



Department of Transportation

BRIDGE DESIGN MANUAL 2020 EDITION

JANUARY 2026

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SECTION 100 – GENERAL INFORMATION

101 INTRODUCTION

101.1 BDM FORMAT

Where the BDM is written in two-column format, the left column represents ODOT structural design specifications and the right column represents commentary. The commentary may include background information; design guidance; or other supplemental information related to a corresponding specification item. Where the BDM is written in full page width format, all content represents ODOT structural design specifications. ODOT structural design specification items shall apply unless the Owner provides a waiver of the item in writing.

101.2 BDM AUDIENCE

The requirements of the two-column format ODOT Bridge Design Manual (BDM) are written to the “Designer of Record” as defined in BDM Section 101.4. The sentences that direct the Designer of Record to perform work are written as commands. For example, a requirement to design bridges for HL-93 loading would be expressed as, “Design all bridges for an HL-93 loading,” rather than “The Designer of Record shall design all bridges for an HL-93 loading.” In the imperative mood, the subject, “the Designer of Record” is understood.

All requirements to be performed by others have been written in the active voice. Sentences written in the active voice identify the party responsible for performing the action. For example, “The Office of Structural Engineering will review designs for non-standard bridge railings.”

Sentences that define terms; describe a product or desired result; or describe a condition that may exist are written in indicative mood. These types of sentences use verbs requiring no action. For example, “All reinforcing steel shall be epoxy coated.”

101.3 PURPOSE

The BDM establishes ODOT policy for the following:

- A. The analysis, design and rating of new and existing bridges.
- B. The analysis and design of new and existing retaining walls and noise walls.
- C. The analysis and design of new and existing pedestrian structures.

C101.1

The design values, specifications, practices, etc. established in the BDM promote uniform, safe and sound designs for bridges and structures in the State of Ohio. During the normal staged review process, the Owner has the authority to require the Designer of Record to justify or otherwise seek recommendation from the Office of Structural Engineering (OSE) when deviation from the design values, specifications, practices, etc. is necessary.

C101.2

C101.3

Consider the practicability of construction with reference to each detail of design.

Constructability is especially critical when new ideas are considered.

101.4 COORDINATION WITH CONTRACT DOCUMENTS

C101.4

Ohio Department of Transportation bridge designs shall be in conformance with the following publications listed in descending order of precedence:

Other ODOT bridge design publications (e.g. Standard Bridge Drawings, Design Data Sheets and Plan Insert Sheets) become ODOT structural design specifications when required by reference from any of the publications listed in BDM Section 101.4.

- A. Scope of Services
- B. ODOT Bridge Design Manual
- C. ODOT [Location and Design Manual, Vol. 1](#), [Location and Design Manual, Vol. 2](#) & [Location and Design Manual, Vol. 3](#); ODOT [Traffic Engineering Manual](#); the ODOT [Pavement Design Manual](#); ODOT [Geotechnical Design Manual](#), and the ODOT CADD Engineering Standards Manual
- D. Proposal Notes and Special Provisions
- E. Supplemental Specifications
- F. ODOT Construction and Materials Specifications
- G. *AASHTO LRFD Bridge Design Specifications* and *AASHTO LRFD Bridge Construction Specifications*

Where complete description or instruction is not provided in the [C&MS](#), provide the description or instruction in the plans to ensure clarity both from a structural and contractual viewpoint.

101.5 TABLE OF ORGANIZATION

An organizational chart for the various sections in the ODOT OSE and a list of bridge contacts is available on the office's [website](#).

101.6 ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ADT	Average Daily Traffic
ADTT	Average Daily Truck Traffic
AISC	American Institute of Steel Construction
AREMA	American Railway Engineering and Maintenance-of-way Association
ASTM	American Society for Testing and Materials
AWS	American Welding Society
BDM	ODOT Bridge Design Manual
CADD	Computer-Aided Drafting and Design
CLP	County Log Point
C&MS	ODOT Construction and Material Specifications
CVN	Acceptable Charpy V-Notch impact testing of steel material is required
DBT	Design-Build Team

DEC	District Environmental Coordinator
EPA	Environmental Protection Agency
FHWA	Federal Highway Administration
GDM	Geotechnical Design Manual published by the ODOT Office of Geotechnical Engineering
IZEU	Three coat steel coating system consisting of Inorganic Zinc primer coat, Epoxy intermediate coat and Urethane finish coat. Typically used for new steel members.
L&D	ODOT Location and Design Manual
LRFD	Load and Resistance Factor Design. When used in combination with AASHTO, this refers to the current edition of the <i>AASHTO LRFD Bridge Design Specifications</i> . References to specific AASHTO LRFD sections throughout this manual will be italicized (e.g. <i>LRFD 11.10.5</i>)
MSE	Mechanically Stabilized Earth. Typically used as earth retaining wall systems.
NCHRP	National Cooperative Highway Research Program
NDT	Non-Destructive Testing
ODOT	Ohio Department of Transportation
OES	Office of Environmental Services
OES-WPU	Office of Environmental Services – Waterway Permits Unit
OHWM	Ordinary High Water Mark
OAC	Ohio Administrative Code
OMM	ODOT Office of Materials Management
ORC	Ohio Revised Code
OSE	ODOT Office of Structural Engineering
OZEU	Three coat steel coating system consisting of Organic Zinc primer coat, Epoxy intermediate coat and Urethane finish coat. Typically used for existing steel members.
PID	Project Identification Number
PN###	Proposal Note
RGP	USACE Regional General Permit for ODOT projects impacting Waters of the U.S.
S####	Supplement
SFN	Structure File Number
SGE	Specifications for Geotechnical Explorations published by the ODOT Office of Geotechnical Engineering
SLM	Straight Line Mileage
SPP	Special Provisions Package
SS###	Supplemental Specification
STD	Standard Temporary Discharge
TAF	Temporary Access Fill
TEM	Traffic Engineering Manual
USACE	U.S. Army Corps of Engineers
USCG	U.S. Coast Guard
USGS	United States Geological Survey
VECP	Value Engineering Change Proposal

101.7 DEFINITIONS

Bridge – The ORC 5501.47(B)(1)(c) defines a bridge as any structure of ten feet or more clear span or ten feet or more in diameter on, above, or below a highway. Refer to the Ohio Bridge Inventory Coding Guide for more specific information.

Bridge Limits – The distance between the bridge ends of the approach slabs measured to the nearest hundredth of a foot along the Centerline of Construction.

Centerline of Construction – The reference line used for construction of a project. Normally located at the median centerline on a divided highway or at the normal crown point location on an undivided highway.

Checker – The Checker is an individual employed by the Design Agency responsible for ensuring correctness, constructability and completeness of designs detailed in the Contract Documents.

Contract Documents – Refer to definition provided in [C&MS](#) 101.03.

Culvert – Unless otherwise noted, a culvert shall be defined herein as a buried structure type, regardless of span, placed on, above or below a highway.

Design Agency – The firm, partnership, association, limited liability company, government entity or corporation that employs the Designer of Record in accordance with ORC 4733.

Design Loading – The envelope of load factors and load combinations encompassing the permanent and transient loads acting on the structure as required by the governing design specifications.

Designer – The Designer is an individual employed by the Design Agency responsible for preparing designs detailed in the Contract Documents.

Designer of Record – The Designer of Record is the individual that professionally endorses (signs and seals) the design in the Contract Documents in accordance with OAC 4733-23-01(C) and as specified in the [L&D, Vol. 3](#) Section 1302.6.1.

Haunch – The distance between the top of a beam/girder and the bottom of the deck.

Invert – The bottom or lowest point of the internal surface of the transverse cross section of a pipe or conduit

Normal Water Elevation – Water elevation within a waterway produced by groundwater flow and not influenced or minimally influenced by surface water runoff. The Normal Water Elevation shown in the plans is approximate and will fluctuate seasonally and from year to year.

Ordinary High Water Mark – Defined by the USACE Regulatory Letter No. 05-05 which is available at: www.usace.army.mil/Missions/CivilWorks/RegulatoryProgramandPermits/GuidanceLetters.aspx. This elevation marks the jurisdictional limit of the USACE.

Owner – The Owner shall be defined as the duly authorized agent of the government agency sponsoring the project. For ODOT sponsored projects, the sponsoring District shall be considered the Owner.

Pedestrian Structure – A structure which is designed for and intended to primarily carry pedestrians, bicyclists, equestrian riders, and light maintenance vehicles, but not designed for and intended to carry typical highway traffic.

Proposal Notes – Specifications that supplement and supersede Supplemental Specifications and the [C&MS](#).

Project Manager – Individual selected by the sponsoring agency responsible for advancing the project through the development process. Refer to ODOT's Project Development Process Manual for specific duties.

Reference Line – A chord drawn from centerline to centerline of abutment bearings at the Centerline of Construction or an extension of a line tangent to the Centerline of Construction at the centerline of abutment bearings.

Rehabilitation – Title 23 Part 650 Section 403 of the Code of Federal Regulations (i.e. 23 CFR 650.403) defines “Rehabilitation” as the major work required to restore the structural integrity of a bridge as well as work necessary to correct major safety defects. Examples include: deck replacement, fatigue retrofit, structural patching, bearing replacement and railing upgrade.

Reviewer – The Reviewer is an individual employed by the Design Agency responsible for the overall evaluation of the design in the Contract Documents for completeness, consistency, continuity, constructability, general design logic and quality.

Skew angle – The skew angle is the angle of deviation of the substructure unit from perpendicular to the centerline of construction or reference chord.

Supplemental Specifications – Specifications that supplement and supersede the [C&MS](#).

Supplements – Individually numbered documents describing necessary information such as laboratory test methods and certification or pre-qualification procedures for materials.

Temporary Access Fill – 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 work pads; and demolition debris.

Thalweg – The line defining the points of lowest elevation along the length of a riverbed. Also, commonly referred to as the Flow Line.

102 PREPARATION OF PLANS C102

Draw all detail views to a scale large enough to be easily read on 11-in x 17-in sheets. Views shall not be crowded on the sheet and shall fit within the prescribed borderlines.

Do not state the scale of the views on the drawings

Reproduction of the drawing prints may distort the scale resulting in inaccuracies.

Place a North Arrow symbol on the Site Plan, General Plan and all plan views.

The north arrow is useful for orienting field personnel using the plan and may eliminate possible construction errors.

Show Elevation views of piers and the forward abutment looking forward along the stationing of the project. Show the rear abutment in the reverse direction. Detail rear and forward abutments on separate plan sheets for staged construction projects or for other geometric conditions that produce asymmetry between abutments.

When describing directions or locations of various elements of a highway project, use the centerline of construction and stationing as a basis for these directions and locations.

Number sheets in the bridge plans in accordance with BDM Figure 102-2.

For structures on a horizontal curve, provide a Reference Line. Show the Reference Line on the General Plan/Site Plan view with the stations of the

The purpose for dimensioning the distance between the Reference Line and centerline of the substructure units is to make a check available to the contractor.

points where the Reference Line intersects the curve. Provide skews, dimensions of substructure elements and superstructure elements from this Reference Line, both on the General Plan /Site Plan and on the individual detail Plan /Site Plan and on the individual detail sheets. Do not providing dimensions from the curve. Dimension the distances between the curve and Reference Line at each substructure units.

For each substructure unit, show the skew angle with respect to the centerline of construction or, for curved structures, to a Reference Line. Measure the skew angle from the centerline of construction or Reference Line to a line perpendicular to the centerline of the substructure unit or from a line perpendicular to the centerline of construction or Reference Line to the centerline of the substructure unit as shown in BDM Figure 102-1.

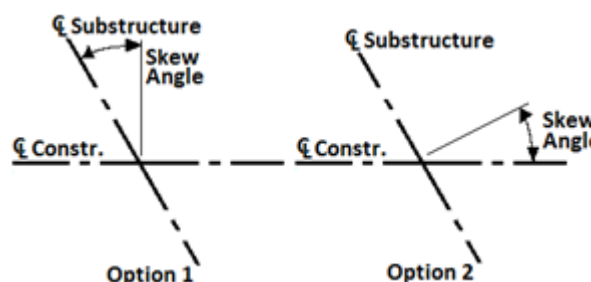


Figure 102-1

In placing dimensions on the drawings, provide sufficient overall dimensions so that it will not be necessary for a person reading the drawings to add up dimensions to determine the length, width or height of an abutment, pier or other element of a structure.

Do not show a detail or dimension in more than one place on the plans.

Unnecessary duplication increases the risk of errors, particularly where revisions are made at a later date.

If, because of lack of space on a particular sheet, it is necessary to place a view or a section on another sheet, provide cross references on both sheets.

Define all abbreviations in a legend provided on the General Notes sheet.

Plan sheets shall be 22-in x 34-in. Margins shall be 2-in on the left edge and 1/2-in on all other edges.

Keep design and check calculations neat and orderly.

This provision allows for calculations that are easily followed and understood by a person other than the preparer.

102.1 BRIDGE PLAN PREPARER QUALIFICATIONS

The Design Agency shall be prequalified in accordance with the “Consultant Prequalification Requirements and Procedures” published by ODOT’s Office of Consultant Services.

102.2 COMPUTER AIDED DRAFTING STANDARDS

For drafting standards, refer to [L&D, Vol. 3](#).

**102.3 DESIGNER, CHECKER, REVIEWER
INITIALS BLOCK**

Provide individual Designer, Checker and Reviewer initials and the date of the final review in the title block of each sheet.

102.4 TITLE BLOCK

See BDM Figure 102-2 for an example of bridge plan sheets.

Provide the project name in CO-Route-Section format. Show the Section as SLM to the nearest 1/100 of a mile.

Provide the bridge number as defined in BDM 905.

A Station is defined as 100-ft. Show Stations to the nearest 1/100 of a foot.

Show the correct SFN in the Existing Structure Block and Title blocks for the existing and proposed structures respectively.

Provide the PID below the project name in the Title Block.

Include the name and address of the Design Agency in the Title Block.

C102.4

The Office of CADD and Mapping provides MicroStation cells for bridge plan sheets in the OHDOT_Sheets.cel library.

Example: MUS-70-10.49

Example: MUS-00070-10.890

Example: STA 895+08.75

The SFN for the existing structure should be included in the Scope of Services. If not, it may be obtained from the Owner.

If a new SFN is required for a proposed structure, contact the Office of Structural Engineering, Bridge Management Section.

It is the Designer of Record's responsibility to contact and confirm the correct SFN with the appropriate office.

Figure 102-2

102.5 ESTIMATED QUANTITIES

Provide plan quantities separately for each bridge SFN. Do not incorporate common bid items between multiple bridge structures in a project.

Furnish electronic files of the structure quantity calculations in accordance with the [L&D, Vol. 3](#). Prepare neat, accurate and complete quantity calculations on standard computation sheets. Arrange the calculations in an orderly fashion so that a person examining them will be able to follow the calculation sequence. Someone other than the original Designer shall independently check the calculations. The Designer and Checker shall each prepare separate sets of calculations to minimize risk of error. Reconcile the two sets of calculations and select one set (either the Designer's or the Checker's) as the official set. Both the Designer and Checker shall initial and date each sheet of the calculations. The results of this official set shall correspond to the quantities shown on the plans.

Provide the Bridge Number; SFN; the date; the individual Designer and Checker initials; and subject on each sheet of quantity calculations.

102.6 STANDARD BRIDGE DRAWINGS

Where required by the Scope of Services and/or BDM, follow current Standard Bridge Drawings. Provide reference to Standard Bridge Drawings using the Drawing Number and latest date of revision or the approval date if there has been no revision. If standards are not adequate for a particular design, then modify the standards, as necessary, by supplying pertinent details, dimensions or material specifications in the plans.

Reference Standard Bridge Drawings on the General Notes sheet and the project plan's Title Sheet.

Do not reference a standard drawing if only one or two small details on the standard are applicable. In such instances, include the required details in the project plans. Do not reference more than one standard drawing for a individual bridge component.

102.7 SUPPLEMENTAL SPECIFICATIONS

Supplemental Specifications are specifications that supplement and supersede the [C&MS](#).

Modifications to Supplemental Specifications require plan notes and "As Per Plan" pay items.

C102.5

Summation of common items cannot be done due to computer tracking of quantities based on Structure File Number.

Since this information may be useful to contractors preparing bids, the Department provides it as a courtesy at the time of advertisement. This information is not considered part of the bidding documents and is for information only.

Sheets that are accidentally misplaced are sometimes very difficult to identify if they do not contain this information.

C102.6

Standard Bridge Drawings provide common details and requirements for structures in Ohio. Most of the drawings require fully detailing the items in the plans to provide clear and bid-worthy plans. The Designer shall be familiar with the standards and know if they are adequate for the particular design situation being addressed. Standard Bridge Drawings can be downloaded from ODOT's Office of Structural Engineering website.

Reference to a standard drawing when very little of the drawing applies to a project can be confusing and lead to contract disputes.

C102.7

Supplemental Specifications are available on ODOT's Division of Construction Management website.

List Supplemental Specifications, like standard bridge drawings, on the General Notes plan sheet. (See BDM Section 600 below). List Supplemental Specifications referenced on the General Notes sheet on the project plan's Title Sheet.

102.8 PROPOSAL NOTES

Proposal Notes are specifications that supplement and supersede Supplemental Specifications and the [C&MS](#).

Do not revise Proposal Notes.

Reference Proposal Notes in the table of Estimated Quantities by adding a note to the end of the applicable bid item (See Proposal Note). Do not refer to Proposal Notes by number.

102.9 SPECIAL PROVISIONS

Special Provisions are project specific specifications with the same order of precedence in the Contract as Proposal Notes. Provide the title and date for all Special Provisions on the Title Sheet of the Plans.

102.10 SUPPLEMENTS

Supplements are individually numbered documents describing necessary information such as laboratory test methods and certification or pre-qualification procedures for materials.

Do not reference Supplements in the bridge plan General Notes or the project plan's Title Sheet.

103 BRIDGE PLAN SHEET ORDER

A set of completed bridge plans shall conform to the following order:

- A. Site Plan
- B. General Plan – A General Plan sheet is required for any of the following:
 - 1. Bridges with variable width and/or curved alignment
 - 2. Bridges with staged construction
 - 3. Bridge deck overlay projects
- C. General Notes
- D. Estimated Quantities
- E. Phase Construction Details
- F. Abutments

C102.8

The Department's numbered Proposal Notes were developed to assure uniform specifications for specific items that may not be required on every project or are either experimental or of an interim status.

Proposal Notes are available on ODOT's Division of Construction Management website.

Ensure the Proposal Note is only specified where applicable.

C102.9

Consider converting General Notes that are lengthy such that multiple plan sheets would be required into Special Provisions.

C102.10

Where necessary, Supplements are referenced as needed by the [C&MS](#) and Supplemental Specifications.

C103

The purpose of the General Plan sheet is to provide bridge information that can get lost in the clutter of information typically shown on a site plan. The General Plan sheet does not require an elevation view.

- G. Piers
- H. Superstructure
- I. Railing Details
- J. Expansion device details
- K. Non-standard approach slab details
- L. Concrete Reinforcement Bar List

104 STRUCTURAL REVIEWS

The purpose of a review for each of the Structure Submissions defined in BDM Section 200 is to ensure compliance with a project's Scope of Services. If a Structure Submission is required by the Scope of Services, do not omit a compliance review of the submission from the Project Development Process.

Upon request of the Project Manager, the Design Agency shall provide project records including, but not limited to: design calculations, software files, survey data, and project correspondence, in a timely manner.

The Design Agency shall provide to the Project Manager, a written response for every review comment provided in a Structure Submission. The Project Manager will provide a copy of the written response to each compliance reviewer. The project shall not proceed to the next stage of the development process until the Project Manager is satisfied that all comments have been resolved. Resolution of a Structure Submission does not absolve the Designer of Record from liability for Errors or Omissions. Ultimately, the Designer of Record bears the responsibility for the content of the design plans.

C104

Compliance with the Scope of Services means compliance with the following: all referenced contract documents; all defined aspects of the project; schedule; and budget. These reviews are not intended as a computational or analysis check of the contents of the design plans. However, the compliance reviewer may perform an independent analysis or request design calculations, software files or other project records, to verify that a design is compliant.

For ODOT-let projects, the performance of the structural reviews is the responsibility of the District Project Manager assigned to administer the project. The Project Manager may: delegate the review to a technical expert at the District; request review services from ODOT's Central Office; or contract review services from a prequalified consulting firm.

The Department typically obligates at least a 30-day window from Stage Submittal to Stage Completion. Within that timeframe the Project Manager will transmit the submittal to various reviewers for comments to be addressed by the Design Agency and resolved to the Project Manager's satisfaction. The Project Manager should be aware that resolution of review comments can delay a project's Staged completion date.

An acceptable Design Agency written response to a structure review is: "We concur with all comments except the following:"

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SECTION 200 – STRUCTURE SUBMISSIONS

201 STAGED REVIEW PROCESS

201.1 STRUCTURE TYPE STUDY

201.1.1 GENERAL

Where a bridge is the most appropriate structure for a particular site, then perform the Structure Type Study to determine the appropriate bridge type.

Provide a separate Structure Type Study for each SFN on a project. For identical parallel bridges that require separate SFN's in accordance with the Ohio Bridge Inventory Coding Guide, develop a full Structure Type Study for one of the bridges. The Structure Type Study for the twin structure shall refer to the full type study to avoid duplication of effort.

201.1.2 SUBMISSION REQUIREMENTS

A Structure Type Study submission shall include the following:

- A. Profile for each bridge alternative, see BDM Section 201.1.2.11
- B. Preliminary Structure Site Plan for preferred bridge alternative, see BDM Section 201.1.2.2
- C. Hydrology and Hydraulics (H&H) Report, see [L&D, Vol. 2](#).
- D. Narrative of Bridge Alternatives see BDM Section 201.1.2.3.
- E. Cost Analysis see BDM Section 201.1.2.4.
- F. Foundation Recommendation see BDM Section 201.1.2.5.
- G. Preliminary Maintenance of Traffic Plan, see BDM Section 201.1.2.6

Include the Structure Type Study in the review submission made directly to the District Office.

201.1.2.1 PROFILE FOR BRIDGE ALTERNATIVE

Draw the profile for each bridge alternative being considered to the same scale as the corresponding plan.

Include the following for each profile:

- A. Existing and proposed profile grade lines

C201.1.1

Refer to BDM 101.7 for the definition of what qualifies as a bridge per the ORC. Study the project site in detail to determine the best structure alternative. Conducting a site visit is highly recommended. In many cases, it can be readily determined whether a bridge or culvert structure should be chosen for a particular site.

The purpose for this requirement is to ensure that every bridge on a project receives an independent Structure Type Study. For bridges that do not require an SFN, a Structure Type Study is still required by the opening paragraph of BDM Section 201.1.1. At the approval of the Project Manager, an independent Type Study may be provided for the twin structure.

C201.1.2.1

This scale is the same as that shown for the preferred alternative on the Preliminary Structure Site Plan. The scales are to be the same for comparative purposes.

- B. Existing ground line
- C. An outline of the structure
- D. Bridges conveying riverine streams or channels require the following hydraulic information:
1. Cross-section of waterway channel
 2. Highest known high-water mark
 3. Normal water elevation
- Refer to BDM 101.7 for Normal Water Elevation, OHWM and Thalweg definitions.
- Normal Water Elevation is an estimate of where the water elevation will be when not influenced by surface water runoff. Plan designer should establish the Normal Water Elevation from waterway physical characteristics and observed water elevations when surface runoff is minimal or not present. No recurrence interval runoff event corresponds to the Normal Water Elevation. The Normal Water Elevation will be lower than the OHWM elevation.
- The Normal Water Elevation, shown in the bridge profile view, provides bidding contractors with understanding of the physical characteristics of a waterway and that water is likely present even when no surface water runoff is occurring. Identification of the NWE, OHWM and design storm event water elevations provides information as to how the channel responds to varying occurrences of surface water runoff and can be critical in planning and bidding in stream work.
4. OHWM
 5. Thalweg elevation
 6. Scour Information
- Examples of scour information would be scour history, scour holes, etc.
7. Design and 100-year water surface elevations (WSE)
 8. Overtopping flood elevation and frequency
- Flood Insurance Studies use the 100-year flood as the baseline. Carry WSE in a FEMA Zone out to two decimal places
- E. Bridges conveying waterbodies that have controlled outlets or spillways require:
1. Cross-section of the channel
 2. Summer pool elevation
 3. Winter pool elevation
 4. Surveillance pool elevation
 5. Record pool elevation
 6. Spillway elevation
 7. Top of dam elevation
- Refer to [L&D, Vol. 2](#) for more information related to these waterways.
- F. Existing and proposed profile grade elevations at 25-ft increments

G Minimum and required vertical and lateral clearances.

H. Vertical clearance of wildlife path(s), if applicable.

I. Spans dimensioned as follows:

1. Spans on straight (tangent) alignments: measure from center to center of bearings along the centerline of construction.
2. Spans on curved alignments: measure along the Reference Line.
3. Spans for standard concrete slab bridges: measure from a line 1-ft behind the face of abutment substructure or breastwall.

J. Existing and proposed (if known) utility lines running perpendicular to the structure.

201.1.2.2 SITE PLAN

The Site Plan scale shall be 1-in = 20-ft. For structures with bridge limits exceeding 400-ft, match-mark the site plan across multiple sheets at the same scale.

The following general information shall be shown on the Preliminary Structure Site Plan:

A. The plan view shall show:

1. Existing structures (using dashed lines)
2. Contours at 2-ft intervals showing the existing surface of the ground (for steep slopes contours at 5-ft or greater intervals are acceptable)
3. Existing utility lines and their disposition. Identify and accurately locate all utilities. Indicate the disposition of existing utilities (e.g. whether they are to remain in place, be relocated or be removed, and for the latter two, by whom).
4. Proposed structure
5. Proposed temporary bridge
6. Proposed channel improvements
7. North arrow
8. Required minimum and actual minimum lateral and vertical clearances and their locations.
9. All other pertinent features concerning the existing topography and proposed work.

C201.1.2.2

For bridges where the 1-in = 20-ft scale is too small to clearly show the Site Plan details, a 1-in = 10-ft scale may be considered.

Avoid placing utilities on bridges where possible. The request to allow utilities on the bridge shall be made through the ODOT District Utilities Coordinator. Refer to the ODOT Utilities Manual. For specific detail requirements refer to BDM 310.4.

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-ft 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 BDM Section 201.1.2.1.
The profile scale shall be the same as the plan view.
- C. Horizontal and vertical curve data.
- D. Bridges conveying riverine streams or channels require the following information:
 - 1. Elevations described in BDM Section 201.1.2.1.D.
 - 2. Size of drainage area
 - 3. Design year flood frequency, discharge and velocity
 - 4. Scour Design Flood & Scour Check Flood
 - 5. 100-yr discharge and velocity. Label discharge as “FIS” when taken from a FEMA Flood Insurance Study.
 - 6. Clearance from the lowest elevation of the bottom of the superstructure to the design year water surface elevation (i.e. freeboard)
- E. Bridges conveying waterbodies that have controlled outlets or spillways require the following information:
 - 1. Elevations described in BDM Section 201.1.2.1.E.
 - 2. Size of drainage area
 - 3. Water body name
 - 4. Owner of outlet or spillway
 - 5. Include a note indicating that the pool elevation is controlled by the outlet or spillway
- F. Buried culvert bridges with an invert: In addition to the information required in BDM Sections 201.1.2.2.D and E, provide the water pH and abrasion level (1-6). The required design service life is 75 years in accordance with L&D Volume 2, Section 1002.2.2.
- G. Proposed Structure block: Located in the lower right-hand corner of the sheet. The block shall be 6-1/2-in wide on a 22-in x 34-in sheet. The block shall include the following information:
 - 1. Type: provide a brief description of proposed bridge type
 - 2. Span(s): provide the span lengths and how measured (c/c of bearings, f/f of abutments) See BDM Section 201.1.2.1.H for additional information.
 - 3. Roadway: provide the roadway and sidewalk widths and how measured (t/t of bridge railing, t/t of curb, or f/f of railing)

- | | |
|--|---|
| <p>4. Vehicular Live Load: Provide the Vehicular Live Load designation used for the proposed design.</p> | <p>If multiple Vehicular Live Load designations are applicable to the proposed structure, list all that apply. For each designation listed, provide the portion of the structure to which it applies. If necessary to conserve space on the Site Plan, place “See General Notes” in the Proposed Structure Block and provide the applicable designations there.</p> |
| <p>5. Future Wearing Surface: Provide the Future Wearing Surface allowance (kip/ft²) used for proposed design</p> | <p>If a future wearing surface is not included for the structure, then report the future wearing surface as zero.</p> |
| <p>6. Skew: provide the skew angle and how measured (right forward or left forward)</p> | |
| <p>7. Approach Slabs: provide the length, thickness, and standard drawing number.</p> | |
| <p>8. Alignment: provide description of horizontal alignment across the bridge (e.g. tangent or degree of curvature and how measured)</p> | |
| <p>9. Crown or Superelevation: provide the rate of cross-slope</p> | |
| <p>10. Coordinates: provide the latitude and longitude for the proposed structure as defined in the Ohio Bridge Inventory Coding Guide and measured to the nearest 0.01 seconds.</p> | |
| <p>11. Deck Area as defined in the Ohio Bridge Inventory Coding Guide and taken to the nearest 1-ft².</p> | |
- H. Existing Structure block: Located in the lower right-hand corner of the sheet above the Proposed structure block. The block shall be 6-1/2-in wide on a 22-in x 34-in sheet. The block shall include the following information:
1. Type: provide a brief description of existing bridge type
 2. Span(s): provide the span lengths and how measured (c/c of bearings, f/f of abutments)
 3. Roadway: provide the roadway width and how measured (t/t of bridge railing, t/t of curb, or f/f of railing)
 4. Vehicular Live Load: Provide the Vehicular Live Load designation used for the original design
 5. Skew: provide the skew angle and how measured (right forward or left forward)
 6. Wearing Surface: provide the type and thickness of the existing wearing surface
 7. Approach Slabs: provide known information regarding length, thickness, std. dwg. number.

8. Alignment: provide description of horizontal alignment across the bridge (e.g. tangent or degree of curvature and how measured)
 9. Crown or Superelevation: provide the rate of cross-slope
 10. Structure File Number: provide the SFN
 11. Date built: provide the year built and year of any major rehabilitations
 12. Disposition: provide the disposition of the existing bridge after the completion of the project
- I. A cross section of the proposed superstructure, including an elevation of the proposed pier type(s) if applicable. This may be shown on a separate sheet and should not be included in subsequent submittals on the site plan.
- J. The design and current average daily traffic (ADT) and the design average daily truck traffic (ADTT). Traffic data is available through the on-line resource: Traffic Monitoring Management System (TMMS). The traffic data provided on the Site Plan shall be reflective of the traffic on the bridge. For example, if the bridge carries one direction of a multi-lane divided highway, the traffic data shall be the directional portion of the total route traffic. Conversely, if the bridge carries two-way traffic, the traffic data shall be the total route traffic. There are many other possibilities, but regardless of the traffic pattern on the bridge, include a clear description of the traffic data provided on the Site Plan to avoid confusion.
- K. In the profile view, for each substructure unit where a bearing is to be used, designate the bearing condition as fixed (FIX), expansion (EXP) or integral (INT). Designate Semi-integral substructures as expansion (EXP) and integral as (INT).
- L. Navigational clearances
- M. A cross section sketch at the abutments to verify bridge limits. This may be shown on a separate sheet and should not be included in subsequent submittals on the site plan.

201.1.2.3 NARRATIVE OF BRIDGE ALTERNATIVES

Include a brief narrative in the Structure Type Study identifying the structure alternatives and their costs. If applicable, include information regarding the use of non-standard bridge railing.

C201.1.2.3

The purpose of the Narrative is to provide insight into why the particular proposed structure was chosen. Factors that need to be considered in selecting a structure for a particular site include geometry, economics, maintainability, constructability, right-of-way constraints, disruption to the traveling public, waterway crossing requirements or grade separations requirements, clearances for railway and highway crossings, foundation considerations, historical and environmental concerns, debris and ice flow problems and appearance.

201.1.2.4 COST ANALYSIS

Include a cost analysis in the Structure Type Study comparing alternative structures. The cost analysis shall consider the initial construction cost and all major future rehabilitation and maintenance costs, converted to present dollars.

201.1.2.5 FOUNDATION RECOMMENDATION

Include a written Foundation Recommendation in the Structure Type Study that consists of the following items:

- A. General foundation type (e.g. Drilled Shafts, Friction Piles, Bearing Piles or Spread Footings)
- B. Preliminary estimates of nominal and factored resistances:

In addition to the above recommendations, include all pertinent geotechnical information gathered for the study area that forms the basis for the Foundation Recommendation.

When the foundation recommendation for the preferred alternative includes MSE wall supported abutments, provide estimates for factored bearing pressure and factored bearing resistance for the in-situ material below the MSE wall.

When considering retaining walls on a project, perform a Retaining Wall Justification, per ODOT [L&D, Vol. 3](#), Section 1407.2, for all retaining walls other than abutments and culvert headwalls. Include an evaluation of feasible retaining wall types in the Retaining Wall Justification, including a comparison of the practicality, constructability and economics of each wall type. See BDM 307 for a description and design guidance for each wall type.

C201.1.2.4

Sufficient preliminary design is necessary for an accurate cost analysis. Construction cost data is available from the ODOT Office of Estimating.

C201.1.2.5

The primary purpose of the foundation recommendation is to identify the probable foundation type for the preferred structure alternative. Specific foundation information such as pile size, estimated pile length, ultimate bearing values, shaft diameter, rock socket dimensions and footing dimensions may be provided, but is not required at this stage.

Performing drilling operations at this stage is not always necessary. Reasonable foundation assumptions for a Structure Type Study may be developed based on existing information gathered during the Office Reconnaissance detailed in the [SGE](#). There may be project locations where sufficient geotechnical information is not available to render an accurate recommendation of foundation type or resistances. For such locations, boring, sampling and field testing according to the [SGE](#) will be necessary at this stage of the project development process

Refer to BDM Section 201.4.1 for additional retaining wall design considerations.

201.1.2.6 PRELIMINARY MAINTENANCE OF TRAFFIC PLAN

Include a Preliminary Maintenance of Traffic Plan in the Structure Type Study.

The various stages of the bridge construction shall match those of the approach roadway, and the nomenclature used to identify the various stages (phases) of construction shall be the same for the roadway and the bridge (Stage 1 and Stage 2 or Phase 1 and Phase 2).

This Plan shall include a transverse section(s) defining all stages of removal and construction with the following information:

- A. The existing superstructure and substructure layout with overall dimensions (field verified) and color photographs.
- B. Type of temporary railing or barrier.
- C. Proposed temporary lane widths, measured as the clear distance between temporary barriers.
- D. Location of cut lines.
- E. Temporary modifications to superelevated sections (existing and/or proposed) on the deck and/or shoulder in order to accommodate the traffic from the phase construction.
- F. Width of closure pour.
- G. Profile grade, alignment, approximate location and width of temporary structures
- H. Location of temporary shoring
- I. Lane closure exceptions.

201.2 BRIDGE PRELIMINARY DESIGN REPORT – STAGE 1

C201.1.2.6

For projects that affect ODOT's Permitted Lane Closure Schedule, evaluate Accelerated Bridge Construction (ABC) techniques to mitigate negative impacts to the travelling public. Access to the Permitted Lane Closure Schedule (maintained by the Office of Roadway Engineering) is available at:

[Permitted Lane Closure Schedule \(PLCS\)](#)

Additional information regarding ABC techniques is available on FHWA's Accelerated Bridge Construction Website:

<http://www.fhwa.dot.gov/bridge/abc/>

For information regarding temporary MOT geometrics refer to ODOT's [TEM](#), Section 640-2.

Evaluate the existing structure to determine where the cut-line can be made to maintain structural integrity.

Refer to BDM 404.5 for additional information on cutline locations for early concrete slab bridges.

Refer to BDM 309.3.8.5 for more information regarding closure pours.

Present alternative concepts when exceptions to the Department's permitted lane closures are necessary.

201.2.1 BRIDGE PRELIMINARY DESIGN REPORT SUBMISSION REQUIREMENTS

Provide a separate Bridge Preliminary Design Report for each SFN on a project. For identical parallel bridges that require separate SFN's in accordance with the Ohio Bridge Inventory Coding Guide, develop a full Bridge Preliminary Design Report for one of the bridges. The Bridge Preliminary Design Report, for the twin structure shall refer to the full report to avoid duplication of effort. However, include a separate Final Structure Site Plan for each SFN. The Bridge Preliminary Design Report shall contain the following:

- A. Final Structure Site Plan, see BDM Section 201.2.1.1
- B. Final Maintenance of Traffic Plan, see BDM Section 201.2.1.2
- C. Foundation Report, see BDM Section 201.2.1.3
- D. Supplemental Site Plan for Railway Crossings, see BDM Section 201.2.1.4
- E. Disposition of comments from the Structure Type Study
- F. Non-standard bridge railing, see BDM 309.4.1.

