 Load Factors: Thermal movement is a major source of the loads on the abutment and abutment piles. As such the load factor for uniform temperature, γ_{TU}, specified in the referenced *AASHTO LRFD Bridge Design Specifications* may yield unconservative loads for this type structure. The load factor for uniform temperature γ_{TU} for Strength limit states shall be taken as 1.0.

AASHTO 3.4

The load factor for passive pressure γ_{EH} shall be taken as 1.25. The load factors for at rest and active earth pressure shall be as specified in the referenced Table of the *AASHTO LRFD Bridge Design Specifications*.

AASHTO Table 3.4.1-2

11.3.3.3.6 Pile to Superstructure Connection: The connection between the pile and the abutment shall be assumed to be rigid. Piles shall be embedded a minimum of 2'-0" into the pile cap or as required to develop the pile plastic moment capacity. The integral abutment shall be adequately designed and detailed to transfer all applied loads from the superstructure. The piles shall be adequately anchored into the abutment cap to prevent any unanticipated uplift movements.

11.3.3.3.7 Pile Design: Integral abutments shall be supported on a single row of flexible H-pile foundation where the piles are oriented with the weak axis perpendicular to the longitudinal axis of the bridge, regardless of skew. To allow for flexibility, the piles shall be driven in oversized pre-augured holes 10 feet deep and filled with pea stone. The minimum diameter of the pre-augured hole shall be the larger of:

- 2'-0" diameter
- The diagonal dimension of the pile plus 10 inches.

Pre-augured holes are not required for bridges with a span length less than 35 feet.

All piles shall be driven to a depth meeting the minimum pile penetration and design requirements of the referenced Article of the *AASHTO LRFD Bridge Design Specifications*.

AASHTO 10.7.1.5 & 10.7.6

The nominal pile structural resistance shall be determined in accordance with the requirements of Article 10.6 of this Manual.

The design of an axially loaded pile subject to lateral forces and/or lateral deformation involves a process which accounts for the soil-pile interaction, in that the pile deflection is dependent on the soil response and the soil response is a function of the pile deflection.

The analysis and design of piles is based on the principle that the pile will behave elastically provided the maximum factored moment at the head of the pile does not exceed the plastic moment capacity of the pile and that adequate pile ductility exists. Once the plastic moment is reached, a plastic hinge will form; no further increase in pile moment capacity will be achieved and there will be a redistribution of forces associated with this inelastic behavior. Therefore, as described below, the design of integral abutment piles may be based on a conventional elastic design approach or an inelastic design approach and shall be as follows:

> Perform an iterative analysis by applying the factored thermal movement (determined in accordance with Article 11.3.3.3.4 of this Manual) and an assumed factored pile head moment. This analysis

should be performed using a computer program such as L-Pile or COM624P. Reanalyze by revising the assumed factored pile head moment until the factored pile head rotation obtained from the analysis is equal to the factored pile head rotation due to the superstructure superimposed dead load and live loads. Compare the factored pile head moment value from the analysis to the pile plastic moment capacity and design the pile according to one of the following:

- *Use a conventional elastic design method if the factored pile head moment from the above analysis is less than the plastic moment capacity of the pile*: The design of the pile shall satisfy the *AASHTO LRFD Bridge Design Specification* combined axial compression and flexural interaction relationship (the factored axial load shall be the applied factored pile axial load and the pile factored design moment shall be the maximum factored bending moment as determined from the above analysis).

- *Use an inelastic design approach if the factored pile head moment from the above analysis exceeds the plastic moment capacity of the pile*: Perform an analysis by applying the pile head plastic moment capacity and the factored thermal movement (determined in accordance with Article 11.3.3.3.4 of this Manual). The design of the pile shall satisfy the combined axial compression and flexural interaction relationship. The factored axial load shall be the applied factored pile axial load and the factored bending moment shall be the maximum factored bending moment between the two points of zero moment closest to the abutment (from the above analysis).

  The piles must have sufficient ductility at the pile head to accommodate the internal force redistribution resulting form the plastic hinge rotation. An accepted criteria used to verify ductility requirements can be found in the publication titled *Rational Design Approach for Integral Abutment Bridge Piles, Transportation Research Record No.1223 (1989), Abendroth and Greimann.*

In the above designs (except for short piles) in determining the pile axial capacity, the length of the pile shall be the length between the two points of zero moment (closest to the abutment). For short piles with only one point of zero moment, the length shall be from the point of zero moment to the pile tip. In all design cases, the pile shall be considered to be pinned at both ends.

The theoretical depth of fixity will be defined as the depth at which the pile is firmly held by the soil (typically the second point of zero lateral deflection)

11.3.3.3.8 Superstructure Design: For the purpose of the superstructure design and analysis, the superstructure shall be assumed to be simply supported between abutments. However, the superstructure design shall consider the compressive loads developed as a result of the passive earth pressures on the abutment backwall. The superstructure design shall also consider the adverse effects of possible beam fixity at the abutments. The connection of the bridge deck to the abutment diaphragm shall be reinforced to resist the moments caused by superstructure rotation under superimposed dead and live loads. The beneficial effects of end fixity shall not be used to reduce the design moments in the beams.

