July 15, 2016

To: Users of the Bridge Design Manual

From: Tim Keller, Administrator, Office of Structural Engineering

By: Sean Meddles, Assistant Administrator, Office of Structural Engineering

Re: 2016 Third Quarter Revisions

Revisions have been made to the ODOT Bridge Design Manual, July 2007. These revisions shall be implemented on all Department projects that begin Stage 2 plan development date after July 15, 2016. Implementation of some or all of these revisions for projects further along the development process should be considered on a project-by-project basis.

This package contains the revised pages. The revised pages have been designed to replace the corresponding pages in the book and are numbered accordingly. Revisions, additions, and deletions are marked in the revised pages by the use of one vertical line in the right margin. The header of the revised pages is dated accordingly.

To keep your Manual correct and up-to-date, please replace the appropriate pages in the book with the pages in this package.

To ensure proper printing, make sure your printer is set to print in the 2-sided mode.

The July 2007 edition of the Bridge Design Manual may be downloaded at no cost using the following link:

http://www.dot.state.oh.us/Divisions/Engineering/Structures/Pages/default.aspx

Attached is a brief description of each revision.
## Summary of Revisions to the July 2007 ODOT BDM

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<td>200 TOC</td>
<td>2-i through 2-ii</td>
<td>Update to BDM Section 200 Table of Contents</td>
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<tr>
<td>201.1</td>
<td>2-1</td>
<td>Removed old references to the Project Development Process</td>
</tr>
<tr>
<td>201.2</td>
<td>2-1</td>
<td>Removed old references to the Project Development Process</td>
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<tr>
<td>202.2.3.1</td>
<td>2-11 through 2-11.2</td>
<td>The section on spread footings was revised to clarify the proper location of the bottom footing elevation for structures founded both inside and outside of the 500-yr flood plain.</td>
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<tr>
<td>203.5</td>
<td>2-24.1 through 2-24.2</td>
<td>Clarification was provided for situations where project sale may occur before the waterway permits are issued.</td>
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<tr>
<td>204.1</td>
<td>2-24.2</td>
<td>Clarification was provided to distinguish spread footings from pile cap and drilled shaft footings.</td>
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<tr>
<td>204.3</td>
<td>2-25</td>
<td>Reference to BDM Section 202.2.3.1 was added for spread footing requirements.</td>
</tr>
<tr>
<td>205.2</td>
<td>2-30</td>
<td>Reference to BDM Section 202.2.3.1 was added for spread footing requirements.</td>
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<tr>
<td>208.1</td>
<td>2-37</td>
<td>A requirement to provide a General Note allowing contractors to design alternate temporary shoring has been removed. Alternate temporary shoring designs will require approval of a Value Engineering Change Proposal in accordance with C&amp;MS 105.19.</td>
</tr>
<tr>
<td>Figure 202.2.3.1.a-1</td>
<td></td>
<td>This new BDM Figure illustrates the proper embedment depth of spread footings outside the limits of the 500-yr flood plain.</td>
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<tr>
<td>300 TOC</td>
<td>3-i through 3-vi</td>
<td>Update to BDM Section 300 Table of Contents</td>
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<tr>
<td>301.2</td>
<td>3-2</td>
<td>Removed old references to the Project Development Process</td>
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<tr>
<td>301.4.4</td>
<td>3-3</td>
<td>This revision updates ODOT policy for seismic design in accordance with the AASHTO LRFD Bridge Design Specifications, 7th Edition.</td>
</tr>
<tr>
<td>301.4.4.1</td>
<td>3-3 through 3-3.1</td>
<td>This new section of the BDM addresses the ODOT requirements for Seismic Performance Zone 1 (SPZ 1).</td>
</tr>
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<tr>
<td>301.4.4.1.a</td>
<td>3-3.1</td>
<td>This new section of the BDM addresses the minimum support length requirements for SPZ 1.</td>
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<tr>
<td>301.4.4.1.b</td>
<td>3-3.1 Through 3-3.2</td>
<td>This new section of the BDM addresses the Horizontal Connection Force requirements for SPZ 1.</td>
</tr>
<tr>
<td>301.4.4.1.c</td>
<td>3-3.2</td>
<td>This new section of the BDM addresses the requirements for bearings in SPZ 1.</td>
</tr>
<tr>
<td>301.4.4.2</td>
<td>3-3.2</td>
<td>This new section of the BDM addresses requirements for existing structures located in SPZ 1.</td>
</tr>
<tr>
<td>301.4.4.2.a</td>
<td>3-3.2 through 3-3.3</td>
<td>This new section of the BDM addresses superstructure requirements for existing structures located in SPZ 1.</td>
</tr>
<tr>
<td>301.4.4.2.b</td>
<td>3-3.3 through 3-3.4</td>
<td>This new section of the BDM addresses substructure requirements for existing structures located in SPZ 1.</td>
</tr>
<tr>
<td>302.2.2</td>
<td>3-13.1</td>
<td>A provision to place shrinkage and temperature reinforcement in the underside of deck overhangs at the minimum concrete cover has been added. Also, references to standard bridge railings have been updated.</td>
</tr>
<tr>
<td>302.4.2.3</td>
<td>3-28</td>
<td>The requirements for detailing crossframe spacings have been revised. Allowing the steel detailer to locate the crossframes has been eliminated. Also, a requirement that crossframe locations be checked to prevent conflicts with bolted splices has been added.</td>
</tr>
<tr>
<td>303.2.2.5</td>
<td>3-52</td>
<td>Reference to BDM Section 202.2.3.1 was added for spread footing requirements.</td>
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<tr>
<td>303.3.2.1</td>
<td>3-58</td>
<td>New Sections for Pier Caps, 303.3.2.1.a, and Pier Columns, 303.3.2.1.b have been added.</td>
</tr>
<tr>
<td>303.3.2.1.a</td>
<td>3-58</td>
<td>This new section of the BDM includes information from the previous BDM Section 303.3.2.1 and provides a new reference for minimum bridge seat widths.</td>
</tr>
<tr>
<td>303.3.2.1.b</td>
<td>3-58</td>
<td>This new section of the BDM enforces the transverse steel requirements of LRFD 5.7.4 &amp; 5.10.11 which were relaxed in the previous editions of the BDM.</td>
</tr>
<tr>
<td>303.3.2.4</td>
<td>3-59</td>
<td>References to BDM Section 202.2.3.1 was added for spread footing requirements.</td>
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<tr>
<td>303.3.2.5</td>
<td>3-60</td>
<td>Reference to BDM Section 301.4.4.1.b has been added to check capped pile pier connections for the seismic horizontal connection force.</td>
</tr>
<tr>
<td>303.3.2.8</td>
<td>3-61</td>
<td>Reference to BDM Section 301.4.4.1.a for minimum seat width requirements as well as reference to the transverse reinforcement requirements in <em>LRFD 5.7.4, 5.10.6.3</em> and <em>5.10.11.4.1</em> have been added.</td>
</tr>
<tr>
<td>303.4.1.1</td>
<td>3-62</td>
<td>This section has been revised to be consistent with BDM Section 202.2.3.1</td>
</tr>
<tr>
<td>305.1</td>
<td>3-77 through 3-78</td>
<td>The purpose for vandal fencing has been clarified.</td>
</tr>
<tr>
<td>305.2</td>
<td>3-78</td>
<td>The scoring system used to determine the need for vandal fencing has been eliminated. A new criteria has been provided. An exemption form located in the listing of Design Data Sheets has also been referenced.</td>
</tr>
<tr>
<td>305.3</td>
<td>3-78 through 3-79</td>
<td>Required locations for fence terminations have been provided.</td>
</tr>
<tr>
<td>305.4</td>
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<td>Aesthetic considerations for fencing have been provided.</td>
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<tr>
<td>305.5.1</td>
<td>3-80</td>
<td>Reference to <em>LRFD 15.8.2</em> has been provided for design loadings.</td>
</tr>
<tr>
<td>Figure 301.4.4.1-1</td>
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<td>Figure 301.4.4.1.a-1</td>
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<td>Figure 301.4.4.1-1-1</td>
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</tbody>
</table>

<p>| 602.3       | 6-4            | The concrete class for drilled shaft concrete has been revised to QC5. |
| 1000 TOC    | 10-i through 10-iv | Update to BDM Section 1000 Table of Contents |
| S3.10.2.1   | 10-6           | This new section provides guidance for locating seismologic data. |
| S3.10.2.2   | 10-6           | This new section provides information on the Site Specific Procedure. |</p>
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<td>10-6</td>
<td>This new section provides information on the Site Class Definitions.</td>
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<td>S3.10.3.2</td>
<td>10-7</td>
<td>This new section provides information on Site Factors.</td>
</tr>
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<td>S3.10.6</td>
<td>10-7</td>
<td>This new section provides information on Seismic Performance Zones.</td>
</tr>
<tr>
<td>S3.10.9.2</td>
<td>10-7</td>
<td>This new section provides information on Seismic Performance Zone 1.</td>
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<tr>
<td>S3.10.9.5</td>
<td>10-7</td>
<td>This new section provides information on Longitudinal Restrainers.</td>
</tr>
<tr>
<td>S4.7.4.1</td>
<td>10-10</td>
<td>This new section provides information on general seismic analysis.</td>
</tr>
<tr>
<td>S4.7.4.4</td>
<td>10-10</td>
<td>This new section references the minimum support length requirements in BDM Section 301.4.1.a.</td>
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<td>S5.7.4.6</td>
<td>10-11</td>
<td>This new section provides a mathematical equation for the ratio of spiral reinforcement in columns.</td>
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<tr>
<td>S5.10.2.2</td>
<td>10-12</td>
<td>This new section provides a definition for plastic hinge zones in columns.</td>
</tr>
<tr>
<td>S6.7.4.1</td>
<td>10-16</td>
<td>This section of the BDM which prohibited skewed crossframes at intermediate supports has been removed.</td>
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<tr>
<td>S6.7.4.2</td>
<td>10-16</td>
<td>This section of the BDM which required intermediate crossframes to be oriented perpendicular to main steel members regardless of skew angle has been removed.</td>
</tr>
<tr>
<td>S14.4.1</td>
<td>10-29</td>
<td>This new section of the BDM allows irreparable damage to bearings resulting from seismic loads.</td>
</tr>
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<td>S14.6.3.1</td>
<td>10-29</td>
<td>This new section of the BDM provides information on horizontal bearing forces.</td>
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<td>This new section of the BDM provides information on the application of seismic loads on bearings.</td>
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<td>This new section of the BDM provides seismic design information related to bearings.</td>
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<tr>
<td>SI4.7.5.3.7</td>
<td>10-30</td>
<td>This new section of the BDM provides design information related to seismic loads on bearing anchorages.</td>
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SECTION 200 - PRELIMINARY DESIGN

201  STRUCTURE TYPE STUDY

201.1  GENERAL

The project site should be studied in detail and evaluated to determine the best structure alternative. A site visit should be made. In many cases, it can be readily determined whether a particular bridge or culvert should be chosen for a particular site. If a bridge is the most appropriate structure for a particular site, then the Structure Type Study needs to be performed to determine the appropriate bridge type.

201.2  STRUCTURE TYPE STUDY SUBMISSION REQUIREMENTS

A Structure Type Study submission should include the following:

A. Profile for each bridge alternative..............................................................................Section 201.2.1
B. Preliminary Structure Site Plan (for preferred bridge alternative)......................Section 201.2.2
C. Hydrology & Hydraulics (H&H) Report .................................................................Section 201.2.3
D. Narrative of Bridge Alternatives.............................................................................Section 201.2.4
E. Cost Analysis ..........................................................................................................Section 201.2.5
F. Foundation Recommendation .................................................................................Section 201.2.6
G. Preliminary Maintenance of Traffic Plan ...............................................................Section 201.2.7

The Structure Type Study shall be included in the review submission made directly to the District Office. A concurrent review submission shall be made to the Office of Structural Engineering if the proposed structure type contains non-standard bridge railing types, non-redundant designs, or fracture critical designs. The Office of Structural Engineering will forward review comments for these items to the responsible District Office.
201.2.1 PROFILE FOR EACH BRIDGE ALTERNATIVE

The profile for each bridge alternative considered shall generally be drawn to a scale of 1”=20’ [1 to 200] and shall generally be taken along the proposed centerline of survey for the full length of the bridge. The profiles shall include: the existing and proposed profile grade lines; existing ground line; the cross-section of channel; an outline of the structure; highest known high water mark; normal water elevation; Ordinary High Water Mark (OHWM); flow line elevation (thalweg); design and 100 year water surface elevations (WSE); overtopping flood elevation and frequency; existing and proposed profile grade elevations at 25 ft [10 m] increments; and minimum and required vertical and horizontal clearances. Note: normal water elevation is the water elevation in the stream which has not been affected by a recent heavy rain runoff and could be found in the stream most of the year. Refer to BDM Section 203.4 for OHWM definition. Carry WSE in a FEMA Zone out to two decimal places.

201.2.2 PRELIMINARY STRUCTURE SITE PLAN

The Site Plan scale generally should be 1” = 20’ [1 to 200]. For some cases to get the entire bridge on one sheet a smaller scale may be provided, if all details can be clearly shown. For bridges where the 1” = 20’ [1 to 200] scale is too small to clearly show the Site Plan details, a 1” = 10’ [1 to 100] scale may be considered. The following general information should be shown on the Preliminary Structure Site Plan:

A. The plan view should show the existing structures (use dashed lines); contours at 2 foot [0.5 meter] intervals showing the existing surface of the ground (for steep slopes contours at 5 foot [2.0 meter] or greater intervals may be used); existing utility lines and their disposition; proposed structure; proposed temporary bridge; proposed channel improvements; a north arrow; and other pertinent features concerning the existing topography and proposed work in an assembled form.

In case of a highway grade separation or a highway-railway grade separation, the required minimum and actual minimum horizontal and vertical clearances and their locations shall be shown in the plan and profile views.

For a bridge over a railway, the vertical clearance shall be measured from a point level with the top of the highest rail and 6 feet [2 meters] from the centerline of those tracks, or greater if specified by the individual railroad. Reference shall be made to Chapter 15, Section 1.2.6(a), AREMA Specifications for increased lateral clearances required when tracks are on a horizontal curve.

B. A profile as described in Section 201.2.1. The profile scale shall be the same as the plan view.

C. Horizontal and vertical curve data.

D. Size of drainage area. The elevation, discharge and stream velocity through the structure and the backwater elevation for the 100-year frequency base flood, the design year flood and if necessary the overtopping flood. Label discharge as “FIS” when taken from a FEMA Flood Insurance Study. The clearance from the lowest elevation of the bottom of the superstructure to the design year water surface elevation (freeboard) should be provided.
unless the footings can be founded on bedrock. Where the scour evaluation has identified a potential problem, the probable scour depths, calculated in accordance with the L&D Vol. 2, Section 1008.10, should be considered in the design of the substructures; the location of the bottom of footings; the minimum tip elevations for piles and drilled shafts; and the factored side resistance of piles and drilled shafts. See BDM Section 202.2.3.2.h for more information.

Where downdrag has been identified as a potential contributor to the total factored load, the estimated downdrag load shall be included in the report. See BDM Section 202.2.3.2.c for more information.