201.2.1.1 FINAL STRUCTURE SITE PLAN

In addition to the Preliminary Structure Site Plan requirements of BDM Section 201.1.2.2, the Final Structure Site Plan shall include the information in the following sections.

201.2.1.1.a PLAN VIEW

The Final Structure Site Plan shall include the following information in the Plan View:

- A. Bridge width and approach pavement widths, showing curb or bridge railing lines and outer limits of the superstructure and substructure units
- B. Skew angle of each substructure unit
- C. The directional bearing of the centerline of construction for tangent alignments or the directional bearing of the Reference Line for curved alignments.

Provide a baseline/workline for structures located on a curve, spiral, or having other complex geometry in the plans along with enough information for the fabricator to meet the requirements of [C&MS 513](#).

- D. Lateral clearances (both the minimum required and the actual) with respect to railroad tracks and/or highways under the proposed structure

C201.2.1

This requirement shall also apply to bridges that do not require an SFN. At the approval of the Project Manager, an independent Bridge Preliminary Design Report may be provided for the twin structure.

C201.2.1.1.a

Refer to BDM Section 201.1.2.2 for additional site plan view information.

[C&MS 513](#) requires the fabricator to include in the Shop Drawings an overall layout with dimensions showing the horizontal position of beam or girder segments with respect to a full-length base or workline. Offsets from this full-length base line are to be provided by the fabricator for each 10-ft of length.

- E. Location of minimum vertical clearance
- F. Treatment of slopes around the ends and under the bridge (location and length of rock channel protection). **Location and width of wildlife path(s), if applicable.**

G. Channel changes

H. Soil boring locations

Use the soil boring symbols for Project Soil Borings and Historic Soil Borings at the correct scale in accordance with the ODOT CADD/ORD cell libraries and [SGE](#) Appendix D. Project Soil Borings are those drilled specifically for the design project; Historic Soil Borings are those drilled previously within the vicinity of the proposed structure.

Provide a Legend adjacent to the Plan View with the same soil boring symbols. Provide the descriptions "Project Boring Location" for Project Soil Borings and "Historic Boring Location" for Historic Soil Borings.

I. Centerline of temporary structure and temporary approach pavement

J. Stationing of Bridge Limits

K. Limits of channel excavation shown by crosshatching with a description provided in a legend

L. The location and description of benchmarks

M. The following earthwork note: EARTHWORK limits shown are approximate. Actual slopes shall conform to plan cross sections.

N. Guardrail stationing

When providing guardrail stationing: for bridges with twin steel tube bridge railing, station the centerline of the first top mounted post on the bridge, for bridges with deep beam railing and concrete bridge railing, station the centerline of the first guardrail post off the bridge. Stationing shall be given to the nearest 1/100th of a foot.

Guardrail stationing is subject to change during the detail design phase. The Site Plan shall be revised accordingly.

This information coincides with the Method of Measurement defined in [C&MS](#) Item 517. These stations are used to determine pay length of roadway approach guardrail which is typically given in multiples of standard w-beam lengths.

201.2.1.1.b PROFILE VIEW

The Final Structure Site Plan shall include the following information in the Profile view:

- A. Profile gradient percent
- B. Embankment slopes and top of slope elevations
- C. Proposed bottom of footing elevations
- D. Type of foundations

C201.2.1.1.b

Refer to BDM Section 201.1.2.2 for additional site plan profile view information.

- E. Top of bedrock elevations at each boring location (if encountered in the boring). Show the top of bedrock elevation with line style "ground" in [ODOT L&D Volume 3](#), Section 1204.2.4, or provide a table of values.)
- F. Shaded areas of the bridge that represent the new bridge components.

201.2.1.1.c SUPERELEVATION TRANSITION

Provide a superelevation transition table or diagram similar to BDM Figure 309-11, if the bridge crown changes across the structure. When detailing the typical bridge transverse section, provide a reference to the table or diagram.

201.2.1.2 FINAL MAINTENANCE OF TRAFFIC PLAN

In addition to the Preliminary Maintenance of Traffic Plan requirements of BDM Section 201.1.2.6 the Final Maintenance of Traffic Plan shall include the following information:

- A. Plan view
- B. Temporary barrier anchorage details and requirements
- C. Location and type of temporary shoring (See BDM Section 310.1)
- D. Location of structural members that require strengthening
- E. Temporary structure design information (See BDM Section 500)
- F. Additional notes and/or details that will be necessary to perform the work

201.2.1.3 STRUCTURE FOUNDATION EXPLORATION REPORT

Include a Structure Foundation Exploration Report in accordance with the [SGE](#) in the Bridge Preliminary Design Report. The Structure Foundation Exploration Report shall include:

- A. Summary of Exploration
- B. Investigational Findings
- C. Analyses and Recommendations
- D. Boring Logs
- E. Test Data

For the scour evaluation, performed in accordance with the [L&D, Vol. 2](#), provide:

C201.2.1.3

Specific design considerations for each foundation type are presented in BDM Section 305.

- A. For granular (cohesionless) soils, D50 values from the particle size analysis
- B. For cohesive soils, a classification package in accordance with [SGE](#) Table 600-19 (includes Visual Description, Water Content Determination, Particle-Size Analysis, Hydrometer, Liquid Limit, Plastic Limit, and Plasticity Index), and a measurement of unconfined compressive strength
- C. For bedrock, the unconfined compressive strength (Qu), Slake Durability Index (SDI), Rock Quality Designation (RQD), total unit weight, Rock Mass Rating (RMR) or Geologic Strength Index (GSI), and an evaluation of bedrock structure in accordance with BDM Section.305.2.1.2.b.

Where the scour evaluation has identified a potential problem, accommodate the probable scour depths, calculated in accordance with [L&D, Vol. 2](#), for: 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.

Where downdrag has been identified as a potential contributor to the total factored load, include the estimated drag load in the report.

See BDM Section 305.3.2.2 for more information regarding downdrag.

201.2.1.3.a SPREAD FOOTING REPORT REQUIREMENTS

For spread footings, provide the size of the footing, external stability analyses, predicted settlement, eccentric load limitations, factored sliding resistance, Service I Limit State bearing pressure, maximum Strength Limit State bearing pressure, and the factored bearing resistance; provide all supporting calculations for these values. Also provide all relevant plan notes for the footing installation including necessary waiting periods.

201.2.1.3.b PILE FOUNDATION REPORT REQUIREMENTS

For driven pile foundations, provide the size and type of driven piles analyzed, the estimated and order length per pile, the nominal and factored loads per pile, the nominal and factored geotechnical and structural resistance per pile, predicted settlement (magnitude and time-rate), horizontal deflections, the pile driving hammer rated energy used for wave equation drivability analyses, and the maximum driving stresses resulting from the drivability analyses. For the geotechnical resistance, provide the following:

- A. Nominal unit tip resistance, qp (ksf)
- B. Nominal unit side resistance, qs for each soil layer contributing to the nominal pile side resistance (ksf)

C201.2.1.3.b

For friction piles, the nominal axial geotechnical resistance is referred to as Ultimate Bearing Value (UBV).

- C. Design methodologies used to determine unit tip and unit side resistances

Provide supporting calculations for all of the above. Also provide all relevant plan notes for the pile installation including necessary waiting periods.

201.2.1.3.c DRILLED SHAFT FOUNDATION REPORT REQUIREMENTS

For drilled shaft foundations, provide the nominal dimensions and the size and type of reinforcement for the drilled shafts analyzed, the nominal and factored loads per shaft, the nominal and factored geotechnical and structural resistance per shaft, predicted settlement (magnitude and time-rate), and horizontal deflections. For the geotechnical resistance, provide the following:

- A. Nominal unit tip resistance, q_p (ksf)
- B. Nominal unit side resistance, q_s for each soil layer contributing to the nominal shaft side resistance (ksf)
- C. Design methodologies used to determine unit tip and unit side resistances
- D. Resistance factor from BDM Table 305-1 for each calculated unit resistance.
- E. Integrity testing methods (if applicable). Provide a justification when specifying drilled shaft integrity testing methods other than Thermal Integrity Profiling (TIP), such as Gamma-Gamma Logging (GGL) or Pile Integrity Testing (PIT), or if specifying Crosshole Sonic Logging (CSL) testing for drilled shafts of less than 84-in nominal diameter.

Provide supporting calculations for all the above. Also provide all relevant plan notes for the drilled shaft installation.

201.2.1.3.d MICRPILES FOUNDATION REPORT REQUIREMENTS

For micropile foundations, provide for review the nominal dimensions and the size and type of reinforcement for the micropiles analyzed, the nominal and factored loads per pile, the nominal and factored geotechnical and structural resistance per pile, predicted settlement (magnitude and time-rate), and horizontal deflections. For the geotechnical resistance, provide the following:

- A. Nominal unit tip resistance, q_p (ksf)
- B. Nominal unit side resistance, q_s for each soil layer contributing to the nominal pile side resistance (ksf)
- C. Design methodologies used to determine unit tip and unit side resistances

- D. Resistance factor from BDM Table 305-1 for each calculated unit resistance.

Provide supporting calculations for all of the above. Also provide all relevant plan notes for the micropile foundation installation.

**201.2.1.3.e CONTINUOUS FLIGHT AUGER
(CFA) PILE FOUNDATIONS
REPORT REQUIREMENTS**

For Continuous Flight Auger pile foundations, provide for review the nominal dimensions and the size and type of reinforcement for the CFA piles analyzed, the nominal and factored loads per pile, the nominal and factored geotechnical and structural resistance per pile, predicted settlement (magnitude and time-rate), and horizontal deflections. For the geotechnical resistance, provide the following:

- A. Nominal unit tip resistance, q_p (ksf)
- B. Nominal unit side resistance, q_s for each soil layer contributing to the nominal pile side resistance (ksf)
- C. Design methodologies used to determine unit tip and unit side resistances
- D. Resistance factor from BDM Table 305-1 for each calculated unit resistance.

Provide supporting calculations for all of the above. Also provide all relevant plan notes for the CFA pile installation.

**201.2.1.3.f MSE WALL REPORT
REQUIREMENTS**

For MSE wall supported abutments provide findings for external stability (*LRFD 11.10.5*) and settlement. Also provide soil reinforcement lengths, and all construction constraints, such as soil improvement methods, that will be required.

Provide supporting calculations for all of the above. Also provide all relevant plan notes for the MSE wall installation.

201.2.1.4 SUPPLEMENTAL SITE PLAN FOR RAILWAY CROSSINGS

For Railway-Highway grade separation structures, provide a Supplemental Site Plan with the Final Site Plan.

Show the following information in the Supplemental Site Plan:

- A. A 1-in = 100-ft scale plan of the alignment of the railroad and the highway extended at least 1000-ft each way from the proposed point of intersection, taken from actual surveys.
- B. Profile of top of all rails, extending at least 1000-ft each way from the proposed intersection.
- C. Minimum lateral and vertical clearances to all tracks.
- D. Sufficient cross sections along the railroad and highway to determine approximate earthwork limits and encroachment on railroad property.
- E. Drainage.
- F. Intersection angle between highway centerline and railroad centerline.
- G. Highway stationing and railroad mile post stationing at intersection.
- H. Railroad right-of-way lines.
- I. Railroad pole lines, signal control boxes, communications relay houses, signal standards and drainage structures.
- J. Centerlines of all tracks and location of switch points.
- K. Location of buildings or other structures within the railroad right-of-way.
- L. Location of all utilities occupying railroad right-of-way and the names of the owners of these utilities.

C201.2.1.4

The railway traffic counts have been removed to avoid duplication and possible conflicts as these are provided in the railroad agreements

201.3 WATERWAY PERMIT DETERMINATION

201.3.1 BRIDGE AND WATERWAY PERMITS

C201.3.1

Impacts to bridges or waterways may require legal authorization in the form of permits or certifications issued by various regulatory agencies, including:

- A1. U.S. Army Corps of Engineers – Section 404, Section 408 and/or Section 10 Permit
- B1. U.S. Coast Guard – Section 9 Bridge Permit
- C1. Ohio EPA – Section 401 Water Quality Certification and/or Isolated Wetland Permit

The U.S. Army Corps of Engineers (USACE) has jurisdiction over “Waters of the United States” and, as noted in ODOT [C&MS](#) 101.03, this includes: rivers, streams, jurisdictional ditches, lakes and wetlands. For rivers, streams and jurisdictional ditches, the jurisdiction begins at the OHWM. The OHWM is defined by the USACE Regulatory Letter No. 05-05 and is available at the following website:

<https://usace.contentdm.oclc.org/utis/getfile/collection/p16021coll9/id/1253>

Coordinate with the DEC and the OES-WPU throughout the waterway permit process to ensure that the final waterway permit applications are indicative of the final project design. For more information refer to the Waterway Permits Manual available on the ODOT Office of Environmental Services website.

Confirm with the project manager that the bridge design plans meet the requirements in the project waterway Special Provisions (SP) and ensure the project waterway SP is submitted with the Final Plan Package.

The ODOT Office of Environmental Services – Waterway Permits Unit (OES-WPU) assumes the responsibility for determining which types of waterway permits apply to ODOT projects, such as, but not limited to, Nationwide Permits (NWP) or ODOT’s Regional General Permit (RGP).

Special Provisions are the method ODOT uses to attach the waterway permit conditions to the project construction plans. The waterway permits Special Provisions is prepared by the OES-WPU and may contain the following:

- A2. All pertinent waterway permit conditions
- B2. Plan sheets and mapping approved by the regulatory agencies.
- C2. Impact quantities authorized in aquatic resources.
- D2. Section 404 Completion Certification form

201.3.2 TEMPORARY ACCESS FILLS**C201.3.2**

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; however, the Department will still provide Waterway Permit Special Provisions prior to project sale indicating anticipated conditions and the pending permit authorizations. 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.

Use the following requirements to estimate the size of TAF's:

- A. The TAF shall provide access to all piers located within the OHWM of the waterway from at least one bank of the waterway.

The Department partnered with the Ohio Contractor's Association to develop these requirements. 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 calculate worst case scenario TAF impacts, including linear footage from upstream to downstream and acreage, to submit in the Permit Determination Request to OES. An example Plan, Profile and Cross-section using the guidance provided above are shown in BDM Figure 201-1. These quantities should be provided to the DEC along with a completed copy of the checklist shown on the OES Waterway Permit Determination Request website. The minimum flow to be maintained during construction should be calculated according to BDM Section 201.3.2.F. Designers will need to estimate whether this flow can be maintained through conduits or if open channels will be required.

In the case of staging, the permit application shall reflect the construction stage that impacts the largest area 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.

- B. Locate the TAF directly beneath the superstructure. The surface width of the TAF shall be equal to the out-to-out width of the superstructure plus 50-ft outboard on one side of the structure and 20-ft outboard on the other side of the structure.
- C. Extend the TAF at least 40-ft beyond the furthest pier accessed by the TAF.
- D. The side slopes of the TAF shall be no steeper than 1.5:1 (H:V).
- E. Locate the top surface of the TAF at least 1-ft above the OHWM.
- F. Design the TAF to maintain the Standard Temporary Discharge (STD) as detailed in [L&D, Vol. 2](#), Section 1012.

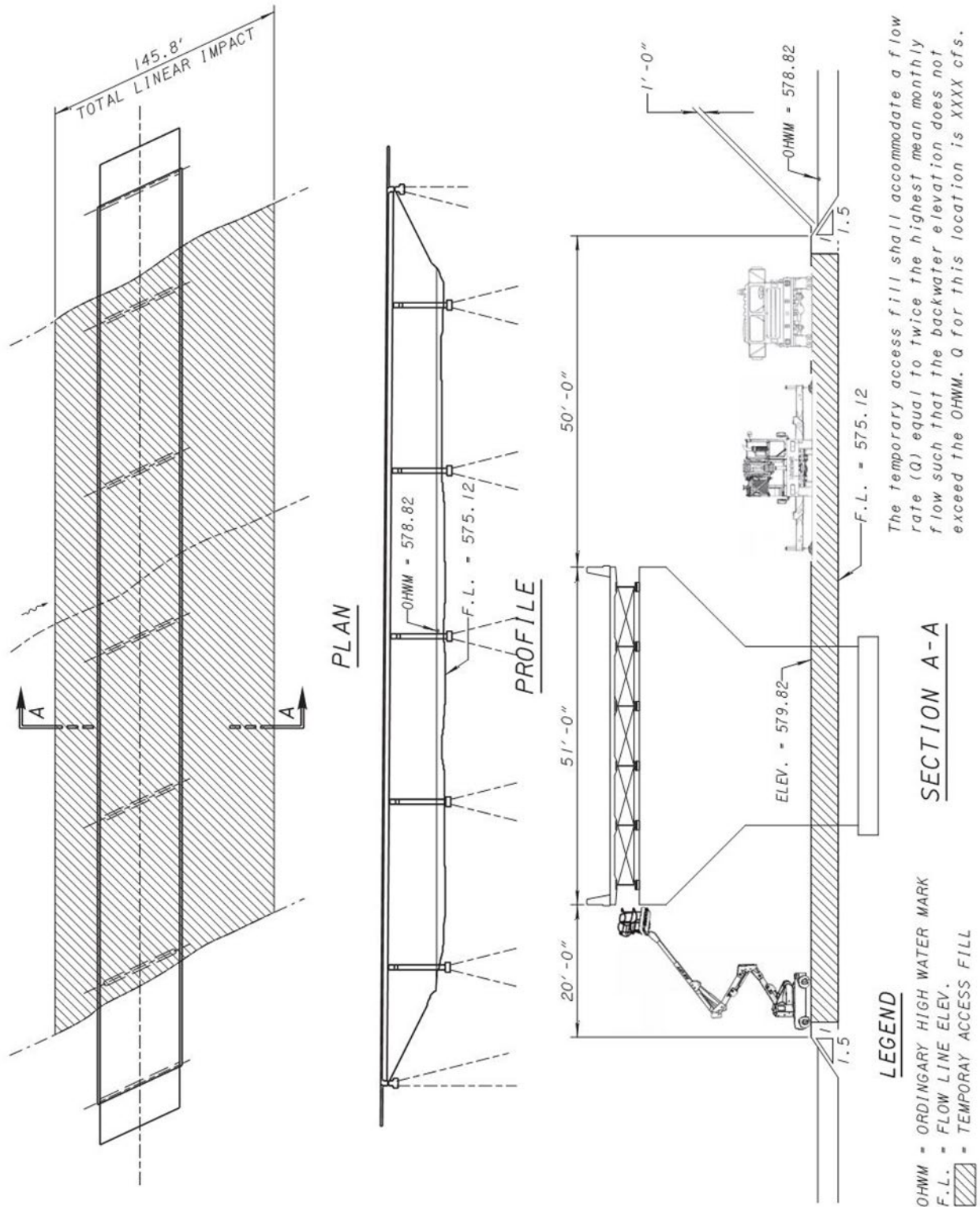


Figure 201-1

201.4 DETAIL DESIGN – STAGE 2

Provide a separate plan set for each SFN on a project. The Stage 2 structure submission for bridges, retaining walls and noise walls shall include a disposition of the comments from the previous submission and the items listed below.

A. Include the following in the Bridge Plans:

1. Site Plan in compliance with all Stage 1 review comments
2. General Plan (if required)
3. General Notes
4. Estimated Quantities showing all pay items and item extensions. Quantity values are not required at this stage.
5. Phase Construction Details (if required)
6. Foundation Plan (if required)
7. Abutment Details with all dimensioning, bar sizes, bar shapes and bar spacings properly shown
8. Pier Details with all dimensioning, bar sizes, bars shapes and bar spacings properly shown (if required)
9. Superstructure Details with all dimensioning, bar sizes, bar shapes and bar spacings properly shown
10. Approach Slab Details with all dimensioning, bar sizes, bar shapes and bar spacings properly shown (if necessary)
11. Concrete Reinforcement Bar Lists. Bar marks, dimensions, quantities and weights are not required at this stage.
12. Other Details as necessary

B. Include the following in the Retaining Wall Plans:

1. General Notes
2. Retaining wall details
3. Other Details as necessary

C. Noise Barrier Plans in accordance with BDM Section 800.

D. Special Provisions

E. Load Rating Reports for bridges

F. Signed Office of Structural Engineering Bridge Stage 2 Plan Review Checklist

C201.4

Not every item listed will apply to every project.

Upon approval of the bridge Owner and Project Manager, plan notes and details for bridge rehabilitation and maintenance work items on multiple bridges may be combined into one plan set with estimated quantities for each bridge shown in the General Summary.

Bridge plans at this stage are essentially complete except for Estimated Quantity values and Concrete Reinforcement Bar Lists. To avoid the potential of wasted effort, completion of these two plan items which are subject to change based on the Stage 2 Plan Review is deferred to Stage 3.

Refer to BDM Section 900 for the Load Rating Report submission requirements at this stage.

201.4.1 RETAINING WALLS**C201.4.1**

Provide a complete design for each retaining wall in accordance with BDM Section 307. The design shall include external stability analysis for the foundations of retaining walls in accordance with BDM Section 305.1.

If specifying a proprietary (prefabricated) retaining wall, provide sufficient information in the plans such that, prior to submitting a bid, the Contractor can select a proprietary company to design the internal stability of the prefabricated retaining wall after the project is awarded.

Detail each wall on a project separately. For MSE wall examples, see BDM Figure 201-2, Figure 201-3, Figure 201-4, Figure 201-5, Figure 201-6, Figure 201-7, Figure 201-8, and Figure 201-9. As a minimum, provide the following information in the project plans for each wall location:

A. Plan View of the wall showing:

1. A wall baseline along the front (exposed) face of the wall
2. For each critical point: station and offset with respect to the centerline of construction
3. All complex geometry information
4. Pay limits for wall and roadway quantities
5. North Arrow
6. Locations of typical sections for (C.) below
7. Locations of abutment footing, piles, utilities, catch basins, and other possible obstructions (Refer to BDM Section 309.7 for drainage and BDM Section 310.4.1 for utility locations)
8. Concrete bridge railing/barrier locations
9. Limits of proposed wall excavation
10. Locations of sheeting and bracing

If sheeting and bracing is required according to BDM Section 310.1, provide a pay item for ITEM 503 – COFFERDAMS AND EXCAVATION BRACING

11. Locations of wall drainage

Locate perforated plastic pipe, [C&MS 707.33](#), wrapped with geotextile fabric as low as possible behind the wall while still providing positive gravity flow in the pipe to an outlet. Locate pipe as near as possible the back side of the leveling pad and near the free end of the soil reinforcement. The pipe shall be continuous and sloped to provide a positive gravity flow to an outlet. Show the approximate location of the outlet on the plan view. Use drainage pipe without perforations outside the wall. See BDM Section 307 for additional guidance on drainage for various wall types. If the wall supports an abutment, provide backfill drainage in accordance with BDM Section 306.2.3.1.

B. Elevation of the wall showing:

1. Station and elevation for each critical point on the wall
2. Finished ground surface elevations for each critical point on the wall
3. Footing or leveling pad showing the minimum dimension from the finished ground line to the top of the footing or pad. If the wall has no footing (e.g. a drilled shaft wall or soil nail wall) show the dimensions of any deep foundation elements and the dimension from the finished ground line to the foundation elevation (or top of deep foundation elements).
4. Locations of abutment footing, piles, utilities, catch basins, and other possible obstructions
5. Backfill drainage
6. Approximate locations of slip joints, if applicable

C. Typical Sections showing:

1. Coping details, if applicable
2. Concrete bridge railing and sleeper slab details
3. Abutment footing details including the dimensions from the back of the wall stem or face to the centerline of bearing at the abutment, dimensions from the back of the wall stem or face to the toe of the abutment footing, and dimensions from the back of the wall stem or face to the centerline of the nearest row of piles.

4. For MSE walls, minimum clearance between the bottom of the footing/sleeper slab and the uppermost wall reinforcement strap. The preferred dimension is 6-in.
 5. Locations of abutment footings, piles, utilities, catch basins, and other possible obstructions
 6. Backfill drainage
 7. For MSE walls, soil reinforcements attached to abutments. These additional reinforcements are necessary to resist horizontal bridge and backwall forces and prevent load transfer to the coping and facing panels. To estimate select granular backfill quantities, Designers may assume these additional reinforcements are the same length as those attached to the facing panels.
Regardless of abutment type and foundation type, one row of soil reinforcements shall be attached to the backside of the abutment footing.
 8. For MSE walls, limits of select granular material.
The top elevation of the select granular backfill shall be at least 6-in above the uppermost layer of soil reinforcement, but not lower than 6-in above the bottom of the abutment footing.
 9. Limits of wall excavation
For MSE walls, [SS840](#) requires a minimum 1-ft undercut beneath the leveling pad elevation. If more undercut is required, show it on the plans. The backfill material is specified in [SS840](#).
For prefabricated modular retaining walls, [SS870](#) requires a minimum 1-ft undercut beneath the wall modules or concrete leveling pad elevation. If more undercut is required, show it on the plans. The backfill material is specified in [SS870](#).
 10. Pay limits of wall and roadway quantities
 11. Pile sleeves (if required)
Show pile sleeves extending from the bottom of the abutment footing to the bottom of the wall excavation.
 12. Location of sheeting and bracing (if required)
 13. Limits of concrete sealer
- D. Requirements for wall surface textures or other aesthetic treatments.
- For prefabricated retaining walls, provide the panel or module size and shape restrictions specific to the project in the plans.

E. Wall design criteria including:

1. Factored bearing resistance at the base of the footing or reinforced soil mass, or at the foundation elevation for other types of walls and wall foundations.
2. The following factored loads applied to the wall or reinforced soil mass from the bridge: vertical dead and live loads, horizontal loads, and total bearing load.

Plan notes for MSE walls are provided in BDM Section 606.5. Plan notes for spread footings (for the wall types in BDM Section 307.2 are provided in BDM Section 606.6.

F. Final copy of the Special Provisions for proprietary wall types, if applicable.

G. Estimated Quantities Table (list each wall on a project separately)

For proprietary wall types, include all necessary pay items listed in the applicable Special Provisions: [SS840](#) for MSE walls; SS851 for precast gravity and semigravity retaining walls, [SS867](#) for wire faced MSE walls; [SS870](#) for prefabricated modular retaining walls. Also include the following pay items as necessary: ITEM 203 EMBANKMENT; ITEM 203 GRANULAR EMBANKMENT; ITEM 503 UNCLASSIFIED EXCAVATION; ITEM 503 COFFERDAMS AND EXCAVATION BRACING; ITEM 509 EPOXY COATED REINFORCING STEEL; ITEM 511 CLASS QC1 CONCRETE WITH QC/QA, FOOTING; ITEM 512 SEALING OF CONCRETE SURFACES (EPOXY URETHANE); ITEM 518 PREFABRICATED GEOCOMPOSITE DRAIN; ITEM 518 CORRUGATED POLYETHYLENE SMOOTH LINED PIPE.