11.3.3.4 Concrete Deck Pour Sequence

A deck pouring sequence shall be specified such that approximately four feet of the deck end at the abutments and the concrete diaphragms are poured last (after the rest of the deck is poured). This will permit all the dead load girder rotations to take place without any rotational forces being transferred to the piles. The concrete deck pouring sequence shall be indicated on the contract drawings.

11.3.3.5 Expansion Joint, Approach Slab and Sleeper Slabs

For abutment movements in excess of ½ inch, movements shall be accommodated with an appropriately designed expansion joint at the free end of the approach slab where a sleeper slab shall be detailed to support the free end and to accommodate the expansion joint detail. For movements less than or equal to ½ inch a saw and seal detail may be used at the end of the sleeper slab. Details of approach and sleeper slabs are included in the Rhode Island Bridge Design Standard Details.

11.3.3.6 Utilities

In general utilities with rigid pipes through integral abutment should be avoided. If they cannot be avoided, then they must be properly detailed (appropriately sleeved) to accommodate the anticipated superstructure translational and rotational movements.

11.3.3.7 Drainage

The embankment behind the integral abutments shall be positively drained with an adequately designed drainage system consisting of an underdrain system.

11.3.3.8 Wingwalls/Return Walls

Wingwalls or return walls shall be U-shaped walls constructed on independent footings. The interface between the two walls shall be adequately detailed and designed to accommodate the anticipated movement and resulting transverse force effects. All lateral forces from the superstructure shall be transferred to the wing wall or return wall foundations through cheek walls.

11.3.4 Semi-Integral Abutments

11.3.4.1 Definition and Detailing

Semi-Integral abutments are abutments with no deck joints. The superstructure is rigidly connected to the backwall (the concrete portion cast integrally with the beam). Semi-integral abutments are similar to integral abutments; except that the bridge rotational and horizontal movements are allowed and accommodated by bearings located at the bottom of the beams/backwall. Semi-Integral abutments must be supported on either spread footings or multiple rows of piles.

Details of semi-integral abutments are shown on the applicable drawings included with the Rhode Island Bridge Design Standard Details.

11.3.4.2 Limitations on Semi-Integral Abutments

The following limitations shall apply when considering the use of semi-integral abutments:

- The bridge skew angle shall be limited to 30 degrees.
- The total expansion length (distance from a fixed bearing to an expansion joint) shall be limited to 175 feet for steel bridges and 300 feet for concrete bridges.
- The maximum grade between abutments shall be 5%.
- The horizontal bridge alignment shall be straight. The criteria used to establish when a bridge is considered "straight" (that is when the effects of curvature may be ignored) are defined in the referenced Articles of the *AASHTO LRFD Bridge Design Specifications.* *AASHTO 4.6.1.2.4b & 4.6.1.2.4c*
- Only steel beams and concrete (I and box) beams shall be used with integral abutments.
- Girder depths shall be limited to six feet.

Any deviation from the above guidelines must be approved by the Managing Bridge Engineer.

11.3.4.3 Analysis and Design Requirements

Semi-Integral abutments shall be designed to resist all dead loads and live loads as well as all horizontal loads and movements. The minimum requirements for loads and their application, the applicable load factors, and the applicable load combinations shall all be in accordance with the requirements of the *AASHTO LRFD Bridge Design Specifications* and as modified in Section 3 of this Manual.

11.3.4.4 Expansion Joint, Approach Slab and Sleeper Slabs

For abutment movements in excess of ½ inch, movements shall be accommodated with an appropriately designed expansion joint at the free end of the approach slab where a sleeper slab shall be detailed to support the free end and to accommodate the expansion joint detail. For movements less than or equal to ½ inch, a saw and seal detail shall be used at the end of the sleeper slab. Details of approach and sleeper slabs are included in the Rhode Island Bridge Design Standard Details.

11.3.4.5 Utilities

In general utilities with rigid pipes through semi-integral abutments should be avoided. If they can not be avoided, then they must be properly detailed (appropriately sleeved) to accommodate the anticipated superstructure translational and rotational movements.

11.3.4.6 Drainage

The embankment behind the semi- integral abutments shall be positively drained with an adequately designed drainage system consisting of an underdrain system.

11.3.5 Seismic Design

11.3.5.1 Abutments

In accordance with Articles 3.6.10 and 3.6.11.1.1 of this Manual, no specific seismic design considerations need be considered for abutments of single span bridges and non-critical bridges classified under Site Class A, B, C, or D.