The Foundation Report for MSE wall supported abutments shall include calculations for external stability (LRFD 11.10.5) and settlement. The report shall also consider the effect of settlement and include all construction constraints, such as soil improvement methods, that may be required.

Specific design considerations for each foundation type are presented in the following sections.

202.2.3.1 SPREAD FOOTINGS

Spread footings shall be designed in accordance with the AASHTO LRFD Bridge Design Specifications, Section 10. The footing dimensions shall be optimized for the controlling Limit State.

The use of spread footings shall be based on an assessment of the following: design loads; depth of suitable bearing materials; ease of construction; effects of flooding and scour analysis; liquefaction and swelling potential of the soils; frost depth; and amount of predicted settlement versus tolerable structure movement. If placing spread footings on soil, consider the effects of primary and secondary settlements on long-term maintenance costs of the structure and appurtenances and the effects on ride quality.

Elevations for the bottom of the footing shall be shown on the Final Structure Site Plan. The size of the footing, predicted settlement, maximum Strength and Service Limit State bearing pressures, eccentric load limitations, factored sliding resistance, factored bearing resistance and overall stability shall be provided for review with the Foundation Report. Adjust the footing size, the amount of predicted settlement and the factored bearing resistance during detail design as the design loads for Service, Strength and Extreme Event Limit State are refined.

The Department will not permit the use of spread footing supported structures on MSE as noted in BDM Section 204.4.

All spread footings not founded on bedrock shall have elevation reference monuments. These monuments allow for the measurement of footing elevations during and after construction for the
purpose of documenting the performance of the spread footings, both short term and long term. See Section 600 for notes and additional guidance.

202.2.3.1.a SPREAD FOOTING ELEVATIONS FOR FOUNDATIONS LOCATED OUTSIDE THE LIMITS OF THE 500-YR FLOOD PLAIN

The following requirements apply to spread footings located outside of the plan view limits of the 500-yr flood plain:

A. The bottom of footings founded on rock, shall be keyed at least 3-in into rock.
B. The top of footings founded on soil shall be embedded at least 1-ft from the nearest soil surface.
C. The bottom of footings founded on soil shall be embedded at least 5-ft from the nearest soil surface.

202.2.3.1.b SPREAD FOOTING ELEVATIONS FOR FOUNDATIONS INSIDE THE LIMITS OF THE 500-YR FLOOD PLAIN

The following requirements apply to spread footings located inside of the plan view limits of the 500-yr flood plain:

A. Locate the footings for 3-Sided Flat Top and Arch Section Culvert structure types (C&MS 706.051 and 706.052) according to L&D Vol. 2, Section 1008.9.
B. Except as noted in BDM Section 202.2.3.1.b.(A), locate the bottom of footings directly on scour resistant rock. If footings require lateral constraint, provide drilled and grouted steel anchors. Scour resistant rock shall have the following properties to an elevation at least 1-ft below the Thalweg:
   1. Unconfined compressive strength $\geq$ 2.5 ksi
   2. Remain intact when immersed in water (e.g. insoluble)
   3. Unit weight $\geq$ 0.15 k/ft$^3$
   4. Joint or bedding plane spacing that define large blocks (> 4-ft)
C. Except as noted in BDM Section 202.2.3.1.b.(A), locate the bottom of footings founded on non-scour resistant rock at least 7-ft below the Thalweg elevation and the top of the footing at least 1-ft below the Thalweg elevation. If historical evidence of scour deeper than 7-ft in the rock foundation material at or near the footing location is available, spread footings foundations shall not be used.
D. Except as noted in BDM Section 202.2.3.1.b.(A), footings founded on soil are prohibited. Substructures shall be founded on piling extending at least 15-ft below the Thalweg elevation or on drilled shafts.
202.2.3.2 PILE FOUNDATIONS

Pile foundations should be considered when spread footing foundations are prohibited or are not feasible.

The type, size and estimated length of the piles for each substructure unit shall be shown on the Final Structure Site Plan. The estimated length for piling shall be measured from the pile tip to
the cutoff elevation in the pile cap and shall be rounded up to the nearest five feet [one meter]. To
determine the estimated length for different pile types, refer to BDM Section 202.2.3.2.a and
202.2.3.2.b. The estimated length may need to be adjusted during detail design as the design loads
for the Service, Strength and Extreme Event Limit States are refined.

202.2.3.2.a      PILES DRIVEN TO REFUSAL ON BEDROCK

When piles are driven to refusal on the bedrock, the plans should specify steel ‘H’ piles.

Refusal is met during driving when the pile penetration is an inch or less after receiving at least 20
blows from the pile hammer. When estimating pile length, the depth to refusal shall be assumed
as the elevation on the nearest soil boring where the rock core begins.

The total factored load ($\sum \eta_i \gamma_i Q_i$) for each pile shall be provided in the structure general notes.
A sample note is provided in BDM Section 600. The plan specified value for total factored load
shall be the factored load for the highest loaded pile at each substructure unit.

The factored resistance for piles driven to refusal on bedrock is typically governed by structural
resistance. The total factored load for any single pile shall not exceed the maximum factored
structural resistance ($R_{R_{\text{max}}}$). The commonly used H-pile sizes and the maximum factored
structural resistance ($R_{R_{\text{max}}}$) allowed for each are listed below:

<table>
<thead>
<tr>
<th>H Pile Size</th>
<th>$R_{R_{\text{max}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HP10X42</td>
<td>310 kips</td>
</tr>
<tr>
<td>HP12X53</td>
<td>380 kips</td>
</tr>
<tr>
<td>HP14X73</td>
<td>530 kips</td>
</tr>
<tr>
<td>HP250X62</td>
<td>1380 kN</td>
</tr>
<tr>
<td>HP310X79</td>
<td>1690 kN</td>
</tr>
<tr>
<td>HP360X108</td>
<td>2360 kN</td>
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</table>

The values listed above for the maximum factored structural resistance assume: an axially loaded
pile with negligible moment; no appreciable loss of section due to deterioration throughout the life
of the structure; a steel yield strength of 50 ksi; a structural resistance factor for H-piles subject to
damage due to severe driving conditions ($LRFD$ 6.5.4.2: $\phi_c = 0.50$); and a pile fully braced along
its length. These values should not be used for piles that are subjected to bending moments or are
not supported by soil for their entire length. Examples include piles for capped pile piers and piles
in soils subject to scour.

For piers, other than capped pile piers, the preferred H-pile size is HP10X42 [HP250X62]. For
information regarding piles for capped pile piers, refer to BDM Section 303.3.2.5.

In order to protect the tips of the steel “H” piling, steel pile points shall be used when piles are
driven to refusal onto strong bedrock. When the depth of overburden is more than 50 feet [15
meters] and the soils are cohesive in nature, piles driven to strong bedrock generally should not
B. Drawings and/or mapping submitted with a permit application
C. Specialized conditions associated with the waterway permits

The designer and the project manager shall confirm that the bridge design plans meet the requirements in the project waterway SPP (e.g. Sections 404 and 401 conditions, and infrequently Sections 9 and 10) and shall ensure the project waterway SPP is submitted with the Final Plan Package.

203.5 TEMPORARY ACCESS FILLS

A Temporary Access Fill (TAF) is a fill or structure that allows a contractor access to work on roads or bridges located within bodies of water. Examples of TAF’s include: cofferdams; temporary structures for maintaining traffic; causeways and workpads; and demolition debris. The placement of all TAF’s in “Waters of the United States” must be performed in accordance with the special provisions for waterway permits.

A contractor’s means and methods of construction will dictate the TAF required for a project. However, the Department must estimate the potential impacts to “Waters of the United States” during project development to enable all permits to be in-place during contract letting. For most projects, the waterway permits are in place prior to sale. There may be instances where unforeseen delays dictate that the waterway permits will not be acquired until after sale and/or award. In those instances it is imperative that the waterway permits be obtained prior to the contractor beginning any work within any Waters of the United States. Furthermore, it is incumbent upon the Department that these permits provide all bidding contractors the ability to construct the project without resulting in expensive delays, change orders or fines. To that end, the Department partnered with the Ohio Contractor’s Association to develop the following guidance to estimate the size of TAF’s:

A. The TAF shall provide access to all piers located within the Ordinary High Water Mark (OHWM) of the waterway from at least one bank of the waterway. Access may be provided by construction staging of the TAF. When considering the constructability of staged TAF’s, typical superstructure erection plans for lifting lengths of 50'-ft or more require two cranes. Unless the access for member delivery is from an adjacent structure, the TAF must provide access to each end of the lift from one bank. In the case of staging, the permit application shall reflect the construction stage that impacts the largest area of the waterway.

B. The TAF shall be located directly beneath the superstructure. The surface width of the TAF shall be equal to the out-to-out width of the superstructure plus 50’-0” outboard on one side of the structure and 20’-0” outboard on the other side of the structure.

C. The TAF shall extend at least 40’-0” beyond the furthest pier accessed by the TAF.

D. Side slopes of the TAF shall be no steeper than 1.5:1 (H:V).

E. The top surface of the TAF shall be located 1’-0” above the OHWM.
F. The TAF shall be designed to maintain a flow equal to two times the highest average monthly flow (i.e. the largest of Q1, Q2, Q3, ...Q12), as reported by the USGS web based application StreamStats (see L&D Vol. 2), such that no rise in the backwater above OHWM is permitted. This information is intended for permit application purposes only and should not be included in the project plan set. However, to assist the OES-WPU in the determination process, Designers should use the guidance above to develop a plan view and cross-section and determine waterway impacts of a TAF. An example plan view and cross-section are shown in Figure 203.5-1. These details should be provided to the DEC along with a completed copy of the checklist shown in Figure 203.5-2. The minimum flow to be maintained during construction should be calculated according to item F above. Designers will need to estimate whether this flow can be maintained through conduits or if open channels will be required.

204 SUBSTRUCTURE INFORMATION

204.1 FOOTING ELEVATIONS

Substructure footing elevations shall be shown on the Final Structure Site Plan. Refer to BDM 202.2.3.1 for Spread Footing elevation requirements. The top of footing (e.g. pile and drilled shaft caps) shall be a minimum of one foot below the finished ground line and shall be at least one foot below the bottom of any adjacent drainage ditch. The bottom of footing (e.g. pile and drilled shaft caps) shall not be less than four feet below and measured normal to the finished groundline.
aesthetically pleasing structure.

The spill-thru slope should intersect the face of abutment a minimum of one foot [300 mm], or as specified in a standard bridge drawing, below the bridge seat for stringer type bridges. For concrete slab and prestressed box beam bridges this distance should be 1'-6" [450 mm].

204.3 ABUTMENT TYPES

Preference should be given to the use of spill-thru type abutments. Generally for stub abutments on piling or drilled shafts the shortest distance from the surface of the embankment to the bottom of the toe of the footing should be at least 4'-0". For stub abutments on spread footing on soil, refer to BDM Section 202.2.3.1 for footing elevations. For any type of abutment, integral design shall be used where possible, see Section 205.8 for additional information.

Wall type abutments should be used only where site conditions dictate their use.

204.4 ABUTMENTS SUPPORTED ON MSE WALLS

When conditions are appropriate, the use of MSE walls to shorten bridge spans and eliminate embankment slopes is acceptable. MSE wall supported abutments shall be supported on piling regardless of the proximity of bedrock to the MSE wall foundation. The Department will not permit the use of spread footing supported abutments on MSE walls because of their susceptibility to loss of bearing caused by erosion during the service life of the structure. Piles require a minimum 15-foot embedment below the MSE wall. If rock exists within the minimum embedment depth, the piles shall be placed in pre-bored holes that extend a minimum of 5-ft into bedrock. The pre-bored holes shall be backfilled with Class QC Misc. concrete up to the top of the leveling pad elevation after pile installation.

Refer to Sections 201.2.6, 202.2.3 and 204.6.2 for the staged review requirements for MSE walls. Consult the Office of Structural Engineering for additional design recommendations.

204.5 PIER TYPES

For highway grade separations, the pier type should generally be cap-and-column piers supported on a minimum of 3 columns. The purpose for this provision is to reduce the potential for total pier failure in the event of an impact involving a large vehicle or its cargo. This requirement may be waived for temporary conditions that require caps supported on less than 3 columns. Typically the pier cap ends should be cantilevered and have squared ends.

For bridges over railroads generally the pier type should be T-type, wall type or cap and column piers. Preference should be given to T-type piers. Where a cap and column pier is located within 25 feet [7.6 meters] from the centerline of tracks, crash walls will be required.

For waterway bridges the following pier type should be used:
A. Capped pile type piers; generally limited to an unsupported pile length of 20 feet [6 meters]. For unsupported pile lengths greater than 15 feet [4.5 meters], the designer should analyze the piles as columns above ground. Scour depths and the embedded depth to fixity of the driven piles shall be included in the determination of unsupported length.

B. Cap-and-column type piers.

C. Solid wall or T-type piers.

Note the use of T-type piers, or other pier types with large overhangs, makes the removal of debris at the pier face difficult to perform from the bridge deck. For low stream crossings with debris flow problems and where access to the piers from the stream is limited, T-type piers, or other similar pier types, should not be used.

For unusual conditions, other types may be acceptable. In the design of piers which are readily visible to the public, appearance should be given consideration if it does not add appreciably to the cost of the pier.

204.6 RETAINING WALLS

In conformance with Section 1400 of the ODOT Location and Design Manual, Volume Three, a Retaining Wall Justification shall be included in the Preferred Alternative Verification Review Submission for a Major Project or in the Minor Project Preliminary Engineering Study Review Submission. A description of the Retaining Wall Justification is provided in Section 1404 of the ODOT Location and Design Manual, Volume Three. Generally, the justification compares the practicality, constructability and economics of the various types of retailing walls listed below:

A. Cast-in-place reinforced concrete
B. Precast concrete
C. Tied-back
D. Adjacent drilled shafts
E. Sheet piling
F. H-piling with lagging
G. Cellular (Block, Bin or Crib)
H. Soil nail
I. Mechanically Stabilized Earth (MSE)

Refer to SS840 for accredited MSE wall systems. Contact the Office of Structural Engineering for modular block wall systems.
(highway or railway) separations or by the requirements for waterway opening at stream crossings. Typically for any given bridge, there are a number of combinations of spans and lengths of spans that can be utilized. Generally a preferred span arrangement that minimizes the number of substructure units should be used (i.e. fewer piers with longer spans).

For grade separation structures spanning any divided highway a two-span bridge with spill-thru slopes is preferred.

For waterway crossings, one or three span bridges are typically used. This span arrangement is preferred so that a pier is not located in the middle of the waterway. Refer to BDM Section 202.2.3.1 for spread footing requirements for a series of precast, three-sided structures used to produce a multiple span structure over a waterway.

When a multiple span arrangement (4 spans or more) is required, the Cost Analysis should examine the most economical number of spans required based on total bridge costs, including a substructure and superstructure cost optimization study. Site conditions will govern the location of substructure units with respect to required horizontal clearances, foundation conditions and appearance.

On structures with steep grades, the designer should account for the load effects of the grade on the substructure units.

### 205.3 CONCRETE SLABS

Cast-in-place concrete slabs are normally used where site geometry dictates a curved alignment or variable superelevation and the use of prestressed concrete box beams is impractical. Since concrete slabs will generally yield the least superstructure depth they should be considered when vertical clearance is limited. For stream crossings where flood waters often inundate the structure, a concrete slab should be considered. When using cast-in-place concrete slabs the construction clearance requirements of the falsework should be considered.