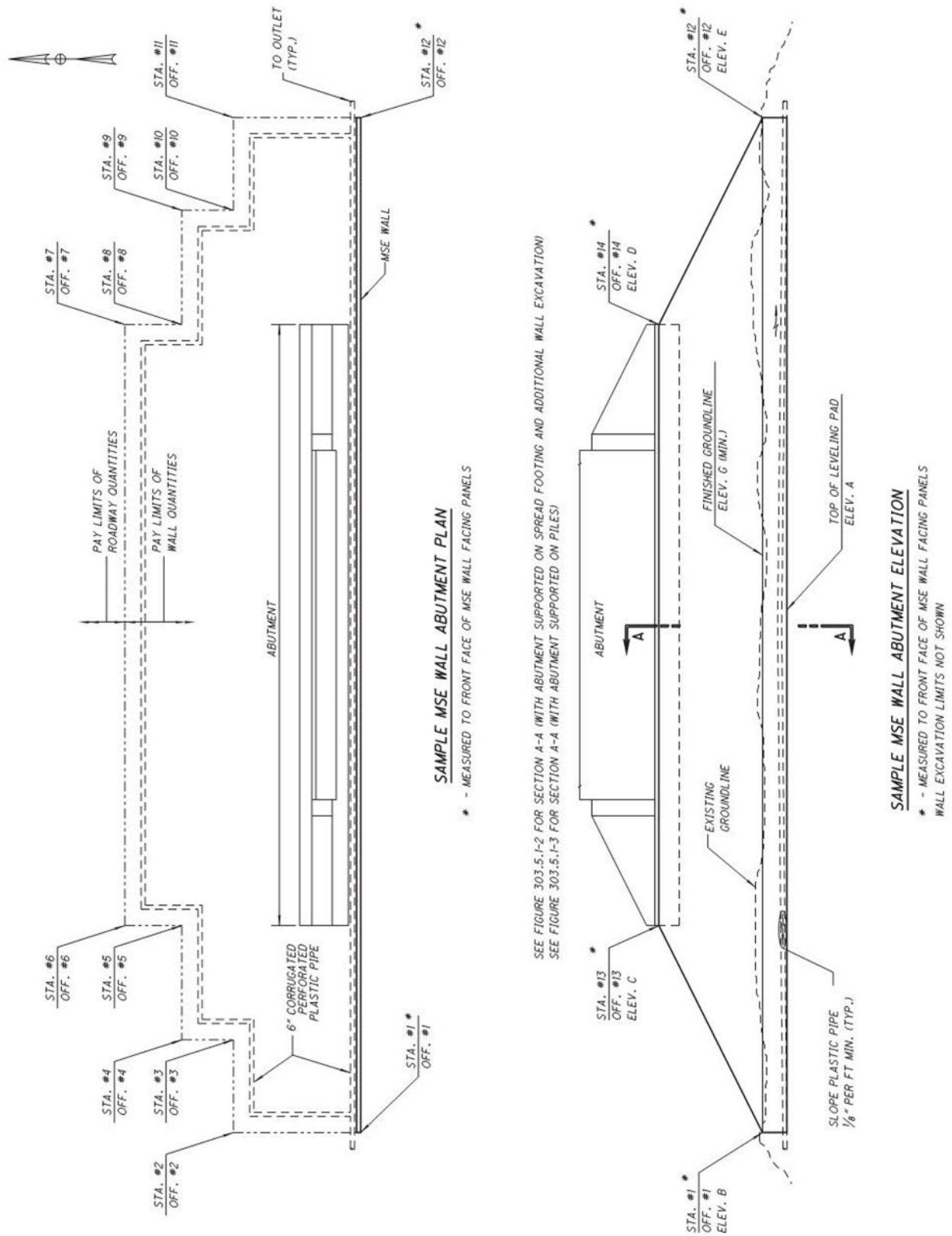


Figure 201-2

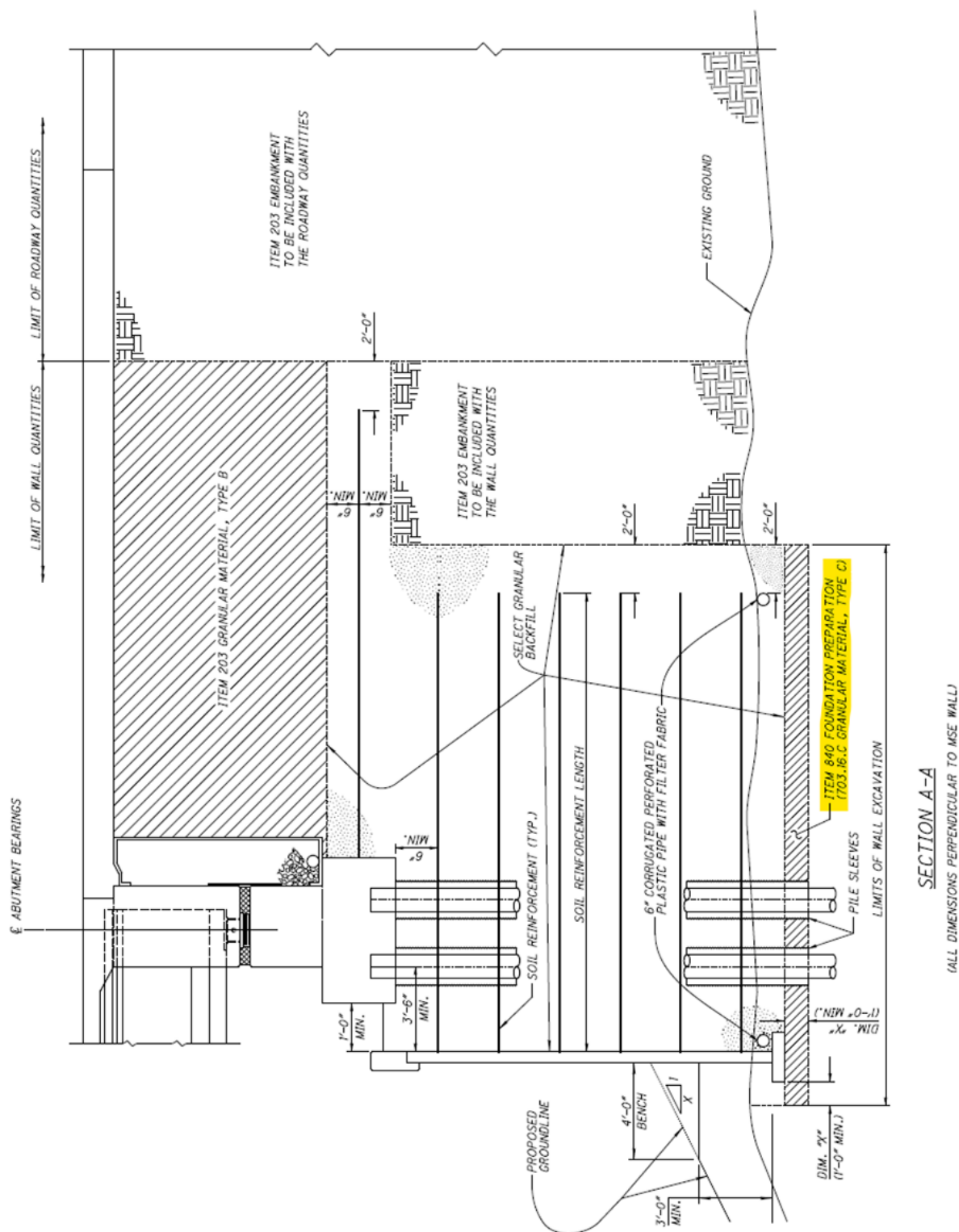


Figure 201-3

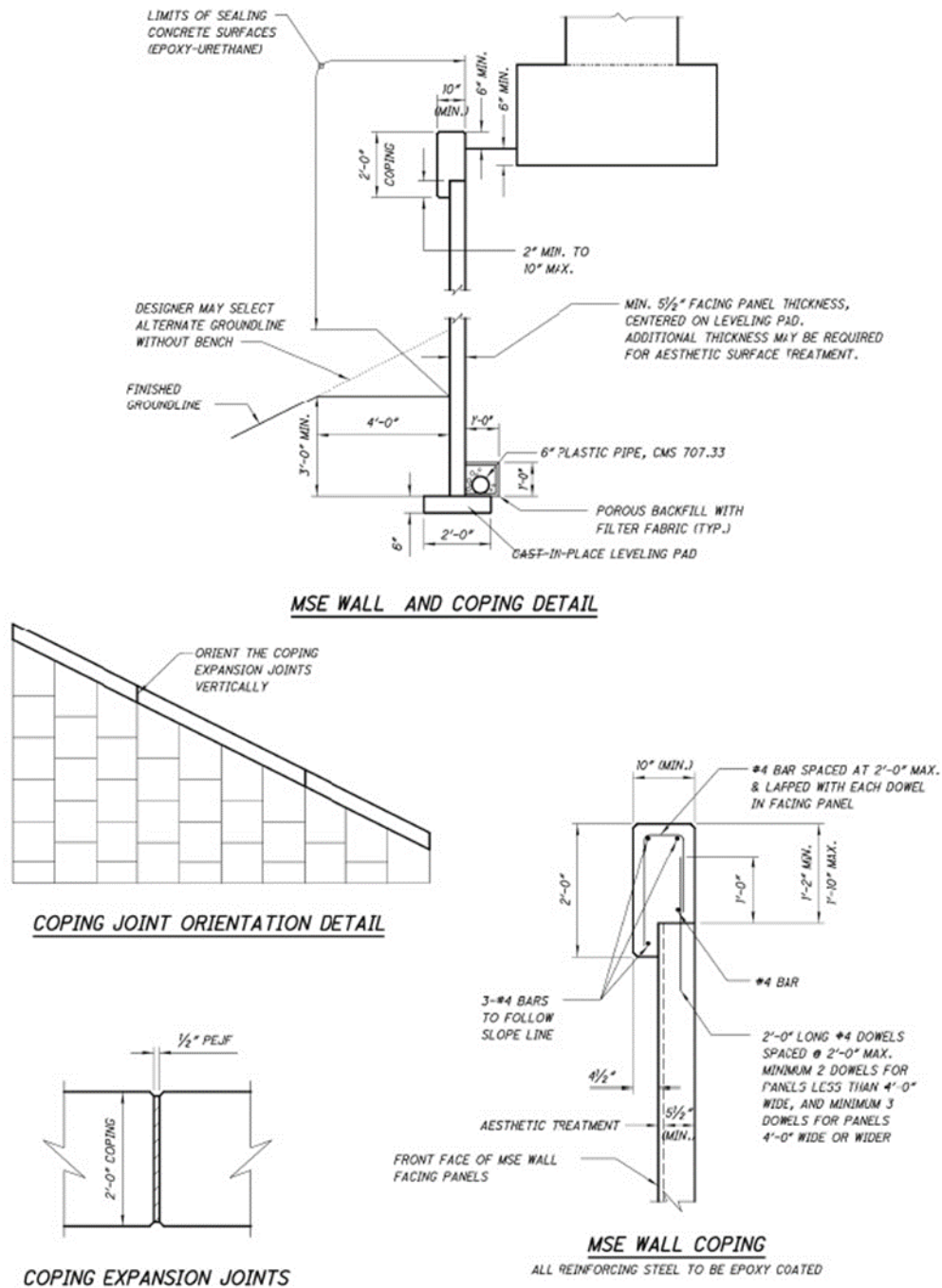


Figure 201-4

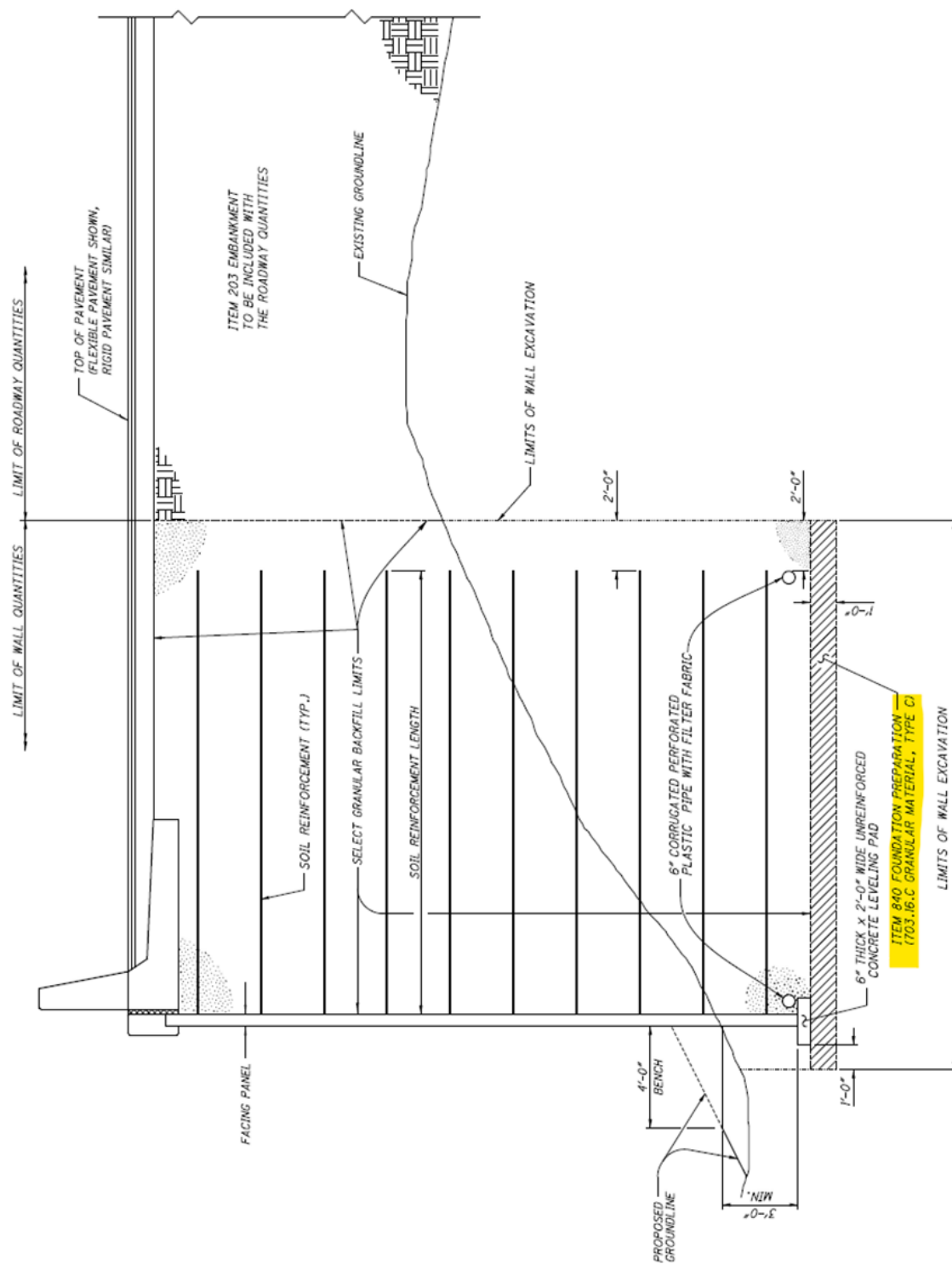


Figure 201-5

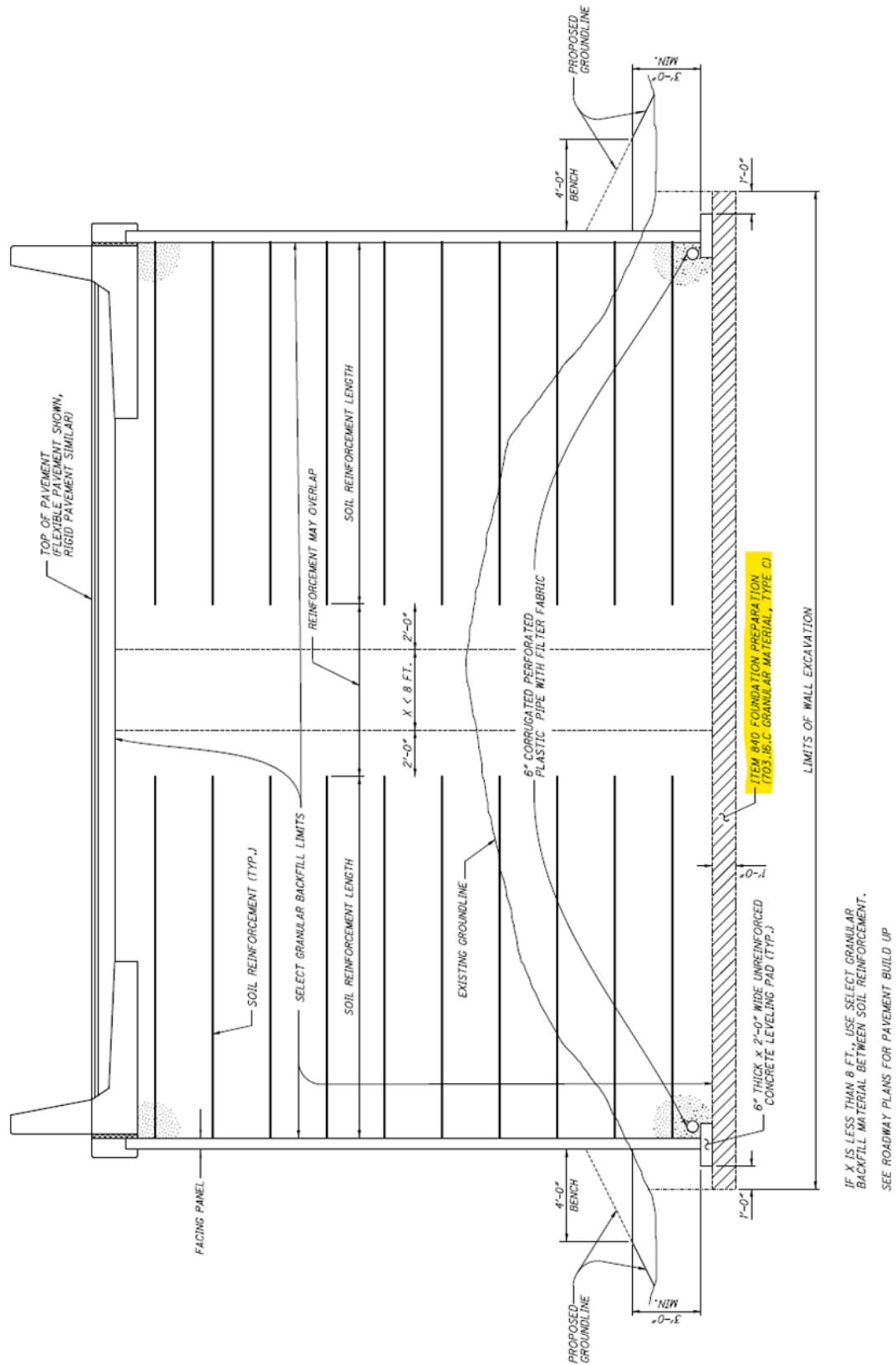


Figure 201-6

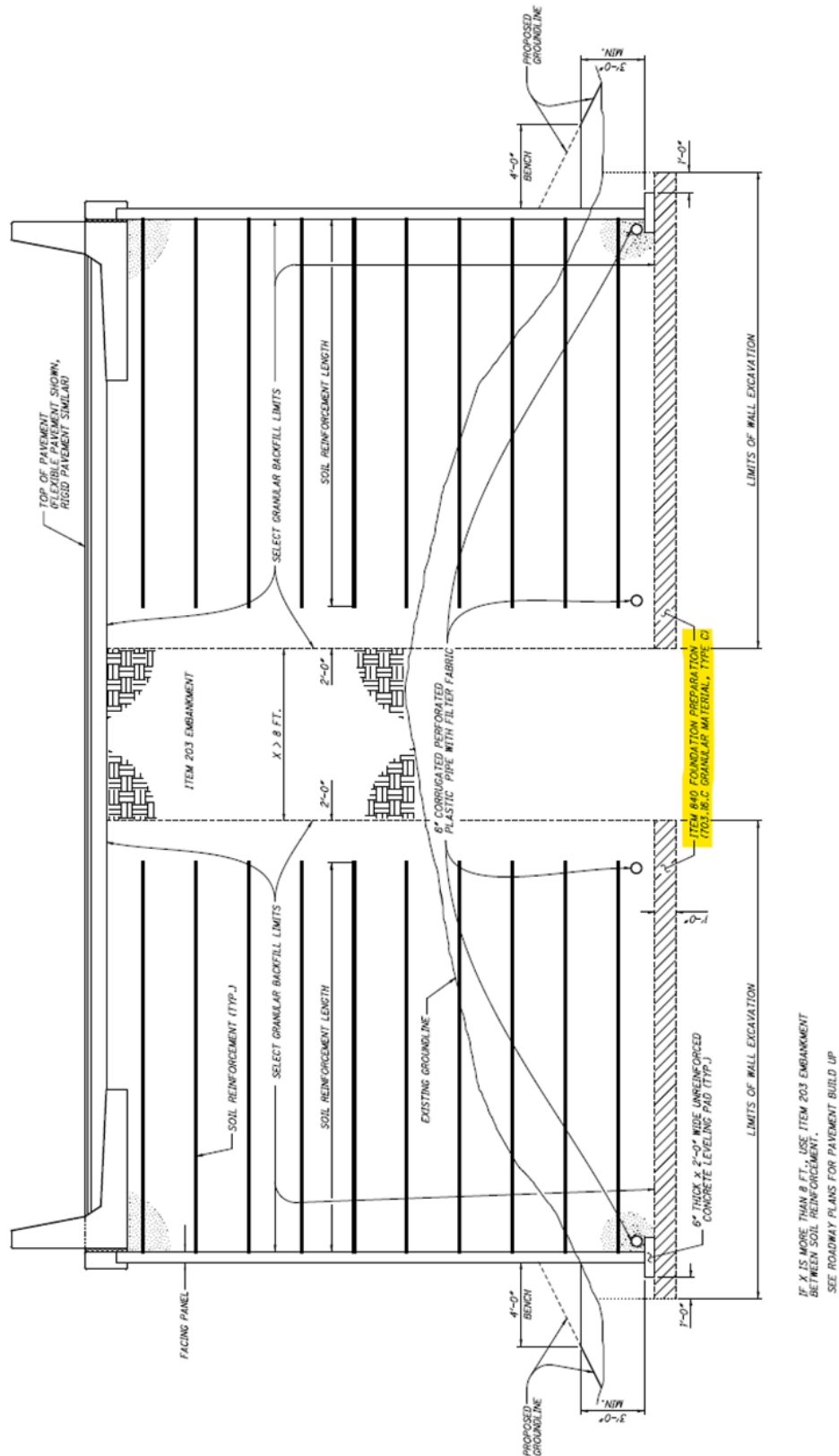


Figure 201-7

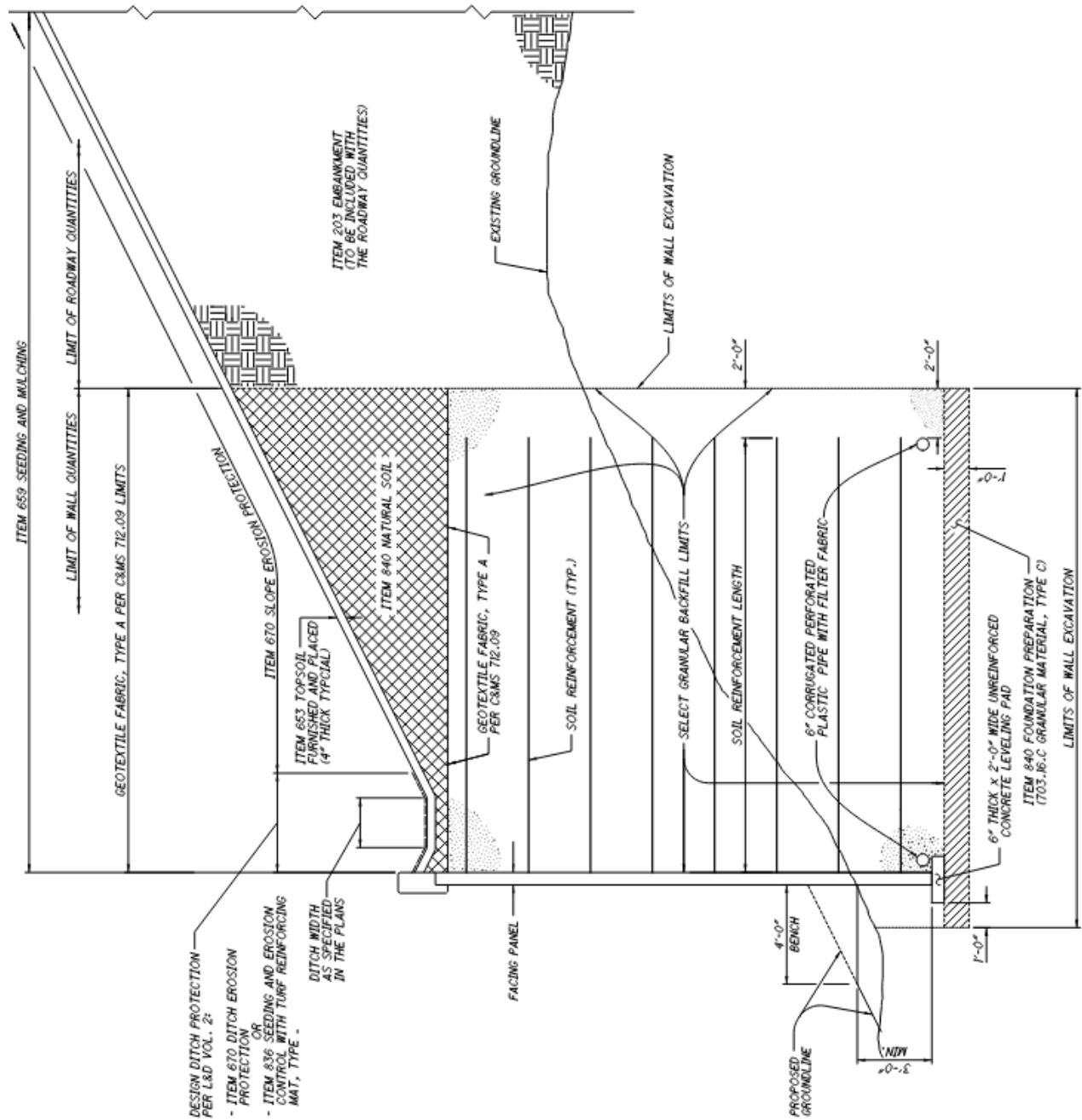


Figure 201-8

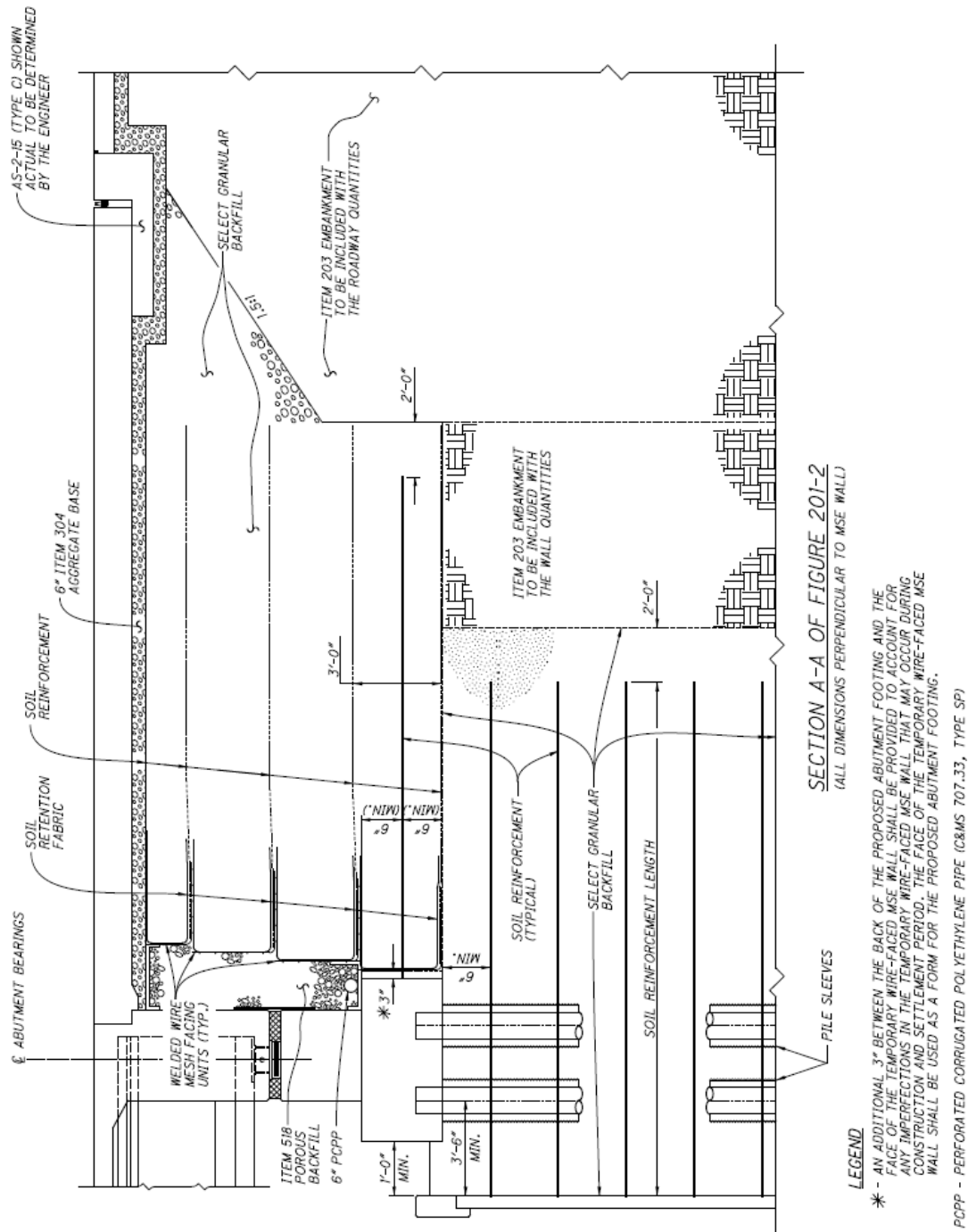


Figure 201-9

201.5 DETAIL DESIGN – STAGE 3

Provide a separate plan set for each SFN on a project. The Stage 3 structure submission for bridges and retaining walls shall include final plans, a disposition of Stage 2 comments and a signed Office of Structural Engineering Bridge Stage 3 Checklist. Estimated Quantities and Concrete Reinforcement Bar Lists shall be complete at this stage. Submit Load Rating Reports for qualifying bridge and project types according to BDM Section 900.

Resolve all Stage 2 comments to the satisfaction of the owner.

201.6 FINAL TRACINGS

Provide a separate plan set for each SFN on a project. The structural submission for final tracings shall include the final bridge plans and any special provisions along with a copy of the Estimated Quantity calculations in accordance with BDM Section 102.5 and LD-4 Estimating Information form in accordance with the [L&D, Vol. 3](#).

202 LIMITED REVIEW PROCESS

Do not utilize the Limited Review Process defined in [L&D, Vol. 3](#) Section 1404.2, for projects involving work on bridges.

203 DESIGN-BUILD REVIEW PROCESS

Refer to the Scope of Services for the project's structure review submission requirements.

C201.5

Upon approval of the bridge Owner and Project Manager, plan notes and details for bridge rehabilitation and maintenance work items on multiple bridges may be combined into one plan set with estimated quantities for each bridge shown in the General Summary.

C201.6

Upon approval of the bridge Owner and Project Manager, plan notes and details for bridge rehabilitation and maintenance work items on multiple bridges may be combined into one plan set with estimated quantities for each bridge shown in the General Summary.

C202

This process does not require ODOT reviews for design activities associated with Stage 1, Stage 2 or Stage 3. As noted in BDM Section 104, structure reviews shall not be omitted from the Project Development Process. If the Project Manager and bridge Owner determine that without staged review oversight the proposed bridge work poses no safety risk and minimal risk for construction cost overruns, then the Limited Review Process may be utilized.

C203

Depending upon the size and scope of a project, structure submissions may be required at Stage 1 only; Stage 1, Stage 2 and Stage 3; or any combination in between sometimes expressed as a completion percentage of the entire design effort (e.g. 50% completion, 90% completion, etc.). These staged submissions may also be provided in buildable units (e.g. foundations, Rear Abutment, Pier 1, Superstructure Unit 1, etc.) rather than complete structures.

**204 LOCAL-LET LOCAL PUBLIC
AGENCY (LPA) PROJECT REVIEW
PROCESS**

A Stage 2 review submission is required for all Local-Let LPA projects. The District LPA Manager will determine if additional review submissions are required for more complex projects.

All Local-Let LPA required Stage submissions shall be in accordance with BDM Section 201.

C204

Local-let LPA projects are those in which the LPA assumes project administration duties. For more information, refer to [L&D, Vol. 3](#) Section 1404.4 and the ODOT Locally Administered Transportation Projects Manual of Procedures.

Structure Type Study, Bridge Preliminary Design Report, Stage 1 and Stage 3 review submissions should be considered for the following project types: new bridge, bridge replacement, superstructure replacement, superstructure rehabilitation, superstructure widening, deck replacement, new retaining wall and retaining wall rehabilitation.

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SECTION 300 – DETAIL DESIGN

301 DESIGN SPECIFICATIONS

Develop Ohio Department of Transportation bridge designs in conformance with the latest edition of the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications.

Refer to BDM Section 101.1 for additional contract document requirements

C301

ODOT's supplement to the AASHTO LRFD specifications are documented in BDM Section 1000.

302 BRIDGE GEOMETRICS

302.1 VERTICAL CLEARANCE

Show the "Required Minimum" and "Actual Minimum" Vertical Clearances and their locations on the Preliminary Structure Site Plan, BDM Section 201.1.2.2.

For grade separation structures over a roadway, the "Required Minimum" Vertical Clearance shall not be less than specified in [L&D, Vol. 1](#), Section 302 unless otherwise specified in the Scope of Services. Prepare a Design Exception in accordance with Section 105 of the [L&D, Vol. 1](#) when the "Actual Minimum" Vertical Clearance is less than the L&D Manual minimum clearance.

302.2 BRIDGE WIDTH

Establish bridge superstructure widths in accordance with [L&D, Vol. 1](#), Section 302, unless specified in the Scope of Services.

302.3 LATERAL CLEARANCE

Provide divided highways having four or more lanes crossing under an intersecting highway with a minimum lateral clearance of 30-ft from the edge of traveled lane to the nearest obstruction.

Establish lateral clearances for other roadway classifications in accordance with [L&D, Vol. 1](#), unless specified in the Scope of Services.

C302.1

The "Actual Minimum" Vertical Clearance is the theoretical overhead clearance provided by the design plans.

C302.3

An obstruction would be a traffic barrier, wall, column or a non-traversable slope. For a non-traversable 2:1 slope, measure the lateral clearance to the point where the 2:1 back slope intersects the radius at the toe of the 2:1 slope. Refer to [L&D, Vol. 1](#), Figure 307-2E for an illustration of safety grading for ditch sections.

302.4 INTERFERENCE DUE TO EXISTING SUBSTRUCTURE

Do not locate a new substructure unit where an existing substructure or its foundation would interfere with the foundation or construction of the proposed substructure unit.

C302.4

This applies particularly where piles are to be driven. It is desirable to avoid the difficulty and expense of removing deep underground portions of the existing substructure and to avoid the resultant disturbance of the ground. Obstructions within the footprint of the new substructure shall be completely removed to avoid interference.

Where existing substructure units are shown on the Site Plan, place the locations precisely. Show the existing substructure configuration based on existing plans or field verified dimensions. Misrepresentation of the location of the existing substructure units has resulted in expensive change orders during construction.

Label existing dimensions as (+/-) plus or minus in the contract documents.

302.5 BRIDGE STRUCTURE, SKEW, CURVATURE AND SUPERELEVATION**C302.5**

During preliminary engineering, investigate alternatives that eliminate the presence of skew angles that exceed 30°, horizontal curves and superelevation transitions within the actual bridge limits.

Skews, curves and superelevation transitions add complexity which can increase costs and also result in poor ride quality.

303 DESIGN LOADING**303.1 HIGHWAY BRIDGES****303.1.1 VEHICULAR LIVE LOAD**

Design all bridges that carry highway traffic for an HL-93 loading as specified in *LRFD 3.6.1.2.1*.

303.1.2 FUTURE WEARING SURFACE

Design all bridges that carry highway traffic for a future wearing surface (FWS) of 0.060-ksf except as noted in BDM Section 401.3

303.1.3 LONGITUDINAL FORCES

Assume force effects transferred through bearings to the substructure occur at the bearing seat elevation.

C303.1.3

Bearings that permit rotation (e.g. elastomeric bearings, rocker bearings, etc.) will develop eccentricities resulting in applied moments as defined in *LRFD 14.6.3.2*. Typically, these eccentricities are negligible with respect to the design of the substructures that support the bearings and may be ignored.

The total factored longitudinal loading applied to the substructure at each expansion bearing shall be the larger of:

- A. The Frictional Force acting on the substructure if the bearing slides.
- B. The bearing's nominal (i.e. unfactored) resistance to longitudinal loading.

A properly designed expansion bearing will utilize friction in lieu of mechanical anchorage to prevent the bearing from sliding under lateral loading. For these bearings, the maximum lateral loading on the substructure unit will be in the form of Friction, *LRFD* 3.13.

For bearings that require mechanical anchorage, the bearing's resistance to lateral loading represents the maximum lateral load on the substructure unit. Resistance in this instance is nominal because it is applied to the substructure as a loading.

303.1.4 SEISMIC

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

Refer to BDM Section 305.1.5 for Seismic Geotechnical requirements.

C303.1.4

This provision is based on Ohio's seismologic data which is explained in BDM Section 305.1.5.

303.1.4.1 SEISMIC PERFORMANCE ZONE 1

Seismic analysis is not required except as noted in BDM Sections 303.1.4.1.a and 303.1.4.1.b.

For bridges founded at locations with $0.10 \leq S_{D1} < 0.15$, specify transverse reinforcement at the top and bottom of columns in accordance with *LRFD* 5.11.4.1.4 and 5.11.4.1.5.

C303.1.4.1

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.

BDM Figure 303-1 provides sample details for a cap and column pier that conform to the requirements of *LRFD* 5.11.4.1.4 and 5.11.4.1.5.

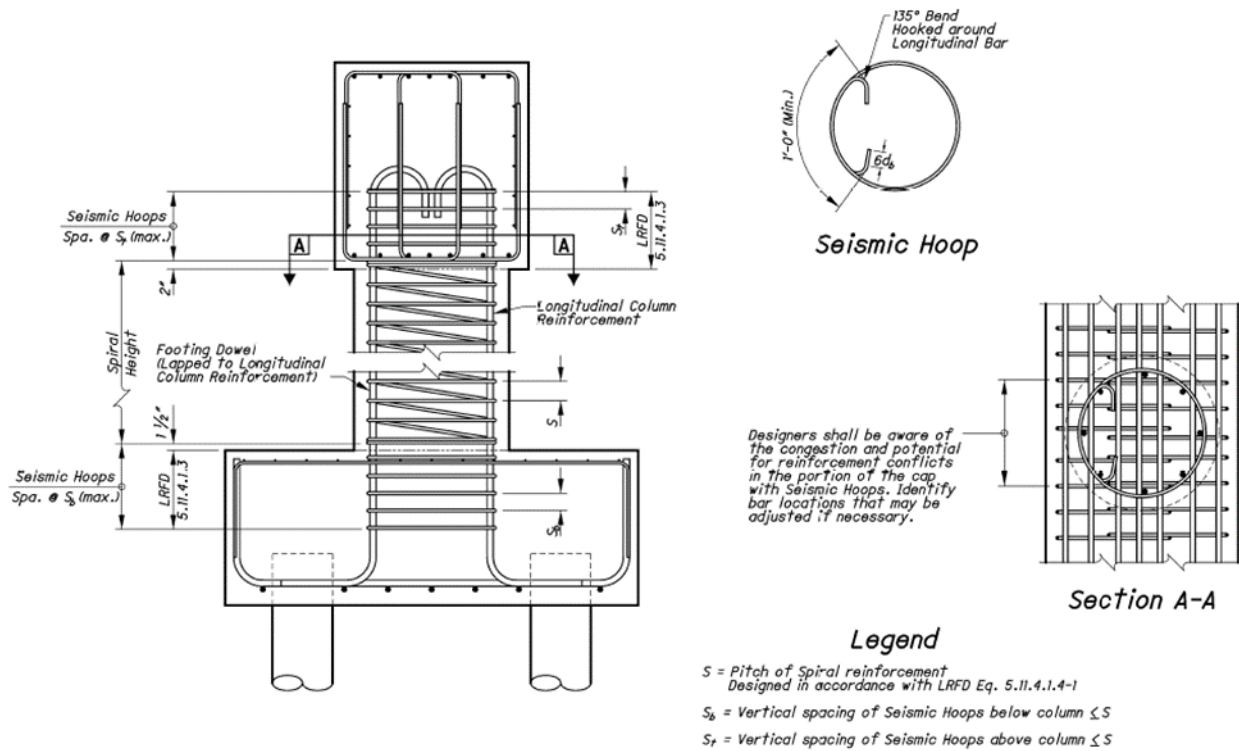


Figure 303-1

303.1.4.1.a MINIMUM SUPPORT LENGTH REQUIREMENTS

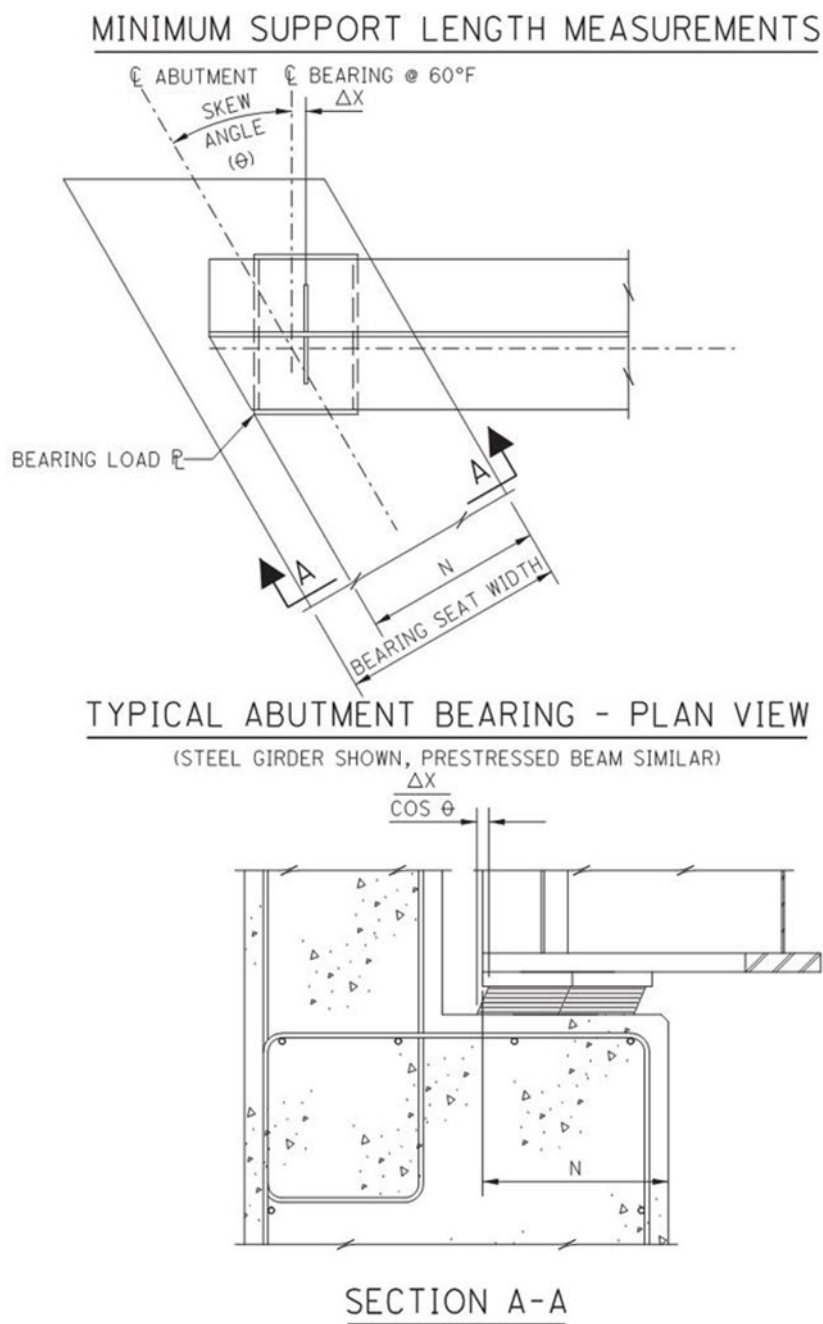
Size the bearing supports according to *LRFD 4.7.4.4*.

As a minimum, the overlapping distance of the superstructure and substructure bearing areas shall meet 100% of the minimal support length, N , calculated according to *LRFD EQ. 4.7.4.4-1*. Measure the minimum support lengths normal to the centerline of supports. Account for any expansion/contraction movements of the bearings when establishing final seat widths.

C303.1.4.1.a

This provision exists to prevent the partial or complete collapse of the superstructure during seismic events.

Refer to BDM Figure 303-2 for the application of support length requirements to a typical expansion elastomeric bearing at an abutment.

**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 303-2

303.1.4.1.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. During a seismic event, design these mechanisms, that prevent the free horizontal translation of the superstructure in any direction relative to the substructure, to transfer an applied horizontal connection force at the Extreme Event I Limit State. Only provide additional restraint for seismic loads (e.g. seismic pedestals) where the mechanisms noted above do not provide sufficient capacity. Refer to BDM Section 303.1.4.1.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*. If sufficient geotechnical information is not available to determine Site Class, assume the magnitude of the connection force is 0.25 times the tributary permanent load.

For restraint provided in multiple directions, *LRFD 3.10.8* applies.

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.

Provide cross-frames, designed to resist Strength and Service load combinations, to create a direct load path from the point of horizontal connection force application to the deck.

C303.1.4.1.b

Examples of mechanisms include fixed bearings, bearing guides, abutment diaphragms, diaphragm guides, wing walls and wind locks.

The load factor for live load, γ_{EQ} , may be taken as 0.0.

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.

These cross-frame do not need to be designed for Extreme Event limit state loading.

303.1.4.1.c REQUIREMENTS FOR BEARINGS

The Department will permit unrestrained bearings that sustain irreparable damage during a seismic event provided loss of span is prevented by the design for the Horizontal Connection Force in BDM Section 303.1.4.1.b.

303.1.4.2 EXISTING STRUCTURES

Evaluate the seismic vulnerability of an existing structure for rehabilitation projects requiring complete deck or superstructure replacements. Design new substructure units in accordance with *LRFD 3.10.9.2*, *4.7.4.4* and BDM Section 303.1.4.2.b. If sufficient geotechnical information is not available, assume:

A. $A_s > 0.05$

B. $S_{D1} < 0.10$

303.1.4.2.a SUPERSTRUCTURE

For projects where seismic vulnerability is evaluated, at bearing locations that will transmit the horizontal connection force from the substructure to the superstructure, provide cross-frames to resist Strength and Service load combinations to create a direct load path to the deck. For supports not in compliance with *LRFD 4.7.4.4*, provide seismic restrainers designed for the Horizontal Connection Force, specified in BDM Section 303.1.4.1.b.