The design of abutments for all other critical bridges, and non-critical bridges which are classified under site Class E or F, shall be designed for seismic-induced lateral forces and abutment inertia effects resulting from earthquake-induced motions in accordance with Article 10.2.5.5 of this Manual. In addition, load transfer of seismic forces from restrained bearings shall also be considered.

For free-standing abutments which may displace horizontally without significant restraint, the seismic horizontal acceleration coefficient K_h shall be taken as specified in Table 11.3.5.1-1.

Table 11.3.5.1-1
Horizontal Acceleration Coefficient K_h

Site Class	Critical	Non-Critical
A	0.10	N/A
B	0.13	N/A
C	0.15	N/A
D	0.20	N/A
E	0.32	0.16
F	SEE 3.6.8	

For abutments which are restrained from horizontal displacement (such as when battered piles are used), the seismic horizontal acceleration coefficient K_h specified in Table 11.3.5.1-1 shall be multiplied by 2.

11.3.5.2 Retaining Walls

Retaining walls supporting critical facilities shall be designed for seismic-induced lateral forces and wall inertia forces resulting from earthquake-induced motions in accordance with Article 10.2.5.5 of this Manual. Critical retaining walls are generally those that support vital links that have to remain open for emergency vehicles or for security/defense purposes immediately following an earthquake. The Department shall determine which retaining walls are to be considered critical.

For free-standing walls which may displace horizontally without significant restraint, the seismic horizontal acceleration coefficient K_h shall be taken as specified in Table 11.3.5.1-1. For walls which are restrained from horizontal displacement (such as when battered piles are used), the seismic horizontal acceleration coefficient K_h specified in Table 11.3.5.1-1 shall be multiplied by 2.

11.3.6 Abutment and Wall Details

11.3.6.1 General

Specific standard details relating to abutments and walls are included in the Rhode Island Bridge Design Standard Details.

11.3.6.2 Beam Seats

The width of the bridge seat shall be determined such that it provides adequate room for the bearings and drainage grooves, and such that it meets the seismic criteria of Article 4.5.3 of this Manual. The preferred minimum beam seat width shall be 2'-6". The minimum allowable distance from a bearing pad or masonry plate to the face of the concrete beam seat shall be 2 inches.

11.3.6.3 Wall Face Batter

The back face of walls shall be detailed to be vertical or battered. The minimum thickness at top of walls shall be 15 inches.

11.3.6.4 Wingwalls and Return Walls

For economy, the preferred wingwall (or return wall) treatment is the use of flared wingwalls. However, in certain restrictive situations, walls parallel to roadways may be employed. Economy can be obtained by stepping the wall footings upward, as the fill slopes allow.

11.3.6.5 Minimum Reinforcing

In order to resist the formation of temperature and shrinkage cracks and to provide reinforcement for distribution of loads, all faces of walls and abutments shall be provided with a minimum horizontal and vertical reinforcing of #5 spaced at 18 inches or #4 spaced at 12 inches.

11.3.6.6 Back of Wall Drainage

Except for integral and semi-integral abutments, weepholes shall be used in all walls or abutments 6 feet tall or higher. When unusual drainage conditions exist, perforated drains in back of the walls shall be considered on all walls and abutments. Bridge seat drains shall be 3" diameter pipes, sloping toward the front of the abutment. Back of wall drainage requirements for integral and semi-integral abutments is discussed in Articles 11.3.3.7 and 11.3.4.6 of this Manual.

11.3.6.7 Wall Joints

Contraction joints should be spaced no more that 30 feet apart and expansion joints shall be spaced no more than 90 feet apart.

The location of vertical and horizontal joints, detailed with appropriate shear keys, should be indicated on the contract drawing. No variations in locations and numbers of vertical and horizontal joints proposed by a contractor will be permitted unless approved as part of the shop drawing review process.

11.4 PIERS

11.4.1 Pier Selection

The most commonly used pier types are the solid wall type, the bent type, and the single column "Hammerhead" type as discussed in the referenced *AASHTO LRFD Bridge Design Specifications*, but the intent is not to limit pier selection to those types only. When specific project conditions dictate, the use of other pier types (not commonly used) may be proposed provided that the selection is based on the consideration of the factors indicated in Article 2.3.1 of this Manual.

AASHTO 11.2

Solid Wall type piers are commonly used in river or stream crossings as well as for crash protection adjacent to and in close proximity of railroad tracks. Piers adjacent to the railroads shall be protected with a solid collision wall. The requirements for the pier protection wall shall be in accordance with the *AREMA Manual* Chapter, Part 2, page 8-2-6.

Bent type piers, commonly used for grade-separation structures, shall consist of reinforced concrete, with round or rectangular columns and with rectangular caps cantilevered at the ends. The minimum diameter of the columns shall be 3'–0" and the minimum projection of the cap beyond the column shall be 6" on each side.