### 205.4 PRESTRESSED CONCRETE BOX BEAMS

The span limits for prestressed, side by side, concrete box beams generally range from 15 to 100 feet. These span limits are based on designs with 0.167 in² low relaxation strands, a concrete 28-day compressive strength of 7.0 ksi, and a release strength of 5.0 ksi. Prestressed box beams of up to 120 foot spans have been designed using 10.0 ksi concrete and larger diameter strands. Concrete compressive strengths should be limited to 5.0 ksi at release and 7.0 ksi at 28-days. Refer to BDM Section 205.5 when considering a deviation from standard prestressed practice in Ohio.

The skew angle should be limited to a maximum of 30 degrees. Consult the Office of Structural Engineering for recommendations prior to designing a box beam structure with a higher degree of skew. For all four lane divided highways or where the design ADTT (one way) is greater than 2500 prestressed box beam superstructures shall not be used. Box beams may be used on curved
D. The use of steel “H” piles with lagging is also a practical solution for some sites. Please note that some railroad companies allow only interlocking steel sheet piling adjacent to their tracks.

E. Where sufficient embedment cannot be attained by driving sheet piling because of the location of shallow bedrock, predrilled holes into the bedrock with soldier “H” piles and lagging should be considered.

For cuts greater than 12-15 feet [3.5-4.5 meters], the “H” piles may need to be anchored.

F. The highway design live loading should be equal to two feet [600 mm] of equivalent soil height as a surcharge.

G. The following items at a minimum should be shown on the detail plans:
   1. Minimum section modulus
   2. Top and minimum bottom elevation of shoring
   3. Limits of shoring
   4. Sequence of installation and/or operations.
   5. Method of payment
   6. If bracing or tiebacks are required, all details, connections and member sizes shall be detailed.

208.2 SUPPORT OF EXISTING STRUCTURE

Whenever temporary support is required for a portion of an existing structure used to maintain traffic, the Design Agency shall provide sufficient information in the plans to allow contractors to prepare bids and construct the project. The feasibility of temporary support of an existing structure should be considered and discussed during the Structure Type Study.

The design shown in the plans should include: permissible locations of temporary support; temporary support loads; construction sequences; construction limitations not otherwise provided in C&MS 501.05; and any remaining plan notes. As a minimum, the plan notes should address method of measurement and basis of payment for temporary support.

209 MISCELLANEOUS

209.1 TRANSVERSE DECK SECTION WITH SUPERELEVATION

If the change in cross slope at the superelevation break point is less than or equal to 7 percent, then no rounding is required. For changes greater than 7 percent the bridge deck surface profile shall be as follows:
A. When the roadway break point is located between roadway lanes (not at the edge of pavement) the bridge cross slope is to extend to the toe of parapet. See “CASE a” in Figure 209.1-1.

B. When the roadway break point is located at the edge of pavement (adjacent shoulder width is less than four feet [1.2 meters]), the bridge cross slope is to be continued past the break point to the toe of deflector parapet. See “CASE b” in Figure 209.1-1.

C. When the roadway break point is located at the edge of pavement (adjacent shoulder width is equal to or greater than four feet [1.2 meters] and less than eight feet [2.4 meters]), a four foot [1.2 meter] rounding distance from the edge of pavement onto the shoulder is used to transition from the bridge cross slope to the 0.5 in. per ft. [0.04] shoulder cross slope. See “CASE c” in Figure 209.1-2.

D. When the roadway break point is located at the edge of pavement (adjacent shoulder width is equal to or greater than eight feet [2.4 meters]), a five foot [1.5 meter] rounding distance
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 07-18-14)
Abutment Founded on Rock

Abutment Founded on Soil

Pier Founded on Soil

Figure 202.2.3.1.a-1
THIS FIGURE HAS BEEN RETIRED
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SECTION 300 – DETAIL DESIGN

301 GENERAL

301.1 DESIGN PHILOSOPHY

Section 300 of this Manual establishes general design guidelines, details, special requirements and reasonable alternatives, which, when incorporated by the engineer in a set of bridge plans, will provide a bridge structure that meets load requirements, provides structural integrity, provides structural efficiency and reduces long term maintenance to a minimum level.

301.2 DETAIL DESIGN REVIEW SUBMISSIONS

The detail design review for structures is conducted as part of the Stage 2 and Stage 3 review submission.

The Stage 2 Detail Design submission should include an updated cost estimate and the items listed below. Not every item listed will apply to every project.

A. Bridge Plans generally consisting of the following:
   1. Site Plan in compliance with all Stage 1 review comments
   2. General Plan (if required)
   3. General Notes
   4. Phase Construction Details
   5. Foundation Plan
   6. Abutment Details with all dimensioning, bar marks and bar spacings properly shown
   7. Pier Details with all dimensioning, bar marks and bar spacings properly shown
   8. Superstructure Details with all dimensioning, bar marks and bar spacings properly shown
   9. Other Details as necessary

B. Retaining Wall Plans generally consisting of the following:
   1. General Notes
   2. Retaining wall details
   3. Other Details as necessary

C. Noise Barrier Plans generally consisting of the following:
   1. General Notes
   2. Plan and Profile Views
3. Noise Barrier Details
4. Foundations Table
5. Subsurface Investigation Plan Sheets
6. Other Details as necessary
D. Special Provisions
E. Load Rating Reports for bridges
F. Signed Office of Structural Engineering Bridge Stage 2 Plan Review Checklist

The Stage 3 Detail Design plan submission should include an updated cost estimate and the following:
A. Stage 2 Detail Design plans in compliance with all Stage 2 review comments.
B. Completed Estimated Quantities Table
C. Completed Reinforcing Steel Schedule
D. Estimated Quantities calculations
E. Load Rating Reports for bridges
F. Signed Office of Structural Engineering Bridge Stage 2 Plan Review Checklist

Refer to Section 1400 of the ODOT Location and Design Manual, Volume Three, for additional staged review submission requirements.

For structures with non-redundant and/or fracture critical design details, a complete Stage 2 Detail Design Review Submission shall be made to the Office of Structural Engineering for concurrent review and comment. The Office of Structural Engineering will forward all comments to the responsible District Office or LPA.

301.3 DESIGN METHODS

Ohio Department of Transportation bridge designs are to be developed in general conformance with the latest edition of the American Association of State Highway and Transportation Officials’ (AASHTO) LRFD Bridge Design Specifications, including all interims. ODOT exceptions to the AASHTO LRFD specifications are documented in BDM Section 1000.

When site conditions require the use of a superstructure type that exceeds the recommended limits set forth by AASHTO and/or this Manual, a special design method may be required using a two-dimensional or three-dimensional model and some type of numerical analysis to solve the model. When this occurs, the designer should place a note in the General Notes section of the detail construction plans listing the type of model used, method of analysis and assumptions made during the design. Examples of special design methods include grillage, finite element, finite strip and classical plate solutions. A sample note can be found in Section 600 of this Manual.

For design of Temporary Structures see Section 500 of this Manual.
301.4 LOADING REQUIREMENTS

301.4.1 HIGHWAY BRIDGES

All bridges designed to carry highway traffic shall be designed for an HL-93 loading as specified in LRFD 3.6.1.2.1 and a future wearing surface (FWS) of 60 psf [2.87 kPa].

301.4.2 PEDESTRIAN AND BIKEWAY BRIDGES

Pedestrian and bikeway bridges shall be designed in accordance with the latest edition of the AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges; ODOT design guidelines; and this Manual. ODOT’s most current design guidelines are available at ODOT’S Office of Local Projects website, www.dot.state.oh.us/local/.

Where sidewalks, pedestrian, and/or bicycle bridges are intended to be used by maintenance and/or other incidental vehicles, an H15-44 [M13.5] vehicle, as shown in Figure 301.4.2-1, shall be included in the design loading. The H15-44 lane loading should not be considered. The vehicle live load shall not be placed in combination with the pedestrian live load and the dynamic load allowance need not be applied to the H15 vehicle.

301.4.3 RAILROAD BRIDGES

Facilities that are operated and maintained by the railroad shall be designed according to the specifications and design standards used by the railroad in its normal practice. Facilities that are operated and maintained by ODOT or other local agency shall conform to the AASHTO LRFD Bridge Design Specifications and this Manual.

301.4.4 SEISMIC DESIGN

Earthquakes arise from the movement of underlying bedrock. Ground motion resulting from the movement of underlying bedrock can be amplified or dampened by the overlying soil profile. Designers shall analyze soil borings to identify overlying soil profiles that can amplify ground motion propagating from underlying rock according to LRFD 3.10.3.1. Bridges located in Site Class D, E and F may require additional design considerations as noted in BDM Sections 301.4.4.1.

301.4.4.1 SEISMIC PERFORMANCE ZONE 1

All bridges in the State of Ohio are located within Seismic Performance Zone 1.

Bridges designed according to the Strength and Service Limit States of the AASHTO LRFD Bridge Design Specifications are assumed to have sufficient capacity to resist Seismic
Performance Zone 1 design loads applied at the Extreme Limit State. Seismic analysis is not required except as noted in BDM Sections 301.4.4.1.a and 301.4.4.1.b.

For bridges located in Site Class D, E and F, Designers shall determine the acceleration coefficient, \( S_{1} \), according to LRFD Eq. 3.10.4.2-6 with \( F_v = 2.4 \). For Ohio, only areas where \( S_1 \geq 0.042 \) and Site Class D, E and F exist, will \( 0.10 \leq S_{DI} < 0.15 \) (See Figure 301.4.4.1-1). For bridges founded at locations with \( 0.10 \leq S_{DI} < 0.15 \), the transverse reinforcement requirements at the top and bottom of columns shall be as specified in LRFD 5.10.11.4.1d and 5.10.11.4.1e. If sufficient geotechnical information is not available to determine Site Class, and the project is located in areas where \( S_1 \geq 0.042 \), then Designers shall assume \( 0.10 \leq S_{DI} < 0.15 \). Otherwise, Designers shall assume \( S_{DI} < 0.10 \).

Designers may use the Seismic maps, LRFD Figure 3.10.2.1-3 or the USGS US Seismic Design Maps web application to determine \( S_1 \) and \( S_{DI} \).

### 301.4.4.1.a MINIMUM SUPPORT LENGTH REQUIREMENTS

To prevent the partial or complete collapse of the superstructure during seismic events, the bearing supports at the end of a superstructure unit shall be sized according to LRFD 4.7.4.4.

As a minimum, the overlapping distance of the superstructure and bearing areas shall meet 100% of the minimal support length, \( N \), calculated according to LRFD Eq. 4.7.4.4-1. The minimum support lengths shall be measured normal to the centerline of supports. Designers shall account for expansion/contraction movements of the bearings when establishing final seat widths. Refer to Figure 301.4.4.1.a-1 for the application of support length requirements to a typical expansion elastomeric bearing at an abutment.

### 301.4.4.1.b HORIZONTAL CONNECTION FORCE

All structures shall have some mechanism to transfer horizontally applied superstructure loads (e.g. vehicular braking force, centrifugal force, vehicular collision force, friction load, water load, wind load, and wind load on live load) to the substructure to ensure structural stability. Examples of mechanisms include fixed bearings, bearing guides, abutment diaphragms, diaphragm guides, wing walls and wind locks. During a seismic event, these mechanisms that prevent the free lateral translation of the superstructure in any direction relative to the substructure, shall be designed to transfer an applied horizontal connection force at the Extreme Event I Limit State. Additional restraint for seismic loads (e.g. seismic pedestals) shall only be provided where the mechanisms noted above do not provide sufficient capacity. Refer to BDM Section 301.4.4.1.c for bearing requirements.

The magnitude of the connection force shall be 0.15 or 0.25 times the tributary permanent load at the location of the restraint as determined in LRFD 3.10.9.2. The load factor for live load, \( \gamma_{EQ} \), may be taken as 0.0. If sufficient geotechnical information is not available to determine Site Class, Designers shall assume the magnitude of the connection force is 0.25 times the tributary permanent load.
permanent load.

For restraint provided in multiple directions, LRFD 3.10.8 applies.

The tributary permanent load defined in LRFD 3.10.9.2 represents the factored dead load of the superstructure applying load to the device or object providing the directional restraint. If every bearing supporting the superstructure provides transverse restraint, the tributary permanent load applied to each restraint would equal the factored dead load reaction at each bearing. If only one transverse restraint was provided at each substructure unit, the tributary permanent load applied to each restraint would equal the sum of the factored dead load reactions for each bearing at the substructure unit. If only one transverse restraint was provided for the entire superstructure unit, the tributary permanent load applied to the restraint would equal the sum of the factored dead load reactions of every bearing. Longitudinal restraint connection forces would be determined similarly.

Because a structure in Seismic Performance Zone 1 is assumed to be able to carry the loads within the elastic strength range of its members, or is assumed to be properly detailed to prevent collapse beyond the elastic strength range of its members, analysis of the superstructure, substructure and foundation for the load effects resulting from the connection force is not required.

Crossframes to resist the horizontal connection force at the Extreme Event Limit state shall be provided to create a direct load path from the point of horizontal connection force application to the deck.

301.4.4.1.c REQUIREMENTS FOR BEARINGS

Unrestrained bearings that sustain irreparable damage during a seismic event are permissible provided loss of span is prevented by the design for the Horizontal Connection Force in BDM Section 301.4.4.1.b.

301.4.4.2 EXISTING STRUCTURES

Seismic vulnerability of a structure shall be considered for rehabilitation projects requiring complete deck or superstructure replacements. New substructure units shall be designed in accordance with LRFD 3.10.9.2, 4.7.4.4 and 5.7.4.6. If sufficient geotechnical information is not available, Designers may assume:

A. $A_5 > 0.05$
B. $S_{dl} < 0.10$.

301.4.4.2.a SUPERSTRUCTURE

For projects where seismic vulnerability is considered, at bearing locations that will transmit the
horizontal connection force from the substructure to the superstructure, crossframes designed to resist the horizontal connection force shall be provided to create a direct load path to the deck. For supports not in compliance with *LRFD 4.7.4.4*, seismic restrainers designed for the Horizontal Connection Force, specified in BDM Section 301.4.4.1.b, shall be provided.

301.4.4.2.b SUBSTRUCTURE

For projects where seismic vulnerability is considered, concrete columns at piers that transfer the seismic horizontal connection force, according to BDM Section 301.4.4.1.b, shall meet the spiral and tie ductility requirements of *LRFD 5.7.4.6*. Designers may consider releasing restraint provided by existing pier bearings as a viable seismic retrofit provided the abutments can accommodate the additional horizontal Strength and Service loadings. Otherwise, Designers shall provide the required confinement of the primary steel in the axially loaded substructure members.

301.4.5 APPLICATION OF LONGITUDINAL FORCES

For bearing types that permit rotation about the transverse axis of the bridge, all longitudinal load types shall be applied at the bearing elevation and moments resulting from eccentricity shall be ignored. The total factored longitudinal loading applied to the substructure at each expansion bearing shall not exceed the bearing’s nominal (i.e. *unfactored*) resistance to longitudinal loading. Resistance in this instance is nominal because it is applied to the substructure as a loading.
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301.5 REINFORCING STEEL

Reinforcing steel - ASTM A615 or A996, Grade 60, $F_y = 60,000$ psi.