303.1.4.2.b SUBSTRUCTURE

For projects where seismic vulnerability is considered, concrete columns at piers that transfer the seismic horizontal connection force, according to BDM Section 303.1.4.1.b, shall meet the spiral and tie ductility requirements of *LRFD Eq. 5.11.4.1.4-1* and *LRFD 5.10.4*. Do not use *LRFD Eq. 5.6.4.6-1*.

C303.1.4.2.b

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, provide the required confinement of the primary steel in the axially loaded substructure members.

LRFD Eq. 5.6.4.6-1 provides erroneous results based on the ratio of A_g/A_c . As the concrete cover increases, impractical areas of transverse reinforcement are required. *LRFD Eq. 5.11.4.1.4-1* provides the volumetric reinforcement requirements for confinement of primary steel at locations where plastic hinges are formed during seismic events.

One acceptable method to increase the amount of confinement provided in an existing concrete column is through the use of Fiber Reinforced Polymer (FRP) wrap systems.

For bridges located in regions with an acceleration coefficient, $S_{D1} < 0.10$, specify a confining stress due to FRP jacket (f_1) of 0.150-ksi for the entire height of the column from the top of the footing/drilled shaft to the bottom of the cap. For bridges located in regions with an acceleration coefficient, $S_{D1} \geq 0.10$, specify a confining stress due to FRP jacket (f_1) of 0.300-ksi in the plastic hinge regions as defined in *LRFD 5.11.4.1.5* and 0.150-ksi in the remaining portions of the columns. The plans shall include an elevation view of the columns with these confining stress regions clearly defined.

303.2 PEDESTRIAN AND BIKEWAY BRIDGES

Design pedestrian and bikeway bridges in accordance with the latest edition of the *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges* and this Manual.

Where sidewalks, pedestrian, and/or bicycle bridges are intended to be used by maintenance and/or other incidental vehicles, include an H15-44 vehicle, as shown in BDM Figure 303-3, in the Design Loading. Do not include the H15-44 lane loading. Do not place the vehicle live load in combination with the pedestrian live load and do not apply the dynamic load allowance to the H15 vehicle.

Position the design truck transversely such that the center of any wheel load is not closer than:

- For the design of the deck overhang – 1.0-ft from the face of curb or railing and
- For the design of all other components – 2.0-ft from the edge of the design lane.

These systems are a viable alternative for dry columns supported on pile caps, spread footings and drilled shafts. Research has shown that providing a confining stress of 0.300-ksi in regions where plastic hinges may form at the top and bottom of columns as defined in *LRFD 5.11.4.1.5* and providing a confining stress of 0.150-ksi outside of the plastic hinge regions is sufficient to prevent buckling of the longitudinal reinforcement.

PN 519 for Composite Fiber Wrap Systems references the International Code Council Evaluation Service website (www.icc-es.org) for acceptable FRP wrap products. Refer to the Designer Notes for plan information associated with this work.

C303.2

The H15-44 vehicle accommodates the largest emergency vehicles that may access a pedestrian facility.

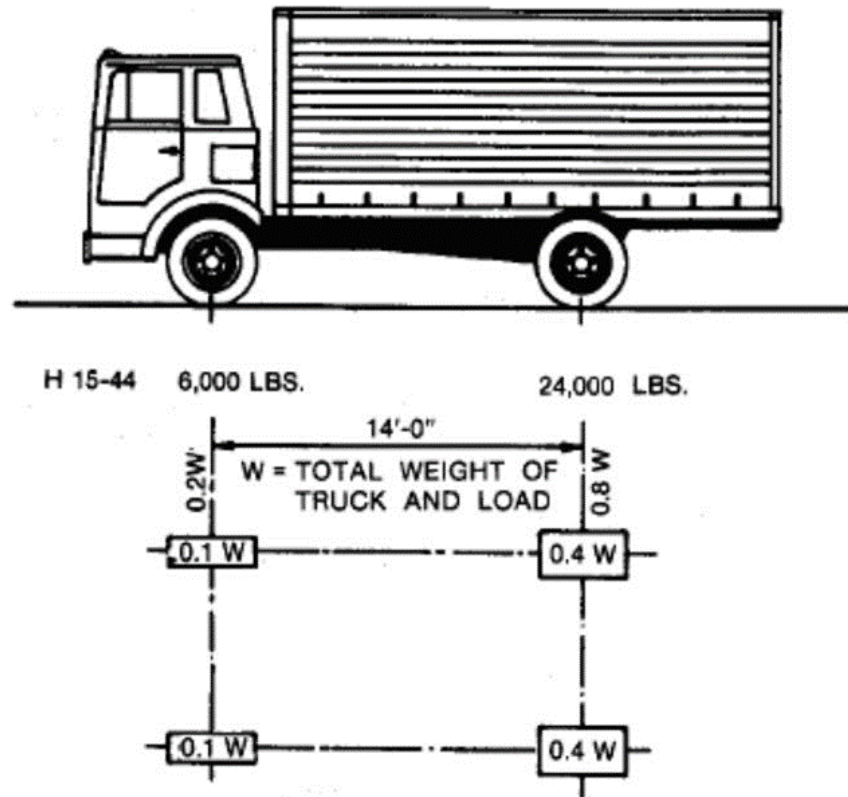


Figure 303-3

303.3 RAILROAD BRIDGES

Design facilities, that are operated and maintained by the railroad, 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.

304 MATERIALS

304.1 STRUCTURAL STEEL

Select structural steel material from the following list:

- A. ASTM A709 Grade 50W
- B. ASTM A709 Grade 50
- C. High Performance Steel (HPS), ASTM A709 Grade 70W

C304.1

There are several systems available for coating steel bridges. These coatings are specified in [C&MS 514](#) or by the plan notes available from the Office of Structural Engineering. See BDM Section 308.2.2.1.d.1 for guidelines on selecting a coating system.

Grade 50W shall be utilized for un-coated weathering steel applications. Grade 50 shall be utilized for coated steel bridges. Grade 70W is an un-coated weathering steel that is most economical when used in the flanges of hybrid girders. Consult the Office of Structural Engineering for recommendations prior to specifying its use. A plan note is available from the Office of Structural Engineering.

Contact OSE if there are no domestic sources available for steel items listed in the Contract Documents.

Common items involving domestic sourcing issues include highly specialized components such as: brakes for lift bridges; steel ferrules for suspension bridge cables; and components in movable bridge mechanisms.

Determine the Estimated Quantities for structural steel pay items in accordance with the Method of Measurement specified in [C&MS](#) 513.29.

Use of contingent quantities for secondary members shall be avoided. If contingencies of any kind are included in the Estimated Quantities, provide a Plan note clarifying how the contingent quantities were determined.

304.2 CONCRETE, CAST-IN-PLACE

304.2.1 CONCRETE DESIGN STRENGTHS

Use the following 28-day concrete strengths (f'c) for **DESIGN** purposes:

- A. Substructure Concrete (Class QC1).....4.0-ksi
- B. Superstructure Concrete (Class QC2).....4.5-ksi
- C. Drilled Shaft Concrete (Class QC5).....4.0-ksi

C304.2.1

The concrete used for drilled shafts is a 4.5-ksi mix design. However, since there is the potential for issues that cause a reduction in strength to arise during placement, a 500-psi reduction in the concrete strength shall be considered for design purposes.

304.2.2 CONCRETE FOR BRIDGES AND STRUCTURES

C304.2.2

The following ODOT concrete types are available for substructure concrete sold under Item 511:

- A₁. Class QC1 Concrete
- B₁. Class QC1 Concrete with QC/QA
- C₁. Class QC3 Special Concrete
- D₁. Class QC4 Mass Concrete

ODOT also has classes of concrete available for moderate setting (Class QC MS) and fast setting (Class QC FS) mixes which are primarily used in pavement applications. For projects where these mix designs may be necessary, use an Item 511 CONCRETE, MISC. pay item and provide a plan note to clarify the project requirements.

The following ODOT concrete types are available for superstructure concrete sold under Item 511:

- A₂. Class QC2 Concrete
- B₂. Class QC2 Concrete with QC/QA
- C₂. Class QC3 Special Concrete
- D₂. Class QC4 Mass Concrete

The following ODOT concrete types are available for drilled shaft concrete sold under Item 524:

- A₃. Class QC4 Mass Concrete
- B₃. Class QC5 Concrete

Specify “Concrete with QC/QA” for the class of concrete when the total concrete quantity for that class exceeds 150-yd³.

The total volume for a class of concrete (which may occur in multiple pay items) used in a single bridge is to be included when determining the need for QC/QA. For example, the Class QC1 concrete from the abutments and piers would need to be quantified together and considered for QC/QA.

Specifying QC/QA for smaller quantities may be cost prohibitive.

Specify Class QC3 Special Concrete when concrete strengths and/or permeability other than the QC1 or QC2 are necessary.

Specify Class QC4 Mass Concrete when the minimum dimension for a concrete component is 5-ft or greater or the diameter of a drilled shaft is 7-ft or greater.

304.2.3 CONSTRUCTION JOINTS

Show all Construction Joint locations in the plans. Label joints as:

- A₁. Construction Joint
- B₁. Optional Construction Joint
- C₁. Construction Joint, Roughened
- D₁. Optional Construction Joint, Roughened

Each plan specified Construction Joint location shall meet the Interface Shear Transfer – Shear Friction requirements specified in *LRFD 5.7.4*.

C304.2.3

A Construction Joint is a plane separating concrete placements that reach initial set at different times. An Optional Construction Joint is one that is placed at the Contractor's discretion. A Construction Joint, Roughened, is one with a finished surface that is intentionally roughened to an amplitude of 0.25-in.

[C&MS](#) 511.09 defines the following Construction Joint surface finishes:

- A₁. Non-finished
 - 1. Applies to horizontal surfaces or any other orientation that does not require forming.
 - 2. For design purposes, the non-finished surface represents concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened as defined in *LRFD 5.7.4.4*.
- B₁. Form-finished
 - 1. Applies to vertical surfaces or any other orientation that requires forming. This finish also applies to the Construction Joint created when placing concrete against an existing concrete surface, placed in a previous project unless otherwise specified in the Plans.
 - 2. For design purposes, the form-finished surface represents concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened as defined in *LRFD 5.7.4.4*.
- C₁. Roughened
 - 1. [C&MS](#) 511.09.A defines the acceptable roughened surface finish for plastic concrete and [C&MS](#) 511.09.B defines the acceptable roughened surface finish for hardened concrete.

2. For design purposes, both [C&MS 511.09](#) roughened surfaces represent concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25-in as defined in *LRFD 5.7.4.4*.
3. [C&MS 515.15](#) defines the acceptable surface finish for composite prestressed beams. For design purposes, the [C&MS 515.15](#) composite surface finish represents a cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25-in as defined in *LRFD 5.7.4.4*.

Joints labeled in the plan as “Construction Joint” or “Optional Construction Joint” will receive a non-finished or form-finished surface depending upon orientation.

Construction Joints may be necessary to facilitate construction; however, joints represent a potential plane of weakness in a concrete member. The best approach to locating joints is to minimize the number and, if possible, locate joints at areas of minimum stress. The recommended design approach to Construction Joints should be as follows:

A₂. For horizontal Construction Joints or any other orientation that does not require forming, design the joints using the following 4-step sequence:

1. Determine if the interface shear capacity of a non-finished surface is sufficient for the specified joint location using the reinforcement and cross-section sized for shear and moment in the member.
2. If additional interface shear capacity is required and the location of the joint cannot be moved to an area of lower stress, reconsider the interface shear capacity with a roughened finish.
3. If additional interface shear capacity is still required, increase the interface shear reinforcement (A_{vf}) crossing the shear plane and reconsider the interface shear capacity with a roughened finish.
4. If interface shear capacity is still not met, a redesign of the cross-section is warranted.

The surface treatment specified for Horizontal Construction Joints in the most recent revision dates of the Standard Bridge Drawings may be used without further analysis of the interface shear capacity.

C₂. For Construction Joints created when joining new concrete to concrete placed in a previous project, and the:

1. Existing concrete surface was formed, troweled, or saw cut, design the joints using the following 3-step sequence:
 - a. Determine if the interface shear capacity of a non-roughened surface is sufficient using non-shrink, non-metallic grout to anchor interface shear reinforcement in accordance with [C&MS 510](#).
 - b. If additional interface shear capacity is required specify the existing concrete surface to be intentionally roughened according to [C&MS 511.09.B](#).
 - c. If interface shear capacity is still not met, a redesign of the cross-section is warranted.
2. Existing concrete surface was prepared with chipping hammers, design the joints using the following 3-step sequence:
 - a. Determine if the interface shear capacity of a roughened surface is sufficient utilizing salvaged reinforcement across the shear plane.
 - b. If additional interface shear capacity is required, specify additional non-shrink, non-metallic grout to anchor interface shear reinforcement in accordance with [C&MS 510](#).
 - c. If interface shear capacity is still not met, a redesign of the cross-section is warranted.

When determining the area of concrete considered to be engaged in interface shear transfer (A_{cv}), do not include the surface area of sawcut surfaces in the calculation of intentionally roughened surface area.

Where practical, avoid horizontal Construction Joints in piers, abutments, and retaining walls. Otherwise locate horizontal joints 1-ft above the OHWM.

If Keyways are specified in Construction Joints, provide a cross-sectional view of the concrete member to completely define all Keyway dimensions.

A Keyway is a Construction Joint requiring formwork to create a “tongue and groove” geometry between the different concrete placements.

Determine the factored shear resistance (V_r) of the Keyway as the least of the following:

Keyways are not an effective mitigation method for water infiltration.

A₂. Factored bearing capacity on the shear key mating surfaces due to shear loading

- B₂. Factored shear capacity on the dimension of shear key parallel to the shear loading
- C₂. Shear load inducing the Cracking Moment (M_{cr}) of the shear key due to the eccentricity of the shear loading located at the centroid of the shear key mating surfaces.

If the factored shear resistance of the Keyway (V_r) is less than the factored interface shear resistance across the depth of the keyless Construction Joint (V_{ri}), then the contribution of the Keyway shall be ignored.

Specify Keyways in unreinforced Construction Joints and joints that serve the purpose of Contraction Joints.

Locate the Construction Joint at the base of columns supported directly on drilled shafts in accordance with BDM Section 305.4.4.

Provide Optional Construction Joints to avoid pier lift heights exceeding 30-ft. Detail reinforcement to be developed each side of the Optional Construction Joint.

Seal all Construction Joints in contact with fill above the footings and below the approach slab on the back of abutments, wingwalls and retaining walls with ITEM 512, TYPE 2 WATERPROOFING, 3-ft wide centered on the joint.

304.3 CONCRETE, PRESTRESSED

Select a 28-day compressive strength ranging from 5.5-ksi to 7.0-ksi. Provide the 28-day compressive strength in the contract plan General Notes.

Select a compressive strength at the time of release ranging from 4.0-ksi to 5.0-ksi. Provide the release strength in the contract plan General Notes.

Cast-in-place concrete used in prestressed concrete superstructures (composite decks, pier diaphragms, intermediate diaphragms, etc.) shall be Class QC2 or Class QC2 with QC/QA.

Consider the use of Construction Joints for the following issues:

- A₃. To avoid or minimize member dimensions defined in BDM Section 304.2.2 that require Mass Concrete provisions.
- B₃. To accommodate standard concrete formwork construction practices and concrete member geometry changes. For example, Construction Joints may allow a Contractor to utilize a prior concrete placement for formwork support to complete a substructure unit.
- C₃. To accommodate construction sequencing. For example, a Construction Joint in a T-type pier stem may allow the removal of cofferdam sheeting which would otherwise be obstructed by pier cap concrete placed directly above.

C304.3

In order to specify a release or 28-day concrete strength that deviates from the ranges provided, contact the Ohio Prestressers Association and/or the PCI Central Region to request documentation that the deviation is acceptable and can be produced within the project timeframe by at least two independent producers prequalified under S1079. ODOT may request verification of this documentation during project reviews.

Select Class QC2 or Class QC2 with QC/QA according to BDM Section 304.2.2.

304.4 CONCRETE REINFORCEMENT**304.4.1 MATERIAL TYPES**

The Department will allow six types of material for use as concrete reinforcement:

- A. Uncoated Steel Reinforcement (USR)
- B. Epoxy Coated Steel Reinforcement (ECSR)
- C. Galvanized Steel Reinforcement (GSR)
- D. Glass Fiber Reinforced Polymer Reinforcement (GFRP)
- E. Chromium Steel Reinforcement (CSR)
- F. Stainless Steel Reinforcement (SSR)

Acceptable designs for each of these materials will yield differing results for reinforcement ratio and/or development lengths. Except as noted in [C&MS](#) 709.16, the Department will require a Value Engineering Change Proposal according to [C&MS](#) 105.19 in order to consider substitution of concrete reinforcement types after the Contract sale date.

The concrete reinforcement material type for new concrete members shall be defined in the Scope of Services. If the Scope of Services does not define the concrete reinforcement material type, then all reinforcement shall be ECSR according to BDM Section 304.4.1.2.

304.4.1.1 UNCOATED STEEL REINFORCEMENT

Uncoated Steel Reinforcement is available for use only in rehabilitation of existing concrete members.

The Department will pay for USR as: ITEM 509 – UNCOATED STEEL REINFORCEMENT

C304.4.1.1

USR is intended for applications where the work consists of repair or rehabilitation of existing concrete members reinforced with uncoated reinforcement. USR may also be considered for replacement of damaged reinforcement after surface preparation for overlays involving hydro demolition or scarification. USR should also be considered for structural doweling applications into existing concrete according to [C&MS](#) 510.

304.4.1.2 EPOXY COATED STEEL REINFORCEMENT

Epoxy coated steel reinforcement shall be Grade 60, $F_y = 60$ -ksi in accordance with [C&MS](#) 709.00 furnished from a [Supplement 1068](#) certified supplier.

Provide epoxy coated steel reinforcement except as noted in BDM Section 405.2 or unless otherwise noted in the Scope of Services.

C304.4.1.2

The advantages of ECSR include:

- A₁. Provides increased corrosion protection above ordinary uncoated steel reinforcement.
- B₁. Lowest cost corrosion protection for steel reinforcement
- C₁. Shortest lead times for concrete reinforcement delivery

The Department will pay for ECSR as: ITEM 509 – D₁. Permits field fabrication (e.g. bending and cutting)
EPOXY COATED STEEL REINFORCEMENT

E₁. Damage to the coating is repairable

F₁. Readily available mechanical splice options

The disadvantages of ECSR include:

A₂. Susceptible to coating damage during shipping, handling, installation and concrete placement.

B₂. Unrepaired damage to the epoxy coating may result in corrosion similar to or in some cases worse than uncoated steel.

C₂. Concrete bond strength to ECSR results in longer development lengths and potentially wider crack widths when compared to ordinary uncoated reinforcing steel.

D₂. ECSR coating will degrade with extended exposure to UV light reducing the coating thickness and causing embrittlement.

E₂. ECSR coating may be completely removed if exposed to the hydro-demolition surface preparation for deck overlays.

304.4.1.3 GALVANIZED STEEL REINFORCEMENT

Galvanized steel reinforcement shall be Grade 60, $F_y = 60$ -ksi in accordance with [C&MS 709.16](#) furnished from a [Supplement 1068](#) certified supplier.

The Department will pay for GSR as ITEM 509 – GALVANIZED STEEL REINFORCEMENT.

C304.4.1.3

Two types of galvanized coating systems are permitted under C&MS 509 and 709.16: ASTM A767 and ASTM A1094.

ASTM A767 reinforcement is normal uncoated reinforcement that is fabricated then sent to a hot-dip galvanizer for coating. ASTM A1094 reinforcement is coated in a continuous galvanizing process before it is sent to the bar fabricator for cutting and bending.

There are some differences between the two galvanized coating systems. Although hot dip galvanized (HDG) reinforcing and continuous galvanized reinforcing (CGR) both have a critical free zinc layer that enhances corrosion protection, CGR (ASTM A1094) is relatively new and lacks in-situ performance data and relies on accelerated lab testing to predict future performance. Whereas HDG reinforcing (ASTM A767) has a longer history of use and has a thicker minimum coating requirement (86-150 microns) when compared to the minimum coating thickness for the CGR (50 microns). Therefore, the HDG reinforcing will likely provide a longer service life relative to CGR. However, there can be initial cost savings when using CGR as the manufacturing process often saves time and freight by eliminating the trip to the galvanizer after fabrication. In the CGR process, the fabricator is responsible for repairing the coating on ends of bars cut during the fabrication process. Should an owner wish to specify only one of the two types of galvanized reinforcement, an “As Per Plan” note will be required.

The advantages of GSR include:

- A₁. Unlike ECSR, unrepaired damage to GSR bars that occurs during shipping, handling, installation, or concrete placement will be protected by galvanic corrosion.
- B₁. Structural development lengths are equal to ordinary uncoated steel reinforcement and shorter than ECSR.
- C₁. The introduction of the continuously galvanized reinforcement has enabled GSR to be more cost competitive with ECSR.
- D₁. Permits field fabrication (e.g. bending and cutting)

The disadvantages of GSR include:

- A₂. Corrosion will occur on improperly coated ends of continuously galvanized reinforcement damaged during fabrication.
- B₂. GSR lead times for delivery longer than ECSR. For hot dipped GSR, fabricated reinforcement needs to be shipped to galvanizer for coating. For continuously GSR, prefabricated reinforcement needs to ship from sole supplier in Oklahoma.
- C₂. Initial cost of GSR higher than ECSR.
- D₂. The galvanized coating may be completely removed if exposed to the hydro-demolition surface preparation for deck overlays.
- E₂. The minimum bend diameters for ASTM A767 bars are larger than ASTM A1094, USR, ECSR, CSR and SSR.

304.4.1.4 GFRP REINFORCEMENT

GFRP reinforcement shall be in accordance with [C&MS 705.28](#) furnished from a [Supplement 1138](#) certified supplier.

GFRP reinforcement shall only be utilized for the following applications:

- A. Reinforcement in concrete decks for Beam-Slab bridges
- B. Longitudinal reinforcement in concrete bridge railings
- C. Spiral and tie reinforcement in axial compression members
- D. Reinforcement in approach slabs

The Department will pay for GFRP reinforcement as: ITEM 509 - NO. — DEFORMED GFRP REINFORCEMENT.

The Department will not permit concrete removal using hydro-demolition for any Corrective Work Plans involving new concrete members reinforced with GFRP reinforcement.

C304.4.1.4

The advantages of GFRP reinforcement include:

- A₁. GFRP reinforcement is much lighter than steel reinforcement resulting in lower shipping, handling, and installation costs.
- B₁. Deterioration of GFRP reinforcement is negligible over the life of the structure.
- C₁. GFRP reinforcement is very cost competitive to ECSR.

The disadvantages of GFRP reinforcement include:

- A₂. GFRP reinforcement does not have a well-defined yield point. This lack of ductility precludes its use as primary tension reinforcement in many applications
- B₂. GFRP reinforcement has more limited availability for bent shapes compared to other material types. Bent bars for sizes #9 and #10 are not available. Also, vertical reinforcement in standard concrete bridge railing includes bend diameters smaller than permitted for GFRP reinforcement and bar shapes not available in GFRP reinforcement.
- C₂. An approved mechanical splice attached directly to GFRP reinforcement is not available. Mechanical splices to steel reinforcement then lapped to GFRP reinforcement provides an acceptable alternative.
- D₂. Concrete repair work (e.g. hydro-demolition and chipping hammers) can irreparably damage GFRP reinforcement. ODOT has performed an acceptable hydro-demolition surface preparation on concrete reinforced with GFRP, but some damage to the reinforcement is likely. Test sections are necessary to determine machine settings for optimal results.
- E₂. GFRP reinforcement will deform more due to its lower elastic modulus than steel reinforcement.
- F₂. Does not permit field fabrication (e.g. bending and cutting)

304.4.1.5 CHROMIUM STEEL REINFORCEMENT

Chromium steel reinforcement is available in three alloy types distinguished by their chromium content:

- A. Type CL (2 - 3.9% Cr)
- B. Type CM (4 – 7.9% Cr)

C304.4.1.5

Chromium steel reinforcement offers a higher level of protection against corrosion than ordinary uncoated steel reinforcement due to its chemical composition. A measure of corrosion resistance available for steel reinforcement is the quantity of chloride in concrete that

C. Type CS (8 – 10.9% Cr)

CSR shall be Grade 100, $F_y = 100$ -ksi in accordance with [C&MS](#) 709.17 furnished from a [Supplement 1068](#) certified supplier. Specify the Type and Grade in the Plans. Do not use CSR Grade 120.

The Department will pay for CSR as: ITEM 509 – CHROMIUM STEEL REINFORCEMENT, TYPE

initiates corrosion commonly referred to as the chloride threshold (CT). Literature for chromium steel reinforcement suggests that Type CL has a CT 1.5 times greater than ordinary uncoated steel reinforcement; Type CM has a CT 2.0 times greater; and Type CS has a CT 4.0 times greater.

[C&MS](#) 709.17 specifies Type CS if not otherwise specified in the Plans.

The advantages of CSR include:

- A1. CSR provides three different levels of corrosion protection that exceed the corrosion resistance of ordinary uncoated steel reinforcement. Type CL provides slightly less protection than undamaged ECSR while Type CM provides slightly more protection.
- B1. CSR's higher strength can be utilized to reduce congestion and development lengths of ordinary reinforcement. CSR is available in Grade 120, $F_y = 120$ -ksi, but AASHTO does not permit steel reinforcement above 100-ksi.
- C1. The bars have no coating that can be damaged during shipping, handling, installation, concrete placement, hand chipping and hydro-demolition.
- D1. Permits field fabrication (e.g. bending and cutting)

The disadvantages of CSR include:

- A2. CSR has a higher unit cost than ECSR. The cost of CSR increases with increases in chromium content. Type CL having the lowest cost and Type CS the highest cost.
- B2. CSR is not produced locally so availability could be an issue and lead times would be longer than ECSR

304.4.1.6 STAINLESS STEEL REINFORCEMENT

Stainless steel reinforcement shall be Grade 75, $F_y = 75$ -ksi in accordance with [C&MS](#) 709.18 furnished from a [Supplement 1068](#) certified supplier.

The Department will pay for SSR as: ITEM 509 – STAINLESS STEEL REINFORCEMENT.

C304.4.1.6

SSR reinforcement weighs slightly less than ordinary steel reinforcement. Refer to ASTM A955 Table 1 for more information.

The advantages of SSR include:

- A₁. SSR offers the highest level of corrosion protection for concrete reinforcement available. The bars have no coating that can be damaged during shipping, handling, installation, concrete placement, hand chipping, and hydro-demolition.
- B₁. Structural development lengths are equal to ordinary uncoated steel reinforcement and shorter than ECSR.
- C₁. Permits field fabrication (e.g. bending and cutting)

The disadvantages of SSR include:

- A₂. AASHTO will not permit the use of SSR above Seismic Zone 1 or in areas where plastic hinging is expected.
- B₂. Equipment used to handle and fabricate stainless steel needs to be kept separate from that used to fabricate carbon steel reinforcement.
- C₂. SSR availability could be an issue and lead times would be longer than ECSR.
- D₂. Requires stainless steel or plastic ties.
- E₂. SSR has the highest initial cost for the reinforcement types listed in BDM Section 304.4.1

304.4.2 MAXIMUM LENGTH

The maximum length of fabricated concrete reinforcement shall not exceed 60-ft.

For bent or radial bars, the maximum dimension from a straight line between the ends of the bar and the apex of the bar shall not exceed 7.5-ft, See BDM Figure 304-1.



Figure 304-1

C304.4.2

The practical maximum length of concrete reinforcement is 40-ft. This limit is for both transit purposes and construction convenience. To facilitate an economical design using 60-ft bar stock, where multiple sets of lapped bars are required (e.g. longitudinal slab reinforcement) consideration should be given to using multiple sets of 30-ft long bars.

304.4.3 BAR MARKS

Provide unique Bar Marks on detail plans to identify the bar's size and general location and to reference the bar to the reinforcing bar list.

Incorporate letters 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.

Provide a note or legend with the bar list to describe each Bar Mark's meaning. Use the following abbreviations to denote the bar material: USR, ECSR, GSR, GFRP, CSR, or SSR. See BDM Figure 304-2.

C304.4.3

Examples:

An A501 bar mark represents a #5 abutment bar.

An SP401 bar mark represents a #4 spiral bar.

A DS901 bar mark represents a #9 drilled shaft bar.

Bar Mark	Mat'rl Type	Number			Length	Weight (lbs.)	Type	Dimensions						
		Rear	Fwd	Total				A	B	C	D	E	R	Inc.
Superstructure (Galvanized Steel Reinforcement – GSR)														
S401	GSR			385	30'-0"	7716	STR							
S402	GSR			77	30'-3"	1556	STR							
S403	GSR			690	7'-10"	3611	16	7'-4"						
S404	GSR			102	3'-5"	233	8	2'-1"	7"	1'-0"				
S501	GSR			425	30'-0"	13299	STR							
S502	GSR			85	32'-9"	2904	STR							
S503	GSR			708	30'-7"	22585	16	30'-0"						
S504*	GSR			708	5'-0"	3693	STR							
S505*	GSR			708	2'-7"	1908	STR							
Superstructure GSR Subtotal						57505								

Bar Mark	Mat'rl Type	Number			Length	Total Length	Type	Dimensions					
		Rear	Fwd	Total				A	B	C	D	E	R
Superstructure (Glass Fiber Reinforced Polymer Reinforcement – GFRP)													
S405	GFRP			88	40'-0"	3520'-0"	STR						
S406	GFRP			22	14'-8"	322'-8"	STR						
S407	GFRP			184	10'-0"	1840'-0"	STR						
S408	GFRP			24	6'-4"	152'-0"	25						
S409	GFRP			24	5'-1"	122'-0"	STR						
Superstructure GFRP Subtotal						5956'-8"							

Bar Mark	Mat'rl Type	Number			Length	Weight (lbs.)	Type	Dimensions						
		Rear	Fwd	Total				A	B	C	D	E	R	Inc.
Abutments (Epoxy Coated Steel Reinforcement – ECSR)														
A501	ECSR	2 Series of 6	2 Series of 6	4 Series of 6	4'-5" To 4'-7"	114	1	3'-3"	1'-4" To 1'-6"					1"
A502	ECSR	80	80	160	3'-0"	501	1	2'-4"	10"					
A503	ECSR	7	7	14	15'-9"	230	2	7'-5"	1'-2"	7'-5"				
A504	ECSR	14	14	28	9'-2"	268	STR							
A801	ECSR	14	14	28	22'-8"	1694	STR							
A802	ECSR	4	4	8	33'-6"	716	STR							
D801	ECSR	44	44	88	4'-10"	1135	18	2'-8"	1'-0"	1'-0"				
Superstructure ECSR Subtotal						4658								

* - Indicates bars with Mechanical Splices

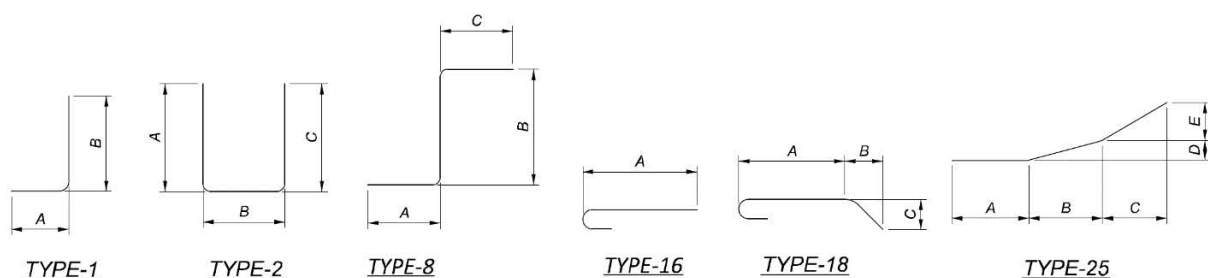


Figure 304-2

304.4.4 LAP SPLICES

Detail bar splice lengths on the plans.

Development and splice lengths shall conform to AASHTO requirements.

C304.4.4

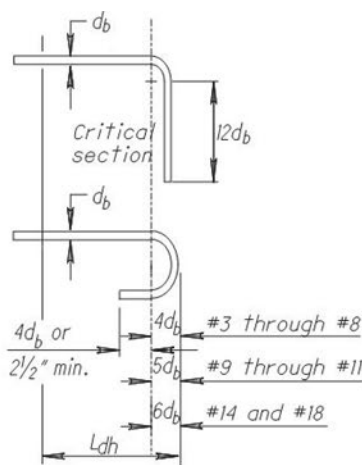
Where a horizontal construction joint is used in a column or pier, the reinforcement should be continuous, and splices avoided if at all possible. An exception to this is the construction joint between a column and a footing, where the reinforcement should be discontinuous and adequate splice length should be furnished.

For compression splice lengths of epoxy coated steel reinforcement, see BDM Figure 304-3.

For tension splice lengths of epoxy coated steel reinforcement, see BDM Figure 304-4.

For development length requirements for epoxy coated steel reinforcement, see BDM Figure 304-3, Figure 304-4 & Figure 304-5.

DEVELOPMENT LENGTH FOR ② EPOXY-COATED STANDARD HOOKS IN TENSION			
BAR NO.	MOD. FACTOR BASIC LENGTH, L_{hb}	CONCRETE COVER	TIES OR STIRRUPS
		(LRFD 5.10.8.2.4b) (1.2) 0.8	(LRFD 5.10.8.2.4b) (LRFD 5.10.8.2.4c) (1.2) 0.8
4	10	10	10
5	12	12	12
6	14	13	13
7	17	16	16
8	19	18	18
9	21	20	20
10	24	23	23
11	27	26	26
14	32	—	—
18	43	—	—



HOOKED-BAR DETAILS
FOR DEVELOPMENT
OF STANDARD HOOKS

COMPRESSION LAP SPLICES (LRFD 5.10.8.4.5a)			
BAR NO.	STANDARD	WITH SPIRALS	WITH TIES
4	15	12	12
5	19	14	16
6	23	17	19
7	26	20	22
8	30	23	25
9	34	25	28
10	38	29	32
11	42	32	35

1. VALUES SHOWN ARE FOR NORMAL-WEIGHT CONCRETE WITH $f'_c = 4$ KSI AND $F_y = 60$ KSI. IF LIGHT-WEIGHT CONCRETE IS USED, THEN CALCULATE DEVELOPMENT LENGTH FOR STANDARD HOOKS IN TENSION PER LRFD 5.10.8.2.4.

2. FOR UN-COATED REINFORCING STEEL, VALUES SHALL BE DIVIDED BY 1.2.

Figure 304-3

Tension Lap Splice Lengths for Epoxy-Coated Bars (in.)												
Bar Size	1.5" Clear Cover			2" Clear Cover			2.5" Clear Cover			3" Clear Cover		
	A	B	C	A	B	C	A	B	C	A	B	C
4	30	23	3.50	30	23	4.50	30	23	5.50	30	23	6.50
5	40	36	3.63	37	29	4.63	37	29	5.63	37	29	6.63
6	48	43	3.75	48	43	4.75	44	34	5.75	44	34	6.75
7	63	56	3.88	56	50	4.88	56	50	5.88	52	40	6.88
8	80	71	4.00	64	57	5.00	64	57	6.00	59	45	7.00
9	99	87	4.13	79	70	5.13	72	64	6.13	72	64	7.13
10	121	107	4.27	98	86	5.27	82	73	6.27	81	72	7.27
11	144	127	4.41	117	104	5.41	99	88	6.41	90	80	7.41

Notes:

- A = Tension lap splice length for horizontal reinforcement, placed such that more than 12" of fresh concrete is cast below the reinforcement, and with c/c bar spacing greater than or equal to "C".
- B = Tension lap splice length for vertical reinforcement or horizontal reinforcement, placed such that no more than 12" of fresh concrete is cast below the reinforcement, and with c/c bar spacing greater than or equal to "C".
- C = Critical c/c bar spacing. If actual c/c bar spacing is less than critical c/c bar spacing, then do not use the table above. Calculate lap splice lengths per LRFD 5.10.8.2.1a and 5.10.8.4.3a.
- Values shown are for Class B lap splices per LRFD 5.10.8.4.3a with $f'_c = 4$ ksi and $F_y = 60$ ksi.
 - The table above applies to normal-weight concrete. If light-weight concrete is used, then calculate lap splice lengths per LRFD 5.10.8.2.1a and 5.10.8.4.3a.
 - The table above assumes that $A_{tr} = 0$, where A_{tr} is the cross-sectional area of transverse reinforcement which crosses the potential plane of splitting. If such transverse reinforcement is present, and the designer wishes to account for this, then calculate lap splice lengths per LRFD 5.10.8.2.1a and 5.10.8.4.3a. One example of such reinforcement is shear stirrups.
 - Where reinforcement is in excess of that required by analysis, the values shown in the table above may be reduced per LRFD Eq. 5.10.8.2.1c-4.