Single Column "hammerhead" piers may be used for narrow superstructures. For wider superstructures (where the cantilever cap carries a larger portion of the reaction) single column piers may result in an excessively deep cantilever and therefore yield an uneconomical pier shape.

11.4.2 Pier Details

11.4.2.1 General

Specific standard details pertaining to piers are included in the Rhode Island Bridge Design Standard Details.

11.4.2.2 Joints in Pier Caps

In order to minimize thermal and shrinkage stresses in column-type bents, concrete caps with lengths exceeding 90 feet shall be made discontinuous by providing a two inch open joint. The open joint in the pier cap shall be located at the same location as the longitudinal open joint in the superstructure deck (see Article 9.6.7 of this Manual).

11.5 APPLICATION OF SUPERSTRUCTURE LOADS TO THE SUBSTRUCTURE

11.5.1 Application of Superstructure Dead Loads to Substructures

All superstructure dead loads shall be transmitted directly to the substructures through the bridge bearings.

11.5.2 Application of Live Load Forces to Substructures

Live load vertical reactions obtained directly from the superstructure design are based on maximum conditions assuming a "single beam analysis". These "single beam" vertical reactions should not be used in the design of substructure elements since they make no allowance for a realistic distribution of the live load across the roadway. In the case of substructures with caps, the AASHTO LRFD live load (HL93) should be placed within the design traffic lane(s) in such manner as to maximize the force effect. Beam live load reactions shall be determined based on the assumption that the deck acts as a simple span between beams. In the case of substructures without caps (such as abutments or solid wall type piers) the AASHTO LRFD live load (HL93) total reaction per lane shall be uniformly distributed to the substructure.

The appropriate number of lanes and load combinations, including the effect of any other applicable factors such as multiple presence factor, superelevation or centrifugal force should be considered.

11.5.3 Application of Longitudinal Horizontal Loads

11.5.3.1 Substructures at Fixed Bearings

The longitudinal horizontal loads transmitted from the superstructures to the substructures (and computed in accordance with the *AASHTO LRFD Bridge Design Specifications* and Section 3 of this Manual) shall be assumed to be resisted equally by all fixed supports (i.e. in substructures where the superstructures are restrained against longitudinal translation). No credit shall be taken for the longitudinal horizontal frictional resistance at the expansion supports of adjacent spans (i.e. in substructures where the superstructures are not restrained against longitudinal translation). However, in no case shall the substructures at the fixed bearings be designed for forces less than the largest frictional forces that can develop at the expansion bearings of all contributing adjacent span(s).

Elastomeric bearings shall be assumed to be fixed if they are prevented from distorting.

11.5.3.2 Substructures at Expansion Bearings

Substructures at expansion supports shall be designed for a force no less than the total frictional force which may develop at the expansion bearing(s).

When elastomeric bearings are used, the maximum “frictional” resistance developed shall be assumed to be equivalent to the shear force induced by the maximum anticipated shear deformation. The force induced by the shear deformation of an elastomeric bearing shall be determined in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*. *AASHTO 14.6.3.1*

11.5.4 Application of Transverse Horizontal Loads

The horizontal loads transmitted from the superstructure to the substructures (and computed in accordance with the *AASHTO LRFD Bridge Design Specifications* and Section 3 of this Manual) shall be assumed to be resisted by the fixed supports (i.e. in substructures where the superstructures are prevented against transverse translation by bearings or by other means such as concrete restraints).

For loads which are applied over the length of the superstructure (such as wind on superstructure or wind on live load) the total transverse horizontal load shall be transmitted to the each individual fixed support utilizing load distribution methods appropriate for the actual transverse continuity configuration.

11.5.5 Alternate Method

When the loads computed using the methods discussed in Articles 11.5.1, 11.5.2, and 11.5.3 are determined to be inappropriate for the situation under consideration (over-estimated or under-estimated), an alternate method utilizing sound engineering judgment may be used.

SECTION 12
BURIED STRUCTURES AND TUNNEL LINERS

12.1 SCOPE

Section 12 and the referenced Article of the *AASHTO LRFD Bridge Design Specifications* cover the design of buried structures. These include reinforced concrete precast or cast-in-place arch, box and elliptical structures; metal and reinforced concrete pipe; flexible thermoplastic pipes; structural plate box structures; and steel (corrugated) structures, all of which may be used for the passage of water, vehicular traffic, or pedestrian traffic below earth embankments. No exception is taken with regard to the criteria in the *AASHTO LRFD Bridge Design Specifications*.

AASHTO 5.14.5

SECTION 13
RAILINGS

13.1 GENERAL SCOPE

The minimum requirements for the application of appropriate loads and forces, load factors, and load combinations for railings on new bridges shall be in accordance with the *AASHTO LRFD Bridge Design Specifications* unless otherwise modified in this Section. All exceptions to or deviations from these criteria or policies must meet the approval of the Managing Bridge Engineer.