Reinforcing steel - ASTM A615M or A996M, Grade 420, $F_y = 420$ MPa

All reinforcing steel shall be epoxy coated.

301.5.1 MAXIMUM LENGTH

Generally maximum length of reinforcing steel should be 40 feet [12.2 meters]. This limit is for both transit purposes and construction convenience. The maximum length before a lap splice is required is 60 feet [18.4 meters]. To facilitate an economical design using 60 foot bar stock, where multiple sets of lapped bars are required (i.e. longitudinal slab reinforcement) consideration should be given to using multiple sets of 30 foot long bars.

The length of the short dimension of L-shaped bars should be limited in order not to extend beyond the sides of a highway vehicle of maximum legal width. The short dimension should preferably be not greater than 7'-6" [2300 mm], and in no case greater than 8'-0" [2450 mm].

301.5.2 BAR MARKS

Bar marks shall be used on detail plans to identify the bar's size and general location and to reference the bar to the reinforcing bar list.

Letters should be incorporated into the bar marks to help identify their location in the detail plans: "A" for abutments, "P" for piers, "S" for superstructure, “SP” for spirals, “DS” for drilled shafts, etc.

The following bar mark represents a #5 [16M] abutment bar ............................................A501 [A16M01]
The following bar mark represents a #4 [13M] spiral bar ........................................... SP401 [SP13M01]
The following bar mark represents a #9 [29M] drilled shaft bar ..........................DS901 [DS29M01]

A note or legend within the bar list sheet in the plans shall describe each bar mark's meaning. See Figure 301.5.2-1.

301.5.3 LAP SPLICES

Bar splice lengths shall be shown on the plans.

Development and splice lengths shall conform to AASHTO requirements.

Reinforcing steel at construction joints should extend into the next pour only by the required
Concrete surfaces that include patches should be sealed with an epoxy-urethane sealer so the concrete color will remain uniform.

The designer should include in the plans actual details showing the position, location and area required to be sealed. A plan note should not be used to describe the location as there can be both description and interpretation problems.

The designer has the option to select a specific type of sealer, epoxy-urethane or non-epoxy. The designer may also use a bid item for sealer, with no preference, and allow the contractor to choose based on cost.

Due to poor performance, epoxy-only sealers shall not be used.

In areas where concrete surfaces have a history of graffiti vandalism, the designer may add a sacrificial or permanent graffiti coating meeting the requirements of Supplement 1083 on top of the epoxy-urethane or non-epoxy sealer. A plan note is available in BDM Section 600. The designer should limit the concrete surfaces that are treated with sacrificial or permanent graffiti coatings to those reachable by easy climbing and visible to the traveling public.

302.2 REINFORCED CONCRETE DECK ON LONGITUDINAL MEMBERS

302.2.1 DECK THICKNESS

For reinforced concrete decks on steel or concrete longitudinal members, the deck thickness shall be computed by the following formula:

\[
T_{\text{min}} \text{(inches)} = \frac{(S + 17)(12)}{36} \geq 8\frac{1}{2}''
\]

\[
T_{\text{min}} \text{(mm)} = \frac{(S + 5200)}{36} \geq 215 \text{ mm}
\]

Where: S is the effective span length in feet [millimeters] determined according to LRFD 9.7.3.2. Tmin shall be rounded up to the nearest one-quarter inch [5 mm].

The one inch [25 mm] wearing thickness, Section 302.1.3.1, is included in the minimum concrete deck thickness but should be excluded in the calculations for structural design of the deck slab.

302.2.2 CONCRETE DECK DESIGN

The concrete deck design shall be in conformance with the approximate elastic methods of analysis specified in the AASHTO LRFD Bridge Design Specifications, latest edition, and the additional requirements specified in this Manual. Refined methods of analysis and the empirical design method, LRFD 9.7.2, are prohibited. The design live load shall be HL-93 and the design dead load shall include an allowance for a future wearing surface equal to 0.06 k/ft².
Shrinkage and temperature reinforcement conforming to *LRFD 5.10.8* shall be placed in the underside of deck overhang with the minimum clear cover measured to the transverse steel.

Deck designs for superstructures with effective span lengths ranging from 7.0 ft. to 14.0 ft. in 0.5 ft. increments are provided in Figures 302.2.2-1, 302.2.2-2 and 302.2.2-3. These designs apply for the full length of the bridge and preclude the need for additional transverse reinforcement at supported deck ends. The design of overhang reinforcement is valid for BR-1-13, SBR-1-13, BR-2-15 and TST-1-99 barrier systems. A complete list of design assumptions is
provided with Figure 302.2.2-1.

Transverse spacing of the top and bottom reinforcing in a deck design shall meet section 302.2.4.2.

302.2.3 DECK ELEVATION REQUIREMENTS

302.2.3.1 SCREED ELEVATIONS

Screed elevations are control elevations for concrete deck finishing machines that account for dead load deflections to ensure that the bridge deck is completed to the correct elevation. To establish screed elevations, the final surface elevations are adjusted for non-composite deflections resulting from deck placement and composite deflections resulting from utility and railing loads. Screed elevations shall not include adjustment for deflections due to the future wearing surface loading. Calculated deflections caused by the weight of the deck concrete should assume a completed placement sequence. Use deflection data from girder lines closest to each screed line to determine elevations. Refer to Figure 302.2.3-1.

If the deflections are determined through a line girder analysis method, the deck load should be distributed evenly to all beams/girders loaded in each construction phase to establish screed elevations. If a refined analysis method is used, establish screed elevations using the individual beam/girder deflections.

The bridge plans shall include a screed elevations table. The locations of all screed elevations in the table should be identified on a transverse section and plan view. Elevations should be provided for all: curblines or deck edges; profile grade points; transverse grade-break lines; and phased construction lines for the full length of the bridge. Screed elevations are not required above beam/girder lines. Bearing points, quarter-span points, mid-span points and splice points shall be detailed as well as any additional points required to meet a maximum spacing between points of 25'-0" [7.5 m].

For bridges with a separate wearing course, the elevations given should be those at the top of the portland cement concrete deck. Provide a plan note stating at what surface the elevations are given in order to eliminate any confusion.

Screed elevations are not required for non-composite box beam bridges or slab bridges. Screed elevations for composite box beam bridges shall meet the same requirements as steel beam, girder and prestressed I-beam bridges.

302.2.3.2 TOP OF HAUNCH ELEVATIONS

Top of haunch elevations represent the theoretical bottom of deck elevation before the concrete deck is placed. Top of haunch elevations should be provided at the centerline of each girder at bearing points, quarter points, mid-span points, splice points and additional points to meet a
completed, the designer should confirm with local galvanizers if a local plant can galvanize the structural members detailed.

Since standard holes may become partially filled with galvanizing, bolted splice designs will require a non-standard hole size equal to the nominal bolt diameter plus 1/8". Bolted crossframes will be required due to field installation issues. Bolted cross frames as detailed in the Standard Bridge Drawing may be specified.

Field welding of end crossframes, intermediate cross frames and bearings is not acceptable because welding onto galvanizing causes damage to the coating and no quality touch-up system is available to handle the number of repairs required.

302.4.2.2 STIFFENERS

Intermediate stiffeners shall only be used when required for cross frames. Stiffeners shall be a minimum 3/8 inch [10 mm] thickness and wide enough to make an adequate and easily accessible cross frame connection. Stiffeners generally should not extend beyond the edge of flange.

Stiffener plates shall have corners in contact with both web and flange clipped. The clip dimensions shall be one inch [25 mm] horizontally and 2½ inches [65 mm] vertically. Dimensions are shown on the Standard Bridge Drawing.

Both sides of the stiffener shall be fillet welded to the beam web and both flanges.

302.4.2.3 INTERMEDIATE CROSS FRAMES

For structures with the stringers placed on tangent alignments, detail cross frames as follows:

A. Cross frames for rolled beams shall be connected directly to the web or to intermediate web stiffeners.

B. Cross frames shall be perpendicular to stringers and be in line across the total width of the structure.

C. Cross frame spacings between points of dead load contraflexure in the positive moment regions shall not exceed 25 ft [7.6 m].

D. Cross frame spacings between points of dead load contraflexure in the negative moment regions shall not exceed 15 ft [4.6 m].

E. Horizontal legs of cross frame angles shall align on both sides of the stringer.

See the General Steel Details Standard Bridge Drawing for standard cross frame configurations.

For structures with flared stringers, the following exceptions apply:
A. If the differential angle between individual stringers is 5 degrees or less, the cross frames shall be perpendicular to one stringer and in line across the total width of the structure.

B. If the differential angle between individual stringers is greater than 5 degrees, the differential angle shall be divided evenly between connections to both stringers.

The design plans shall show:

A. The cross frame spacing for each region along the length of the stringer.

B. The typical cross frame details or reference to the General Steel Details Standard Bridge Drawing for standard cross frame configurations.

The designer shall ensure crossframe locations do not conflict with bolted splices or shall provide appropriate details to attach crossframes at bolted splice locations.

A detail showing a completely bolted connection for cross frame to the steel member is shown in the Standard Bridge Drawing.

Holes for erection bolts are normally provided in the connection of cross frames to stiffeners. Refer to the Standard Bridge Drawing for details.

In phased construction of new steel structures cross frames should not be permanently attached between phases until all deadload (deck, parapet, etc.) has been applied to the members. The crossframes can then be permanently attached and a deck closure pour can be completed to finish the superstructure. See Section 302.2.9.

For curved or flared bridges with “dog-legged” stringers, cross frames should be placed near the bend points. The cross frames should be located approximately 1 foot [300 mm] from the bend point but not interfere with the splice material. The cross frame should be placed normal to the stringer used to set the 1 foot [300 mm] clearance dimension and should be connected to the adjacent stringer only on the same side of the centerline of the splice. The cross frame units should be similar to standard cross frames but should have an additional horizontal angle near the top flange of the stringers.

See Figure 302.4.2.3-1 for plan view layout of cross frames for dog-legged stringers.

Cross frames for curved stringers may be one of the types shown on the Standard Bridge Drawing with an additional top strut. The designer shall confirm that the standard cross frames and their connections meet the additional loading developed in a curved member design. Since
the concrete (1 to 3 percent per yard [meter] of the abutment, pier or wall)

D. Generic formliner patterns shall be specified. An alternative of at least three suppliers listed. Listing of a formliner pattern only available from one supplier will not be accepted.

303.2.2.1.a COUNTERFORTS FOR FULL HEIGHT ABUTMENTS

For full height abutments exceeding 30 feet [10 000 mm] in height, counterforts should be considered.

Reinforcing steel in the back, sloping, face of the counterfort should be placed in two rows with a 6 inch [150 mm] clearance between rows. Reinforcing steel splices should be staggered a minimum of 3'-0" [1000 mm], by row.

Reinforcing extending from the footing of a counterforted wall into the highly reinforced areas of the counterforts shall have reinforcing steel splices staggered.

In counterforted walls, each pocket formed by the intersection of the counterfort and wall shall be drained.

303.2.2.1.b SEALING STRIP FOR FULL HEIGHT ABUTMENTS

Use an impervious fabric across the expansion joints in full height abutments or retaining walls to eliminate leakage. The impervious fabric should be CMS 512 Type 2 Waterproofing, 3 feet [1000 mm] wide, centered over, and extending the full length of the joint to the top of the footing. See Section 303.2.5 on requirements for expansion joints in abutments.

303.2.2.2 CONCRETE SLAB BRIDGES ON RIGID ABUTMENTS

For a continuous concrete slab bridge supported on rigid abutments, eliminate the A801 bar shown in CPA-1-08; trowel smooth the joint between the deck slab and the top of the abutment; and recess a continuous strip of elastomeric material into the abutment seat before placement of the superstructure concrete. This bearing system should conform to temperature movement and bearing design requirements of this Manual.

303.2.2.3 STUB ABUTMENTS WITH SPILL THRU SLOPES

If a stub abutment is to support a bridge having provision for relative movement between the superstructure and the abutment, two rows of piles are required and the front row shall be battered 1:4.

Where two rows of piles are used, the forward row shall have approximately twice the number of piles as the rear row, with the rear piles placed directly behind alternate front piles.
The perpendicular distance from the surface of the embankment to the bottom of the toe of the footing should be at least 4'-0" [1200 mm].

The maximum spacing of piles in a single row or in the front row of a double row shall be 8 feet [2500 mm].

For phased construction projects, do not design an abutment phase supported on less than three (3) piles or two (2) drilled shafts.

303.2.2.4 CAPPED PILE STUB ABUTMENTS

For capped pile stub abutments that do not provide for relative movement between the superstructure and the abutment, one row of vertical piles shall be used.

The construction joint at the top of the footing for cap pile abutments should be shown as optional.

For phased construction projects, do not design an abutment phase supported on less than three (3) piles or two (2) drilled shafts.

To avoid non-redundant designs, capped pile abutments should be supported on at least 5 piles. Refer to LRFD 10.5.5.2.3 for more information.

303.2.2.5 SPREAD FOOTING TYPE ABUTMENTS

Where foundation conditions warrant the use of an abutment on a spread footing, the footing shall be located in accordance with BDM Section 202.2.3.1.

303.2.2.6 INTEGRAL ABUTMENTS

Integral Abutment use is limited as defined in Section 200 of this Manual. Integral design should not be used with curved main members or main members that have bend points in any stringer line.

For an integral design to work properly, the geometry of the approach slab, the design of the wingwalls, (see section 303.2.4) and the transition parapets must be compatible with the freedom required for the integral (beams, deck, backwall, wingwalls and approach slab) connection to rotate and translate longitudinally.
on bedrock.

The minimum width of footing supported by a drilled shaft is the diameter of the shaft.

Where piling is used to support free-standing piers, the distance between centers of outside piles, measured across the footing, generally shall be not less than one-fifth the height of the pier.

Widths greater than the above shall be provided if required for proper bearing area or to accommodate the required number of piles.

The height of the pier shall be measured from the bottom of the footing to the bridge seat.

For multiple span bridges with continuity over piers, where the height of pier is more than 50 percent of the length of superstructure from the point of zero movement to such pier, it may be assumed that the pier will bend or tilt sufficiently to permit the superstructure to expand or contract without appreciable pier stress. This assumption is not permissible if the piers are skewed more than 30 degrees. The above rule does not apply to rigid frame or arch bridges.

Slender columns of either concrete or steel may be designed to bend sufficiently to permit the superimposed superstructure to expand and contract, but the resulting bending stresses shall not exceed the allowable.

During phased construction of a capped pile pier, do not design a pier phase to be supported on less than three (3) piles. For cap and column piers, do not design a phase to be supported on less than two (2) columns.

For a new or replacement structure, individual free-standing columns without a cap are not permitted.

303.3.1 BEARING SEAT WIDTHS

Pier bearing seat widths for reinforced concrete slab bridges should conform to Standard Bridge Drawing CPP-1-08. Also see Section 303.3.2.5 of this Manual.

Pier caps on piles, drilled shafts or on columns are normally a minimum of 3'-0" [915 mm] wide. This is the standard width used for continuous span prestressed box beams and I-beams. Bearing seat widths of 3'-0" [915 mm], while normally adequate must be verified by the designer of the structure. Large bearings, skew angle, intermediate expansion devices, AASHTO earthquake seat requirements, etc. may require additional width.