Figure 304-4

Development Lengths for Epoxy-Coated Bars in Tension (in.)												
Bar Size	1.5" Clear Cover			2" Clear Cover			2.5" Clear Cover			3" Clear Cover		
	A	B	C	A	B	C	A	B	C	A	B	C
4	23	18	3.50	23	18	4.50	23	18	5.50	23	18	6.50
5	31	27	3.63	29	22	4.63	29	22	5.63	29	22	6.63
6	37	33	3.75	37	33	4.75	34	26	5.75	34	26	6.75
7	49	43	3.88	43	38	4.88	43	38	5.88	40	31	6.88
8	62	54	4.00	49	44	5.00	49	44	6.00	45	35	7.00
9	76	67	4.13	61	54	5.13	56	49	6.13	56	49	7.13
10	93	82	4.27	75	67	5.27	63	56	6.27	63	55	7.27
11	111	98	4.41	90	80	5.41	76	67	6.41	70	61	7.41

Notes:

- A = Development length for horizontal reinforcement, placed such that more than 12" of fresh concrete is cast below the reinforcement, and with c/c bar spacing greater than or equal to "C".
- B = Development length for vertical reinforcement or horizontal reinforcement, placed such that no more than 12" of fresh concrete is cast below the reinforcement, and with c/c bar spacing greater than or equal to "C".
- C = Critical c/c bar spacing. If actual c/c bar spacing is less than critical c/c bar spacing, then do not use the table above. Calculate development lengths per LRFD 5.10.8.2.1a.
- Values shown are for $f'_c = 4$ ksi and $F_y = 60$ ksi.
 - The table above applies to normal-weight concrete. If light-weight concrete is used, then calculate development lengths per LRFD 5.10.8.2.1a.
 - The table above assumes that $A_{tr} = 0$, where A_{tr} is the cross-sectional area of transverse reinforcement which crosses the potential plane of splitting. If such transverse reinforcement is present, and the designer wishes to account for this, then calculate development lengths per LRFD 5.10.8.2.1a. One example of such reinforcement is shear stirrups.
 - Where reinforcement is in excess of that required by analysis, the values shown in the table above may be reduced per LRFD Eq. 5.10.8.2.1c-4.

Figure 304-5

Lap spliced reinforcement shall extend through a phased construction joint at least the required splice length.

Consider additional extension for bars extending through a vertical construction joint to allow for placement tolerance.

Do not extend concrete reinforcement through expansion and contraction joints.

304.4.5 MECHANICAL SPLICES

Mechanical splices are an acceptable alternative to lap splices. Mechanical splices shall be in accordance with the requirements of [C&MS](#) 509.

When mechanical splices are required, designate which bar(s) will have mechanical splice(s) in the bar list.

Specify only mechanical type splices for bar sizes #14 and larger. The Department will not permit lap splices for these bar sizes.

C304.4.5

When specifying mechanical splices in congested areas, consider staggering splice locations in order to meet the minimum spacing of concrete reinforcement according to *LRFD 5.10.3.1*.

The Department will not permit splicing of reinforcing by welding unless otherwise noted.

304.4.6 MECHANICAL ANCHORING

Mechanical anchoring, such as headed reinforcing, is permitted. Design headed anchor in accordance with *ACI 318-14* 25.4.4.

C304.4.6

The *AASHTO LRFD Bridge Design Specification* does not currently have a formal specification for mechanical anchorage, however their use is permitted in *LRFD* 5.10.8.3.

304.4.7 LENGTHS AND WEIGHTS OF REINFORCEMENT

Calculate concrete reinforcement lengths to the nearest 1-in. Use the criteria specified in [C&MS](#) 509 to calculate standard bend lengths.

Calculate and include the weight of the additional 1-1/2 coils of spiral required at the end by *LRFD* 5.10.4.2 in the estimated quantities.

C304.4.7

The length or height of a spiral is defined as the distance out-to-out of coils, including the finishing turns at top and bottom.

For one, #4 spiral with a 4-1/2-in pitch, the weight, including the 1½ coils at each end, is given by the following formula:

$$W = 0.148\pi H \sqrt{\left(\frac{4.5}{2}\right)^2 + (D - 0.5)^2} + 0.167\pi(D - 0.5)$$

Where:

W = Spiral Weight (lb)

D = Outside Diameter of the Spiral (in)

H = Height or Length of the Spiral (ft)

See BDM Figure 304-6 for area, weight and diameter of standard reinforcing. See BDM Figure 304-7 for bar bending data. See BDM Figure 304-8 for standard bar length deductions of common bends.

<i>ECSR, GSR AND CSR REINFORCING BARS</i>				<i>SSR REINFORCING BARS</i>			
BAR SIZE DESIGNATION	NOMINAL AREA SQ. INCHES	NOMINAL WEIGHT POUNDS PER FT.	NOMINAL DIAMETER INCHES	BAR SIZE DESIGNATION	NOMINAL AREA SQ. INCHES	NOMINAL WEIGHT POUNDS PER FT.	NOMINAL DIAMETER INCHES
# 3	0.11	0.376	0.375	# 3	0.11	0.374	0.375
# 4	0.20	0.668	0.500	# 4	0.20	0.679	0.500
# 5	0.31	1.043	0.625	# 5	0.31	1.048	0.625
# 6	0.44	1.502	0.750	# 6	0.44	1.495	0.750
# 7	0.60	2.044	0.875	# 7	0.60	2.038	0.875
# 8	0.79	2.670	1.000	# 8	0.79	2.685	1.000
# 9	1.00	3.400	1.128	# 9	1.00	3.396	1.128
# 10	1.27	4.303	1.270	# 10	1.27	4.312	1.270
# 11	1.56	5.313	1.410	# 11	1.56	5.296	1.410
# 14	2.25	7.65	1.693	# 14	2.25	7.640	1.693
# 18	4.00	13.60	2.257	# 18	4.00	13.590	2.257

Figure 304-6

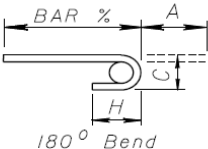
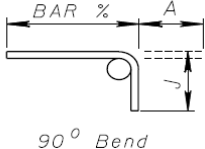
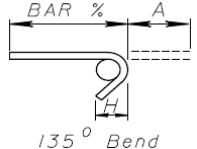
Bar Bending Data (Dimensions in inches)											
Bar Size	 180° Bend				 90° Bend				 135° Bend		
	D	A	H	C	D	A	J	J-A	D	A	H
3	2.25	5	4.00	3.00	2.25	5.0	6	1.0	1.50	4.00	2.50
4	3.00	6	4.50	4.00	3.00	7.0	8	1.0	2.00	4.50	3.00
5	3.75	7	5.00	5.00	3.75	8.5	10	1.5	2.50	5.50	3.75
6	4.50	8	5.75	6.00	4.50	10.0	12	2.0	4.50	8.00	4.50
7	5.25	10	7.50	7.00	5.25	12.0	14	2.0	5.25	9.00	5.25
8	6.00	11	8.00	8.00	6.00	13.5	16	2.5	6.00	10.50	6.00
9	9.50	15	10.00	11.75	9.50	15.5	19	3.5	-	-	-
10	10.75	17	11.50	13.25	10.75	18.0	22	4.0	-	-	-
11	12.00	19	12.75	14.75	12.00	20.0	24	4.0	-	-	-
14	18.25	27	17.25	21.75	18.25	25.0	31	6.0	-	-	-
18	24.00	36	23.25	28.50	24.00	33.0	41	8.0	-	-	-
Bending Tolerances: Refer to ACI 318 & CRSI Manual of Standard Practice											

Figure 304-7

STD. BAR LENGTH DEDUCTIONS FOR COMMON BENDS, (IN)								
BAR NO.	STANDARD BEND (DEGREES)							
	D	45	D	90	D	135	D	180
*3	1½"	¼"	2¼"	1"	1½"	1"	2¼"	1⅞"
*4	2"	¼"	3"	1"	2"	1¼"	3"	2½"
*5	2½"	⅜"	3¾"	1½"	2½"	1⅝"	3¾"	3⅞"
*6	3"	⅜"	4½"	2"	3"	2"	4½"	3¾"
*7	3½"	½"	5¼"	2"	3½"	2¼"	5¼"	4⅞"
*8	4"	½"	6"	2½"	4"	2½"	6"	5"
*9	6⅜"	⅝"	9½"	3½"	6⅜"	3⅜"	9½"	6⅞"
*10	7⅞"	¾"	10¾"	4"	7⅞"	3¾"	10¾"	7¾"
*11	8"	¾"	12"	4"	8"	4¼"	12"	8⅝"
*14	12⅛"	1"	18¼"	6"	12⅛"	5⅝"	18¼"	12"
*18	16"	1⅜"	24"	8"	16"	7½"	24"	15¾"

NOTE:

"D" IS THE DIAMETER OF THE BEND PER CONSTRUCTION
AND MATERIAL SPECIFICATIONS ITEM 509.05

Figure 304-8

304.4.8 CONCRETE REINFORCEMENT BAR LIST C304.4.8

Include the following in the bar lists:

A sample bar list is provided in BDM Figure 304-2.

A1. Bar Mark

B1. Number of bars required

C1. Overall length of each bar

D1. Total weight or length for each bar mark

E1. Column for type/shape of bar:

1. "ST" for straight, or
2. Number assigned to "Bending Diagram"

F1. "Number" and "Series" for series bars

G1. Identify bars with mechanical splices

Identify each bent bar by a unique number and illustrate in the "Bending Diagram". Define dimensions for the bent bars by letters A through Z.

Also include spiral reinforcing in the detail plan's bar list. Provide the following information on the bar list:

A₂. Core diameter

B₂. Pitch

C₂. Mark

D₂. Number

E₂. Height

F₂. Weight

304.4.9 CORROSION PROTECTION

Unless noted in the Scope of Services or as noted in BDM Section 405.2, provide epoxy coated steel reinforcement in all cast-in-place concrete.

Provide epoxy coated reinforcing steel for all approach slabs.

C304.4.9

The concrete reinforcement material for approach slabs should match that used in the superstructure concrete. The design provided by [AS-1-15](#) does not apply to approach slabs reinforced with GFRP, Stainless Steel or Chromium Steel reinforcement.

304.4.10 MINIMUM COVER FOR CONCRETE REINFORCEMENT

The minimum concrete cover shall be as follows:

- A. Bridge decks*, slab bridges* and sidewalks (top surface) 2.50-in
- B. Bridge decks and slab bridges (bottom surface) 1.50-in
- C. Footings (bottom surface) 3-in
- D. Approach slabs (top & bottom surfaces) 3-in
- E. Column ties or spirals 3-in
- F. Drilled shaft ties/spirals (dia. ≤ 3-ft) 3-in
- G. Drilled shaft ties/spirals (3-ft < dia. < 5-ft) 4-in
- H. Drilled shaft ties/spirals (dia. ≥ 5-ft) 6-in
- I. Prestressed I-beams & Box beams (side & bottom surfaces) 1.25-in
- J. Prestressed Box Beams (inside surfaces) 1-in
- K. All other concrete surfaces 2-in

* - The 1.0-in monolithic wearing surface is included in the minimum cover shown

Provide clearances in the detail plans for areas not defined in [C&MS](#) 509.04, [C&MS](#) 524 or referenced Standard Bridge Drawing.

304.4.11 MINIMUM CONCRETE REINFORCEMENT

Near exposed surfaces of walls and slabs not otherwise reinforced, provide minimum reinforcement for shrinkage and temperature according the AASHTO requirements.

304.5 PRESTRESSING STEEL**304.5.1 TYPE & SIZE**

Use the following type and size strand for prestressed box beam members:

Low-relaxation 0.520-in diameter ($A_s = 0.167\text{-in}^2$) seven wire uncoated strands, ASTM A416, Grade 270.

Use the following type and size strand for prestressed I-beam members:

Low-relaxation, 0.600-in diameter ($A_s = 0.217\text{-in}^2$) seven wire uncoated strands, ASTM A416, Grade 270.

Provide the prestressing strand type and size in the contract plan General Notes.

304.5.2 STRESSES

Initial prestressing loads for low-relaxation strand shall be as follows:

Initial stress $0.75 f_s = 202.5\text{-ksi}$

Initial tension load:

A. $A_s = 0.167\text{-in}^2$ 33.750-kip/strand

B. $A_s = 0.217\text{-in}^2$ 43.950-kip/strand

305 FOUNDATIONS**305.1 GENERAL DISCUSSION**

Analyze all structure foundations for external stability in the Strength Limit State and for vertical and horizontal movements in the Service Limit State according to *LRFD Section 10 or 11*, as applicable.

Meet all the subsurface exploration requirements for structures according to Section 300 of the [SGE](#).

For footings supported on deep foundations, neglect bearing resistance of the footing concrete on soil.

305.1.1 OVERALL STABILITY

Analyze overall (global) stability for each structure at critical locations according to *LRFD 10.5.5.2.1* and *11.6.3.7*, with the following exceptions:

C305.1

External stability analyses include: bearing resistance; eccentric load limitations; overturning analyses; sliding resistance; and overall stability.

C305.1.1

Factored loads are incompatible with limit equilibrium analysis.

A. Do not use factored loads for the analyses

B. Use the following resistance factors for the analyses:

Design all bridge substructure units and tiered retaining wall systems for overall stability using a resistance factor of $\phi = 0.65$.

Design all other retaining walls for overall stability using a resistance factor of $\phi = 0.75$.

Resistance factors as shown in *LRFD 11.6.2.3* should be taken as the inverse of the following factors of safety (FS) for the case of limit equilibrium slope stability analysis only:

A. $\phi = 0.75$ corresponds to FS = 1.3

B. $\phi = 0.65$ corresponds to FS = 1.5

305.1.2 LATERAL LOADING ON DEEP FOUNDATIONS

Perform a *p-y* analysis when analyzing deep foundations (such as driven piles, drilled shafts, micropiles, or continuous flight auger piles) under lateral loading.

C305.1.2

A *p-y* analysis should be performed using a derivative of the COM624P program (e.g. LPILE). The *p-y* analysis provides the expected lateral displacement, as well as shear and moment load effects in the deep foundation element due to applied lateral loading, moment, and displacement.

For abutments supported on MSE walls that utilize MSE reinforcements attached to the back of the abutment footing to resist lateral loading, the *p-y* analysis is not necessary.

If a *p-y* analysis on vertical piles indicates the need for battered piles, and the battered piles are designed to resist the calculated lateral loads, an additional *p-y* analysis of the battered piles is not necessary. See BDM Section 305.3.5.8 regarding the use of battered piles.

For stub abutment walls of 10-ft or less in height measured from the roadway grade to the bottom footing elevation, no *p-y* analysis is necessary as long as there is no scour extending below the bottom of footing, since the lateral load will not be enough to substantially affect the piles.

If a *p-y* analysis of the piles in the construction condition (without the superstructure installed) shows excessive stresses or deflection in the piles, provide plan notes regarding sequence of construction and limiting the placement of backfill behind the abutments before placement of the superstructure. For rehabilitation of existing full-height integral superstructures, provide plan notes to require excavation of the backfill prior to removal of the superstructure.

In the case of full-height integral abutments, the critical state will occur when there is earth pressure on the back of the abutment prior to having the superstructure in place to act as a lateral brace. [C&MS 516.07](#) will allow backfilling to the level of the beam seat prior to the completion of bearings connections. For integral construction, the superstructure will not act as a brace until the end diaphragms are complete. Designers should verify that the deep foundation elements have the capacity to resist this loading condition.

For all integral abutments in the in-service condition (with the superstructure installed), if the piles have the minimum length specified in [ICD-1-20](#) or [ICD-2-18](#), no *p-y* analysis is necessary. Otherwise, perform *p-y* analyses combining the thermal deflection of the superstructure with the lateral earth and surcharge loads to check if the piles will experience excessive stresses.

For deep foundation elements spaced closer than 5 diameters center to center, include group effects. Apply group effects only within the soil overburden length of the deep foundation. Do not apply to rock sockets.

Group effects for a single row of deep foundation elements under lateral loads (with the load oriented perpendicular to the axis of the row) can be ignored when the deep foundation elements are placed at a center-to-center spacing of 3.75 diameters or greater. For deep foundation elements spaced closer together, utilize a reduction factor as calculated in the following equation, per “Analysis and Design of Shallow and Deep Foundations” (Reese, Isenhowe, and Wang, 2006):

$$P_m = 0.64 (S/B)^{0.34} \text{ for } 1.0 \leq S/B \leq 3.75$$

Where:

P_m = p-multiplier

S = center-to-center spacing of deep foundation elements

B = diameter of deep foundation element

For multiple rows of deep foundation elements, see *LRFD 10.7.2.4* for values of P_m . See FHWA [GEC 10](#), Section 11.5.1, Table 11-1 regarding p-multipliers with a more comprehensive table of values for drilled shafts; use these same P_m for other drilled deep foundation elements (CFA piles or micropiles).

When lateral loads are controlling the design of drilled shafts, perform lateral load testing according to the Scope of Services.

Consult the Office of Geotechnical Engineering regarding specifications for lateral load testing in the Scope of Services.

To evaluate lateral deflection in the Strength Limit State (overturning resistance) see BDM Section 307.1.4. Do not analyze deep foundations with the “Geotechnical Strength Limit State” analysis, per publication FHWA-NHI-10-016, Geotechnical Engineering Circular 10 ([GEC 10](#)) “Drilled Shafts: Construction Procedures and LRFD Design Methods,” Section 12.3.3.3.1.

The “Geotechnical Strength Limit State” analysis is over-conservative for consideration of overturning stability of deep foundations.

To evaluate lateral deflection in the Service Limit State, see BDM Section 305.1.3.

305.1.3 VERTICAL AND HORIZONTAL MOVEMENTS

C305.1.3

Analyze foundation settlement according to *LRFD 10.5.2.2*.

In addition to the requirements for serviceability provided in *LRFD C10.5.2.2*, use the following criteria to establish acceptable settlement criteria:

- A. The angular rotation caused by differential settlement between adjacent substructures shall not exceed 0.004 radians.
- B. The maximum differential settlement (Δ) between adjacent substructures supporting continuous steel superstructures shall be:

$$\Delta = 0.55 \frac{L}{S} - 2.6 < 8.0$$

In which:

Δ = Maximum differential settlement (in)

L = Shortest adjacent span length (ft)

S = Beam/Girder spacing (ft)

- C. The maximum differential settlement (Δ) between adjacent substructures supporting prestressed concrete beams with a continuous deck shall be:

$$\Delta = 0.13 \frac{L}{S} - 0.17 < 2.5$$

In which:

Δ = Maximum differential settlement (in)

L = Shortest adjacent span length (ft)

S = Beam/Girder spacing (ft)

If long-term settlements encroach upon the required minimum vertical clearance, make necessary adjustments to the vertical profile.

Vertical and horizontal movements of abutment walls shall meet the limits of BDM Section 307.1.6.

For lateral displacement of foundations that support bridge structures, bridge serviceability limits shall control.

Consider the following methods for remediation to limit structural deformations due to settlement: increasing the foundation dimensions, excavating and replacing the foundation soils, adjusting the construction sequence, specifying waiting periods, application of surcharge loads, the use of deep foundations, or ground improvement. Wick drains or other subsurface drainage may be utilized to accelerate the rate of settlement. Select the method for settlement remediation based on the type of structure and time, space, and cost considerations. Refer to BDM Section 600 for applicable plan notes. A Special Provision for Installation of Wick Drains is available from the Office of Geotechnical Engineering.

Reference:

Moon, F.L., N. Romano, D. Masceri, J. Braley, N. Samtani, T. Murphy, M. Lopez de Murphy, and D.R. Mertz. 2017. *Bridge Superstructure Tolerance to Total and Differential Foundation Movements*. National Highway Cooperative Research Program (NCHRP), Web-only Document 245:

<http://www.trb.org/Publications/Blurbs/177297.aspx>

The limiting values of 8.0-in and 2.5-in are provided due to the range of applicability noted in the report referenced above for the equations shown.

305.1.4 GROUND IMPROVEMENTS**C305.1.4**

If analyses demonstrate inadequate foundation stability (bearing, sliding, or lateral resistance; overall stability; or settlement) consider using ground improvement techniques to improve the foundation soils. Refer to the Transportation Research Board SHRP2 Geotech Tools website at <http://www.geotechtools.org/> for a comprehensive list of ground improvement techniques. Refer to [FHWA-NHI-16-027/FHWA-NHI-16-028](#), Geotechnical Engineering Circular 13 (GEC 13) “Ground Modification Methods Reference Manual” for guidance on design for ground improvement techniques. Consider ground improvements for resistance of horizontal forces for structures with shallow foundations only after the improvements recommended in BDM Section 305.2.1.3 or BDM Section 307.1.3 are found to be not cost effective. Consult with the Office of Geotechnical Engineering if considering ground improvement techniques.

305.1.5 SEISMIC DESIGN**C305.1.5**

Analyze soil borings to identify overlying soil profiles that can amplify ground motion propagating from underlying rock and use the Site Class Definition as specified in *LRFD 3.10.3.1* for determination of seismic design parameters on a site-by-site basis. Use blow counts corrected to an equivalent rod energy ratio of 60%, N_{60} , as defined in the [SGE](#) for the average SPT blow count $\bar{N}$. In the absence of sufficient geotechnical information, assume Site Class D for the project soil profile. Refer to BDM Section 303.1.4.1 for additional design requirements for bridges located in Site Class D, E or F.

Use the Seismic maps, *LRFD Figures 3.10.2.1-1* through *3.10.2.1-3* or the USGS US Seismic Design Maps web application using latitude and longitude to determine the seismologic data for a project location (PGA, SS, and S1). 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. In other words, for the entire State of Ohio, assume $F_{pga} \leq 1.6$, $F_a \leq 1.6$, and $F_v \leq 2.4$. The site-specific procedure, per *LRFD 3.10.2.2* is not required for Ohio. For all other Site Class types, use Site Factors as defined in *LRFD 3.10.3.2*.

The entire State of Ohio is assumed to have a Peak Ground Acceleration Coefficient (PGA) above 0.03g but less than 0.06g, a Short-Period Horizontal Response Spectral Acceleration Coefficient (S_S) above 0.072g but less than 0.134g, and a Long-Period Horizontal Response Spectral Acceleration Coefficient (S_I) above 0.03g but less than 0.05g. In other words, per the provisions of *LRFD 3.10.3.2*, $PGA < 0.10$, $S_S < 0.25$, and $S_I < 0.1$ for the entire State of Ohio.

If sufficient geotechnical information is not available to determine Site Class for new substructure units, and the project is located in areas where $S_I \geq 0.042$, then assume $0.1 \leq S_{DI} < 0.15$. Otherwise, assume $S_{DI} < 0.10$ and $A_S \geq 0.05$.

For Ohio, only areas where $S_I \geq 0.042$, as shown in BDM Figure 305-1, and Site Class D, E or F exist, will $0.10 \leq S_{DI} < 0.15$. In Seismic Performance Zone 1, for $0.10 \leq S_{DI} < 0.15$, according to *LRFD 5.11.2*, refer to BDM Section 303.1.4.1 for additional connection and reinforcement detailing requirements. In Seismic Performance Zone 1, according to *LRFD 3.10.9.2*, the magnitude of the horizontal design connection force in the restrained directions is defined in relation to whether A_S is greater or less than 0.05.

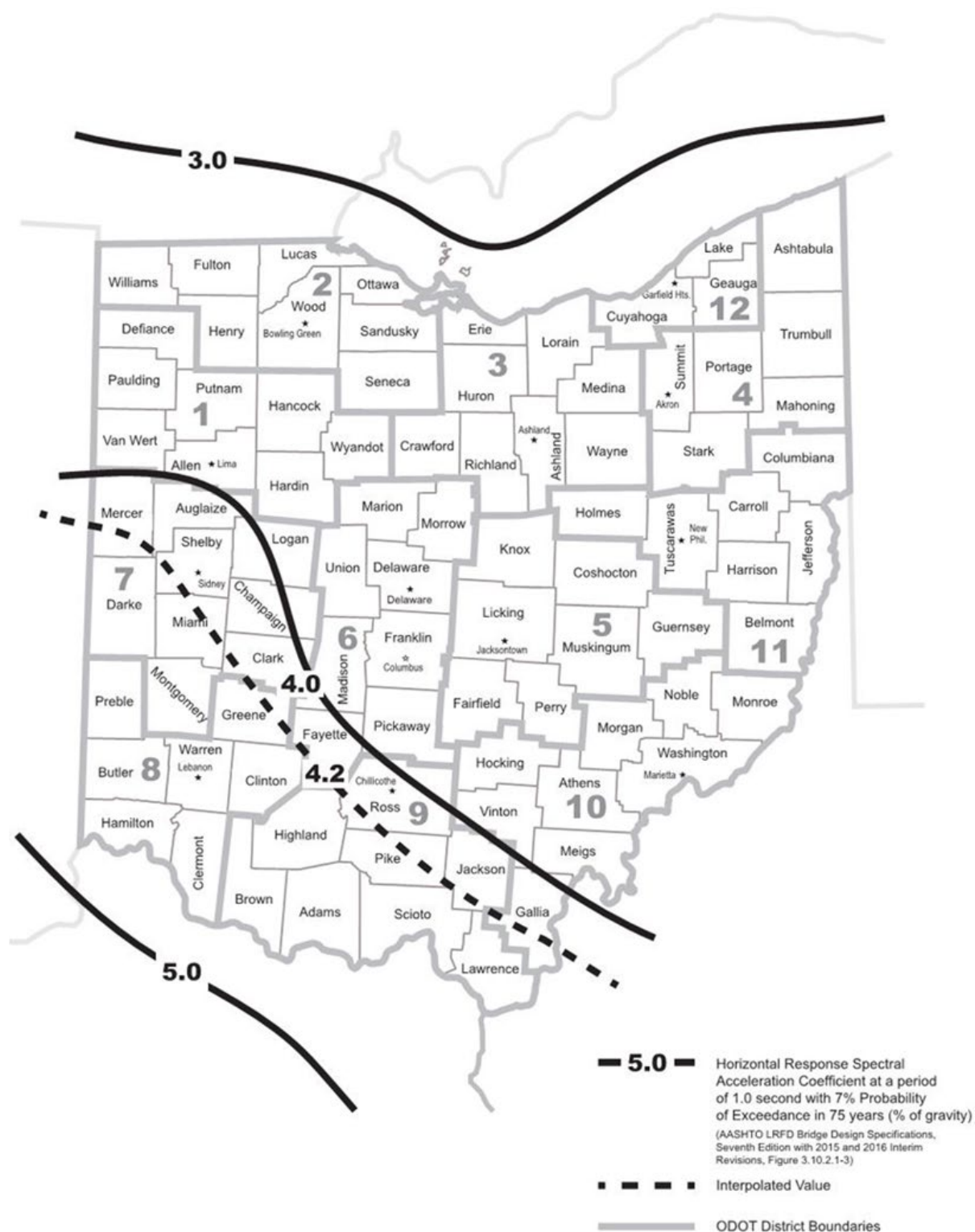


Figure 305-1

If sufficient geotechnical information is not available to determine Site Class for rehabilitation projects refer to BDM Section 303.1.4.2.

See BDM Section 303.1.4.1 for additional seismic analysis and design requirements.

305.1.6 SCOUR

All foundation requirements shall be satisfied for the changes in conditions resulting from the scour design flood at the strength and service limit states and the scour check flood at the extreme event limit states.

Determine the design flood frequency and the check flood frequency for scour, in accordance with [L&D, Vol. 2](#), Section 1008.10.5. Ignore Rock Channel Protection in the prediction of scour depths.

305.1.7 RESISTANCE FACTORS

Use resistance factors for different types of deep foundation systems for design at the strength limit state as specified in BDM Table 305-1. Additional resistance factors can be found in *LRFD 10.5.5.2*. Where resistance factors in BDM Table 305-1 differ from those in *LRFD 10.5.5.2*, the values in BDM Table 305-1 override.

C305.1.6

The changes in foundation conditions resulting from scour do not apply to the overall (global) stability

Table 305-1

Method/Soil/Condition		Resistance Factors			
		Driven Piles	Drilled Shafts	Micro-Piles	CFA Piles
Nominal Axial Compressive Resistance of Single Elements, ϕ_{stat}	Side resistance in clay	0.35	0.45	0.55	0.45
	Tip resistance in clay	0.35	0.40	N/A	0.40
	Side resistance in sand	0.45	0.55	0.55	0.50
	Tip resistance in sand	0.45	0.50	N/A	0.45
	Side resistance in cohesive IGMs	N/A	0.60	0.55	0.50
	Tip resistance in cohesive IGMs	0.45	0.55	N/A	0.45
	Side resistance in rock	N/A	0.55	N/A	0.50
	Tip resistance in rock	0.45	0.50	0.50	0.45
Block Failure, ϕ_{bl}	Clay	0.60	0.55	0.60	0.50
Uplift Resistance of Single Elements, ϕ_{up}	Clay	0.25	0.35	0.55	0.30
	Sand	0.35	0.45	0.55	0.40
	Rock	N/A	0.40	0.55	0.35
Group Uplift Resistance, ϕ_{ug}	All Materials	0.50	0.45	0.50	0.40
Horizontal Geotechnical Resistance of Single Element or Group	All Materials	1.00	1.00	1.00	0.90
2% Dynamic Load Test, ϕ_{dyn}	All Materials	0.70	0.60	0.60	0.60
100% Dynamic Load Test, ϕ_{dyn}	All Materials	0.75	0.70	0.70	0.70
Static Load Test (compression), ϕ_{load}	All Materials	0.75	0.70	0.70	0.70
2% Dynamic with Static Load Test, ϕ_{dyn}	All Materials	0.80	0.75	0.75	0.75
100% Dynamic with Static Load Test, ϕ_{dyn}	All Materials	0.85	0.80	0.80	0.80
Static Load Test (uplift), ϕ_{upload}	All Materials	0.60	0.60	0.60	0.60
Structural Resistance Factors		Driven Piles	Drilled Shafts	Micro-Piles	CFA Piles
Permanent Cased Length ²	Tension, ϕ_{TC}^1	0.80	0.90	0.80	N/A
	Compression, ϕ_{CC}	0.60	0.75	0.75	N/A
	Flexure, ϕ_f	1.00	varies ⁴	1.00	N/A
	Shear, ϕ_v	1.00	0.90	1.00	N/A
Uncased Length ³	Tension, ϕ_{TU}^1	0.80	0.90	0.80	0.80
	Compression, ϕ_{CU}	0.50	0.75	0.75	0.70
	Flexure, ϕ_f	1.00	varies ⁴	1.00	varies ⁵
	Shear, ϕ_v	1.00	0.90	1.00	0.85
¹ Consider only resistance of the steel section for Deep Foundation Elements in tension					
² Permanent Cased Length for Driven Piles represents CIP Pipe Pile					
³ Uncased Length for Driven Piles represents Steel H-piles					
⁴ Check net tensile strain per <i>AASHTO LRFD 5.6.2.1</i> and calculate ϕ_f per Figure C5.5.4.2-1 ($\phi_f = 0.75$ to 0.90)					
⁵ Check net tensile strain per <i>AASHTO LRFD 5.6.2.1</i> and calculate ϕ_f per Figure C5.5.4.2-1, except subtract 0.05 ($\phi_f = 0.70$ to 0.85)					

305.2 SPREAD FOOTINGS**305.2.1 GENERAL DISCUSSION**

Design spread footings for piers in accordance with *LRFD Section 10*. Design spread footings for abutments and retaining walls in accordance with *LRFD Section 11*. Optimize spread footing dimensions for the controlling Limit State.

Refer to BDM Section 201.2.1.3 for information to include in the Structure Foundation Exploration Report.

For the calculation of the depth correction factor d_q used in *LRFD Equation 10.6.3.1.2a-3*, use the following equation:

$$d_q = 1 + 2 \tan \phi_f (1 - \sin \phi_f) 2 \arctan (D_f/B)$$

Where:

d_q = depth correction factor (dim)

ϕ_f = angle of internal friction of soil (deg.)

D_f = footing embedment depth (ft)

B = footing width (ft)

$\arctan (D_f/B)$ is in radians.

Use this equation without any limits on the values of ϕ_f , D_f , B , or D_f/B , regardless of the limits imposed in *LRFD 10.6.3.1.2a*.

For the case of vertical or inclined eccentric loading, assume the values of $i_c = i_q = i_\gamma = 1.00$. For the case of inclined centric loading, use *LRFD Equations 10.6.3.1.2a-5* through *10.6.3.1.2a-9* for the calculation of i_c , i_q , and i_γ .

Do not use spread footing supported structures on MSE walls.

305.2.1.1 SETTLEMENT

For all spread footings, perform a settlement analysis according to *LRFD 10.6.2.4*. See BDM Section 305.1.3 for limits on vertical movement with respect to a supported bridge structure.

C305.2.1

The use of spread footings should be based on the following: depth and strength of suitable bearing materials; ease of construction; potential effects of phased construction; effects of flooding and scour; 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.

LRFD Equation 10.6.3.1.2a-1 clarification of γ_q and γ_f is no longer necessary. This change is incorporated in the *LRFD Bridge Design Specifications, 9th Edition (2020)*.

The equation is per “A Revised and Extended Formula for Bearing Capacity” (Hansen, 1970).

Spread footings are susceptible to loss of bearing caused by erosion during the service life of the structure.

C305.2.1.1

Where significant settlement of a spread footing is predicted that would affect the serviceability of the supported structure, provide elevation reference monuments. See BDM Section 600 for notes and additional guidance.

Significant settlement is defined as that exceeding the serviceability limits in BDM Sections 305.1.3 or 307.1.6. Elevation reference monuments are for the purpose of measuring footing elevations during and after construction to document the performance of the spread footings, both short term and long term.

305.2.1.2 MINIMUM DEPTH AND SCOUR CONSIDERATIONS

305.2.1.2.a SPREAD FOOTING ELEVATIONS FOR FOUNDATIONS LOCATED OUTSIDE THE LIMITS OF THE 100-YR FLOOD PLAIN

The following requirements apply to spread footings located outside of the plan view limits of the 100-yr flood plain:

- A. For footings founded on rock, key the bottom of footings at least 3-in into rock.
- B. Embed the tops of footings founded on soil at least 1-ft from the nearest soil surface.
- C. Embed the bottoms of footings founded on soil at least 4-ft from the nearest soil surface.
- D. Embed the bottoms of footings founded on embankment fill at least 5-ft from the nearest soil surface. See BDM Figure 305-2.
- E. In no case shall the bottom of the footings in existing soil or on embankment fills be above the frost line.

Refer to BDM Figure 305-3 to determine the frost depth at the project site in inches. If the project site lies between two contour lines, choose the contour line with the greater frost penetration depth.