The criteria for selection of railings for rehabilitation of existing structures shall be in accordance with Section 16 of this Manual.

Any reference(s) in this Section to railings shall encompass all bridge traffic or combination barrier systems.

13.2 TRAFFIC RAILINGS

13.2.1 Test Level Criteria

AASHTO 13.7.2 & Table 13.7.2-1

All railings systems shall meet the full-scale crash-test criteria as established in the NCHRP Report 350. The test levels and the selection criteria are described in the *AASHTO LRFD Bridge Design Specifications*. The following identifies these test levels and general applications:

- Test Level TL-5 is generally considered acceptable for the majority of applications on high speed, high traffic volume interstate highways and roadways where large trucks make up a significant portion of the average daily traffic and when unfavorable site conditions exist. The barrier systems meeting Test Level TL-5 have been designed and tested to the following criteria:

  1,800 lbs. small passenger automobiles at 60 mph, 20º crash angle
  4,500 lbs. pickup trucks at 60 mph, 25º crash angle
  18,000 lbs. single-unit van truck at 60 mph, 15º crash angle
  80,000 lbs. van-type tractor-trailer at 50 mph, 15º crash angle

- Test Level TL-4 is generally considered acceptable for the majority of applications on high speed, high traffic volume interstate highways and roadways with a mixture of trucks and heavy vehicles. The barrier systems meeting Test Level TL-4 have been designed and tested to the following criteria:

  1,800 lbs. small passenger automobiles at 60 mph, 20º crash angle
  4,500 lbs. pickup trucks at 60 mph, 25º crash angle
  18,000 lbs. single-unit van truck at 60 mph, 15º crash angle

- Test Level TL-3 is generally considered acceptable for a wide range of high speed arterial highways with a very low mixture of heavy vehicles and with favorable site conditions. The barrier systems meeting Test Level TL-3 have been designed and tested to the following criteria:

  1,800 lbs. small passenger automobiles at 60 mph, 20º crash angle
  4,500 lbs. pickup trucks at 60 mph, 25º crash angle

- Test Level TL-2 is generally considered acceptable for low-speed local and collector roads with a small number of heavy vehicles and with favorable site conditions. The barrier systems meeting Test Level TL-2 have been designed and tested to the following criteria:

  1,800 lbs small passenger automobiles at 45 mph, 20º crash angle
  4,500 lbs pickup trucks at 45 mph, 25º crash angle

13.2.2 Test Level Selection

13.2.2.1 Test Level Selection Guidelines

- TL-4 shall be the minimum acceptable test level, except as described below.

- Unless otherwise directed by the Department, test level TL-5 shall be the minimum test level for all bridges on interstate highways (Interstate Route 95, Route 195 and Route 295).

  TL-5 should also be considered for other non-interstate, high-volume, high-speed roadway bridges with a high percentage of truck traffic <u>or</u> when unfavorable site conditions may be present. Unfavorable conditions which may warrant the use of a TL-5 system include high occupancy land uses below the bridge, deep water below the bridge, steep profile grades on or approaching the bridge, high curvature along the alignment of the bridge, anticipated excessive number of van-type tractor trailers, or any other set of conditions which, through sound engineering judgment, may justify a higher level of railing resistance. The AASHTO *1989 Guide Specification for Bridge Railings,* as discussed in Article 13.2.2.2, should be used to evaluate an appropriate test level.

- TL-2 or TL-3 may be considered for roadways with ADT less than 500, ADTT less than or equal to 5%, design speed less than 40 mph, and with favorable site conditions (horizontal alignment with no curve and with bridge deck height above ground elevation or water surface less than 28 feet).

13.2.2.2 Test Level Selection Guidelines using the AASHTO Guide Specifications

When required or when directed by the Department, the AASHTO *1989 Guide Specification for Bridge Railings* may be used to assist in determining an appropriate test level. These specifications provide railing selection procedure guidelines based on three performance levels (PL-1, PL-2, and PL-3). The evaluation process considers various design elements consisting of the type of roadway, the bridge vertical and horizontal geometry, the design speed, the adjacent land uses, the truck percentage and the bridge railing offset.

When available, truck percentage should be obtained from weigh-in-motion (WIM) data. Truck percentage shall include all medium to heavy trucks (Class 4 and above). Alternatively, when WIM data is not available, the percentage of truck traffic may be based on actual counts or based on the available truck flow maps. Available digital maps of the latest traffic and truck flow maps may be viewed through the Rhode Island Department of

Transportation engineering website. Printed versions of these maps may also be ordered through the same website.

The crash test requirement equivalency shall be as follows:

AASHTO LRFD	1989 AASHTO GUIDE
TL-5	PL-3
TL-4	PL-2
TL-2	PL-1

13*.2.2.3 Documentation of Recommended Test Level*

Based on the above requirements (including the optional AASHTO *1989 Guide Specification for Bridge Railings a*s discussed in Article 13.2.2.2), the Consultant must evaluate and present recommendations for an appropriate railing test level. The recommendations shall be included in the Bridge Type Study Report. The final decision on the appropriate test level shall be made by the Department.