303.3.1.2 PIER PROTECTION IN WATERWAYS

See Section 200 of this Manual for piling protection requirements and Section 600 for a plan note to be added to design drawings when the Capped Pile Pier Standard Bridge Drawing is not referenced.
303.3.2 TYPES OF PIERS

303.3.2.1 CAP AND COLUMN PIERS

303.3.2.1.a CAPS

When designing the cantilever portions of cap and column piers, the design moments shall be calculated at the actual centerline of the column.

The uppermost layers of longitudinal reinforcing steel in the pier cap shall not be lap spliced at the centerline of a column.

Cap dimensions should be selected to meet strength requirements and to provide necessary bridge seat widths according to BDM Section 301.4.4.1.a. Caps should be cantilevered beyond the face of the end column to provide approximately balanced factored dead load moments in the cap. The end of the cantilevered caps should be formed perpendicular to the longitudinal centerline of the cap to allow for uniform development lengths for the reinforcing steel. Cantilevered pier caps may have the bottom surface of the cantilever sloped upward from the column toward the end of the cap. Cantilevered caps may be eliminated for waterway crossing where debris removal access is an issue.

303.3.2.1.b COLUMNS

Columns shall be designed as compression members according to LRFD 5.7.4 and 5.10.11.

Round columns are preferred. The minimum column diameter is 36”.

303.3.2.2 CAP AND COLUMN PIERS ON PILES

Piers supported on piles shall have separate footings under each column.

Column piers shall have at least 4 piles per footing.

For grade separation structures, the top of the pier's footings should be a minimum of 1'-0" [300 mm] below the level of the bottom of the adjacent ditch. This applies even though the pier is
located in a raised earth median barrier.

303.3.2.3 CAP AND COLUMN PIERS ON DRILLED SHAFTS

Where columns are supported on a drilled shaft foundation, the drilled shaft should be at least 6 inches [150 mm] larger in diameter than the column. This is to allow for field location tolerances of the drilled shaft. A drilled shaft foundation is defined as starting 1 foot [0.3 meter] below ground level or 1 foot [0.3 meter] above normal water.

303.3.2.4 CAP AND COLUMN PIERS ON SPREAD FOOTINGS

Cap and column piers on spread footings, placed on existing soils or on embankment fills, shall have continuous footings which should extend beyond the center of the end column a distance equal to approximately 1/3 of the distance between the end column and the adjacent column, in order to provide approximately balanced moments.

Cap and Column piers with spread footings on bedrock shall have separate footings under each column.

For grade separation structures, the top of pier footings shall be located in accordance with BDM 202.2.3.1. In no case shall the bottom of the footings in existing soil or on embankment fills be above the frost line.

The width of footing for a free-standing pier shall be not less than one-fourth the height of the pier where founded on soil and not less than one-fifth the height of the pier where founded on bedrock.

303.3.2.5 CAPPED PILE PIERS

Steel H piles shall be a minimum HP12x53 [HP310 x 79]. The piles should be shown on the plans with the flanges of the H-section perpendicular to the face of the pier cap.

The distance from the edge of a concrete pier cap to the side of a pile shall be not less than 9 inches [230 mm].

The diameter of the exposed portions of cast-in-place reinforced concrete piles generally should be 16 inches, but if exposed length, design load or other conditions make it necessary, larger diameter cast-in-place piles should be used. Cast-in-place piles shall be reinforced with a reinforcement cage composed of 8-#6 reinforcing bars with a 12 inch outside diameter, #4 spiral, with a 12 inch pitch. The cage length should extend from the finished top of the pile to 15 feet below the assumed point of fixity (minimum 15-ft below ground level). The reinforcing steel shall be shown in the structure's reinforcing bar list and be included in item 507 for payment.

The use of cast-in-place piles greater than 16 inches in diameter will require an increase in the width of the cap of Standard Bridge Drawing CPP-1-08. See Section 303.4.2.3.
Exposed H piles and unreinforced concrete piles shall have pile protection. Refer to the description in Standard Bridge Drawing CPP-1-08. A plan note is also available. Also See Section 200 for a description of pile protection.

For pile embedment requirements into concrete, see Section 303.4.2.3.

An optional construction joint shall be shown at the top of pier caps for reinforced concrete slab bridges. This joint is optional as some machine finishing equipment for slab bridge decks require a uniform depth of freshly placed concrete in order to obtain best results.

For phased construction projects, do not design a pier or abutment phase to be supported on less than three (3) piles.

To avoid non-redundant designs, capped pile piers should be supported on at least 5 piles. Refer to LRFD 10.5.5.2.3 for more information.

The connection between the pile cap and the superstructure shall be designed for the horizontal connection force specified in BDM 301.4.4.1.b. When using Standard Bridge Drawing, CPP-1-08, Designers shall verify the spacing of the P501 bars to resist this force. No additional seismic analysis for capped pile piers is required.

303.3.2.6 STEEL CAP PIERS

If at all possible this alternative should not be selected. This is a fracture critical design that has historically shown both steel member and weld metal cracking problems. As specified in Section 301.2, these structure types require a concurrent detail design review to be performed by the Office of Structural Engineering.

If a steel box girder is required as a pier cap, the design shall allow reasonable access to the interior for maintenance, inspection and repair purposes. The physical dimensions of the box shall be large enough to allow access to the interior for inspection. Access hatches of the box girder should be bolted and sealed with a neoprene gasket. Access hatches should also be light enough for an inspector to easily remove them. One recommended lightweight material is ABS plastic.

Designers shall ensure that all governmental agency regulations regarding to enclosed spaces, ventilation, lighting, etc. are complied with within any enclosed steel pier cap design.

Box designs with cut away webs to allow for stringers to continue through the box are generally not considered acceptable alternatives.
Situations that require stringers to be continuous through, and in the same plane with a steel pier cap or crossbeam should be avoided if at all possible.

Designers should review all weld details for possible fatigue problems. Consult the Office of Structural Engineering for assistance in this area.

303.3.2.7 POST-TENSIONED CONCRETE PIER CAPS

Where vertical clearance or geometric considerations require stringers to be continuous through and/or in the same plane as the pier cap, a post-tensioned concrete cap should be investigated as a first option in lieu of a steel pier cap. However, this is a non-redundant design, and, as specified in Section 301.2, these structure types require a concurrent detail design review to be performed by the Office of Structural Engineering.

303.3.2.8 T-TYPE PIERS

In the cap of a T-type pier, the top layer of reinforcing bars shall extend the full length of the cap and be turned down at the end face the necessary development length. The second layer of reinforcing steel shall extend into the stem of the pier at least the necessary development length plus the depth of the cantilever at its connection to the stem. Cap widths shall provide sufficient bearing seat width according to BDM Section 301.4.4.1.a.

Stems of T-type piers and wall type piers shall be designed as compression members according to LRFD 5.7.4. Wall type piers are characterized by the absence of clearly defined cap member.

Lateral ties shall be provided according to LRFD 5.10.6.3. For bridges located in areas where $0.10 \leq S_{D1} < 0.15$, transverse steel at the top and bottom of concrete compression members shall meet the requirements of LRFD 5.10.11.4.1d and 5.10.11.4.1e.

303.3.2.9 PIER USE ON RAILWAY STRUCTURES

For clearance requirements see Section 200 of this Manual. Items listed in Section 200 are only general rules and vary from railroad to railroad. The designer shall confirm with the individual railroad the actual physical dimension and design requirements.

303.3.2.10 PIERS ON NAVIGABLE WATERWAYS

Piers in the navigation channel of waterways, unless protected from collision by an adequate fendering system, shall be designed to resist collision forces based on AASHTO Guide Specification for Vessel Collision Design of Highway Bridges.

303.3.2.11 PIER CAP REINFORCING STEEL STIRRUPS

Stirrups for concrete beams of constant depth, such as pier caps, should be detailed using either 2 “U” bars with the vertical legs long enough to furnish the required lap length or a single bar closed type stirrup with 135° bends at both ends of the rebar. The single bar closed type stirrup
should only be selected when minimum required lap lengths cannot be provided with the “U” type stirrup. The corner with the 135° bends of the closed type stirrup should be placed in the compression zone of the concrete beam.

303.4 FOUNDATIONS

303.4.1 FOOTINGS

303.4.1.1 MINIMUM DEPTH OF FOOTINGS

Refer to BDM Section 202.2.3.1 for Spread Footing Elevation requirements.

Pile and drilled shaft footings shall be founded as follows:

A. For footings partially or totally located inside the plan view of the OHWM, the top of the footing shall be at least 1-ft below the Thalweg.

B. For footings located entirely outside the plan view of the OHWM:
   1. The top of footing shall be a minimum of 1-ft below the finished ground line. The top of footing shall be at least 1-ft below the bottom of any adjacent drainage ditch.
   2. The bottom of footing shall not be less than 4-ft below, measured normal to, the finished groundline.

303.4.1.2 FOOTING RESISTANCE TO HORIZONTAL FORCES

For spread footings, if the frictional or shearing resistance of the supporting material as specified in LRFD 10.6.3.4 is inadequate to withstand the horizontal forces, one or more of the following means, listed in order of preference, shall provide additional resistance:

A. Increase the footing width.

B. Use footing key and utilize the passive pressure acting on the key. Keys should be located within the middle-half of the footing width.

C. Use footing struts, sheeting or anchors.
On projects where maintaining minimum lane widths during a construction phase is not possible due to limited bridge width, the use of a top mounted steel post and tubular steel rail system, similar to the Twin Steel Tube bridge guardrail, may be justified. The railing, post and anchorage designs of these systems are to be in accordance with the *AASHTO LRFD Bridge Design Specifications, Sections A13.1-3.*

304.3.5 BRIDGE SIDEWALK RAILING WITH CONCRETE PARAPETS (BR-2-98)

This railing system is for use on bridges with sidewalks at least 5'-0" wide and a curb height of 8 inches. Although this system is essentially a combination railing system, it may also be used without a sidewalk in applications where pedestrian traffic is not a concern.

Where Vandal Protection Fencing is required, the fencing shall be installed behind the steel tubing as shown in Figure 0-2. However, the steel tubing may be omitted if the concrete parapet height is 32" or greater. See Figure 0-1. If the tubing is omitted, the fencing should extend the full length of the concrete parapet and the additional 18" parapet height at each end, as detailed in the standard, is not required.

The concrete parapet shall be designed and detailed as follows:

A. All horizontal reinforcing steel shall be detailed as continuous for the total length of the structure.

B. Crack control joints shall be sawed into the concrete parapets. The distance between the saw-cut joints on the structure shall be between 6'-0" and 10'-0". The detailed locations of the crack control joints and vertical reinforcing bars shall be shown in the contract plans.

C. The saw-cut crack control joint shall be detailed as 1 ¼ inch deep and shall be filled with a polyurethane or polymeric material conforming to ASTM C920, Type S. The bottom one-half inch of both the inside and outside face shall be left unsealed to allow any water that enters the joint to escape. This requirement is established in the Standard Bridge Drawing; however, a plan note is required for special designs. See Section 600.

305 FENCING

305.1 GENERAL

The primary purposes of protective fencing are to provide for the security of pedestrians and to discourage the throwing or dropping of objects from bridges onto traffic below.
The Vandal Protection Fencing Standard Bridge Drawing provides standard details for fencing attached to bridges. The designer may need to enhance this standard to deal with requirements for the specific structure.

### 305.2 WHEN TO USE

Fencing shall be installed on all bridges over vehicular traffic except as noted herein. Fencing shall be installed on bridges over rail traffic if required in an agreement with the affected railroad. Bridges that carry vehicular traffic over county/township routes shall be exempt from fencing. For existing bridges, fencing shall be provided when new concrete or refaced concrete barriers are installed. At locations where fencing will adversely affect public safety (e.g. reduced sight distance), submit a written request for exemption to the Deputy Director Engineering. The request for exemption shall include supporting documentation.

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* Bridges carrying only pedestrian/bicycle traffic shall have vandal protective fencing.

### 305.3 FENCING CONFIGURATIONS

For structures without sidewalks, the top of fence shall be a minimum height of 8-ft above the pavement surface. For structures with sidewalks, the top of fence shall be a minimum height of 8-ft above the sidewalk. For a greater degree of protection against objects being thrown from the bridge, the fence may be curved to overhang the sidewalk. For curved fence the posts shall be vertical for at least 8-ft above the sidewalk before curving inward over the sidewalk. The overhang shall be at least 1-ft less than the width of the sidewalk. See Figures 0-1 & 0-2.

For pedestrian bridges, use bent pipe frames with pipe bend radii of 24" at the upper corners and the start of the radii about 8-ft above the sidewalk surface. The fabric shall start at the deck line, top of curb or parapet and may stop at the upper end of the bent portion of the frame. Fabric on the top horizontal area of the frame is not required to prevent an individual from walking on the top of the enclosure. See Figure 0-3 for an illustration of this configuration. Alternatively, the frame may be designed to form a peak at the center of the structure, similar to a house roofline.

The maximum gap at the bottom of the fence shall be 1-in. A detail to close the bottom of a fencing section is included on the standard bridge drawing.
Posts and frames may be either plumb or perpendicular to the longitudinal grade of the bridge, subject to considerations of aesthetics or practicality of construction. Complete details of base plates, pipe inserts or other types of base anchorage shall be provided on the plans. If applicable to the specific project, details from the standard bridge drawing may be referred to in the project plans.

Fencing on the bridge shall extend between its End Posts placed at the locations selected from the following list that creates the shortest length:

A. 30-ft ± 2.5-ft beyond the under bridge route’s edge of traveled way nearest the fence terminal
B. The centerline of the abutment expansion joint (-2.5-ft, +0-ft)
C. The end of the bridge barrier (-2.5-ft, +0-ft)

Designers shall also place fence on structures parallel to traffic and that carry sidewalks located 30-ft or less from the nearest edge of the traveled way below.

For bridges where a snooper truck will be used for inspection, use only straight fence with the top of the fence located 10-ft or less above the deck.

### 305.4 SPECIAL DESIGNS

The design loading for non-standard fence designs shall be in accordance with LRFD 15.8.

For fence installation projects on new structures, the installation of a traffic railing (steel tubing) is not required if the top concrete parapet or concrete wall is 36-in above roadway for structures without sidewalks or 36-in above the top of sidewalk for structures with sidewalks. See Figure 0-1.

Where the standard gray chain link fence mesh detracts from a project’s aesthetic enhancements, designers may select an optional color from the following: green, olive green, brown and black. Designers shall consider the welded wire fabric, BDM Section 0.B, for additional color options. Color coating of posts and rails shall utilize a two coat shop applied epoxy/urethane system in accordance with C&MS 708.02. Plan notes for this coating system are available from OSE upon request.

For special fence designs, plan notes shall be required to define non-standard color, materials, traffic maintenance, construction procedures and other requirements. The designer should follow the example of standard bridge drawing for development of required notes.

### 305.5 FENCE DESIGN GENERAL REQUIREMENTS

Fencing mesh shall consist of either of the following materials:
A. Chain-link wire mesh with one inch [25 mm] diamonds. The core wire shall be 11 gage [3.05 mm] with a Polyvinyl chloride coating. (C&MS 710.03)

B. Welded wire fabric with $\frac{1}{2}'' \times 3''$ [12 mm x 75 mm] opening size. The core wire shall be 10.5 gage [3.25 mm]; galvanized after welding (1.2 oz zinc/ft²), and PVC coated (10 mil [0.25 mm]).