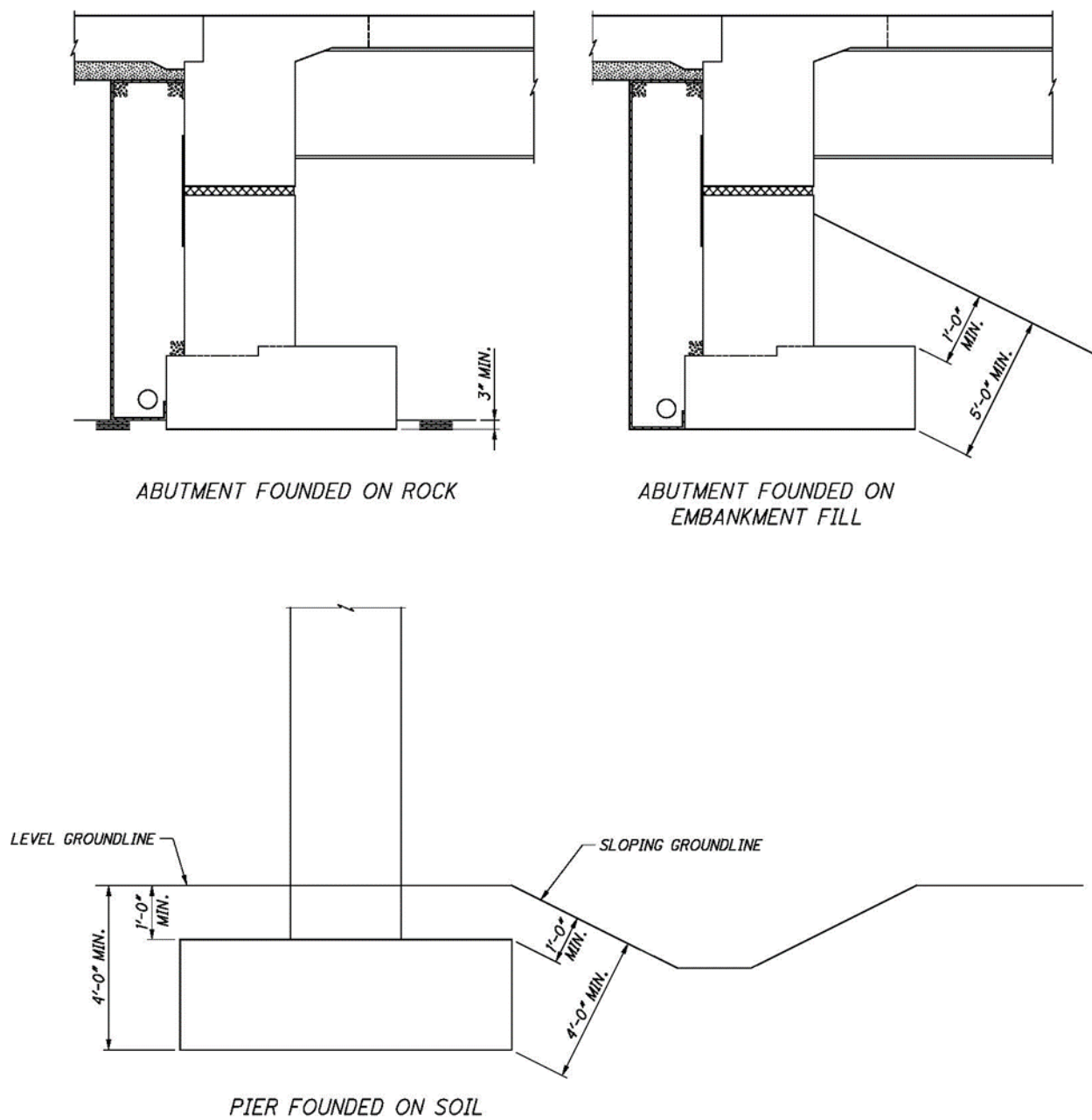


Figure 305-2

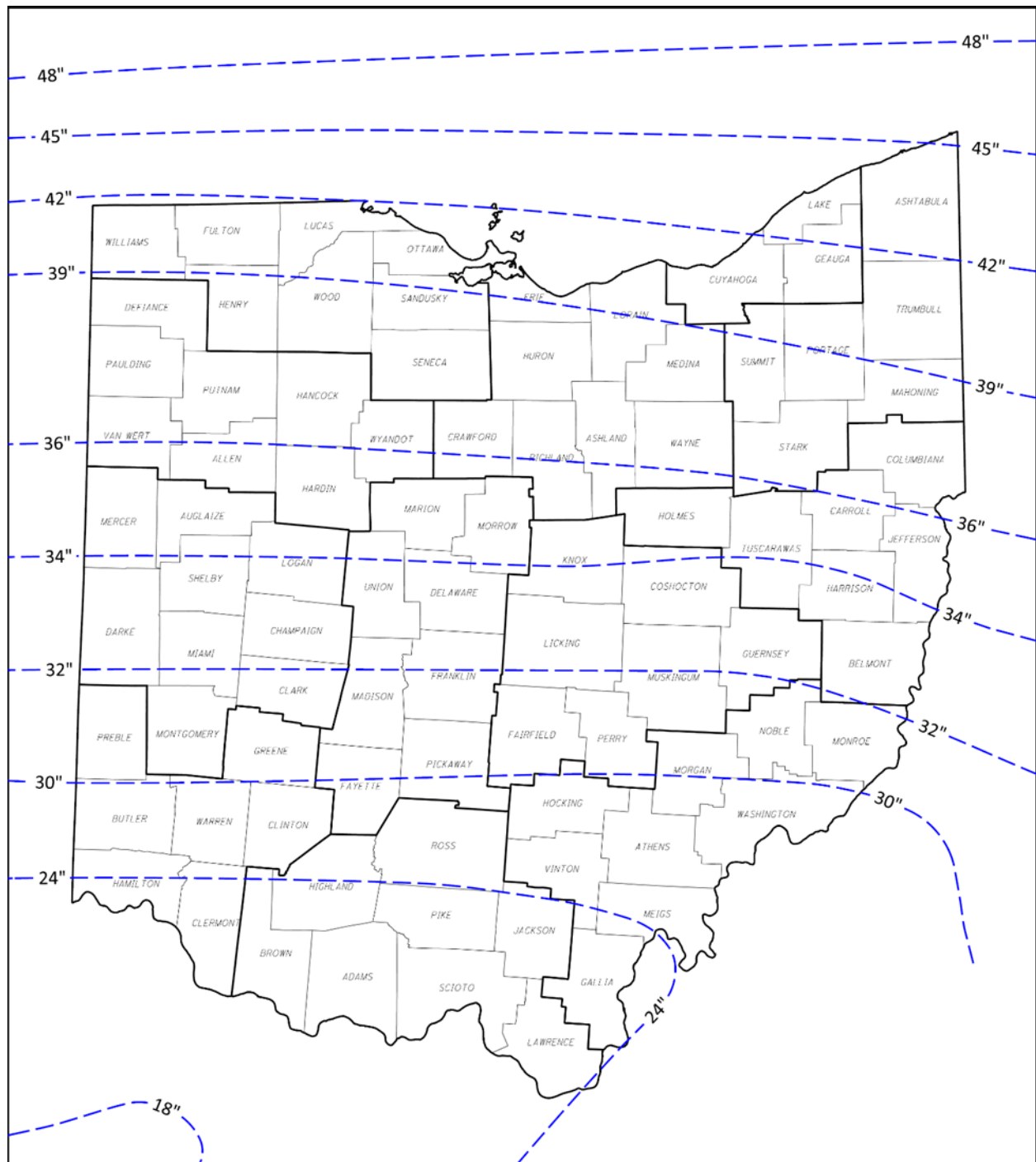


Figure 305-3

**305.2.1.2.b SPREAD FOOTING ELEVATIONS
FOR FOUNDATIONS INSIDE THE
LIMITS OF THE 100-YR FLOOD
PLAIN**

C305.2.1.2.b

The following requirements apply to spread footings located inside of the plan view limits of the 100-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 [GDM](#) Section 1400.
- B. Except as noted in BDM Section 305.2.1.2.b.(A) and **305.2.1.2.b.(C)**, locate the bottom of footings directly on scour resistant rock. If footings require lateral restraint, provide drilled and grouted steel anchors. Scour resistant rock shall have the following properties to an elevation at least 4-ft below the Thalweg:
 1. Unconfined compressive strength, $Q_u \geq 2500$ -psi, per ASTM D7012, Method C
 2. Slake Durability Index, $S_{DI} \geq 90\%$, per ASTM D4644
 3. Rock Quality Designation, $RQD \geq 65\%$, per [SGE](#) Section 6
 4. Total Unit weight ≥ 150 -pcf
 5. Rock Mass Strength Properties:
 - a. Rock Mass Rating, $RMR \geq 75$ or
 - b. Geologic Strength Index, $GSI \geq 75$ with Very Good or Good Joint Surface Conditions and Massive or Blocky Structure
 6. Erodibility Index, $K \geq 100$, per publication FHWA-HIF-12-003 (HEC 18) "Evaluating Scour at Bridges," Section 4.7.2, where:
 - a. $K = (M_s)(K_b)(K_d)(J_s)$, $K_b = RQD/J_n \geq 0.10$, and $K_d = J_r/J_a$
 - b. For the Intact Rock Mass Strength Parameter, M_s , use the following equations:
 - i. $M_s = Q_u$ for $Q_u \geq 10$ -MPa, or
 - ii. $M_s = (0.78) Q_u^{1.05}$ for $Q_u < 10$ -MPa, where
 - iii. Q_u is in units of MPa, per USBR publication "Best Practices in Dam and Levee Safety Risk Analysis," Chapter IV-1.

Where $RQD = 0$, Block Size Parameter $K_b = RQD/J_n = 0$, and subsequently Erodibility Index $K = 0$. In the scour calculations that depend on K , it is in the denominator, and $K = 0$ will result in a divide by zero error.

- c. See GDM Section 1302.1.4 for recommended values of Rock Joint Set Number, J_n , Joint Roughness Number, J_r , Joint Alteration Number, J_a , and Relative Joint Orientation Parameter, J_s . Conservative limit values for these parameters are $J_n \leq 5$, $J_r \geq 1$, $J_a \leq 5$, and $J_s \geq 0.4$.
 - 7. For interbedded rock formations, consider only the weaker material.
 - 8. No Ordovician bedrock formation may be considered as scour resistant rock.
- C. Except as noted in BDM Section 305.2.1.2.b.(A) and 305.2.1.2.b.(C), locate the bottom of footings founded on non-scour resistant rock at least one foot below the rock scour depth as calculated according to HEC 18. If historical evidence of scour deeper than the calculated depth 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 305.2.1.2.b.(A), footings founded on soil are prohibited. For substructures founded on deep foundations not socketed into bedrock, extend the piles or drilled shafts to at least 15-ft below either the Thalweg elevation or the maximum estimated scour depth, whichever is deeper. For drilled shafts socketed into bedrock, see BDM Section 305.4.1.1.
- E. At the design flood scour condition, evaluate the foundations at the Strength I Limit State for structural strength and external stability.

**305.2.1.2.c 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. At the check flood, evaluate the foundations at the Extreme Event II Limit State for structural strength and external stability.
- B. Include live loads on the structure and foundations (LL and LS), and water load from stream pressure (WA).
- C. Include the maximum estimated scour depth at the check flood, considering any streambed or bank material displaced by scour from the check flood.
- D. All Resistance Factors shall be assumed to be 1.00 for the Extreme Event II Limit State analysis with the check flood.

305.2.1.3 RESISTANCE TO HORIZONTAL FORCES

If the sliding resistance of the foundation material is inadequate to withstand the horizontal forces, as determined in accordance with *LRFD 10.6.3.4*, provide additional resistance using one or more of the following means, listed in order of preference:

- A. Increase the footing width.
- B. Use a footing shear key and utilize only the passive pressure acting on the key. Locate the shear key within the middle two-thirds of the footing width.
- C. Use deep-foundation supported footings. For footings supported on deep foundations, neglect frictional resistance of the footing concrete on soil.
- D. Use ground anchors, struts, deadmen, or soil reinforcements. Extend these elements outside the active failure wedge of the footing or retaining wall. Use only the portion of the horizontal resistance located outside of the active wedge.

Do not utilize passive resistance against sliding unless the foundation penetrates below frost depth or soil disturbance. See *LRFD 11.6.3.5*. If utilities or pavement will exist on the passive resistance side of the foundation, assume a minimum depth of soil disturbance of 3 feet.

305.2.1.4 CONCRETE REINFORCEMENT

Locate bottom transverse footing reinforcement (i.e. reinforcement parallel to the centerline of the substructure) at the minimum concrete cover. Locate bottom longitudinal footing reinforcement (i.e. reinforcement perpendicular to the centerline of the substructure) above the bottom transverse footing reinforcement.

Locate the hooks for footing dowels in the plane of the bottom mat of footing concrete reinforcement.

For piers located within embankment slopes, the minimum footing dowel reinforcement ratio shall be 0.79-in²/ft. Otherwise the minimum footing dowel reinforcement ratio shall be 0.44-in²/ft.

Provide a minimum reinforcement ratio of 0.20 in²/ft at the following footing locations:

- A. The bottom of the toe and the top of the heel of a spread footing for a cantilever retaining wall or abutment.
- B. The top of all pier footings.

C305.2.1.3

Soil reinforcements are generally geogrids or metallic reinforcements that are typically used for mechanically stabilized earth structures.

The opposite is also the case: passive resistance can be used if the foundation penetrates below frost depth or probable soil disturbance.

C305.2.1.4

Footings for this provision include spread footings and footings supported on deep foundations. Refer to BDM Section 304.4.10 for concrete cover requirements.

Footing dowels are the reinforcement that laps with the primary (vertical) steel in the substructure unit. This provision provides a location to tie the footing dowel reinforcement.

305.2.1.5 DESIGN CONSIDERATIONS

For cap and column piers on spread footings placed on existing soils or on embankment fills, provide continuous footings which 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.

For cap and column piers with spread footings on bedrock provide separate footings under each column

For grade separation structures, locate the top of pier footings in accordance with BDM Section 305.2.1.2.

The spread footing width for a pier shall not be 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.

The minimum width of abutment and retaining wall spread footings founded on soil shall be one-half the wall height (H). There is no minimum width for abutment and retaining wall spread footings founded on bedrock.

305.2.2 SPREAD FOOTINGS ON COHESIONLESS SOILS

For spread footings founded on cohesionless soils, determine settlement due to the incremental loading during the construction sequence.

305.2.3 SPREAD FOOTINGS ON COHESIVE SOILS

Do not found spread footings for piers and abutments on cohesive soils with Undrained Shear Strength (S_u) less than 2.0-ksf. There is no limitation on S_u for foundation soils under spread footings for all other types of retaining walls.

Do not found spread footings on cohesive soils when secondary settlements will directly affect the performance of the structure over its service life.

305.2.4 SPREAD FOOTINGS ON BEDROCK

Use $\phi_\tau = 0.9$ as the sliding resistance factor for pier footings founded on weak or very weak bedrock ($Q_u < 1.5$ -ksi).

C305.2.1.5

The continuous footing requirement provides resistance to differential settlement between columns. The footing dimension requirement provides approximately balanced moments.

Measure the pier height from the bottom of the footing to the top of the highest beam seat.

Refer to *LRFD Figure 11.6.3.2-1* for a definition of (H). For semi-integral abutments, measure the height to the beam seat for this purpose.

C305.2.3

This limitation also does not apply to culvert headwall and wing wall spread footing foundations. Refer to [GDM](#) Section 1400 for guidance.

C305.2.4

A sliding resistance check may be non-performed for footings keyed 3-in into slightly strong or stronger bedrock ($Q_u \geq 1.5$ -ksi), as long as there is no evidence of features of the bedrock structure that may constitute planes of weakness such as laminations or interbedding.

305.3 DRIVEN PILES**305.3.1 GENERAL DISCUSSION**

Design driven piles in accordance with *LRFD 10.7* and install in accordance with [C&MS](#) 507 and 523.

Refer to BDM Section 201.2.1.3 for information to include in the Structure Foundation Exploration Report.

For the piles in each substructure unit, provide the following in the structure general notes:

- A. For piles driven to refusal on bedrock, the total factored load in axial compression.
- B. For friction piles, the Ultimate Bearing Value (UBV).
- C. For laterally loaded piles, externally applied lateral loads and bending moments, and maximum values of internal shear and moment.

Provide a unique number designation for each pile in the plans.

305.3.1.1 PILE TYPES

Steel H-piles shall meet the requirements of ASTM A572 Grade 50.

Cast-in-place (CIP) reinforced concrete pipe piles shall meet the requirements of ASTM A252 for steel pipe piles Grade 2 ($F_y = 35$ -ksi) or Grade 3 ($F_y = 45$ -ksi). Do not use Grade 1 steel pipe piles. Closed-end CIP reinforced concrete pipe piles shall not exceed 24-in nominal diameter. The Department will define large-diameter open-ended pipe piles as having a minimum 36-in nominal outside diameter.

Specify the wall thickness and Grade of closed-end CIP reinforced concrete pipe piles. The minimum wall thickness is 0.250-in. Increase the wall thickness as necessary to avoid overstressing the pile during driving, as determined by the wave equation drivability analysis required in BDM Section 305.3.1.2.

C305.3.1

Consider driven piles when they become economically viable, or if they are needed for resistance against lateral and uplift loads. If artesian conditions are encountered, consult with the Office of Geotechnical Engineering regarding pile type selection.

C305.3.1.1

The commonly available steel H-pile sizes are HP10x42, HP12x53, and HP14x73. Refer to the AISC Steel Construction Manual for section properties of available H-piles.

The commonly available closed-end CIP reinforced concrete pipe pile sizes are 12-in, 14-in, and 16-in nominal diameter. Refer to publication [FHWA-NHI-16-009/FHWA-NHI-16-010](#), Geotechnical Engineering Circular 12 (GEC 12) “Design and Construction of Driven Pile Foundations” for section properties of available CIP reinforced concrete piles.

Start the drivability analysis for CIP piles with Grade 2 material and the minimum wall thickness. Before increasing the wall thickness, switch to Grade 3 material. Choose the appropriate sample Plan note in BDM Section 600 to specify the wall thickness. Specify the material Grade and strength in the Plan note for Design Data provided in BDM Section 602.

Detail large-diameter open-ended pipe piles as filled with Class QC1 concrete to the clean out elevation shown on the plans, after pile installation. Specify the Grade and the wall thickness and increase the thickness as necessary to avoid overstressing the pile during driving, as determined by the wave equation drivability analysis required in BDM Section 305.3.1.2.

The Department does not restrict the use of precast reinforced concrete piles or precast prestressed concrete piles.

Do not use timber piles.

305.3.1.2 PILE DRIVING HAMMERS

Perform a wave equation drivability analysis to determine whether the pile can be driven without overstressing the pile, utilizing a pile driving hammer commonly available in Ohio, to achieve refusal on bedrock, the design Ultimate Bearing Value, or the pile tip elevation. If the analysis indicates blow counts higher than 100 bpf prior to reaching the required bearing depth or resistance, consider this as practical refusal, and choose a hammer of higher rated energy.

For point bearing piles on bedrock, select a hammer that is capable of reaching and penetrating bedrock for the specified pile type and size. Refusal criteria is met during driving when the pile penetrates into bedrock 1-in or less after receiving at least 20 blows from the pile hammer.

For friction piles, select a pile hammer large enough to achieve the specified Ultimate Bearing Value. Perform a dynamic load test to verify that the Ultimate Bearing Value is achieved. Refer to BDM Section 305.7 for specific pile testing requirements.

It is preferable to specify a single wall thickness across smaller structures (one to three spans) whenever the piles are of the same diameter, so that the piles cannot be mixed up and the wrong piles installed at a substructure unit. If a structure has different size piles at the abutments and the pier(s) – for example 12-inch and 16-inch respectively – then different pile wall thicknesses can be reasonably specified for the two sizes without any worry of mixing up the piles by the pile installer.

If considering large-diameter open-ended pipe piles, consult with the Office of Geotechnical Engineering. The soil inside large-diameter open-ended pipe piles should be removed to the elevation shown on the plans. Clean out requirements should be specified on the plans. Formation of the pile plug often does not occur in large-diameter open-ended pipe piles without providing a steel plate annulus to facilitate intentional plugging. Estimation of pile lengths should be based on an iterative, rigorous analysis, considering potential of pile plug formation during driving.

Currently no construction specifications exist for precast concrete piles.

C305.3.1.2

Single-acting diesel pile driving hammers are commonly available in Ohio up to a rated energy of up to 44,000-ft-lb which corresponds roughly to a Delmag D 19-42 hammer. Wave equation analyses should typically start with this size hammer. Other types and larger sized pile driving hammers have higher mobilization costs, which should be considered in the economics of the design.

A wave equation analysis is not necessary for point bearing piles on bedrock when all three of the following apply:

- A. Bedrock is 50 ft or less in depth.
- B. All granular soils are medium dense or less in density ($\text{SPT } N_{160} \leq 30$ bpf).
- C. All cohesive soils are very stiff or less in consistency ($\text{SPT } N_{60} \leq 30$ bpf).

Determine the limiting driving stress on the pile (σ_{dr}) in accordance with LRFD 10.7.8 using $\phi_{da} = 1.00$.

In which:

ϕ_{da} = resistance factor for driven piles, drivability analysis (LRFD Table 10.5.5.2.3-1).

If the analysis indicates overstress to the pile, either 1.) increase the yield strength of the pile steel and provide a plan note specifying the new minimum yield strength, 2.) choose a thicker pile wall and provide a plan note specifying the minimum pile wall thickness, or 3.) try a smaller hammer and provide a plan note specifying the pile driving hammer **maximum** rated energy. If analysis with a Delmag D 19-42 hammer indicates that the required bearing depth or resistance cannot be reached without driving refusal, use a larger hammer and provide a plan note specifying the pile driving hammer **minimum** rated energy.

A sample note for hammer maximum or minimum rated energy is provided in Section 600.

305.3.2 LOAD EFFECTS

For each point bearing pile on bedrock, provide the total factored axial pile load in the plan General Notes.

Determine the total factored axial pile load (Q_p) using BDM Eq. C305.3.2-1.

C305.3.2

ODOT [C&MS](#) 507.04 provides three criteria to specify proper pile installation depth:

A. Refusal

The Refusal criteria is defined in BDM Section 305.3.1.2 and by plan note in BDM Section 600 as pile penetration of an inch or less after receiving at least 20 blows from a pile hammer sufficiently sized to achieve the required depth to bedrock. For piles driven to Refusal, the total factored axial load applied to a point bearing pile on bedrock (Q_p) is resisted by the factored axial structural compressive resistance of the pile (P_r) determined according to *LRFD 6.15.3.1* Determine the total factored axial pile load as follows:

$$Q_p = \sum \eta_i \gamma_i Q_i + \eta_i \gamma_p DD \leq P_r \dots \dots \dots (C305.3.2-1)$$

Where:

$\sum \eta_i \gamma_i Q_i$ = Sum of factored loads for highest loaded pile at each substructure unit (kips)

$\eta_i \gamma_p DD$ = Factored drag load per pile (kips); see BDM Section 305.3.2.2.

η_i = Load modifier relating to ductility, redundancy and operational classification (see *LRFD 1.3.2.1*)

γ_i = Load factor (see *LRFD 3.4*)

γ_p = Load factor for drag load = 1.05

P_r = factored axial structural compressive resistance of the pile (*LRFD 6.15.3*). Consider the depth of scour as an unbraced length. If including drag load,

For each friction pile, provide the Ultimate Bearing Value (UBV) in the plan General Notes.

Determine the UBV using BDM Eq. C305.3.2-3.

use the resistance factor for steel piles in compression under good driving conditions (*LRFD* 6.5.4.2). Otherwise, use the resistance factor for steel piles in compression and subject to damage due to severe driving conditions (*LRFD* 6.5.4.2 and BDM Table 305-1).

B. Ultimate Bearing Value (UBV)

The driving criteria for *UBV* is established in [C&MS](#) 507.05 using the results of dynamic pile testing specified in [C&MS](#) 523. The Ultimate Bearing Value is the final maximum unfactored resistance that an individual pile is expected to supply. The *EOID* is the unfactored pile resistance at the end of initial driving defined as R_{ndr} in *LRFD* 10.3.

Determine the *EOID* and *UBV* as follows:

$$EOID = R_n + R_{Ssc} - R_{Ssu} \dots\dots\dots (C305.3.2-2)$$

And:

$$UBV = EOID + R_{Ssu} \dots\dots\dots (C305.3.2-3)$$

In which:

R_n = Nominal Pile Bearing Resistance [*LRFD* 10.3]

$$R_n = Q_p / \phi_{dyn} \dots\dots\dots (C305.3.2-4)$$

R_{Ssc} = Nominal amount of side resistance (kips) that must be overcome during driving through soil in the scour zone calculated using static analysis methods, per *LRFD* 10.7.3.8.6.

R_{Ssu} = Additional amount of resistance (kip) to account for setup of the side friction after *EOID*. See BDM Sections 0 and 305.3.5.9 for how and when to utilize this value in design. Typically, driven piles are not designed to account for setup and $R_{Ssu} = 0$. In which case, $UBV = EOID$.

Where:

Q_p = Total factored axial pile load (kip) for the highest loaded pile at each substructure unit. In the case of friction piles, drag load is considered separately according to BDM Section 305.3.2.2.

$$Q_p = \sum \eta_i \gamma_i Q_i \leq P_r \dots\dots\dots (C305.3.2-5)$$

ϕ_{dyn} = Resistance factor for driven piles, dynamic analysis and static load test methods (BDM Table 305-1) taken as 0.70 for piles installed according to [C&MS](#) 507 using dynamic test methods according to [C&MS](#) 523 (dim).

For piles with uplift refer to BDM Section 305.3.2.3 and specify a pile tip elevation in the Plans.

Provide the estimated lengths of piles on the Final Structure Site Plan.

Determine the estimated length for friction piles using static analysis methods to calculate the length of pile necessary to develop the *UBV* defined according to BDM Eq. C305.3.2-6.

When performing static analysis methods for friction piles, do not use the Tomlinson 1979 model in the analyses.

Only use the Tomlinson 1980 overlying sand model if the overlying layer has (Combined Gravel and Coarse Sand Sizes $\geq 35\%$) AND ($P_{200} \leq 30\%$) AND ($PI \leq 8\%$). If a "sand" layer does not meet the above qualifications, or if it is intermittent, consider modeling the layer as cohesive.

Only use the Tomlinson 1980 overlying soft clay model if the overlying layer has an undrained shear strength less than or equal to 750 psf, and an undrained shear strength less than or equal to 1/4 of the undrained shear strength less of the underlying layer.

305.3.2.1 SCOUR

Where scour is predicted, neglect the pile resistance provided by soil in the scour zone. Use the depth of scour resulting from the design flood, per *LRFD* 2.6.4.4.2, with Strength and Service Limit State checks. Use the depth of scour resulting from the check flood with Extreme Event II Limit State checks.

See BDM Section 306.1.1 for pile supported footing elevation requirements.

If the design flood scour reaches top of rock, do not use driven piles

C. Pile tip elevation

A pile tip elevation is required where a specific length of side resistance is required or where a pile needs to bear on a specific subsurface profile material.

Estimate pile length for friction piles using the following equation:

$$UBV = R_p + R_s + R_{ssc} \dots\dots\dots (C305.3.2-6)$$

Where:

R_p = Nominal pile tip resistance (*LRFD* 10.7.3.8.6a) (kip)

R_s = Nominal pile side resistance (*LRFD* 10.7.3.8.6a) (kip) for soil below the scour zone

The Tomlinson 1979 model was superseded by the Tomlinson 1980 models; furthermore, *LRFD* recognizes only the 1980 models.

The Tomlinson 1980 overlying sand model represents the frictional resistance of a cohesive layer as increased by a significant amount by granular material being freed from the overlying sand or sandy gravel layer. This model should not be used for cohesive soil layers below granular layers that consist primarily of fine sands or silts, or which contain a significant portion of fines or a high plasticity, such that a significant amount of coarse sand or gravel cannot be freed from the matrix of the soil. In these cases, the predicted effect is considered unconservative.

The Tomlinson 1980 overlying soft clay model represents the frictional resistance of a cohesive layer as reduced by softer overlying clay being drawn down during driving to lubricate the pile. The overlying clay needs to be soft enough to draw down, and significantly weaker than the underlying layer to have an effect.

C305.3.2.1

The maximum estimated scour depth may occur with the design flood, check flood or an overtopping flood of lesser recurrence interval.

For friction piles, include the side resistance from the soil within the scour zone (R_{sc}) in the Ultimate Bearing Value. Determine the UBV according to BDM Section 305.3.2.

A plan note is available in BDM Section 600.

Because the pile will lose support along the scour depth, investigate the structural capacity of the pile considering the depth of the scour as an unbraced length. Also perform a p-y analysis on the pile according to BDM Section 305.1.2 to demonstrate:

- A. Lateral stability against overturning failure in the Strength Limit State with the maximum estimated scour depth at the design flood.
- B. Lateral stability against overturning failure in the Extreme Event II Limit State with the maximum estimated scour depth at the check flood
- C. Excessive deflection in the Service Limit State at the design flood.

Extend the piles to penetrate a minimum of 15-ft below the maximum estimated scour depth.

The plan note in BDM Section 600 specifies driving to the UBV or a tip elevation, whichever is deeper.

305.3.2.2 DOWNDRAG & DRAG LOAD

C305.3.2.2

Assume downdrag to act if ground settlement of greater than or equal to 0.4-in will occur after pile installation at the substructure unit in question.

When soil moves downward relative to the pile, it creates a drag load on, and therefore within, the pile. The downward soil movement creates the potential for downward pile movement. This downward pile movement is referred to as downdrag. The subsurface conditions, pile installation methods, pile loading sequences, as well as the pile and ground surface configuration determine the magnitude of the drag load and the downdrag movement.

For all friction piles subject to downdrag, calculate the location of the neutral plane per the Goudreau and Fellenius (1994) method as described in [FHWA-NHI-16-009/FHWA-NHI-16-010](#), Geotechnical Engineering Circular 12 (GEC 12) “Design and Construction of Driven Pile Foundations,” Section 7.3.5.7. For all point bearing piles on bedrock, consider the neutral plane to be at the top of bedrock.

Analyze downdrag and drag load according to the Siegel et al. (2013) method as described in [GEC 12](#), Section 7.3.6.1, except as modified below.

The factored structural axial resistance of the pile at the Strength Limit State shall equal or exceed the combined effect of the factored drag load and the sum of factored loads for the highest loaded pile at each substructure using BDM Eq. C305.3.2.2-1.

Use the following equation:

$$Q_p = \sum \eta_i \gamma_i Q_i + \eta_i \gamma_p DD \leq P_r \quad (\text{C305.3.2.2-1})$$

Where:

Q_p = Total factored load (kips) in axial compression per pile for highest loaded pile at each substructure unit. This will include the loads imposed by the supported infrastructure and the drag load.

$\sum \eta_i \gamma_i Q_i$ = Sum of factored loads for highest loaded pile at each substructure unit (kips)

η_i = Load modifier relating to ductility, redundancy and operational classification (see LRFD 1.3.2.1)

γ_i = Load factor (see LRFD 3.4)

DD = Nominal drag load (downdrag load) per pile (kips)

γ_p = Load factor for drag load = 1.05

P_r = Factored structural axial resistance per pile (kips), calculated per BDM Section 305.3.3 and LRFD 6.15.3.1.

When calculating the factored axial structural compressive resistance of the pile (P_r), use the resistance factor for steel piles in compression under good driving conditions (LRFD 6.5.4.2) if including drag load. Otherwise, use the resistance factor for steel piles in compression and subject to damage due to severe driving conditions (LRFD 6.5.4.2 and BDM Table 305-1).

The reasoning for using the resistance factors for piles in compression under good driving conditions when considering drag load is that the maximum load in the pile when including drag load will occur somewhere in the middle of the pile, not at the head or toe of the pile. The highest driving stresses occur at the extreme ends of the pile – either at the head or toe – where driving damage is more likely. Therefore, the lesser resistance factors for piles in compression subject to damage due to severe driving conditions are to only be used for consideration of factored structural axial resistance per pile versus the total factored axial load per pile excluding drag load.

If greater resistance to drag load is necessary, consider using larger piles or increasing the number of piles and reducing the applied load per pile. To reduce or eliminate downdrag, consider preloading the soil so settlement occurs before pile installation. Also consider installing wick drains and an additional earth surcharge load to decrease the amount of time required for settlement to occur. Plan notes regarding pile driving waiting periods are available in BDM Section 600. A Special Provision for Installation of Wick Drains is available from the Office of Geotechnical Engineering.

Drag load is the total sum of skin friction in the downwards direction (negative skin friction) for the length of the pile in contact with the soil above the neutral plane. Calculate skin friction and drag load using static analysis methods, according to *LRFD 10.7.3.8.6* assuming 100% mobilization of tip (toe) resistance. Use a structural resistance factor for piles in compression and subject to damage due to severe driving conditions according to *LRFD 6.5.4.2*.

Do not specify battered piles when drag load is anticipated.

Evaluate the downdrag movement, as it contributes to pile head settlement, at the geotechnical Service Limit State. Consider the pile downdrag movement (S_{dd} = settlement due to downdrag) equal to the settlement of the foundation soil at the location of the neutral plane, plus the elastic compression of the pile above the location of the neutral plane due to the effects of the permanent load at the pile head plus the drag load. Calculate settlement of the foundation soils according to *LRFD 10.6.2.4*. Calculate elastic compression of the pile according to [GEC 12](#), Section 7.3.5.1. Exclude transient loads from the calculation of settlement and elastic compression.

For point bearing piles on bedrock, include both transient loads and drag load in the computation of total factored axial load per pile, and provide this value in the structure general notes.

For friction piles, do not include both transient loads and drag load in the computation of total factored axial load per pile. If transient loads exceed the value of drag load, include only transient loads. If transient loads are less than the value of drag load, include only the drag load.

305.3.2.3 UPLIFT

When a pile must resist uplift loads, calculate the uplift resistance in accordance with *LRFD 10.7*. Use static analysis methods to determine the nominal uplift resistance due to side resistance.

Where the estimated pile length is controlled by the required uplift resistance, specify a pile tip elevation

If site constraints or other factors make permanent tensile loads on driven piles unavoidable, use a redundancy load modifier of $\eta_R = 1.05$ for the permanent tensile loads on the foundation elements.

Alternately, tip resistance versus displacement curves (t - z and q - z curves), along with a comparison to the predicted settlement of the foundation soil, may be used in a more refined iterative approach.

The additional bending forces imposed on the piles by the drag load can cause pile deformation and damage.

C305.3.2.3

Driven piles under permanent tensile loads are generally not permitted.

A plan note is available in BDM Section 600.

305.3.2.4 SETUP

When pile setup is addressed according to BDM Section 305.3.5.9.A, determine the components of the nominal pile side resistance according to BDM Eq. C305.3.2.4-1. Determine the *EOID* and *UBV* values according to BDM Eq. C305.3.2.4-4 and C305.3.2.4-5 respectively and perform both a “driving resistance” analysis, taking into account the driving losses to side frictional resistance experienced by the pile during driving and prior to setup, and an “ultimate resistance” analysis for the long-term condition after pile setup has occurred. Use BDM Table 305-2 to determine the setup factor for each soil layer based on Soil Class. If the estimated driving resistance indicates driving losses that would increase the length of the pile during driving by more than 10-ft with *EOID* compared to the *UBV*, then account for pile setup in the design. Refer to BDM Section 600 for a sample plan notes.

Perform dynamic load testing on the piles to *EOID* and measure the *EOID* resistance. Specify a waiting period and restrike dynamic testing of the piles to field-verify that the *UBV* resistance is achieved.

C305.3.2.4

Dissipation of excess pore water pressure in foundation soils after driving can result in increased soil shear strength and additional side friction commonly referred to as Setup.

Typically, driven piles are not designed to account for setup, therefore, $R_{Ssu} = 0$ and the *EOID* resistance equals *UBV* resistance. However, if soft cohesive soils are encountered at the site, and pile bearing resistance analysis for the “driving resistance” indicates long estimated pile lengths at *EOID*, it may prove beneficial to consider setup in the design. Refer to publication [FHWA-NHI-16-009](#) for guidance on the potential for pile setup or relaxation for various soil types.

As noted in BDM Section C305.3.2, determine the Nominal pile side resistance for soil below the scour zone, R_S , using static analysis methods according to *LRFD 10.7.3.8.6a*. Determine the components of this side resistance as follows:

$$R_S = R_{Sndr} + R_{Ssu} \dots \dots \dots (C305.3.2.4-1)$$

In which:

$$R_{Ssu} = (1 - f_{su}) R_S \quad (C305.3.2.4-2)$$

$$R_{Sndr} = (f_{su}) R_S \quad (C305.3.2.4-3)$$

Where:

R_{Sndr} = The side friction component of the nominal pile driving resistance at end of initial driving (kip)

R_{Ssu} = The additional resistance to account for setup of the side friction after end of initial driving (kip)

f_{su} = Setup factor according to BDM Table 305-2

Determine *EOID* and *UBV* values as a function of tip and side friction resistances as follows:

$$EOID = R_P + R_{Sndr} + R_{Ssc} \dots \dots \dots (C305.3.2.4-4)$$

$$UBV = EOID + R_{Ssu} \dots \dots \dots (C305.3.2.4-5)$$

Table 305-2

Pile Driving Recommended Setup Factor (fsu) for Side Friction
 based on [FHWA-NHI-16-009](#) Table 7-16 (after Rausche et. al., 1996)

ODOT Class	Soil Type Description	Recommended Setup Factor (fsu)	DRIVEN % Driving Strength Loss	APILE Reduction Factor
A-1-a	Gravel and/or Stone Fragments	1.0	0%	1.00
A-1-b	Gravel and/or Stone Fragments w/ Sand	1.0	0%	1.00
A-2-4	Gravel and/or Stone Fragments w/ Sand and Silt	1.2	17%	0.83
A-2-5		1.2	17%	0.83
A-2-6	Gravel and/or Stone Fragments w/ Sand, Silt and Clay	1.2	17%	0.83
A-2-7		1.2	17%	0.83
A-3	Fine Sand	1.2	17%	0.83
A-3a	Coarse and Fine Sand	1.2	17%	0.83
A-4a	Sandy Silt, Non-Plastic	1.2	17%	0.83
A-4a	Sandy Silt, Plastic	1.5	33%	0.67
A-4b	Silt	1.5	33%	0.67
A-5	Elastic Silt and Clay	1.5	33%	0.67
A-6a	Silt and Clay	1.5	33%	0.67
A-6b	Silty Clay	1.75	43%	0.57
A-7-5	Elastic Clay	1.75	43%	0.57
A-7-6	Clay	2.0	50%	0.50
A-8a	Organic Silt	1.75	43%	0.57
A-8b	Organic Clay	1.75	43%	0.57

305.3.3 POINT BEARING PILES ON BEDROCK

Do not specify precast reinforced concrete piles or precast prestressed concrete piles for point bearing pile applications.