13.2.3 Railing Details

In general, crash-tested railings shall be detailed in accordance with the Rhode Island Bridge Standard Details. On a case by case basis, such as when aesthetics is a prime design consideration, the Consultant must consider the use of crash-tested railings systems other than those shown in the Rhode Island Bridge Standard Details. The FHWA Bridge Railing website contains drawings or sketches of numerous railings systems meeting Test Levels TL-2, TL-3, TL-4 and TL-5.

Details of all railing systems for historic bridges (or those considered eligible for listing) must be coordinated with the Department and the appropriate Historic Agencies.

When minor detail changes or improvements are made to railing systems which have already been crash-tested, engineering judgment and/or analysis should be used to determine the need for additional crash-testing. This shall also apply to the crash-tested railing details included in the Rhode Island Bridge Standard Details. Any detail changes or improvements to any crash-tested railing must be approved by the Managing Bridge Engineer.

AASHTO C13.7.3.1.1

SECTION 14
JOINTS AND BEARINGS

14.1 GENERAL SCOPE

The minimum requirements for the design of joints and bearings shall be in accordance with the *AASHTO LRFD Bridge Design Specifications* unless otherwise modified or further clarified in this Section.

14.2 MOVEMENTS AND LOADS

14.2.1 General

Deck joints and bearings shall be designed to accommodate movements (including rotation) and to resist loads at the Service, Strength, and/or the Fatigue limit states in accordance with the referenced Article of the *AASHTO LRFD Bridge Design Specifications*.

AASHTO 14.4

For very wide bridges, horizontally curved bridges and bridges with large skews, the impacts of transverse movement and forces shall be carefully considered.

14.2.2 Design Requirements

The minimum thermal movements shall be computed for the extreme temperatures referenced in the *AASHTO LRFD Bridge Design Specifications.*

AASHTO 14.4.2 & 3.12.2

For bridges consisting of concrete deck with concrete or steel beams, the use of Procedure B is preferred. The minimum and maximum temperature ranges for these bridges shall be in accordance with Article 3.8 of this Manual.

14.2.3 Elastomeric and Multi-Rotational Bearings

The maximum unfactored service rotation for elastomeric-type bearings and the maximum strength limit state rotations for multi-rotational bearings must include an allowance of 0.005 radians for uncertainties, regardless of whether a smaller value can be justified.

AASHTO 14.4.2.1 & 14.2.2.2

14.3 BRIDGE JOINTS

14.3.1 Details

Superstructure expansion and fixed joint details are included in the Rhode Island Bridge Design Standard Details.

14.3.2 Selection

In general and when feasible, roadway expansion joints shall be eliminated in accordance with the guidelines provided in Articles 2.4.1 and 11.3.1.1 of this Manual. However, when roadway joints are required the following shall apply:

1. For anticipated movements less than or equal to 3/8 inch, fixed joint details shall be used.

2. For the selection of expansion joints (for anticipated movements in excess of 3/8 inch), the following guidelines are provided:

- The use of asphaltic expansion joints on high traffic volume interstate and expressway bridges is not permitted. Otherwise, asphaltic expansion joints may be used only when anticipated movements are less than 1 inch and when the bridge skew angle is less than 10 degrees. Asphaltic expansion joints may also be used for shorter span secondary roadway bridges (with anticipated movements less than 5/8 inch and with skew angles of up to 30 degrees).

- Strip seal expansion joints may be specified for anticipated movements up to 5 inches.

- Modular Bridge Joint Systems (MBJS) shall be specified for anticipated movements between 5 inches and 28 inches.

14.3.3 Bridge Joint Design

The referenced *AASHTO LRFD Bridge Design Specifications* clearly outlines the various factors to be considered in the design of bridge joints for force effects and movements. Designers should carefully consider these factors, specifically with respect to the impacts due to lateral horizontal displacements and/or bridge rotation on heavily skewed bridges, horizontal curved structures or very wide bridges. The limitation with respect to movement (and shear forces) parallel to the joint resulting from racking must be considered when selecting and sizing the joint system.

AASHTO 14.5.1.2

14.3.4 Bridge Joint Installation

The contract drawings must include a table providing the joint opening for installation temperatures at 15, 30, 45, 60, 75, and 90 degrees.

14.4 REQUIREMENTS FOR BRIDGE BEARINGS

14.4.1 Details

Superstructure bearings details are included in the Rhode Island Bridge Design Standard Details.