Brace and bottom rails shall be clamped to posts or post frames.

The top rail, if any, of a free-standing fence shall be continuous over two or more posts and suitable cap fittings provided.

Bent pipe frames for narrow pedestrian bridges are permitted. Bent pipe frames for narrow pedestrian bridges shall be fabricated in two or more sections and field spliced at the top with sleeves bolted to the frame sections.

To prevent pipe blow-ups during galvanizing, both ends of pipe shall be open. Therefore base plates shall have holes in them almost equal to the pipes’ inside diameter.

### 305.5.1 WIND LOADS

The design wind loading for non-standard fence designs shall be in accordance with LRFD 15.8.2.

The projected area for wind forces on 11 gage polyvinyl chloride coated with one inch diamond wire mesh shall be 20% of the gross horizontally projected area.

Additional area for posts, rails and other hardware need not be considered.

### 306 EXPANSION DEVICES

#### 306.1 GENERAL

Expansion devices should provide a total seal against penetration and moisture. Standard bridge drawings are available for expansion devices for typical bridge superstructure types.

For fabricated steel expansion devices, the designer should specify the type of steel required. Type of steel should be included as a plan note if requirements in the plans are not covered by a selected standard bridge drawing.

To protect steel expansion devices, metallizing of the exposed surfaces with a 100% zinc coating shall be specified. Standard bridge drawings define the requirements for metallizing. The design agency will need to develop plan notes for special expansion devices, such as finger joints and modular joints. Use the note for shop-applied metallizing located in the appendix as a guideline. Consult the Office of Structural Engineering for recommendations prior to completion of the project plans.
306.1.1 PAY ITEM

Expansion devices, except as specifically listed in this section, shall be paid for as Item 516.

For sealed expansion devices the elastomeric seal, either strip or compression, shall be included in the pay Item 516.

The plans shall clearly show what components are included with the expansion devices, Item 516. As an example, cross frames, which are field welded to both the superstructure girders and the expansion devices, are part of the 513 structural steel item. The seal is considered part of the expansion device and should be included in the 516 pay item.

306.1.2 EXPANSION DEVICES WITH SIDEWALKS

On structures with sidewalks, the expansion devices shall be the same type as furnished for main bridge deck expansion joint.

Sidewalk details for standard expansion devices (strip seals) are shown on the standards. For non-standard devices, a curb plate and sidewalk cover plate will be required. The Curb and sidewalk plates should be separated at the interface of the sidewalk and curb. See details on Standard Bride Drawings: EXJ-2-81, EXJ-3-82, EXJ-4-87, EXJ-5-93 and EXJ-6-95 for sidewalk plates.

306.1.3 EXPANSION DEVICES WITH STAGE CONSTRUCTION

On projects involving stage construction, joints in the seal armor must be located and shown in the plans. At the stage construction lines, expansion devices should require complete penetration welded butt joints. If butt welds will be in contact with a sealing gland the butt-welded joint shall be ground flush at the contact area.

306.2 EXPANSION DEVICE TYPES

306.2.1 ABUTMENT JOINTS IN BITUMINOUS CONCRETE, BOX BEAM BRIDGES

This poured joint seal system is capable of small expansion movements, up to 3/16". Refer to AS-1-15 Sheet 2, Detail A. This joint system requires including the following bid item in the structure estimated quantities: Item 409 - Sawing and Sealing Bituminous Concrete Joints.
MINIMUM SUPPORT LENGTH MEASUREMENTS

TYPICAL ABUTMENT BEARING - PLAN VIEW
(STEEL GIRDER SHOWN, PRESTRESSED BEAM SIMILAR)

ΔX

COS θ

SECTION A-A

NOTES:
ΔX IS THE MAXIMUM MOVEMENT ALLOWED BY THE ELASTOMERIC BEARING BEFORE SLIDING OCCURS.
N AS DEFINED BY LRFD EQ. 4.7.4.4-1.

FIGURE 301.4.4.1.α-1
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**Type-1**

**Type-5**

**Type-6**

**Type-12**

**Type-17**

**Type-18**

The bar size number is specified on the plans in the bar mark column. The first digit where three digits are used, and the first two digits where four are used, indicates the bar size number. For example, P601 is a No. 6 bar. Bar dimensions shown are out to out unless otherwise indicated. R indicates inside radius, unless otherwise noted, "STD." written in place of a dimension indicates a standard bend at the end of the bar.

All reinforcing steel to be epoxy coated.

Figure 301.5.2-1
602.1  LRFD LOAD MODIFIERS

For bridges with non-redundant components, the following note shall be included:

[602.1-1]  REDUNDANCY: The following item(s) were considered non-redundant for design and include a load modifier equal to 1.05 in accordance with the AASHTO LRFD Bridge Design Specifications, Article 1.3.4:

NOTE TO DESIGNER:
Include a list of all items considered non-redundant for design in accordance with BDM Section S1.3.4.

For bridges with non-redundant foundation components, the following notes shall be included:

[602.1-2]  REDUNDANCY: The piles supporting the following substructure(s) were considered non-redundant for design and include a modified resistance factor equal to (1) in accordance with the AASHTO LRFD Bridge Design Specifications, Article 10.5.5.2.3:

[602.1-3]  REDUNDANCY: The drilled shafts supporting the following substructure(s) were considered non-redundant for design and include a modified resistance factor equal to (1) in accordance with the AASHTO LRFD Bridge Design Specifications, Article 10.5.5.2.4:

NOTE TO DESIGNER:
Include a list of all substructures with pile foundations or drilled shafts considered non-redundant for design in accordance with AASHTO LRFD 10.5.5.2.3 & 10.5.5.2.4.

(1) Provide the modified resistance factor value. This should be equal to 80% of the resistance factor used for design on redundant pile foundations.

For all bridges the following note shall be included:

[602.1-4]  OPERATIONAL IMPORTANCE: A load modifier of __ has been assumed for the design of this structure in accordance with the AASHTO LRFD Bridge Design Specifications, Article 1.3.5 and the ODOT Bridge Design Manual, 2007.

NOTE TO DESIGNER:
Refer to BDM Section S1.3.5 for guidance.

602.2  DESIGN LOADING

For bridges designed for highway loads, the design loading shall be:

[602.2-1]  DESIGN LOADING: HL-93
Future Wearing Surface (FWS) of 0.060 kips/ft\(^2\).

For bikeway/pedestrian bridges that will not accommodate vehicular traffic the design loading shall be:

[602.2-2] DESIGN LOADING: 0.090 kips/ft\(^2\)

For bikeway/pedestrian bridges subject to vehicular traffic the design loading shall be:

[602.2-3] DESIGN LOADING: 0.090 kips/ft\(^2\) and H15-44 vehicle

602.3 DESIGN STRESSES

A. General Design Data:

[602.3-1] DESIGN DATA:

Concrete Class (1) - compressive strength 4.5 ksi (superstructure)
Concrete Class (2) - compressive strength 4.0 ksi (substructure)
Concrete Class QC5 - compressive strength 4.0 ksi (drilled shaft)
Reinforcing steel - minimum yield strength 60 ksi
Structural Steel - ASTM A709 Grade (3) - yield strength (3) ksi
Steel H-piles - ASTM A572 - yield strength 50 ksi

NOTE TO DESIGNER:

Modify note [602.3-1] as necessary. Delete references that are not applicable to project.

(1) Class QC2 Concrete for superstructure
(2) Class QC1 Concrete for substructure
(3) Grade 50 - yield strength 50 ksi, or
   Grade 50W - yield strength 50 ksi, or
   Grade HPS70W - yield strength 70 ksi, or
   Grade 36 - yield strength 36 ksi

If more than one grade of steel is selected, the description shall clearly indicate where the different grades are used in the structure.

B. Additional Design Data for Prestressed Concrete Members:

Provide the following note in addition to note [602.2-1].
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SECTION 1000 - ODOT SUPPLEMENT TO THE LRFD BRIDGE DESIGN SPECIFICATIONS

This section of the Bridge Design Manual is the ODOT Supplement to the current edition of the AASHTO LRFD Bridge Design Specifications. Designers shall use this section of the Bridge Design Manual as a complement to the AASHTO LRFD Bridge Design Specifications. This section contains ODOT exceptions and commentary to various provisions as well as recommendations for optional provisions. Supplemented AASHTO articles are identified by the letter “S” preceding the article number (e.g. S1.3.3 DUCTILITY, SA13.4.1 DESIGN CASES, etc.). References to AASHTO articles are presented in italics (e.g. 1.3.3 DUCTILITY, A13.4.1 DESIGN CASES, etc.). References to ODOT Bridge Design Manual sections are always preceded with the initials BDM (e.g. BDM Section 201.2).

1001 LRFD SECTION 1 - INTRODUCTION

S1.3.3 DUCTILITY

For bridges and bridge components designed in accordance with the AASHTO LRFD Bridge Design Specifications, apply a ductility load modifier ($\eta_D$) equal to 1.00 for all limit states.

S1.3.4 REDUNDANCY

Non-redundant designs should be avoided.

For the strength limit state only, apply a redundancy load modifier ($\eta_R$) equal to 1.05 for all elements and components designated as non-redundant. For elements and components designated as redundant, apply a redundancy load modifier ($\eta_R$) equal to 1.00 for all limit states.

The main members of superstructure types (a) and (k) as defined in Table 4.6.2.2.1-1 consisting of three or fewer longitudinal girder lines shall be considered non-redundant. The main members of type (a) and (k) superstructures consisting of four longitudinal girder lines spaced at 12.0 ft. or more shall be considered non-redundant. Type (a) and (k) superstructures with four longitudinal girder lines spaced at less than 12.0 ft and type (a) and (k) superstructures with five or more longitudinal girder lines regardless of spacing shall be considered redundant. NCHRP Report 406, Redundancy in Highway Bridge Superstructures offers additional guidance for determining redundancy of other superstructure types.

The columns of single-column and two-column piers shall be considered non-redundant. The columns of cap-and-column piers with three or more columns shall be considered redundant. The stems of T-type piers with a stem height-to-width ratio of 3-to-1 or greater shall be considered non-redundant. Stems of wall-type and T-type piers, except as noted above, shall be considered redundant. NCHRP Report 458, Redundancy in Highway Bridge Substructures offers additional
guidance for determining redundancy of other substructure types.

Refer to *LRFD 10.5.5.2.3* for more information regarding redundancy of driven pile foundations. Refer to *LRFD 10.5.5.2.4* for more information regarding redundancy of drilled shaft foundations.

When determining redundancy in members, consideration shall only be given to the final design condition; temporary construction phases should be ignored.

The redundancy modifier shall be applied at the component level. For example, for a two-girder Type (a) or (k) superstructure, the 1.05 load modifier would apply only to the design of the girders. The redundancy modifier applied to the two-girder superstructure’s deck, crossframes, expansion joints, bearings, substructure and foundation elements should be considered independently.

Designers are required to submit the Structure Type Study review submission and the Stage 2 review submission for bridge designs that include non-redundant or fracture critical elements to the Office of Structural Engineering. Refer to BDM Section 201.2 and BDM Section 301.2 for more information.

**S1.3.5 OPERATIONAL IMPORTANCE**

A bridge shall be considered a typical bridge with an operational importance load modifier ($\eta_I$) equal to 1.00 for all limit states except as noted below.

For bridges meeting one of the following criteria an operational importance load modifier ($\eta_I$) equal to 1.05 shall be applied at the strength limit states to all components except: railings; concrete slab-type superstructures; and concrete decks on beams and girders. An importance load modifier ($\eta_I$) equal to 1.00 shall be used for all other limit states.

A. Design ADT of 60,000 or greater, or
B. Detour length of 50 miles or greater, or
C. Any span length of 500 ft. or greater.

For bridges meeting both of the following criteria, an operational importance load modifier ($\eta_I$) equal to 0.95 may be applied at the strength limit states to all components except concrete decks on beams and railings. An importance load modifier ($\eta_I$) equal to 1.00 shall be used for all other limit states.

A. Design ADT of 400 or less, and
B. Detour length of 10 miles or less.

The Detour length shall be shortest route available to emergency vehicles if the bridge is taken out of service.
**1002 LRFD SECTION 2 – GENERAL DESIGN AND LOCATION FEATURES**

**S2.3.2.2 PROTECTION OF USERS**

For routes with design speeds in excess of 45 mph, pedestrian traffic and vehicular traffic shall be separated by a crash tested barrier system. For routes with design speeds of 45 mph or lower, the Department requires a crash tested barrier to separate vehicle and pedestrian traffic when the pedestrian railing does not meet NCHRP 350 crash testing requirements. Refer to BDM Section 304 for more information.

**S2.3.3.2 HIGHWAY VERTICAL**

The Department’s requirements for vertical clearance are provided in the ODOT Location & Design Manual, Section 300. ODOT’s “Preferred” vertical clearances include 6.0 in. for possible future overlays.

Apply the additional 1.0 ft. of vertical clearance provided for sign supports and pedestrian overpasses to ODOT’s “Preferred” vertical clearance.

**S2.5.2.3 MAINTAINABILITY**

For structures with High Load Multi-rotational (HLMR) bearings, the plans shall show the location of permanent or temporary jacking points and provide jacking forces in accordance with LRFD 3.4.3.1. Both the superstructure and substructure shall be designed for the location and forces provided. Jacking points and forces are not required for other bearing types.

**S2.5.2.4 RIDEABILITY**

Where concrete decks without an initial overlay are used, the top 1.0 in. of thickness shall be considered sacrificial to permit a maximum correction of the deck profile by grinding of 0.5 in. and to compensate for a maximum thickness loss due to abrasion of 0.5 in. This top 1.0 in. is commonly referred to as the monolithic wearing surface. Refer to BDM Section 302.1.3 for more information.

**S2.5.2.6.2 CRITERIA FOR DEFLECTION**

Designers shall apply the deflection limits shown. Do not include the stiffness contribution of railings, sidewalks and median barriers into the design of the composite section.
S2.5.2.6.3 **OPTIONAL CRITERIA FOR SPAN-TO-DEPTH RATIOS**

Designers shall apply the minimum span-to-depth ratios shown in *Table 2.5.2.6.3-1*.

S2.6.6.3 **TYPE, SIZE AND NUMBER OF DRAINS**

Refer to Section 1103 of the Location and Design Manual, Volume 2 for ODOT’s design criteria for deck drainage.

S2.6.6.4 **DISCHARGE FROM DECK DRAINS**

ODOT requires the minimum projection of scuppers below the lowest adjacent superstructure component to be 8.0 in. Refer to Standard Bridge Drawing GSD-1-96 for more information.
LRFD SECTION 3 – LOADS AND LOAD FACTORS

S3.4.1 LOAD FACTORS AND LOAD COMBINATIONS

The load combinations and load factors specified in Table 3.4.1-1 shall apply. If a bridge design warrants the use of a special design vehicle analysis, the scope of services will provide the necessary information. Otherwise, the Department does not require an analysis using a special design vehicle, and the Strength II limit state will not apply.