C305.3.3

When piles are to be driven to refusal on bedrock, consider steel H-piles as the first option. An H-pile generally has greater steel cross-sectional area than a comparable sized pipe pile, unless the pipe pile has a very thick wall. Therefore, the H-pile usually has greater survivability driving directly onto or into the top of bedrock when a refusal blow count of 20 blows per inch must be achieved. However, cast-in-place reinforced concrete pipe piles (including large-diameter open-ended pipe piles) may be considered for point bearing piles on bedrock, provided that all design and construction considerations are adequately addressed.

Refer to BDM Section 305.3.5.6 for pile point requirements.

Provide the total factored load (Q_p) in the structural General Notes in accordance with BDM Section 305.3.2.

Determine the factored axial structural compressive resistance of the pile (P_r) according to *LRFD* 6.15.3. If including drag load, use the resistance factor for steel piles in compression under good driving conditions (*LRFD* 6.5.4.2). Otherwise, use the resistance factor for steel piles in compression and subject to damage due to severe driving conditions (*LRFD* 6.5.4.2 and BDM Table 305-1). Assume piles are unbraced along the predicted scour depth.

A sample note is provided in BDM Section 600.

If the total factored load for piles that are to be driven to refusal on bedrock is appreciably less than the maximum factored structural resistance, consideration should be given to increasing the pile spacing or reducing the pile size.

The commonly used H-pile sizes and the factored structural resistance (P_r) for each are listed below:

H-Pile Size	P_r
HP10X42	310 kips
HP12X53	380 kips
HP14X73	530 kips

The values listed above for the factored structural resistance are calculated in accordance with *LRFD* 6.9.4.1 assuming: 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 ($\phi_c = 0.50$); and a pile fully braced along its length ($l = 0$ -in). Please note that as l approaches zero, P_e approaches infinity which means P_n equals P_o in *LRFD* Eq. 6.9.4.1.1-1 and $P_o = F_y A_g$.

So:

$$P_r = \phi_c F_y A_g \dots\dots\dots (C305.3.3-1)$$

The commonly used pipe pile sizes and the factored structural resistance (P_r) for each are listed below:

In which:

$$F_e = F_y + 0.85 f'_c (A_c / A_s) \dots\dots\dots (C305.3.3-3)$$

Where:

- F_y = yield strength of structural steel (ksi)
- f'_c = 28-day compressive strength of the concrete (ksi)
- A_c = cross sectional area of concrete (in²)
- A_s = cross sectional area of steel pile (in²)

For piles under combined axial compression and flexure, determine factored structural resistance in accordance with *LRFD* 6.9.2.2.

Perform wave equation drivability analysis according to BDM Section 305.3.1.2 to determine whether the pile can be driven to refusal on bedrock without overstressing the pile utilizing commonly available pile driving hammers.

305.3.4 FRICTION PILES

Design friction piles in accordance with GDM Section 1304.

Provide additional concrete reinforcement inside the pile for capped pile piers as described in BDM Section 306.3.3.2.

Provide the *UBV* for each substructure unit in the structural General Notes in accordance with BDM Section 305.3.2.

Perform wave equation drivability analysis according to BDM Section 305.3.1.2 to determine whether the pile can be driven to the *UBV* without overstressing the pile utilizing commonly available pile driving hammers.

C305.3.4

Piles not driven to refusal on bedrock develop their geotechnical resistance by a combination of soil friction or adhesion along the sides of the pile and end bearing on the pile tip. These piles are typically referred to as friction piles.

When friction piles are to be driven, consider closed-end cast-in-place reinforced concrete pipe piles as the first option. Generally, H-piles should not be specified as friction piles. Experience demonstrates that H-piles tend to drive longer than pipe piles, generally from 10 to 20% longer, but sometimes more. In a full granular profile, H-piles may drive twice as long. However, if driving in very stiff soils where a minimum length must be achieved (e.g. for scour or uplift resistance), since H-piles do tend to drive longer, this may be a desirable trait.

This reinforcement provides structural capacity for the concrete pile if the steel shell completely deteriorates. For other pipe pile applications where significant deterioration of the steel shell is not anticipated, the steel pipe provides sufficient reinforcement, and no additional internal concrete reinforcement is required.

Every pile in a single substructure unit shall be driven to the same *UBV* as the pile with the maximum factored load in the substructure unit. A sample note is provided in BDM Section 600. The *UBV* may need to be adjusted during detail design as the design loads for the Service, Strength and Extreme Event Limit States are refined.

The commonly used pipe pile sizes and the typical maximum *UBV* for each are listed below:

Closed-end CIP Reinforced Concrete Pipe Pile Diameter	Maximum <i>UBV</i>
12-in	330-kip
14-in	390-kip
16-in	450-kip

These maximum *UBV* values for closed-end CIP reinforced concrete pipe piles may be significantly less than the structural capacity of a pile in axial compression (P_r). The maximum *UBV* values listed in the above table are based on pile drivability analyses using commonly available pile hammers, assuming a 3/8-in pile wall thickness and ASTM A252 Grade 2 steel.

When H-piles are used as friction piles, the maximum *UBV* for commonly used H-pile sizes are the same as the P_r values listed in 305.3.3.

Because higher strength steels are now often used (particularly in Design Build projects) and since there is a large degree of variability inherent in the estimation of these typical maximum values, these values should be used only as a guide for preliminary design

If driven piles in cohesive soils are specified at a center-to-center spacing of 6 pile diameters or more, or if driven piles in cohesionless soils are specified at a center-to-center spacing of 2.5 pile diameters or more, ignore group effects for axial loading.

For guidance on group effects for pile groups in axial compression, refer to *LRFD 10.7.3.9*.

The group efficiency factors for pile groups as given in *LRFD 10.7.3.9* for cohesive soils (clay) shall apply to pile groups in soft or very soft soils regardless if the pile cap is in firm contact with the ground.

305.3.5 DESIGN CONSIDERATIONS

305.3.5.1 MINIMUM PILE SPACING, CLEARANCE AND EMBEDMENT INTO CAP

Maximize the pile spacing based on the selected pile type and size.

C305.3.5.1

Perform an economic analysis when selecting the pile type and size and the pile driving hammer (including cost of mobilization). Keep in mind that friction H-piles tend to drive longer than pipe piles, particularly in granular soils.

The following maximum center-to-center pile spacings by structure type may be used as a guide:

- A. In capped pile piers, 7.5-ft.
- B. In capped pile abutments, 8-ft.
- C. In stub abutments, front row, 8-ft.
- D. In wall type abutments and retaining walls, front row, 7-ft.

Cap and column piers shall have at least 4 piles per individual footing.

For minimum center-to-center spacing of the piles, refer to *LRFD 10.7.1.2*.

Reinforce the pile cap to resist bending and shear based on the proposed center-to-center spacing of the piles.

Piles supporting capped pile piers shall be embedded 1.5-ft into the concrete cap. For other substructure units on a single row of piles, the piles shall be embedded 2-ft into the concrete. A 1-ft embedment depth into the concrete footing is required for all other cases. Perform a punching shear analysis to determine the necessary concrete thickness over the top of pile. In every case, there shall be at least 1.5-ft cover over top of pile.

The distance from the edge of a footing to the center of a pile shall be not less than 1.5-ft. The distance from the edge of a concrete pier cap to the side of a pile shall be not less than 9-in.

305.3.5.2 ESTIMATED PILE LENGTH

C305.3.5.2

When estimating pile length for point bearing piles on bedrock, except as noted, assume the pile tip elevation as the elevation on the nearest soil boring where either bedrock coring begins or where SPT refusal blow count occurs with a recovered sample visually classified as bedrock. If using piles placed in prebored holes in bedrock, use the bottom of the prebored hole elevation as the pile tip elevation. If sufficient boring data is available, analyze the dip and strike of the bedrock in all three dimensions. If exploration reveals that the rock dips more than 8H:1V across the width of the unit, specify more than one pile length for the unit in divisions of 5-foot lengths. If there are no borings within 50 feet of the substructure unit, locate the top of rock at the substructure unit by interpolation between borings located to either side of the unit; provide a discussion to this effect in the Analyses and Recommendations section of the Foundation Report.

When estimating pile length for friction piles, use static analysis methods to determine the depth of pile penetration necessary to develop the required Ultimate Bearing Value as described in BDM Section 305.3.2.

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

Calculate the following pile lengths:

- A. $\text{Estimated Length} = \text{Pile Cutoff Elevation} - \text{Pile Tip Elevation}$
Round Estimated Length up to the nearest 5-ft. Provide the Estimated Length on the Site Plan.
- B. $\text{Order Length} = \text{Estimated Length} + 5\text{-ft}$
Provide the Order Length for each pile in the Structure General Notes.
- C. $\text{Furnished Length} = \text{Order length} \times \text{No. of Piles}$
Include Furnished Length in the Estimated Quantities.
- D. $\text{Driven Length} = \text{Estimated Length} \times \text{No. of Piles}$
Include Driven Length in the Estimated Quantities.

Note that pile cutoff elevation includes the embedment into the pile cap per BDM Section 305.3.5.1 and free-standing length for capped pile piers. If rounding up to the nearest 5-ft for Estimated Length adds less than a foot, increase to the next 5-foot interval.

305.3.5.3 CORROSION AND PROTECTION C305.3.5.3

If the subsurface exploration identifies soil or site conditions considered indicative of potential pile deterioration or corrosion from environmental conditions according to *LRFD 10.7.5*, verify conditions with laboratory testing of soil samples. Consider soils with an organic content of 4 percent or more as “high organic content”.

For soils that are not indicative of a potential pile corrosion problem, ignore corrosion for steel not exposed to atmospheric conditions over the design life of the structure. Provide pile encasement for portions of piles exposed to atmospheric conditions. The pile encasement shall extend a minimum of 3-ft below the ground line/stream bottom.

For soils that are indicative of a potential pile corrosion problem, determine the appropriate corrosion loss rate for carbon steel per Eurocode 3, Part 5, Section 4.4 for the specific environmental conditions at the site. Apply the appropriate corrosion loss rate to all surfaces of the piles in the respective exposure area.

Design the steel pile section to retain the required factored structural resistance after discounting corrosion loss and provide a plan note that addresses the amount of additional pile section specified to account for the corrosion loss. Alternately, provide corrosion protection for the piles.

Corrosion protection for the pile steel shall consist of a zinc coating or concrete encasement. For zinc coatings, estimate the corrosion loss rate as 1/2 the respective loss rate for carbon steel. The minimum thickness for zinc coating is 4 mils. Treat steel piles with precast concrete encasement as precast reinforced concrete piles for the purposes of geotechnical resistance, pile handling and installation, and limits on driving stresses. For concrete piles or concrete encasement, if the environmental conditions indicate a soil chloride content ≥ 500 ppm, a sulfate content ≥ 500 ppm, or a pH < 5 , then retain the services of a corrosion specialist in order to develop an appropriate corrosion-inhibiting mix design or concrete cover thickness over the steel, per *LRFD 5.10.1*, *5.14.2*, and *C10.7.5*.

305.3.5.4 VERTICAL AND HORIZONTAL MOVEMENTS

Perform settlement and horizontal pile foundation movement analyses according to *LRFD 10.7.2*. Consider the effects of downdrag according to BDM Section 305.3.2.2. Refer to BDM Section 305.1.3 for limits on vertical movement with respect to a supported bridge structure.

A form of pile encasement is detailed on Standard Bridge Drawing [CPP-1-08](#). The top of the encasement shall be located no more than 1-ft from the bottom of the pile cap and the concrete fill shall be sloped to drain.

305.3.5.5 BUCKLING AND LATERAL STABILITY

For free-standing piles, analyze the piles for buckling and lateral stability as unsupported columns above the point of fixity with scour depths included.

Determine the depth to the point of fixity in accordance with *LRFD 10.7.3.13.4*.

305.3.5.6 STEEL PILE POINTS OR SHOES

To protect the tips of H-piles, CIP pipe piles and open-ended pipe piles, provide steel points, conical points or cutting shoes respectively for each of the following criteria:

- A. Piling is driven to refusal on bedrock with an unconfined compressive strength ≥ 7.5 -ksi, **AND**
 - 1. Overburden of any thickness consisting primarily of granular soils, **OR**
 - 2. Overburden less than 50-ft consisting primarily of cohesive soil.
- B. Piling is driven through overburden containing boulders
- C. Piling is driven through very dense granular soils of 10 feet or greater in thickness
- D. Piling is driven to refusal on bedrock meeting the following criteria:
 - 1. Unconfined compressive strength ≥ 1.5 -ksi, **AND**

C305.3.5.5

Buckling and lateral stability analysis applies particularly to capped pile piers, large-diameter open-ended pipe piles supporting single pier columns, and conventional piles subject to scour.

The equations in *LRFD C10.7.3.13.4* may be used to determine depth to fixity for preliminary design.

C305.3.5.6

Sample plan notes for each of the pile requirements are provided in BDM Section 600. Generally, H-piles with pile points should not be specified as friction piles. With points, H-piles typically run deeper than estimated, because the pile points are slightly larger than the pile and pile side resistance is reduced.

Steel points need not be specified when the piles are driven to refusal on weaker bedrock because driving stresses at the tip should not be damaging. Refer to [SGE Table 600.10](#) for more information regarding the strength of bedrock.

When the overburden is greater than 50-ft consisting primarily of cohesive soils, there should be enough side friction to prevent damaging stress at the pile tips.

Also give consideration to include steel pile points or shoes if the overburden contains significant cobbles (encountered widespread, difficult drilling noted, or cobbles larger than 6 inches noted). Generally, H-piles should not be specified as friction piles for driving in boulders and cobbles; the pile tips are often deformed and damaged by the cobbles and boulders. H-piles also tend to get cobbles and boulders jammed in the web, warping the flanges, and causing the pile to “corkscrew” or bend, or in some extreme cases, to form a U-shape turning back towards the surface. A thick-walled pipe pile with a conical steel driving point can shed cobbles and boulders and will generally drive through them with little effect. Typically, specify CIP pipe piles with conical points as friction piles for driving in boulders and cobbles.

This assumes that the pile would be going through a substantial thickness of dense granular soils, such that pile driving stresses at the tip would prove excessive. Experience demonstrates that H-piles may drive **up to** twice as long as pipe piles in a fully granular soil profile.

This provision helps to facilitate penetration of piles into the bedrock without skipping off the surface.

2. Angle between inclined bedrock and axis of the pile is less than 60°, AND
3. Overburden immediately above bedrock is:
 - a. Stiff or weaker cohesive soil, or
 - b. Granular soil

305.3.5.7 MINIMUM PILE PENETRATION REQUIREMENTS

C305.3.5.7

When driving piles through new embankment, the piles shall penetrate a minimum distance of 10-ft into the underlying in-situ soils. If piles are to be driven through 15-ft or more of new embankment, specify prebored holes, in accordance with [C&MS 507.11](#). Clearly indicate the locations and lengths of all prebored holes in the plans. For design purposes, ignore the effect of skin friction along the length of the prebored holes for bearing resistance; for drivability analyses, include skin friction along the prebored length as if it is sand with a 28° friction angle. The specified length of the prebored hole shall be the height of the new embankment at each pile location.

For sleeved piles placed through MSE fill, ignore the effect of skin friction along the length of the sleeve for bearing resistance and for drivability analyses.

There are two potential types of sleeve in general use (a sand-filled corrugated polyethylene smooth lined pipe or a prefabricated friction reduction pile sleeve). If pile driving will occur after sleeving, they must be considered in the design. For the sand-filled smooth lined pipe sleeve, skin friction along the sleeved length is as for sand with a 28° friction angle. For the prefabricated friction reduction pile sleeve, skin friction along the sleeved length is as for sand with a 10° friction angle.

When placing driven pile foundations through an MSE wall backfill, the piles shall penetrate a minimum distance of 15-ft into the underlying in-situ soils below the leveling pad elevation. If bedrock exists within the minimum penetration length, place the prebored in drilled holes that extend a maximum of 5-ft into bedrock.

Where bedrock is encountered in the penetration length:

- A. For in-situ soil depth $\leq$ 10-ft, extend 5-ft into rock.
- B. For in-situ soil depth $>$ 10-ft, total penetration depth including into rock = 15-ft

For applications other than MSE walls, the piles shall penetrate a minimum distance of 10-ft into in-situ material. If bedrock exists within the 10-ft length, place the piles in prebored holes that extend a minimum of 5-ft into bedrock. If bedrock unconfined compressive strength $<$ 1.5-ksi, extend the prebored holes a minimum of 10-ft into bedrock.

Unlike under MSE walls, this provision accommodates the possibility of a single row of piles with significant lateral loads and bending moments. The 5-ft extension into rock should create a point of fixity for pile stability where shallow rock is encountered.

Place the piles in the prebored holes without driving the piles. For integral abutment piles, place Class QC Misc. concrete in the bottom 5 feet of the prebored hole; otherwise place the concrete in the prebored holes to the top of bedrock. When piles are placed in drilled holes, determine the nominal structural resistance of the piles according to BDM Section 305.3.3 using the resistance factor for axial compression, $\phi_c = 0.95$ [*LRFD 6.5.4.2* for steel columns].

305.3.5.8 BATTERED PILES

Only use battered piles if a *p-y* analysis in accordance with BDM Section 305.1.2 predicts excessive deflection or inadequate fixity, such that the piles fail to meet the serviceability requirements of BDM Section 305.1.3.

Do not extend battered piles beyond the limits of the right-of-way. Ensure that the location and alignment of the proposed piles do not conflict with deep foundation elements from the adjacent or existing substructure units.

Do not specify battered piles when drag load is anticipated, unless piles can be driven after the construction of the embankment, the specified minimum waiting period for consolidation of in-situ soils has concluded, and the remaining settlements are anticipated to be less than 0.4-in.

For skewed bridges, batter abutment piles normal to the centerline of bearings.

Do not batter piles less than 15-ft in length and driven to refusal on bedrock.

305.3.5.9 PILE SETUP AND RELAXATION

If the subsurface geotechnical conditions at the site indicate the potential for pile setup on friction piles, and the estimated driving resistance indicates driving losses that would increase the length of the pile during driving by more than 10-ft with *E_{OID}* compared to the *UBV*, then account for pile setup in the design. Utilize one of the following three alternatives for the design:

- A. Drive all piles to the full Estimated Length, perform dynamic load testing on the first two piles driven and then restrike testing on the same two piles after a specified waiting period, in order to verify setup and the *UBV* capacity as shown in the plans, or
- B. Specify the performance of a site-specific construction evaluation utilizing variable length driving of the first four production piles and restrike testing in order to determine a specific pile tip elevation to be used during construction, or

Include the prebored holes with [C&MS](#) ITEM 507 PREBORED HOLES, AS PER PLAN for payment. The Department will include the backfill concrete with the prebored holes for payment. The Department will pay for the piles under Item 507 Steel Piles HP_x_, Furnished, As Per Plan with the placement into the prebored holes incidental to the pay item. Do not include an ITEM 507 PILES DRIVEN pay item. Sample plan notes are provided in BDM Section 600.

C305.3.5.8

Battered piles are not preferred, as they are more expensive and difficult to construct than vertical deep foundation elements.

Piles battered transverse to the centerline of a bridge should be avoided. A batter of 1:4 (H:V) is considered desirable, but in cases where the lateral resistance of the piles is inadequate to counteract the horizontal forces transmitted to the foundation, a batter of 1:3 (H:V) may be specified.

The additional bending forces imposed on the piles by the drag load can cause pile deformation and damage.

C305.3.5.9

Refer to publication [FHWA-NHI-16-009/FHWA-NHI-16-010](#), Geotechnical Engineering Circular 12 (GEC 12) “Design and Construction of Driven Pile Foundations” for guidance on the potential for pile setup or relaxation for various soil types.

Pile setup can potentially result in upwards of a six-fold increase in the *UBV* compared to *E_{OID}*. Conversely, if the loss of resistance during driving is not realized in design, this can result in substantial pile quantity overruns during construction. Realization of pile setup in design can therefore result in substantial pile quantity savings. A site-specific pre-construction evaluation utilizing test piles (Alternative C) should be considered if over 10,000-ft of piles are specified.

- C. Specify the performance of a site-specific pre-construction evaluation utilizing test piles and restrike testing in order to determine values of *E_{OID}* and *UBV* along with pile driving criteria to be used during construction.

If pile setup is realized in the design, specify restrike dynamic testing of the piles to field verify the *UBV* in axial compression.

If the subsurface geotechnical conditions at the site indicate the potential for pile relaxation for friction piles, make no change to the design regarding the nominal resistance provided at the pile tip. Setup may still be considered for the nominal resistance provided along the side of the pile.

If the potential for pile setup or relaxation is indicated for point bearing piles on bedrock, make no change to the design.

Plan notes for all three of the above alternatives are available in BDM Section 600.

Pile relaxation can significantly reduce *UBV* compared to *E_{OID}* in heavily over-consolidated clays and in dense saturated silts and fine sands. Research has shown relaxation related losses as much as 40%. However, the amount of displacement to re-mobilize the full *UBV* is considered small, and it is routinely neglected in design.

For point bearing piles on bedrock, while pile setup will doubtless increase the overall pile bearing resistance some unknown amount, it is ignored in design, since soil friction resistance is not included in the pile bearing resistance due to the large difference in displacement necessary to mobilize side resistance compared to tip resistance. Conversely, while relaxation in the bedrock at the pile tip will doubtless decrease the overall pile bearing resistance, this is also ignored, as the additional displacement necessary to re-mobilize the tip resistance is considered negligible.

305.3.6 VIBRATION MONITORING

If pile driving with a hammer having a rated energy of 44,000-ft-lb or smaller is to be performed within 200-ft of an existing building, perform a vibration impact assessment in accordance with the guidelines provided in the U.S. Federal Transit Administration (FTA) guidance manual, “Transit Noise and Vibration Impact Assessment,” Final Report FTA-VA-90-1003-06. In this event, use $PPV_{ref} = 0.644$ in/sec.

For the vibration impact damage assessment, use the equation provided in Section 7.1 of NCHRP 25-25/Task 72 “Current Practices to Address Construction Vibration and Potential Effects to Historic Buildings Adjacent to Transportation Projects”, with the following values of the attenuation exponent, *n*:

C305.3.6

As noted in BDM Section C305.3.1.2, single-acting diesel pile driving hammers having a rated energy of up to 44,000-ft-lb are commonly available in Ohio. The 200-ft proximity limit is reasonably conservative for conventional pile driving using commonly available pile hammers. For driving of large-diameter piles or the use of higher rated hammers, the proximity limit would need to be extended and an increase in PPV_{ref} would also be necessary.

The recommended values of “*n*” are within the range of published data and are considered conservative. For additional guidance and commentary on the use of this equation and performance of the vibration impact assessment, see CalTrans Transportation and Construction Induced Vibration Guidance Manual.

Cohesive Soils

Relative Consistency	Attenuation Exponent, n
Very Soft	1.75
Soft	1.5
Medium Stiff	1.4
Stiff	1.3
Very Stiff	1.2
Hard	1.1

Granular Soils

Relative Compactness	Attenuation Exponent, n
Very Loose	1.75
Loose	1.5
Medium Dense	1.3
Dense	1.2
Very Dense	1.1

Specify vibration monitoring in the plans when the vibration impact assessment indicates:

- A. Potential for damage to nearby structures, or
- B. Unacceptable levels of vibration annoyance with respect to human response, or
- C. Interference with building function

Include a preconstruction condition survey in the plans when the buildings within the proximity limits of the assessed vibration impact are residential, historic, museums, fragile, or otherwise vibration sensitive structures.

Plan notes are available in BDM Section 600.

305.3.7 EMBANKMENT CONSTRUCTION CONSTRAINTS

Refer to BDM Section 305.3.2.2 regarding downdrag and drag load effects from new embankment settlement, and potential methods of remediation. Refer to BDM Section 305.3.5.7 regarding pile resistance and embedment through new embankments. Refer to BDM Section 305.3.5.8 regarding new embankment settlement effects on battered piles.

C305.3.7

When driving piles through new embankment, special considerations with respect to settlement, downdrag, additional loading effects, and resistance of the pile may be necessary.

305.4 DRILLED SHAFTS

Design drilled shafts in accordance with *LRFD 10.8* and construct in accordance with [C&MS](#) 524.

Refer to BDM Section 201.2.1.3 for information to include in the Structure Foundation Exploration Report.

Do not specify drilled shafts with an enlarged base (bell, or underream).

305.4.1 LOAD EFFECTS

Provide the total factored load as well as the side and tip resistance for each drilled shaft in the structure General Notes. For laterally loaded drilled shafts, provide the total factored externally applied lateral and moment bending loads, and the maximum factored values of the internal shear and moment.

Determine the total factored axial drilled shaft load (Q) using BDM Eq. C305.4.1-1. For consideration of geotechnical resistance in rock-socketed drilled shafts and for structural resistance in drilled shafts subject to downdrag, include the drag load (DD); for geotechnical resistance in friction drilled shafts, DD = 0.

C305.4

Drilled shafts should be considered when their use would:

- A. Prevent the need for cofferdams
- B. Become economically viable due to high design loads
- C. Improve the structural stability in a scour event.
- D. Provide resistance against lateral and uplift loads
- E. Accommodate sites where the depth to bedrock is too short for adequate pile embedment but too deep for spread footings
- F. Accommodate the site concerns associated with the pile driving process (vibrations, interference due to battered piles, etc.).

Belled shafts are difficult to construct under water or with slurry and the bell will often collapse in non-cohesive soils. Cleaning of the base of the drilled shaft and inspection within the bell is also difficult to conduct

C305.4.1

Sample plan notes are provided in BDM Section 600.

Determine the total factored axial drilled shaft load as follows:

$$Q_p = \sum \eta_i \gamma_i Q_i + \eta_i \gamma_p DD \leq R_g \leq P_r \dots \dots \dots (C305.4.1-1)$$

In which:

Q_p = Total factored load (kips) in axial compression per shaft for highest loaded drilled shaft at each substructure unit. This will include the loads imposed by the supported infrastructure and the drag load. This is resisted by the factored fully mobilized geotechnical resistance of the shaft; factored load versus the axial structural resistance of the shaft will be considered separately (See BDM Section 305.4.1.2).

$\sum \eta_i \gamma_i Q_i$ = Sum of factored loads for highest loaded drilled shaft at each substructure unit (kips)

$\eta_i \gamma_p DD$ = Factored drag load per drilled shaft (kips) (See BDM Section 305.4.1.2)

η_i = Load modifier relating to ductility, redundancy and operational classification (see *LRFD 1.3.2.1*)

γ_i = Load factor (see *LRFD 3.4*)

γ_p = Load factor for drag load = 1.05

R_R = Factored geotechnical resistance at the Strength Limit State (kips)

P_r = Factored structural axial resistance per drilled shaft (kips), calculated according to *LRFD 5.6.4.4*.

Determine the factored geotechnical resistance (R_R) for axial compression of drilled shafts using BDM Eq. C305.4.1-2

The factored geotechnical resistance in axial compression of drilled shafts required to support the factored load in axial compression is provided by the tip and side resistance in only the rock socket for rock-socketed drilled shafts, and by the tip and side resistance in the overburden soil for friction drilled shafts (see BDM Section 305.4.1.2).

$$R_R = \phi R_n = \phi_{qp} R_P + \phi_{qs} R_S \dots \dots \dots (C305.4.1-2)$$

In which:

R_n = Nominal resistance in axial compression (kips)

R_P = Nominal drilled shaft tip resistance (kips)

ϕ_{qp} = Resistance factor for tip resistance

R_S = Nominal drilled shaft side resistance (kips)

ϕ_{qs} = Resistance factor for shaft side resistance

Determine the design depth for drilled shafts using static analysis methods to calculate the depth of drilled shaft necessary to develop the factored resistance in axial compression.

When a substructure is supported on only a single drilled shaft, apply a reduction factor of 0.8 to R_R . When a substructure is supported on more than four drilled shafts, apply a redundancy factor of 1.2 to R_R .

305.4.1.1 SCOUR

Neglect the shaft resistance provided by soil or non-scour resistant rock in the scour zone. Assess scour resistant rock according to BDM Section 305.2.1.2.b. Use the depth of scour resulting from the design flood, with Strength and Service Limit State checks. Use the depth of scour resulting from the check flood with Extreme Event II Limit State checks.

C305.4.1.1

The design flood and check flood are defined in *LRFD 2.6.4.4.2*.

Extend friction drilled shafts to penetrate a minimum of 15-ft below the controlling scour elevation. If socketed into non-scour resistant bedrock, when the rock has $Q_u < 2.5$ -ksi, extend drilled shafts to penetrate a minimum of 10-ft below the controlling scour elevation in the bedrock; when the rock has $Q_u \geq 2.5$ -ksi, extend drilled shafts to penetrate a minimum of 5-ft below the controlling scour elevation in the bedrock. If socketed into scour resistant bedrock, design the length of the bedrock socket based solely on the requirements of axial resistance in accordance with *LRFD 10.8.3.5.4* and of lateral stability.

Because the drilled shaft will lose support along the scour depth, investigate the structural capacity of the shaft considering the depth of the scour as an unbraced length. Also perform a p-y analysis on the drilled shaft according to BDM Section 305.1.2 to demonstrate lateral stability against:

- A. Overturning failure in the Strength Limit State with the maximum estimated scour depth at the design flood, and
- B. Overturning failure in the Extreme Event II Limit State with the maximum estimated scour depth at the check flood, and
- C. Excessive deflection in the Service Limit State.

305.4.1.2 DOWNDRAG AND DRAG LOAD

Assume downdrag to act if ground settlement ≥ 0.4 -in will occur after drilled shaft installation at the substructure unit in question.

For all rock-socketed drilled shafts or drilled shafts founded on top of bedrock, consider the neutral plane to be at the top of bedrock.

C305.4.1.2

When soil moves downward relative to the drilled shaft, it creates a drag load on, and therefore within, the shaft. The downward soil movement creates the potential for downward drilled shaft movement. This downward drilled shaft movement is referred to as downdrag. The subsurface conditions, shaft installation methods, shaft loading sequences, as well as the shaft and ground surface configuration determine the magnitude of the drag load and the downdrag movement.

If greater resistance to drag load is necessary, consider using larger drilled shafts or increasing the number of drilled shafts and reducing the applied load per drilled shaft. To reduce or eliminate downdrag, consider preloading the soil so settlement occurs before drilled shaft installation. Also consider installing wick drains and an additional earth surcharge load to decrease the amount of time required for settlement to occur.

A Special Provision for Installation of Wick Drains is available from the Office of Geotechnical Engineering.

Do not use friction drilled shaft foundations when the downdrag is calculated to induce settlements on the structure that will exceed the acceptable settlement criteria as described in BDM Section 305.1.3.

Downdrag is a very complex phenomenon for friction drilled shafts, involving partial mobilizations of side and tip resistances in both the positive and negative directions, and can only be fully analyzed with advanced computerized solutions; even in this case, caution must be used, as research has not definitively established the soil-structure interaction relationships to a point where accurate t-z and q-w curves can be predicted. For these reasons, ODOT prefers not to use friction drilled shafts subject to downdrag.

For friction drilled shafts subject to downdrag, use either the simplified hand-calculation method described in FHWA-NHI-10-016, Geotechnical Engineering Circular 10 ([GEC 10](#)) “Drilled Shafts: Construction Procedures and LRFD Design Methods,” Section 13.6.3 to calculate the location of the neutral plane, or utilize a more refined computerized analysis utilizing soil and structure springs with t-z and q-w curves established from a computerized solution.

The maximum total factored axial load per shaft will be at the location of the neutral plane.

Analyze downdrag and drag load according to the method described in [GEC 10](#), Section 13.6, and as described below.

Evaluate the total factored axial load per shaft, including drag load at the structural Strength Limit State using BDM Eq. C305.4.1-1.

Drag load is the total sum of skin friction in the downwards direction (negative skin friction) for the full length of the shaft in contact with the soil above the neutral plane. Calculate skin friction and drag load using static analysis methods, according to *LRFD 10.8.3.5* and [GEC 10](#), Section 13.6. In this calculation, assume an initial base settlement w_b and associated mobilization of tip (base) resistance either based on integration of the load displacement curves for both side resistance and end bearing according to *LRFD 10.8.2.2* or by utilizing advanced numerical modelling to integrate a series of t-z and q-w curves generated through a computerized solution according to [GEC 10](#), Appendix D. Use a structural resistance factor for drilled shafts as reinforced concrete columns in compression in accordance with *LRFD 5.5.4.2* (typically, $\phi_c = 0.75$ for a compression-controlled section with spirals).

Evaluate the downdrag movement, as it contributes to drilled shaft head settlement, at the geotechnical Service Limit State. Consider the shaft downdrag movement (S_{dd} = settlement due to downdrag) equal to the settlement of the foundation soil at the location of the neutral plane, plus the elastic compression of the shaft above the location of the neutral plane due to the effects of the permanent load at the pile head plus the drag load. Calculate settlement of the foundation soils according to *LRFD 10.6.2.4*. Calculate elastic compression of the drilled shaft according to [GEC 10](#), Section 14.4.2.1. Exclude transient loads from the calculation of settlement and elastic compression.

For rock-socketed drilled shafts or drilled shafts founded on top of bedrock, include both transient loads and drag load in the computation of total factored axial load per shaft, and provide this value in the structure general notes.

For friction drilled shafts, do not include both transient loads and drag load in the computation of total factored axial load per shaft. If transient loads exceed the value of drag load, include only transient loads. If transient loads are less than the value of drag load, include only the drag load.

305.4.1.3 UPLIFT

Calculate drilled shaft uplift resistance according to *LRFD 10.8* using static analysis methods to determine the nominal uplift resistance due to side resistance.

As uplift is a Strength Limit State consideration, include the self-weight of the drilled shaft, factored by $\gamma_{DC,min} = 0.90$, and the effect of buoyancy factored by $\gamma_{WA} = 1.00$.

Where the drilled shaft length is controlled by the required uplift resistance, specify a drilled shaft tip elevation.

If site constraints or other factors make permanent tensile loads on drilled shafts unavoidable, use a redundancy load modifier of $\eta_R = 1.05$ for the permanent tensile loads on the foundation elements.

305.4.2 ROCK-SOCKETED DRILLED SHAFTS

For competent bedrock with a minimum rock socket length equal to 1.5 times the rock socket diameter, use *LRFD Eq. 10.8.3.5.4c-1* to evaluate the nominal unit tip resistance of the bedrock.

C305.4.1.3

The minimum load factor is used for self-weight of the drilled shaft because it provides resistance.

A plan note is available in BDM Section 600.

The Department will generally not permit the use of Drilled shafts under permanent tensile loads.

C305.4.2

Competent bedrock is defined as rock that does not contain adverse jointing, open solution cavities, or joints that are filled with weathered material that would affect the bearing resistance of the rock, within a distance equal to two socket diameters below the tip of the drilled shaft rock socket.

For all other rock, provide a justification in the Foundation Report and evaluate the nominal unit tip resistance of the rock mass through Geological Strength Index (GSI) and *LRFD Eq. 10.8.3.5.4c-2*.

Neglect side resistance for rock sockets with a length less than 1.5 times the rock socket diameter. Otherwise, neglect the contribution to skin friction provided by the top 2-ft of the rock socket.

When drilled shafts are socketed into bedrock, do not specify the tip elevation. Instead, provide the approximate top of the bedrock elevation and the length of the bedrock socket in the profile view on the Final Structure Site Plan.

If a rock socket is not necessary either for bearing or lateral resistance, and the bedrock is competent slightly strong or stronger rock, drilled shafts may be founded on top of the bedrock, or utilize a rock socket length of less than 1.5 times the rock socket diameter. In this case, use *LRFD Eq. 10.8.3.5.4c-2* to evaluate the nominal unit tip resistance of the bedrock.

For rock-socketed drilled shafts or drilled shafts founded on top of bedrock, neglect the contribution to side resistance from the soil overburden.

For drilled shafts socketed into interbedded rock, use a composite of the unconfined compressive strength of the interbedded rock types to calculate the side and tip resistance.

If the bedrock is weak or disintegrated, and the designer anticipates difficulty verifying the top of bedrock during construction, a tip elevation may be specified.

This type of drilled shaft will usually not have a reduction in diameter for a rock socket, in which case, the drilled shaft diameter in the soil overburden should be considered as the rock socket diameter for determining its length.

The amount of deformation necessary to mobilize the tip resistance in rock is much less than that to mobilize the side resistance in soil

To calculate the composite strength, multiply the unconfined compressive strength of each interbedded rock type by the percentage of that rock type in the interbed, and then sum the result.

305.4.3 FRICTION DRILLED SHAFTS

Perform load testing at each substructure unit supported by friction drilled shafts. Specify:

- A. a number of high-strain dynamic load tests in accordance with BDM Table 305-1, or
- B. a number of static load tests based on site variability recommendations in *LRFD C10.5.5.2.3* and *C10.5.5.2.4*, or a minimum of one per structure, or
- C. a combination of high-strain dynamic and static load tests in accordance with BDM Table 305-1.

Use a resistance factor ϕ_{dyn} for geotechnical axial resistance in accordance with the specified number of high-strain dynamic and static load tests, based on the appropriate row in BDM Table 305-1.

Show the tip elevation of friction drilled shafts on the Final Structure Site Plan.