14.4.2 Selection

In general, the bearing type selection will be dependent on the type and span length of the structure. The following guidelines, in conjunction with the requirements of the referenced Article of the *AASHTO LRFD Bridge Design Specifications,* must be considered in the selection of the bridge bearing type:

AASHTO 14.6.1

Plain (unreinforced) Elastomeric Pads:

- The use of plain (unreinforced) elastomeric pads is permitted only for short simple span bridges with total vertical reactions of less than 50 kips.

Steel-Reinforced Elastomeric Bearings:

- Steel-reinforced elastomeric bearings may be considered for simple and continuous steel beam or girder bridges; prestressed and cast-in-place concrete beam or girder bridges; and horizontally curved girder bridges, all with total vertical reactions of up to approximately 250 to 300 kips. For reactions in excess of 300 kips, steel-reinforced elastomeric bearings may be considered when it is determined that they are an economically feasible alternative to multi-rotational bearings and provided that the bearing assemblies are not excessively large.

- Only round elastomeric bearings may be specified for horizontally curved girder bridges and bridges with skew angles greater than 30 degrees.

Multi-Rotational Bearings may be specified:

- When steel-reinforced elastomeric bearings are not feasible or cost effective
- As an alternate to steel-reinforced elastomeric bearings.
- When high load capacity bearings (in excess of approximately 250 to 300 kip reactions) are required.
- For horizontally curved girder bridges or bridges with skew angles of 30 degrees or more.

14.4.3 Horizontal Forces and Movements

14.4.3.1 General

Bearings may be detailed to provide restraint in all direction (fixed bearings), to provide restraint to control the direction of translation (guided bearings), or to provide no restraint and to permit translation in all directions (non-guided bearings). For guided bearings, consideration shall be given to the potential for unequal participation from each bearing due to construction and detailing tolerances, misalignment and bridge skew. For bridge structures with skew angles in excess of 30 degrees or for horizontally curved girders, no more than two fixed or guided bearings shall be assumed to resist the sum of the horizontal loads. As an alternate to specifying bearings requiring a larger horizontal restraint capacity, the use of an independent restraint system (such as concrete cheek walls or concrete restraints cast on top of beam seats) in conjunction with the bearing restraint system is preferred.

Except as stated above, a minimum of two fixed or guided bearings shall be specified per girder support line to provide for redundancy.

On very wide bridges where significant transverse movement may occur, consideration must be given to not fixing or guiding all the bearings along a bearing line. The bearings with guided restraints should be located only at the interior girders or beams where large transverse movements do not occur.

Bridges with skew angles greater than approximately 40 degrees may require a refined analysis to better estimate the anticipated thermal movements and rotations.

The maximum horizontal force transfer for non-guided bearings shall be those induced by sliding friction (multi-rotational bearings) or by shear deformation (elastomeric bearings) computed in accordance with the procedure outlined in the referenced Article of the *AASHTO LRFD Bridge Design Specifications.* *AASHTO 14.6.3.1*

Application of horizontal loads to substructures shall be in accordance with Article 11.5.2 of this Manual.

14.4.3.2 Additional Requirements for Horizontally Curved Girder Bridges

For horizontally curved girder bridges, the proposed bridge support bearings shall provide vertical and horizontal stability and shall accommodate the anticipated thermal movements and rotations under service conditions as well as during the girder erection and construction phase. Bearings for horizontally curved girder bridges must allow rotation in all directions.

The expansion and contraction movement for each girder or girder system can be assumed to occur along the chord line, running from the fixed bearing (or point of horizontal restraint) to the bearing under consideration. Guided bearings shall be oriented to accommodate movement along this chord line. An independent (back-up) horizontal restraint system, comprised of shear blocks or cheek walls cast on top of the beam seats at each substructure should also be provided.

14.4.4 Bridge Bearing Design and Installation

14.4.4.1 Elastomeric Bearings

Elastomeric bearings shall be designed in accordance with Article 14.5.3 of this Manual and shall be detailed on the contract drawings.

14.4.4.2 Multi-Rotational Bearings

The design and detailing of the multi-rotational bearings shall be the responsibility of the manufacturer. The schematic details of pot and disc bearings are included in the Rhode Island Bridge Design Standard Details and should be shown on the contract drawings. The design and detailing of the masonry plates, anchor bolts, and sole plates shall be the responsibility of the Designer. The Designer shall also closely coordinate with at least two manufacturers of multi-rotational bearing devices during the preliminary and final design stage to assure proper bearing fit. The contract drawings must include a table providing the required geometric and loading requirements. As a minimum for the controlling load combination for each limit state, the following information must be included:

1. The type of bearing (fixed, guided or non-guided).
2. The maximum and minimum dead load and live load vertical loads.
3. The maximum and minimum horizontal transverse and longitudinal loads.
4. The total bearing heights assumed in establishing beam seat elevations.
5. The maximum design rotation (including tolerances) and the horizontal displacement requirements.

The contract drawings must also include a table, providing the bearing installation setting at temperatures of 15, 30, 45, 60, 75, and 90 degrees. The table should indicate the position of the top sole plate relative to the bottom base masonry plate.