S3.5.1 DEAD LOADS: DC, DW, AND EV

In lieu of more specific information, the assumed unit weight of normal weight reinforced concrete shall be 0.150 kcf.

Design all bridges for a future wearing surface of 60 psf applied to the clear roadway width between curbs and/or barriers. Refer to BDM Section 301.4 for more information.

S3.6.1.3.1 GENERAL

The investigation of load effects produced by two tandem vehicles spaced from 26.0 ft. to 40.0 ft as specified in Article C3.6.1.3.1 is not required.

S3.6.1.3.2 LOADING FOR OPTIONAL LIVE LOAD DEFLECTION EVALUATION

The live load deflection criteria specified in Article 2.5.2.6.2 applies.

S3.6.1.3.3 DESIGN LOADS FOR DECKS, DECK SYSTEMS, AND THE TOP SLABS OF BOX CULVERTS

Use the approximate strip method of analysis. Do not apply the Empirical Design Method specified in Article 9.7.2. Refer to BDM Section 302.2.2 for more information.

S3.6.1.3.4 DECK OVERHANG LOAD

This article does not apply. Design deck overhangs in accordance with BDM Section 302.2.2.

S3.6.1.4.2 FREQUENCY

The ADTT shall be estimated as follows:

\[ ADTT = ADTT_{20} \times 4 \]
Where:

\[ ADTT = \text{the number of trucks per day in one direction averaged over the design life} \]

\[ ADTT_{20} = \text{the number of trucks per day in one direction occurring in the design year (year 20)} \]

**S3.6.1.6 PEDESTRIAN LOADS**

For bridges that can accommodate service vehicles, refer to BDM Section 301.4.1 for loading requirements.

**S3.6.2.1 GENERAL**

For deck joints at all limit states, the Dynamic Load Allowance, IM, shall be taken as 125% of the static effect of either the design truck or the design tandem.

**S3.6.5.1 PROTECTION OF STRUCTURES**

Use the flow chart provided in BDM Figure S3.6.5.1-1 to determine protection requirements for substructures against vehicle collisions. Roadway geometry and/or accident experience, either at the site or at a comparable site, may be used to override the flow chart to determine inclusion or omission of protection.

**S3.10.2.1 GENERAL PROCEDURE**

When required by BDM Section 301.4.4 to determine the seismologic data for a project location, Designers may use the Seismic maps, *LRFD Figure 3.10.2.1-3* or the USGS US Seismic Design Maps web application using latitude and longitude.

**S3.10.2.2 SITE SPECIFIC PROCEDURE**

This procedure is not required for Ohio.

**S3.10.3.1 SITE CLASS DEFINITIONS**

In the absence of sufficient geotechnical information, Designers shall assume Site Class D for the project soil profile.

Designers shall use blow counts corrected to an equivalent rod energy ration of 60%, \( N_{60} \) as defined in the ODOT Specifications for Geotechnical Explorations for the average SPT blow count \( 
\bar{N} \).
S3.10.3.2 SITE FACTORS

Use the Site Factors for Site Class D in lieu of the Site Factors provided in LRFD 3.10.3.2 for Site Class E and F. All other Site Class types shall use Site Factors as defined in LRFD 3.10.3.2.

S3.10.6 SEISMIC PERFORMANCE ZONES

All bridges in the state of Ohio are located within Seismic Performance Zone 1.

S3.10.9.2 SEISMIC ZONE 1

For bridges assigned to Seismic Performance Zone 1, determine the minimum support length according to BDM Section 301.4.4.1.a and the horizontal design connection force according to LRFD 3.10.9.2 and BDM Section 301.4.4.1.b.

S3.10.9.5 LONGITUDINAL RESTRAINERS

For bearing areas of existing structures, especially at intermediate superstructure hinges, not meeting the minimum support length requirements defined in BDM Section 301.4.4.1.a, longitudinal restrainers are required. Restrainers shall be capable of permitting movements at service and strength limit states before engaging. Restraint loading shall be in accordance with BDM Section 301.4.4.1.b.

S3.11.2 COMPACTION

The Department typically ignores the effect of additional earth pressure from mechanical compaction equipment on retaining walls. For situations requiring special compaction equipment by plan note, proposal note or special provision, contact the Office of Structural Engineering for additional guidance.

S3.11.6.5 REDUCTION OF SURCHARGE

Do not reduce the Live Load Surcharge regardless of the presence of an approach slab.
S3.11.8    DOWNDRAG

Refer to BDM Section 202.2.3.2.c for more information.

S3.12.2    UNIFORM TEMPERATURE

To determine the thermal effects for all bridges, use the following ranges of temperatures:

A. Steel or Aluminum ................................................................. -30° to 120°F
B. Concrete ............................................................................... 15° to 95°F
C. Wood .................................................................................... 0° to 75°F

The base construction temperature assumed for design shall be 60°F.
LRFD SECTION 4 – STRUCTURAL ANALYSIS AND EVALUATION

S4.4 ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS

This Manual identifies various design conditions that require specific methods of analysis. Where analysis methods are dictated by this Manual, the Designer shall provide justification during the staged review process for designs that utilize alternative analysis methods. This justification shall include impacts to project cost and schedule; safety; constructability; etc. The Department reviewer may consult with the Office of Structural Engineering to determine the appropriateness of the justification. The Department is not responsible for engineering costs incurred as a result of unjustified alternative analysis methods. Where analysis methods are not dictated by this Manual, the selection of an appropriate analysis method utilized for the design of new structures is the responsibility of the Designer.

Regardless of the analysis method utilized for design, superstructures are required to be load rated in accordance with BDM Section 900. At the inventory level, the minimum rating factor for the HL-93 loading shall be 1.0.

Listing design software used for structural analysis in the structure general notes is not required.

S4.5.1 GENERAL

Do not include the stiffness contribution of structurally continuous composite railings, curbs, elevated medians and barriers in the structural analysis.

S4.6.2.1 APPLICATION

Use the following live load distribution factor application guidelines for Table 4.6.2.2.1-1 and typical ODOT bridge types:

<table>
<thead>
<tr>
<th>Typical ODOT Bridge Type</th>
<th>Applicable Table 4.6.2.2.1-1 Cross-section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel beam/girder</td>
<td>(a)</td>
</tr>
<tr>
<td>Concrete I-beam</td>
<td>(k)</td>
</tr>
<tr>
<td>Composite Box beam</td>
<td>(f)</td>
</tr>
<tr>
<td>Non-composite box beam</td>
<td>(g)</td>
</tr>
</tbody>
</table>

* - Use distribution factors that assume beams are connected only enough to prevent relative displacement at the interface. The tie rods specified in Standard Bridge Drawing PSBD-1-93 do not supply sufficient force to ensure units act together.

The 3.0 ft. limit specified for the roadway part of the overhang, $d_e$, does not apply to the determination of the interior distribution factor for cross-sections (a) and (k).
**S4.6.2.5 EFFECTIVE LENGTH FACTOR, K**

In the absence of a refined analysis, the following values for G, as defined in C4.6.2.5, may be assumed:

A. For spread footings on rock ................................................................. G = 1.5
B. For spread footings on soil ................................................................. G = 5.0
C. For footings on multiple rows of piles or drilled shafts:
   - End Bearing ....................................................................................... G = 1.0
   - Friction ............................................................................................. G = 1.5
D. For footings on a single row of drilled shafts or friction piles ................. G = 1.0
E. For footings on a single row of end bearing piles .................................. refined analysis required

For columns supported on a single row of drilled shafts or friction piles, the effective column length shall include the unbraced length above grade and the depth below grade to the point of fixity. Refer to Article 10.7.3.13.4 to determine the depth to the point of fixity. For drilled shafts socketed into rock, the point of fixity should be no deeper than the top of rock.

The list above assumes that typical spread footings on rock are anchored when the footing is keyed at least 3 in. into rock including unweathered shale.

**S4.6.3 Refined Methods of Analysis**

Refer to S4.4 for limitations placed on refined analysis methods.

**S4.6.4.3 Approximate Procedure**

The approximate procedure for moment distribution as described in Appendix B6 is permitted. Moment redistribution as described in Article 5.7.3.5 is not permitted.

**S4.7.4.1 General**

If the Designer elects to determine the seismic effects based on the stiffness of the structure and ground acceleration data in lieu of using the connection forces defined in LRFD 3.10.9.2 and BDM Section 301.4.4.1.b, modal analysis shall be performed in accordance with LRFD 4.7.4.3.2.

**S4.7.4.4 Minimum Support Length Requirements**

The minimum support length requirements shall be applied according to BDM Section 301.4.4.1.a.
LRFD SECTION 5 – CONCRETE STRUCTURES

S5.4.2.3.3 SHRINKAGE

Designers shall assume the relative humidity to be 70% in the absence of more precise data.

S5.4.3.3 SPECIAL APPLICATIONS

BDM Section 301.5 and C&MS 509.02 specify all reinforcing steel for structures to be epoxy coated.

S5.4.4.2 MODULUS OF ELASTICITY

Designers shall assume the modulus of elasticity for prestressing strand to be 28,500 ksi in the absence of more precise data.

S5.5.3.1 GENERAL

Fatigue need not be investigated for the design of longitudinal edge beams of slab bridges.

S5.7.3.4 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT

Unless otherwise noted below, the exposure factor ($\gamma_e$) for reinforcing steel in cast-in-place concrete bridge decks and cast-in-place slab bridges shall be 0.75. The one inch monolithic wearing surface, BDM Section 302.1.3.1, shall be deducted from both $d_c$ and $h$.

S5.7.3.5 MOMENT REDISTRIBUTION

Moment redistribution is not permitted.

S5.7.4.6 SPIRALS AND TIES

The ratio of spiral reinforcement to total volume of the concrete core ($\rho_s$) shall be taken as:

$$\rho_s = \frac{4A_S\sqrt{p^2 + (\pi d_S)^2}}{\pi p d_C^2}$$

Where:
- $A_S$ = Area of spiral reinforcement (in$^2$)
- $p$ = Spiral pitch (in)
- $d_S$ = Centerline diameter of the spiral reinforcement = $d_C$ – Bar dia. (in)
- $d_C$ = Diameter of the concrete core measured out-to-out of spiral reinforcement (in)
S5.8.4.2 COHESION AND FRICTION

The top surface of composite prestressed concrete beams produced under Item 515 is intentionally roughened to an amplitude of 0.25 in.

S5.9.4.2.2 TENSION STRESSES

Designers shall assume a severe corrosive environment to determine the tensile stress limit for components with bonded prestressing tendons in non-segmentally constructed bridges.

S5.9.5.3 APPROXIMATE ESTIMATE OF TIME-DEPENDENT LOSSES

Approximate methods to determine time-dependent losses utilizing Eq. 5.9.5.3-1 should be used for the detail design of prestressed members without post-tensioning. Values from Table 5.9.5.3-1 may be used for preliminary design purposes only.

S5.9.5.4 Refined Estimates of Time-Dependent Losses

The refined estimates for time-dependent losses presented in this article may be used for detail design of prestressed members without post-tensioning.

In the absence of more precise data, for prestressed members without post-tensioning, designers may assume the following ages:

A. Age at transfer (t_i) ........................................................................................................ 0.75 days
B. Age at deck placement (t_d) .......................................................................................... 45 days
C. Final age (t_f) ............................................................................................................. 10,000 days

S5.10.2.2 SEISMIC HOOKS

Transverse reinforcement in plastic hinge zones of structures assigned to areas with $0.10 \leq S_{D1} \leq 0.15$ shall be detailed in accordance with LRFD 5.10.11.4.1d. Plastic hinge zones shall be assumed to extend from the top and bottom of the column a distance taken as the greater of:

A. The maximum cross-sectional dimension of the column,
B. One-sixth of the clear height of the column, or
C. 18.0-in.
S5.10.3.1.1 CAST-IN-PLACE CONCRETE

The maximum aggregate size permitted for structural concrete according to C&MS 499 is 1-inch. When a maximum aggregate size required for design purposes differs from C&MS 499, the Designer shall provide a plan note and specify the concrete pay item “As Per Plan”.

S5.10.3.1.2 PRECAST CONCRETE

For prestressed concrete mixes, C&MS 515 allows the use of the following aggregate gradations: No. 57, 6, 67, 68, 7, 78 or 8. Unless more precise data is provided, assume the maximum aggregate size according to the No. 57 gradation shown in Table S5.10.3.1.1-2.

When a maximum aggregate size required for design purposes differs from the gradations specified in C&MS 515, the Designer shall provide a plan note and specify the prestressed concrete pay item “As Per Plan”.

S5.10.3.3.1 PRETENSIONING STRAND

The minimum spacing of pretensioning strand shall be 2.0 in. measured center-to-center of the strands.

S5.10.6.3 TIES

Ties are required for T-type and wall-type piers. Refer to BDM Section 303.3.2.8 for more information.

S5.11.4.3 PARTIALLY DEBONDED STRANDS

Refer to BDM Section 302.5.2.2.d for additional debonded strand requirements.

S5.12.3 CONCRETE COVER

The minimum concrete cover for reinforcing steel shall be provided according to BDM Section 301.5.7. No modification for W/C ratio shall be made.
S5.13.2.2 DIAPHRAGMS

Refer to BDM Section 302.5.2.6 for additional information.

S5.13.4.5.2 REINFORCING STEEL

For 12.0 in., 14.0 in. and 16.0 in. diameter cast-in-place piles, the minimum wall thickness requirements of C&MS 507.06 provide sufficient longitudinal reinforcement to meet Article 5.13.4.5.2. Except as noted in BDM Sections 202.2.3.2.b and 303.3.2.5 for capped pile piers, no additional reinforcement is required. The additional steel required for capped pile piers shall extend from the pier cap to a minimum of 15 ft. below the finished ground line or flow line, but is not required to extend 10.0 ft. below the plane where the soil provides adequate lateral restraint.

The cast-in-place concrete piling clear distance requirements specified in Article 5.13.4.5.2 do not apply to drilled shafts or piles for Capped Pile Piers. Refer to BDM Section 303.4.3 for reinforcing steel requirements in drilled shafts.
LRFD SECTION 6 – STEEL STRUCTURES

S6.4.1 STRUCTURAL STEELS

Refer to BDM Section 302.4.1.1 for steel selection criteria.

S6.4.3.1 BOLTS

The use of ASTM A 490 bolts is prohibited.

S6.6.1.2.3 DETAIL CATEGORIES

All components or details shall be designed for infinite life using the Fatigue I load combination.

S6.6.1.2.5 FATIGUE RESISTANCE

As noted in BDM Section 1006, S6.6.1.2.3 above, all components and details shall be designed for infinite life using the Fatigue I load combination. Use of the Fatigue II load combination for finite life shall be avoided.

S6.6.2 FRACTURE

The CVN requirements specified in C&MS 711.01 meet Temperature Zone 2.

The CVN requirements for HPS 70W steels are not provided in C&MS 711.01, but are included in BDM Appendix note AN-10.
**6.7.2 DEAD LOAD CAMBER**

Design camber shall not include an allowance for deflections caused by future wearing surface.

C&MS 513.06 requires lateral bracing to be detailed to fit in the steel dead load condition with the webs of the primary members plumb.

**6.10.1.1.1b STRESSES FOR SECTIONS IN POSITIVE FLEXURE**

Use the modular ratio (n) values provided in Article C6.10.1.1.1b.

**6.10.1.7 MINIMUM NEGATIVE FLEXURE CONCRETE DECK REINFORCEMENT**

Refer to BDM Section 302.2.4.1 for negative moment deck reinforcement requirements on non-composite members.

**6.10.3.4 CONCRETE PLACEMENT**

The minimum compression flange width requirement specified in C6.10.3.4 shall apply. For additional flange width requirements, refer to BDM Section 302.4.3.3.a.

**6.10.7.3 DUCTILITY REQUIREMENT**

The design haunch should not be included in the determination of $D_p$ or $D_t$.

**6.10.10.1 GENERAL**

All composite designs for new steel beam and girder superstructures shall have shear connectors for the full length of the members.

**6.10.10.1.1 TYPES**

The use of channel sections as shear connectors is not permitted. Refer to BDM Section 302.4.1.15 for more information.

**6.10.10.1.4 COVER AND PENETRATION**

A detail for deep haunches is provided in the 2004 ODOT BDM Section 400.
**S6.10.11.1** **GENERAL**

Violation of the $6t_w$ requirement of this article due to the C&MS 513.13 requirements for clipping stiffeners and stiffener weld terminations is acceptable.

Refer to BDM Sections 304.4.2.2 and 304.4.3.4 for more information.

**S6.10.11.3** **LONGITUDINAL STIFFENERS**

BDM Section 302.4.3.1 prohibits the use of longitudinal stiffeners.

**S6.13.2.4.1** **TYPE**

Refer to BDM Section 302.4.1.14 for hole requirements of galvanized members.

**S6.13.2.6.6** **EDGE DISTANCES**

Minimum edge distances shall be measured from the center of a fastener. Use the edge distances defined in BDM Section 302.4.1.14.b in lieu of those provided in Table 6.13.2.6.6-1.

**S6.13.2.8** **SLIP RESISTANCE**

Refer to BDM Section 302.4.1.14 for more information.

**S6.13.6.1.4a** **GENERAL**

Holes larger than standard holes are required for galvanized members. Refer BDM Section 302.4.2.1 for more information.
LRFD SECTION 7 – ALUMINUM STRUCTURES

No ODOT comments have been made to this article.
1008  LRFD SECTION 8 – WOOD STRUCTURES

No ODOT comments have been made to this article.
LRFD SECTION 9 – DECKS AND DECK SYSTEMS

S9.4.1 INTERFACE ACTION

For non-composite decks, no physical connection method is required.

S9.5.1 GENERAL

Designers shall ignore the structural contribution of concrete appurtenances for all limit states.

S9.6.1 METHODS OF ANALYSIS

The approximate elastic method of analysis specified in Article 4.6.2.1 shall be used. The empirical and refined methods of analysis are prohibited.

S9.7.1.1 MINIMUM DEPTH AND COVER

The minimum depth of a concrete deck is 8.5 inches as specified in BDM Section 302.2.1.

The minimum cover shall be in accordance with BDM Section 301.5.7.

S9.7.1.3 SKewed DECKS

BDM Section 302.2.4.2 reduces this skew limitation to 15°. This allowance does not apply to all superstructure types. Refer to BDM Section 302.2.4.2 for more information.

S9.7.2 EMPIRICAL DESIGN

The Empirical methodology of concrete deck design is prohibited.

S9.7.4 STAY-IN-PLACE FORMWORK

BDM Section 302.2.6 prohibits the use of stay-in-place formwork.

S9.7.5.1 GENERAL

If precast deck slabs are used, the minimum depth (BDM Section 302.2.1) and cover (BDM Section 301.5.7) requirements apply.
1010 LRFD SECTION 10 – FOUNDATIONS

S10.4.2 SUBSURFACE EXPLORATION


Table 10.4.2-1 is superseded by the “Specifications for Geotechnical Explorations”.

S10.5.5.2.3 DRIVEN PILES

For the purpose of determining redundancy in pile groups, a pile group shall be defined as the piles supporting an entire substructure.

The resistance factor ($\phi_{dy,n}$) for a single driven pile in axial compression, installed according to C&MS 507 and 523 shall be 0.70.

Refer to BDM Section 202.2.3.2 for more information.

S10.5.5.2.4 DRILLED SHAFTS

For the purpose of determining redundancy of drilled shaft foundations, entire substructures supported on one or two shafts shall be considered non-redundant. For entire substructures supported on 5 or more drilled shafts, no increase to the resistance factors provided in Table 10.5.5.2.4-1 shall be made.

S10.5.5.3.2 SCOUR

The foundation shall be designed so that the resistance remaining after the scour resulting from the check flood provides adequate foundation resistance to support the Extreme Event II Limit State loads with a resistance factor of 1.0. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.8. The loads applied to the substructure shall include any debris loads occurring during the flood event, but shall not include any loading from ice or collision forces.

S10.6.1.2 BEARING DEPTH

Minimum footing depth guidelines are provided in BDM Section 303.4.1.

S10.7.1.2 MINIMUM PILE SPACING, CLEARANCE, AND EMBEDMENT INTO CAP

Refer to BDM Section 303.4.2.2 for maximum pile spacings for typical ODOT substructure types.
Refer to BDM Section 303.3.3 for specific pile embedment and clearance requirements for typical ODOT substructure types.

**S10.7.1.3 PILES THROUGH EMBANKMENT FILL**

Refer to BDM Section 202.2.3.2.g for pre-drilling requirements.

**S10.7.1.4 BATTER PILES**

Refer to BDM Section 303.4.2.4 for more information.

**S10.7.2.4 HORIZONTAL PILE FOUNDATION MOVEMENT**

Where possible, use battered piles to resist the potential movement due to horizontal forces. Use Article 10.7.2.4 design procedures when Service I loads exceed the horizontal resistance of battered piles.

For drilled shafts, horizontal movement shall be determined according to Article 10.7.2.4. Battered drilled shafts are not permitted.

**S10.7.3.2.3 PILES DRIVEN TO HARD ROCK**

For piles driven to refusal on bedrock, acceptable refusal occurs at a blow count of at least 20 blows per inch.

**S10.7.3.3 PILE LENGTH ESTIMATES FOR CONTRACT DOCUMENTS**

Determine pile lengths for contract documents in accordance with BDM Section 202.2.3.2.

**S10.7.3.6 SCOUR**

Determine the required pile resistance and estimated lengths for piling driven through material subjected to scour in accordance with BDM Section 202.2.3.2.h.

**S10.7.3.7 DOWNDRAG**

Determine the required pile resistance and estimated lengths for piling subjected to downdrag loading in accordance with BDM Section 202.2.3.2.c.
S10.7.3.8.2  STATIC LOAD TEST

Refer to C&MS 506 to determine the axial pile resistance for the test piles.

S10.7.9  TEST PILES

Refer to BDM Sections 303.4.2.5 and 303.4.2.6 for specific ODOT pile testing requirements. Refer to BDM Section 303.4.2.1 to determine pile order lengths.

S10.8.1  GENERAL

Refer to BDM Section 202.2.3.3 for specific ODOT drilled shaft considerations.

S10.8.1.2  SHAFT SPACING, CLEARANCE, AND EMBEDMENT INTO CAP

The minimum center-to-center spacing of axially-loaded, rock-socketed drilled shafts is 2.0 diameters. The minimum center-to-center spacing of axially-loaded, friction drilled shafts is 3.0 diameters. No further evaluation of axially-loaded shafts for interaction effects is required.

The interaction effects for laterally-loaded shafts shall be considered according to Article 10.8.2.3.

Refer to C&MS 524 for construction requirements for closely spaced drilled shafts.

S10.8.1.4  BATTERED SHAFTS

The use of battered shafts is prohibited.
LRFD SECTION 11 – ABUTMENTS, PIERS, AND WALLS

S11.5.1 GENERAL
The design life for MSE walls shall be 100 years.

S11.6.1.3 INTEGRAL ABUTMENTS
The maximum structure length for integral abutments shall be in accordance with BDM Section 205.8.

S11.6.1.6 EXPANSION AND CONTRACTION JOINTS
BDM Section 303.2.5 does not require contraction joints in abutments.

S11.7.2.2 COLLISION WALLS
Refer to BDM Section 209.8 for more information.

S11.10 MECHANICALLY STABILIZED EARTH WALLS
Refer to BDM Sections 204.6 and 303.5 for more information.

S11.10.2.1 MINIMUM LENGTH OF SOIL REINFORCEMENT
BDM Section 204.6.2.1 further defines the minimum length of soil reinforcement as the larger of: 70% of the wall height or 8’-0”. The reinforcement length shall be uniform for the entire height of the wall facing.

S11.10.2.2 MINIMUM FRONT FACE EMBEDDMENT
The minimum depth shall be as defined in BDM Section 204.6.2.1. Table C11.10.2.2-1 does not apply.

S11.10.8 DRAINAGE
Impervious membranes shall not be used.
11.10.11 MSE ABUTMENTS

The minimum distances from facing panels to abutments and footings do not apply. Refer to BDM Section 204.6.2.1 for more information.

Refer to BDM Sections 204.4 and 204.6.2.1 for additional information.
LRFD SECTION 12 – BURIED STRUCTURES AND TUNNEL LINERS

No ODOT comments have been made to this article. Refer to the ODOT Location and Design Manual, Volume Two.
LRFD SECTION 13 – RAILINGS

S13.4 GENERAL

Refer to BDM Section 304.1 for ODOT bridge railing warrants and railing design considerations.

S13.7.2 TEST LEVEL SELECTION CRITERIA

Acceptance levels for standard ODOT bridge railings are listed in BDM Section 304.2.

S13.8.1 GEOMETRY

If vandal protection fence is required, the fabric mesh shall have 1 inch maximum openings. Refer to BDM Section 305 for more information.

SA13.4.1 DESIGN CASES

Observations made during full-scale crash testing indicate that the wheel closest to the point of impact loses contact with the surface of the pavement during the impacting event. For the duration of the impacting event, the vertical component of vehicle weight for the wheel closest to the point of impact is negligible. For Design Case 1 and 2, the load factor for vehicular live load (LL), including dynamic load allowance (IM), acting on the overhang shall be taken as 0.0 for the wheel load closest to the barrier and 1.0 for the wheel load furthest from the barrier. The position of the truck shall be in accordance with Article 3.6.1.3.1.

SA13.4.2 DECKS SUPPORTING CONCRETE PARAPET RAILINGS

For Design Case 1, the deck overhang shall be designed to resist a vehicular impact moment, $M_{CT}$, and coincidental axial tension force, $T_{CT}$, calculated as follows:

$$M_{CT} = \frac{RH}{L_c + 2H + 2X} \quad \text{and} \quad T_{CT} = \frac{R}{L_c + 2H + 2X}$$

Where:

- $R$ = Barrier resistance to lateral impact force (kips)
- $H$ = Height of the barrier (ft.)
- $L_c$ = Critical length of yield line failure pattern (ft.)
- $X$ = Lateral distance from toe of barrier to deck design section (ft.)
Assume the barrier resistance (R) to be the lesser of:

A. 1.33 times the transverse force ($F_t$) specified in *Table A13.2-1*, or
B. The calculated parapet resistance specified in *Article A13.3.1*

The transverse force selected for design shall be that which corresponds to the barrier’s crash tested acceptance level (i.e. Test Level). The following table provides design overhang data for standard ODOT barrier types:

<table>
<thead>
<tr>
<th>Barrier System</th>
<th>$L_C$ (ft.)</th>
<th>R (kip)</th>
<th>H (ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBR-1-12</td>
<td>12.7</td>
<td>165.0</td>
<td>3.5</td>
</tr>
<tr>
<td>42” BR-1-13</td>
<td>12.4</td>
<td>165.0</td>
<td>3.5</td>
</tr>
<tr>
<td>36” BR-1-13</td>
<td>8.8</td>
<td>72.0</td>
<td>3.0</td>
</tr>
<tr>
<td>BR-2-98</td>
<td>10.0</td>
<td>72.0</td>
<td>2.5 $(^1)$</td>
</tr>
</tbody>
</table>

(1) For BR-2-98, this height represents the maximum effective height of the railing resistance ($\bar{Y}$). Refer to *Article A13.3.3* for more information.

**SA13.4.3.1 OVERHANG DESIGN**

Refer to BDM Figures 302.2.2-1 and 302.2.2-3 for the deck overhang reinforcement requirements for the TST-1-99 railing system. Alternative railing systems shall be considered for projects that do not meet the design assumptions for BDM Figure 302.2.2-1.
S14.4.1 GENERAL
Irreparable damage in accordance with BDM Section 301.4.4.1.c is permitted.

S14.5.6 CONSIDERATIONS FOR SPECIFIC JOINT TYPES
Refer to BDM Section 306 for standard ODOT joint types and applications.

S14.6 REQUIREMENTS FOR BEARINGS
Refer to BDM Section 307 for preferred ODOT bearing types and applications.

S14.6.3.1 HORIZONTAL FORCE AND MOVEMENT
Bearings without directional seismic restraint are not required to accommodate seismic movements in the unrestrained direction. Irreparable damage due to seismic movement in accordance with BDM Section 301.4.4.1.c is permitted.

S14.6.3.2 MOMENT
For elastomeric bearings utilizing 4 anchor bolts connecting load plates to the bearing seat, \( M_u \), shall be taken as specified by Eq. 14.6.3.2-3.

For elastomeric bearings without anchor bolts and those with 2 anchor bolts centered at the centerline of bearing, no moment will be transferred from the superstructure to the substructure.

S14.6.5.2 APPLICABILITY
A design strategy to reduce seismic design forces that permits the bearing to deform beyond its elastic limits during a seismic event without restraint is acceptable provided loss of span is prevented. As noted in BDM Section 301.4.4.1.b, Designers shall design the mechanisms for the Horizontal Connection Force that transfer horizontally applied superstructure loads to the substructure. These mechanisms are not required at every bearing location.

S14.6.5.3 DESIGN CRITERIA
For bearings without directional seismic restraint, design for the horizontal connection force is not required.
Analysis of the load path for resistance to seismic design forces is not required.

S14.7.5 STEEL-REINFORCED ELASTOMERIC BEARINGS – METHOD B

The preferred design of elastomeric bearings is Method A. Method B is recommended for use when specialized bearings are being considered. Since Method B designs have additional testing requirements versus Method A designs, these additional costs shall be factored into cost comparisons for Method A designs versus Method B designs versus specialized bearing designs.

The contract plans shall specify the method of bearing design. A sample plan note is provided in BDM Section 700.

S14.7.5.3.2 COMPRESSIVE STRESS

The effect of impact shall be ignored.

S14.7.5.3.4 SHEAR DEFORMATION

Designers shall assume the ambient temperature during setting is 60°F [15°C] to calculate \( \Delta_0 \).

S14.7.5.3.7 SEISMIC AND OTHER EXTREME EVENT PROVISIONS

Anchor bolts for restrained elastomeric bearings shall be designed for the horizontal connection forces in the restrained directions in accordance with BDM Section 301.4.4.1.b. Elastomeric bearings, not otherwise requiring anchor bolts, do not require seismic anchoring and irreparable damage is permitted under BDM Section 301.4.4.1.c.

S14.9 CORROSION PROTECTION

Refer to C&MS 516.03 for standard bearing corrosion protection requirements.