14.4.5 Seismic Provisions

All bearings shall be designed and detailed to accommodate the effects of earthquakes in accordance with the provisions of the *AASHTO LRFD Bridge Design Specifications* and Articles 3.6.10, 3.6.11 and 3.6.12 of this Manual. For the specific requirements for the design and detailing of seismic isolation bearings, refer to Article of this 3.6.15 of this Manual.

14.5 SPECIAL DESIGN PROVISIONS FOR BRIDGE BEARINGS

14.5.1 Metal Rocker and Roller-Type Bearings

The use of rocker type or roller-type bearings is not permitted.

14.5.2 PTFE Sliding Surfaces

In the absence of specific information regarding the frictional coefficients, the service limit state design coefficient of friction for PTFE sliding surfaces shall be in accordance with the table provided in the referenced Article of the *AASHTO LRFD Bridge Design Specifications*.

AASHTO 14.7.2.5

14.5.3 Elastomeric Bearings

The design of steel-reinforced elastomeric bearings shall be in accordance with Method A of the referenced Article of the *AASHTO LRFD Bridge Design Specifications.* The use of Method B may be considered on large projects when cost savings can justify the additional testing and quality control which is required for this method. Use of Method B must be approved by the Managing Bridge Engineer.

AASHTO 14.7.6

Some typical details of steel-reinforced elastomeric bearings are included in the Rhode Island Bridge Design Standard Details.

SECTION 15
STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES AND TRAFFIC SIGNALS

15.1 GENERAL SCOPE

15.1.1 Applicable Specifications

All structural supports herein listed shall be designed and detailed in accordance with the latest edition and interims to the *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*, and with the applicable sections of the Rhode Island Standard Specifications for Road and Bridge Construction, except as modified herein. Such supports shall include:

- Frames mounted on vehicular bridges.
- Independent supports founded on the ground, including both overhead bridge structures with vertical supporting systems at each end, and cantilevered structures.
- Span pole and wire systems for traffic signals.
- Poles for traffic lighting and luminaires, including top-mounted, high-level, and single or twin davit types.
- Poles for traffic cameras

15.1.2 Fatigue Provisions of the Specifications

The fatigue provisions of the AASHTO Specifications shall be applied to cantilevered structures, as well as structures supporting variable message signs, dynamic message signing overhead span structures, and high level light structures.

15.1.3 Component Requirements

- All cantilevered sign structures shall consist of dual arms without planar truss-work. The arms shall have a minimum 0.14 inch per foot taper.
- All overhead bridge support structures shall consist of single horizontal steel members supported by single steel columns at either side.
- The support structure for dynamic message signs shall be steel box (“quadric-chord) trusses.
- Vibration mitigation devices will not be allowed.

15.2 DESIGN

15.2.1 Fatigue Category

The Basic Wind Speed, V, used in the determination of the design wind pressure shall be 130 mph.

All sign and luminaire structures on interstate or limited access type facilities must comply with fatigue category 1 requirements, including galloping, vortex shedding (if applicable), natural wind gusts, and truck-induced gusts. The truck induced loading shall be based on a 65 mph velocity.

All sign, traffic signal, and luminaire structures on all other roadways must comply with fatigue category 2 requirements, including galloping, vortex shedding (if applicable), natural wind gusts, and truck-induced gusts. The truck induced loading shall be based on a 30 mph velocity.

15.2.2 Alternate Wind Provisions

The use of Appendix C of the AASHTO Specifications as an alternative wind load determination is not allowed.

15.2.3 Truck-Induced Fatigue Exemption

Traffic signal structures on roadways with limited truck traffic may be declared exempt from truck-induced fatigue loading, upon approval of the Department on a case-by-case basis.

15.2.4 Anchor Bolts

The design of anchor bolts shall be based on a ductile steel failure prior to any sudden brittle failure of the concrete.

15.2.5 Offset Clearance

When the clearance between the bottom of the leveling nuts and the top of the concrete is equal to or greater than one bolt diameter, bending stresses in the anchor bolts shall be considered in the design.

15.3 CONSTRUCTION DETAILING

The following notes shall be accounted for in all designs and shall be included on all plans and/or shop drawings:

- Pretensioning of all anchor nuts is required and shall be accomplished by tightening to 1/6th turn beyond the snug-tight position.

- The maximum clearance between the bottom of the leveling nuts and the top of the concrete is critical and shall not exceed the amount specified on this drawing.

- The use of grout under base plates shall generally not be permitted. If specific conditions warrant its use, the grout shall not be considered load-carrying, and the loads shall be considered to be directly supported by the anchor bolts. Adequate drainage shall be provided.

SECTION 16
EXISTING BRIDGE EVALUATION AND REHABILITATION

THIS SECTION IS UNDER DEVELOPMENT