C305.4.3

For friction drilled shafts in cohesive soils, except as noted below, neglect the contribution of side resistance for portions of the length of the drilled shaft as specified in *LRFD 10.8.3.5.1*. Do not neglect the upper 5-ft if the top of the shaft is buried greater than 5-ft, or there is a buried drilled shaft cap above the drilled shaft.

If friction drilled shafts in cohesive soils are specified at a center-to-center spacing of 6 drilled shaft diameters or more, or if friction drilled shafts in cohesionless soils are specified at a center-to-center spacing of 4 drilled shaft diameters or more, ignore group effects for axial loading.

Ignore the effects of self-weight of the drilled shaft foundation and of buoyancy for consideration of bearing resistance.

For guidance on group effects for axially loaded friction drilled shafts, refer to *LRFD 10.8.3.6*.

The equations for bearing resistance of deep foundations inherently already include self-weight of the foundation element and buoyancy, as they are based on load tests applied to the top of piles or shafts, and are measuring the net resistance of the system including the effects of self-weight and buoyancy.

305.4.4 DESIGN CONSIDERATIONS

Provide a unique number designation for each drilled shaft in the Plans.

For pier columns supported directly on drilled shafts, the top of the drilled shaft shall be 1 foot above the OHHM for piers in water, and 1 foot below the ground surface for piers not in water.

For drilled shafts with 4.5-ksi Class QC 4 or Class QC 5 concrete, assume a concrete strength of 4.0-ksi in the design. For a concrete strength above 4.5-ksi, assume a strength of 0.90 f'_c in the design.

Drilled shafts shall be plumb. Do not batter drilled shafts.

Where casings are to be left in place, provide a plan note on either the Estimated Quantities or General Notes sheet.

Provide a construction joint between the top of a drilled shaft and the bottom of the supported substructure element.

C305.4.4

The designer may choose to number the drilled shafts on the individual substructure plan sheet or on a separate drilled shaft foundation layout sheet.

The reason for the strength reduction in design is to account for potential poor placement control during construction, and the inability to inspect the placement.

305.4.4.1 DRILLED SHAFT SPACING, CLEARANCE, AND EMBEDMENT INTO CAP

C305.4.4.1

Maximize the drilled shaft spacing based on the factored resistance in axial compression, considering redundancy and group effects.

For guidance on center-to-center spacing of drilled shafts, refer to *LRFD 10.8.1.2*.

The drilled shaft cap shall be adequately reinforced based on the proposed center-to-center spacing of the drilled shafts. Perform a punching shear analysis of the drilled shaft cap to determine the necessary cap thickness.

The distance from the edge of a drilled shaft cap to the side of a drilled shaft shall be not less than 6-in. Shafts shall be embedded sufficiently into the cap to develop the required structural resistance.

305.4.4.2 DRILLED SHAFT SIZE

Drilled shafts that directly support pier columns shall be 6-in larger in diameter than the diameter of the supported pier column. The minimum diameter for drilled shafts that directly support pier columns shall be 42-in. The minimum diameter for drilled shafts that support all other substructure types shall be 36-in. The rock socket diameter shall be 6-in less than the diameter of the drilled shaft in the soil overburden

The design diameter of the drilled shaft shall include an allowance for minimum concrete cover over the concrete reinforcement. For socketed drilled shafts, the minimum cover applies within the rock socket which will in turn correspondingly increase the concrete cover in the soil overburden. The minimum concrete cover is as follows:

- A. 3.0-in for shafts $\leq 3'-0"$ diameter
- B. 4.0-in for $> 3'-0"$ but $< 5'-0"$ diameter
- C. 6.0-in for shafts $\geq 5'-0"$ diameter

Show the Drilled shaft diameters on the Final Structure Site Plan.

305.4.4.3 CONCRETE REINFORCEMENT

The clear distance between parallel longitudinal bars and between transverse reinforcement shall be five times the maximum aggregate size.

C305.4.4.2

The shaft diameter is specified to be 6-in larger than the column diameter to allow for the misalignment of drilled shafts that directly support pier columns. Drilled shafts founded on top of the bedrock or utilizing a rock socket length of less than 1.5 times the rock socket diameter need not have a reduction in diameter for a rock socket

The increase in concrete cover with respect to increasing drilled shaft diameter is due to allowance for placement tolerance given potential misalignment or “wander” in the drilled shaft tooling.

C305.4.4.3

The maximum aggregate size for an ODOT QC5 mix design according to [C&MS](#) Table 499.03-1 is 3/8-in or 1-in. If mass concrete provisions govern, please note that QC4 mix designs have a nominal 1-in maximum aggregate size requirement according to [C&MS](#) 499.03. A plan note modifying the nominal maximum coarse aggregate size in the drilled shaft QC5 or QC4 mix design is available in BDM Section 600.

Where reinforcement spacing cannot be met between parallel longitudinal bars, consideration may be given to bundling of bars or to specifying an inner and outer reinforcing cage. Inner and outer reinforcing cages should only be considered when bundling of bars is impractical. Consideration may also be given to embedding a steel beam section within the drilled shaft. A steel beam is particularly useful for increasing lateral resistance.

Provide either spirals or ties as the transverse reinforcement for drilled shafts.

For temporarily cased or uncased drilled shafts, meet the requirements of *LRFD Eq. 5.11.4.1.4-1* and use a minimum #4 bar for the transverse reinforcement. Do not use *LRFD Eq. 5.6.4.6-1*.

LRFD Eq. 5.6.4.6-1 provides erroneous results based on the ratio of A_g/A_c . As the concrete cover increases, impractical areas of transverse reinforcement are required. *LRFD Eq. 5.11.4.1.4-1* provides the volumetric reinforcement requirements for confinement of primary steel at locations where plastic hinges are formed during seismic events. This provision of the BDM applies *LRFD Eq. 5.11.4.1.4-1* to the entire length of the drilled shaft.

When using the minimum concrete cover in BDM Section 305.4.4.2 in combination with *LRFD Eq. 5.11.4.1.4-1* and the clear distance provision described in BDM Section 305.4.4.3 the spiral reinforcing becomes standardized. BDM Table 305-3 below provides the spiral pitch and bar size for each drilled shaft diameter. It should be noted that when the drilled shaft diameter is greater than 72-in the spiral would need to be a #6 bar. Not all rebar fabricators carry spools of #6 bar. Therefore, when the drilled shaft is greater than 72-in ties should be considered rather than spiral reinforcement.

Table 305-3

Column/Shaft Diameter	Column/Shaft Reinforcing			
	Cover	Bar	Pitch	Clear
30"	3"	#4	4.00"	3.50"
36"	3"	#5	5.00"	4.38"
42"	4"	#5	4.50"	3.88"
48"	4"	#5	3.75"	3.13"
54"	4"	#5	3.25"	2.63"
60"	6"	#5	3.13"	2.50"
66"	6"	#5	2.88"	2.25"
72"	6"	#5	2.50"	1.875"
84"	6"	#6	3.00"	2.25"
96"	6"	#6	2.63"	1.875"

For drilled shafts with permanent casing, use a #4 bar size with a pitch or spacing of 12-in for the transverse reinforcement.

Provide a construction joint between the top of the drilled shaft and the bottom of any supported substructure element.

In the case of a directly supported column, the designer should specify concrete reinforcement that incorporates the required lap splices at the construction joint. The developed lap splice should allow for both the required lap and minimum cover arising from the misalignment of the drilled shaft with respect to the column. One possible alternative that incorporates a lap splice is shown in BDM Figure 305-4. In this example, the use of ties above the construction joint allows for 3-in of misalignment of the drilled shaft and column as shown in BDM Figure 305-4 but does require an analysis considering a smaller core area.

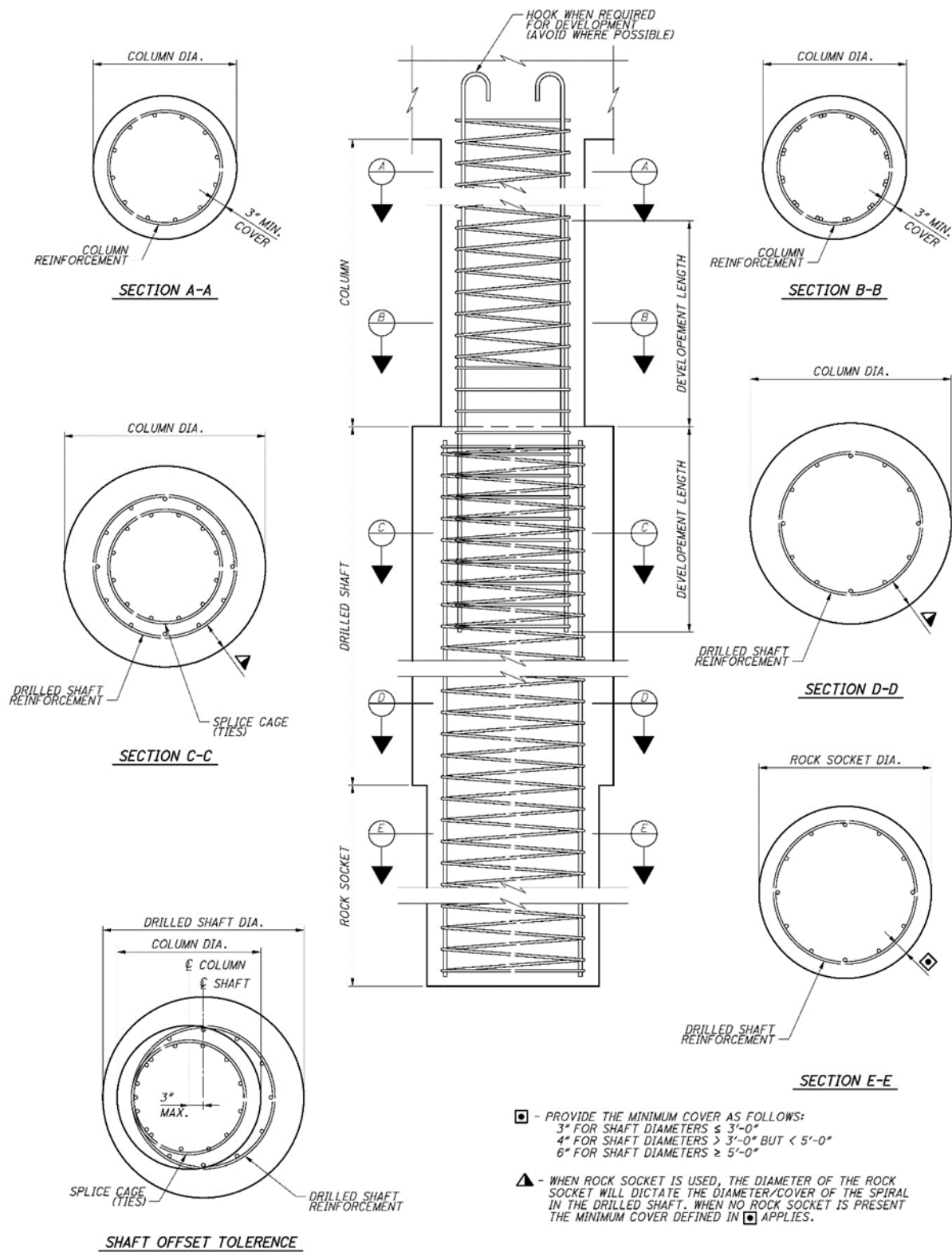


Figure 305-4

When the exposed length of the pier column is relatively short, a single full-length concrete reinforcement cage extending from the bottom of the drilled shaft into the pier cap may be specified while maintaining a minimum 3-in concrete cover within the pier column.

305.4.4.4 DRILLED SHAFT DESIGN DEPTH

When determining design depth for rock-socketed drilled shafts, assume the top of the bedrock socket as the elevation on the nearest soil boring where the top of rock (TR) is identified.

When determining design depth for friction drilled shafts, use static analysis methods to determine the penetration depth necessary to develop the required factored resistance in axial compression as described in BDM Section 305.4.1 using geotechnical resistance factors as specified in BDM Table 305-1.

When placing drilled shafts through new embankment, the shafts shall penetrate a minimum distance of 10-ft into the existing soils.

When placing drilled shafts through an MSE wall backfill, the shafts shall penetrate a minimum distance of 15-ft into the existing soils below the leveling pad elevation. If bedrock exists within the minimum penetration length, the drilled shafts shall be socketed into bedrock a minimum of 5-ft.

If bedrock is within 10-ft of the ground surface, or within 10-ft of the bottom of the drilled shaft cap, the drilled shafts shall be socketed into bedrock a minimum of 5-ft.

305.4.4.5 VERTICAL AND HORIZONTAL MOVEMENTS

Perform drilled shaft foundation settlement and horizontal movement analyses according to *LRFD 10.8.2*. Consider the effects of downdrag in accordance with BDM Section 305.4.1.2. See BDM Section 305.1.3 for limits on vertical movement with respect to a supported bridge structure. Additionally, the Department will consider vertical deflections of greater than 5% of the drilled shaft diameter to be failure.

305.4.4.6 DEMONSTRATION DRILLED SHAFTS

Specify a demonstration drilled shaft for drilled shaft structure foundations meeting any of the following conditions:

- A. Nominal shaft diameter $\geq$ 84-in
- B. Depth $\geq$ 100-ft

C305.4.4.4

If a series of soil borings indicate a sloping top of rock, interpolation between borings may be used.

The design depth may need to be adjusted during detail design as the design loads for the Service, Strength and Extreme Event Limit States are refined.

C305.4.4.5

For more guidance in estimating vertical movements, see the load displacement curves in *LRFD 10.8.2.2* and in publication FHWA-NHI-10-016, Geotechnical Engineering Circular 10 ([GEC 10](#)) “Drilled Shafts: Construction Procedures and LRFD Design Methods”.

C305.4.4.6

A demonstration drilled shaft is used to verify the proposed construction methods for the production drilled shafts. A plan note is available in BDM Section 600.

- C. Substructure supported by a single drilled shaft.
- D. Shaft constructed in open water without the use of a causeway
- E. Shaft constructed using the Wet Construction Method.
- F. Shaft construction using post grouting methods
- G. Unusual behavior of foundation materials encountered in the geotechnical exploration soil borings
- H. Artesian conditions or heaving sands encountered in the geotechnical exploration soil borings
- I. Karstic or mine void conditions encountered in the geotechnical exploration soil borings

The wet method consists of using water or slurry to contain seepage and groundwater movement and placing concrete using a tremie or concrete pump.

Unusual behavior of foundation materials include: discrepancies between in-situ test results and expected behavior based on index properties; difficulty in obtaining representative samples; extremely high spatial variability in subsurface conditions; and expansive or collapsing behavior.

305.4.5 INTEGRITY TESTING OF DRILLED SHAFTS

Specify Thermal Integrity Profiling (TIP) testing according to ASTM D7949 Method B for a minimum of 10% of all drilled shaft structure foundation elements, including at least one shaft per substructure unit.

Specify TIP testing for 100% of all drilled shaft structure foundation elements meeting any of the following conditions:

- A. Nominal shaft diameter > 60-in
- B. Substructure supported by a single drilled shaft.
- C. Shaft constructed using the Wet Construction Method or Temporary Casing Construction Method

For drilled shaft structure foundation elements with nominal diameters $\geq$ 84-in, also specify Crosshole Sonic Logging (CSL) testing in accordance with ASTM D6760.

When specifying TIP Testing by Method B, provide the number of embedded thermal sensors and their locations in the plans. Place embedded thermal sensors in string arrays at a one foot spacing between sensors on each string. When specifying CSL Testing, provide the number of access tubes and their locations in the plans.

C305.4.5

TIP testing can be waived for very shallow, permanently cased or dry shafts. In the first case, very shallow shafts are defined as 30-ft or less in total length or 30-ft or less to rock and a rock socket length of 20-ft or less. In the second case, dry shafts are defined as construction being anticipated by the Dry Construction Method in accordance with [C&MS](#) 524.04.A.

See BDM Section C305.4.4.6 for definition of the Wet Construction Method; see [C&MS](#) 524.04.C for a description of the Temporary Casing Construction Method.

TIP is performed per Supplemental Specification 894. A Special Provision for CSL is available from the Office of Geotechnical Engineering. Additional drilled shaft integrity testing methods not listed here may be considered based upon a justification study. Refer to BDM Section 201.2.1.3.c for more information.

Plan notes are available in BDM Section 600.

305.4.6 EMBANKMENT CONSTRUCTION CONSTRAINTS

Refer to BDM Section 305.4.1.2 regarding downdrag and drag load effects from new embankment settlement, and potential methods of remediation. Refer to BDM Section 305.4.4.4 regarding drilled shaft embedment through new embankments.

305.5 MICROPILES

Consult with the Office of Geotechnical Engineering before considering micropiles. Design micropiles in accordance with *LRFD 10.9*. The resistance factor for geotechnical axial resistance shall be as specified in BDM Table 305-1 for micropiles installed according to Section 33 of the *AASHTO LRFD Bridge Construction Specifications*.

Refer to BDM Section 201.2.1.3 for information to include in the Structure Foundation Exploration Report.

Specify the number of verification tests and show the test locations in the plans. Select the number of tests by dividing the site into zones where subsurface conditions are relatively uniform using engineering judgment and performing one test in each zone including at least one test per structure. Show verification test locations in the plans. Set the Verification Test Load (VTL) as the nominal axial resistance of the micropile. Specify the VTL in the plans.

Specify proof tests on a minimum of five percent of the production micropiles, or a minimum of one per substructure unit. Set the Proof Test Load (PTL) as 80 percent of the nominal axial resistance of the micropile. Specify the PTL in the plans.

C305.4.6

When placing drilled shafts through new embankment, special considerations with respect to settlement, downdrag, additional loading effects, and resistance of the shaft may be necessary. Plan notes are available in BDM Section 600 for construction constraints regarding drilled shaft installation and embankment construction.

C305.5

Only consider micropiles as a structure foundation (after acceptance of the Office of Geotechnical Engineering) in order to address a design constraint which would make installation of a different foundation type impractical or not cost effective. Examples may include: locations where difficult access or limited headroom preclude use of other deep foundation systems; when underpinning or retrofitting existing foundations; or at locations where foundations must bridge over or penetrate subsurface voids.

Micropiles are limited in the amount of bearing resistance they can develop by several factors:

- A. Their small diameter results in a low surface area for side resistance.
- B. Tip resistance is typically neglected due to the small end area.
- C. Elastic compression of micropiles limits the effective bonded length to approximately 50-ft. Beyond this, there is no load transfer from the micropile to the surrounding foundation materials.

For micropiles used as structure foundation elements, specify an option to allow either tension or compression load tests at the preference of the construction contractor. For micropiles subject to both tension and compression loads, or micropiles used as tensile elements in slope stabilization projects, specify both tension and compression load tests. If performing both tension and compression load tests, perform the tension load test prior to the compression load test. For micropiles used as tensile elements in slope stabilization projects, specify tension load tests. Compression load tests shall conform to ASTM D1143, “Standard Test Methods for Deep Foundations Under Static Axial Compressive Load,” Procedure A: Quick Test. Tensile load tests shall conform to ASTM D3689 “Standard Test Methods for Deep Foundations Under Static Axial Tensile Load,” Procedure A: Quick Test.

Provide the total factored load in axial compression and the side and tip resistance for each micropile in the structure general notes. For laterally loaded micropiles, also provide externally applied lateral and moment bending loads, and the maximum values of the internal shear and moment.

Include Special Provision for Micropile Load Testing in the plans.

Provide a unique number designation for each micropile in the plans.

When the same micropile is to be tested in both tension and compression, it is recommended that the tension test be conducted first. This will allow the pile to be reseated during compression testing in the event some net upward residual movement occurs during the tension test.

The Special Provision for Micropile Load Testing is available from the Office of Geotechnical Engineering.

305.6 CONTINUOUS FLIGHT AUGER (CFA) PILES

Design CFA piles in accordance with [SS893](#) and BDM Section 305.4, with the exception of Sections 305.4.4.6 and 305.4.5. Use Resistance Factors for CFA Piles provided in BDM Table 305-1.

Use *LRFD Eq. 10.8.3.5.4c-2* for tip resistance for CFA piles bearing on bedrock. Ignore side resistance in bedrock.

Refer to BDM Section 201.2.1.3 for information to include in the Structure Foundation Exploration Report.

Extend concrete reinforcement for CFA piles to the bottom of the pile.

C305.6

These deep foundation elements are commonly referred to as auger-cast piles, continuous flight auger (CFA) piles, augered cast-in-place (ACIP) piles, and screw piles.

The following items must be considered before deciding to use CFA piles for foundation applications:

- A₁. Quality control and structural integrity
- B₁. Verticality of the installed CFA piles
- C₁. Subsurface geotechnical conditions
- D₁. Use of grout versus concrete

CFA piles usually cannot penetrate rock (no rock sockets) except for very weak rock. There is no consideration for the use of *LRFD Eq. 10.8.3.5.4c-1* for tip resistance.

Do not construct CFA piles on a batter. Use only vertical CFA piles.

Provide the total factored load in axial compression and the side and tip resistance for the CFA piles in each substructure unit in the structure general notes. For laterally loaded CFA piles, also provide externally applied lateral and moment bending loads, and the maximum values of the internal shear and moment.

Provide a unique number designation for each CFA pile in the plans.

305.7 FIELD VERIFICATION OF NOMINAL RESISTANCE

Forward the reports, electronic records, calculations, and all electronic instrumentation data files from all field verification testing of nominal resistance to the Office of Geotechnical Engineering.

305.7.1 DYNAMIC TESTING

When driven friction piles are specified, for each individual structure, specify one dynamic load testing item for each pile size. If multiple Ultimate Bearing Values are required for a given pile size, specify one dynamic load testing item for each Ultimate Bearing Value.

When friction drilled shafts are specified as structure foundation elements, specify a number of high-strain dynamic tests in accordance with BDM Table 305-1 and Sections 305.4.3 and 305.7.3.

For CFA piles, specify high-strain dynamic tests in accordance with [SS893](#).

C305.7

The Department intends to collect this data in a database, and to use this data for future calibration of State-specific resistance factors.

C305.7.1

The Department requires dynamic load testing to establish the driving criteria (i.e. blows per foot of penetration) for all friction piles. If the designer determines that dynamic load testing should be considered for point bearing piles on bedrock, consult with the Office of Geotechnical Engineering prior to including this requirement in the plans. Dynamic load testing is performed in accordance with [C&MS 523](#) and ASTM D4945, “Standard Test Method for High-Strain Dynamic Testing of Deep Foundations.” One dynamic load testing item consists of performing successful tests on two piles (which provide adequate data to provide pile driving criteria as described in [C&MS 523.04](#)) and performing a signal matching analysis of the dynamic test data on one of the two piles. One restrrike item consists of performing dynamic testing on two piles and performing a signal matching analysis of the dynamic test data on one of the two piles. Restrikes are a useful tool to determine if a driven pile gains or loses capacity over time.

In accordance with BDM Section 305.7.3, rapid load testing is an acceptable alternative to high-strain dynamic testing.

See Section 305.7.3 for a description of high-strain dynamic testing.

Whenever setup or relaxation are realized in the design, specify restrike testing of the same piles that are dynamically load tested (specify a minimum of one restrike item for each dynamic load testing item on the structure). For a site-specific pre-construction evaluation, specify two restrike items for each dynamic load testing item (or more, if the plan note is modified to include more than two restrikes).

Whenever a static load test is specified for driven piles, also specify dynamic and restrike testing according to BDM Section 305.7.2.

305.7.2 STATIC LOAD TEST

When driven friction piles are specified, specify a static load test when the total pile order length for an individual structure exceeds 10,000-ft for piling of the same size and Ultimate Bearing Value. Specify one subsequent static load test for each additional 10,000-ft increment of pile order length. For each static load test, specify two dynamic testing items and two restrike items. Limit this requirement by the following criteria:

- A. No more than one static load test per substructure unit.
- B. A maximum number of static load tests based on site variability recommendations in *LRFD C10.5.5.2.3*.

If a static load test is specified, use a resistance factor for design of driven piles in accordance with BDM Table 305-1.

When large-diameter open-ended pipe piles are specified, specify a number of static load tests based on site variability recommendations in *LRFD C10.5.5.2.3*, or a minimum of one per structure. For each static load test, specify one dynamic testing item and one restrike item.

When friction drilled shafts are specified as structure foundation elements, specify a number of static load tests in accordance with BDM Table 305-1 and Section 305.4.3.

For micropiles, specify verification tests and proof tests in accordance with BDM Section 305.5.

Do not specify static load testing for foundation elements driven to refusal on bedrock, founded on top of bedrock, or socketed into bedrock.

305.7.3 SPECIAL LOAD TESTS

Special Load Tests include the high-strain dynamic testing, bi-directional load testing, lateral load testing, and rapid load testing.

C305.7.2

Static load testing for driven friction piles is performed in accordance with [C&MS](#) 506 and ASTM D1143, “Standard Test Methods for Deep Foundations Under Static Axial Compressive Load,” Procedure A: Quick Test. See BDM Section 600 for plan note addressing the disposition of the associated dynamic load tests.

The maximum number of static load tests could be increased for bridges of higher complexity or operational importance.

For each of these dynamic and restrike items, perform one test on the static load test pile and perform one on one of the anchor piles.

In accordance with BDM Section 305.7.3, bi-directional load testing is an acceptable alternative to a static load test.

Micropile verification tests and proof tests are required in place of any other axial load testing.

C305.7.3

Consult with the Office of Geotechnical Engineering before specifying any special load tests.

Perform the high-strain dynamic testing in accordance with ASTM D4945, “Standard Test Method for High-Strain Dynamic Testing of Deep Foundations.”

High-strain dynamic testing is typically performed on drilled foundation elements not to verify the total nominal axial resistance, but as a proof test: to prove that the installed foundation element meets the design load-deflection criteria and verify resistance adequate to support the factored load. Mobilization of the full nominal resistance would require a magnitude of displacement that would render a drilled foundation element unusable. Perform a signal matching analysis on the dynamic test data for each high-strain dynamic test. See Special Provision: High-Strain Dynamic Testing Of Drilled Shafts and [SS893](#) for details regarding this testing on drilled shaft and CFA pile foundations, respectively. Due to cost, high-strain dynamic tests are preferred over static load tests for drilled shaft foundation elements.

Perform the bi-directional load testing in accordance with ASTM D8169, “Standard Test Methods for Deep Foundations Under Bi-Directional Static Axial Compressive Load.”

Bi-directional load testing is also commonly known as the Osterberg load cell test or the “O-Cell test.” This test is typically utilized to verify the tip and side resistance for drilled shaft foundations that are designed to support high factored loads, or for structures with high importance factors as defined in BDM Section 100, or for substructure units supported on a single drilled shaft.

Perform Lateral load testing in accordance with ASTM D3966, “Standard Test Methods for Deep Foundations Under Lateral Load.”

The deflection data gathered during the performance of the test is used in combination with a flexural strength and stiffness model of the drilled shaft to perform a reverse p-y analysis to determine the lateral loading characteristics of the site soils.

Perform rapid load testing in accordance with ASTM D7383, “Standard Test Methods for Axial Rapid Load (Compressive Force Pulse) Testing of Deep Foundations.”

Rapid load testing is also commonly known as the Statnamic test. Similar to high-strain dynamic testing, rapid load testing is typically performed on drilled foundation elements as a proof test. Reduce the data collected during the test using the unloading point method (UPM) for drilled shafts less than 80-ft in length or the segmental unloading point method (SUPM) for longer drilled shafts.

306 SUBSTRUCTURE

306.1 GENERAL

306.1.1 FOOTING ELEVATIONS

Show substructure footing elevations on the Final Structure Site Plan. Refer to BDM Section 305.2.1.2 for Spread Footing elevation requirements. The top of footing (e.g. pile and drilled shaft caps) shall not be less than 1-ft below and measured normal to the finished ground line. The bottom of footing (e.g. pile and drilled shaft caps) shall not be less than 4-ft below and measured normal to the finished groundline. Evaluate the footing elevations for scour according to BDM Section 305.1.6.

306.1.2 SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Specifications for the sealer are defined in [C&MS](#) 512. Seal concrete surfaces with a concrete sealer as follows:

- A. Seal the front face of abutment backwalls, from top to bridge seat, the bridge seat and the breastwall down to the groundline with an epoxy-urethane or non-epoxy sealer.

Sealing of the backwall is not required on prestressed box beam bridges because the beams are installed before the backwall is placed.
- B. Seal the exposed surfaces of all wingwalls and retaining walls, exclusive of abutment type, that are within 30-ft of a pavement edge, with an epoxy-urethane sealer.
- C. Seal ends and sides of piers exposed to traffic-induced deicer spray, from any direction, with either an epoxy-urethane or non-epoxy sealer. Top of pier caps need only be sealed if there is an expansion joint or the tops are subject to exposure to deicer-laden water.
- D. Seal the total vertical surface of piers within 30-ft of a pavement edge with either an epoxy-urethane or non-epoxy sealer.
- E. Seal the total vertical surface of piers supporting weathering steel superstructures with either an epoxy-urethane or non-epoxy sealer

Include in the plans actual details showing the position, location and area required to be sealed. Do not use a plan note to describe the position as there can be both description and interpretation problems.

See BDM Figure 306-1, Figure 306-2 & Figure 306-3.

C306.1.2

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

In areas where concrete surfaces have a history of graffiti vandalism, the designer may add a permanent graffiti coating meeting the requirements of S1083 on top of the epoxy-urethane or non-epoxy sealer. A plan note is available in BDM Section 602.6. The designer should limit the concrete surfaces that are treated with permanent graffiti coatings to those reachable by easy climbing and visible to the traveling public.

In accordance with [C&MS](#) 516.07, the beam seats under proposed bearing locations are not sealed.

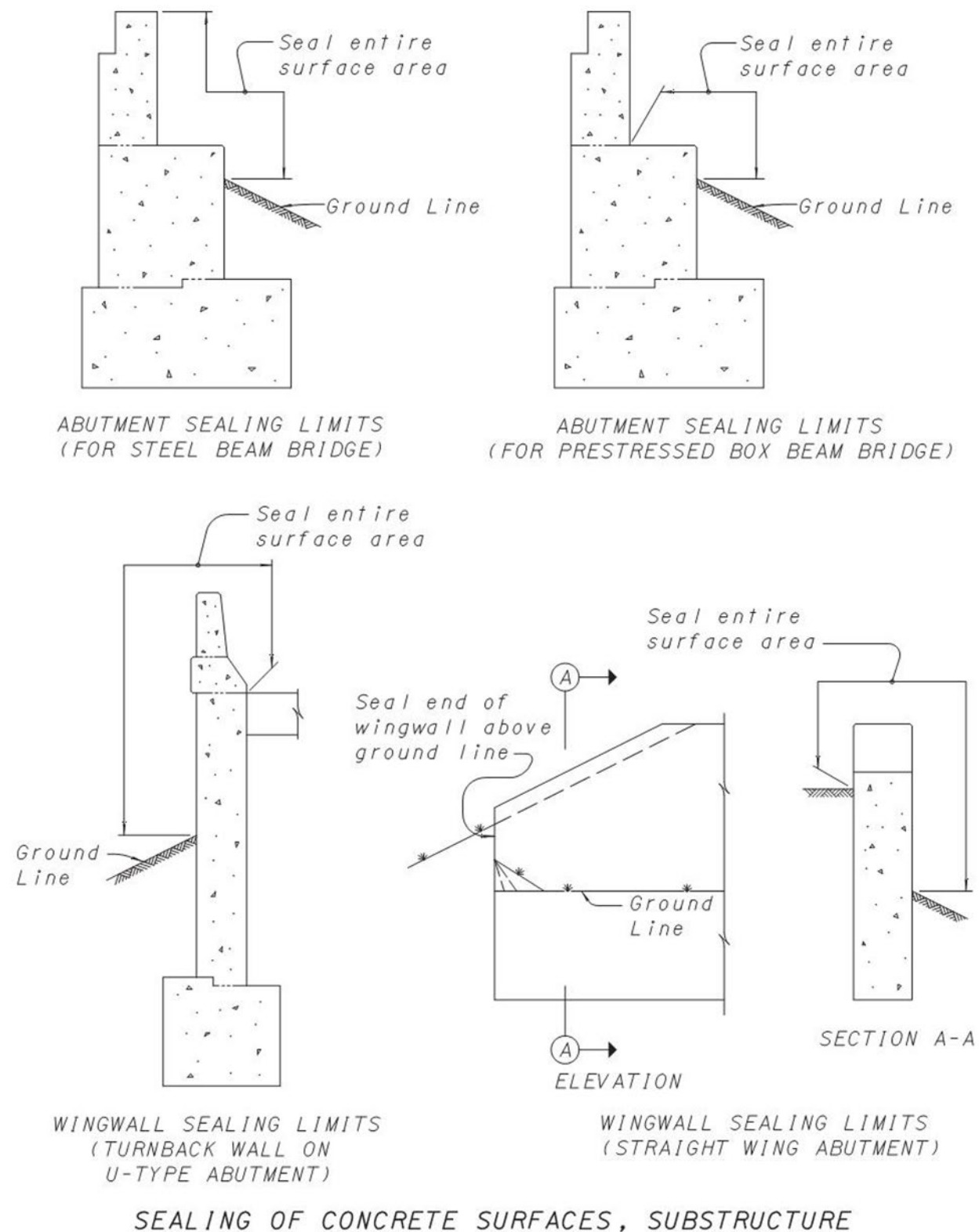
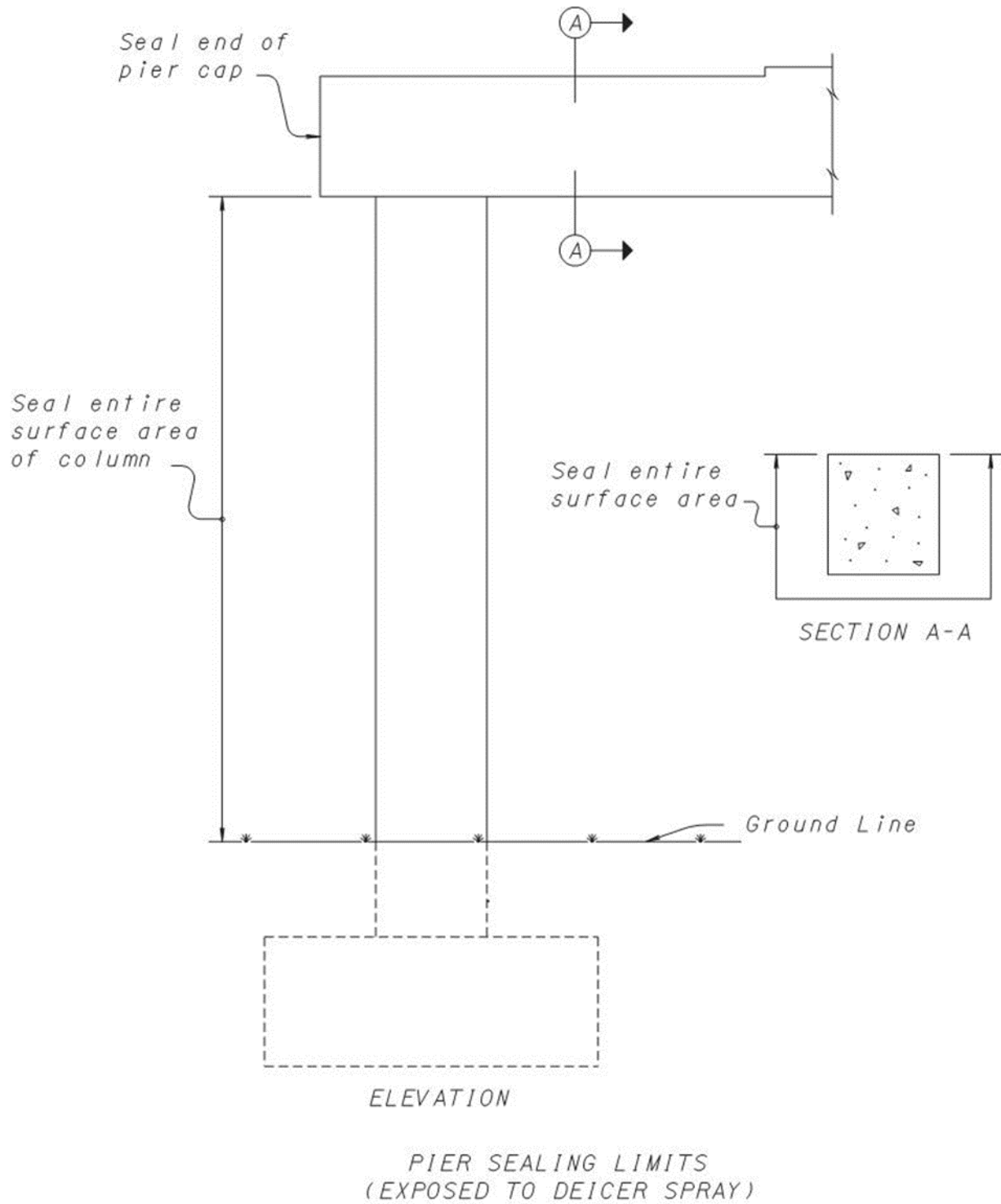
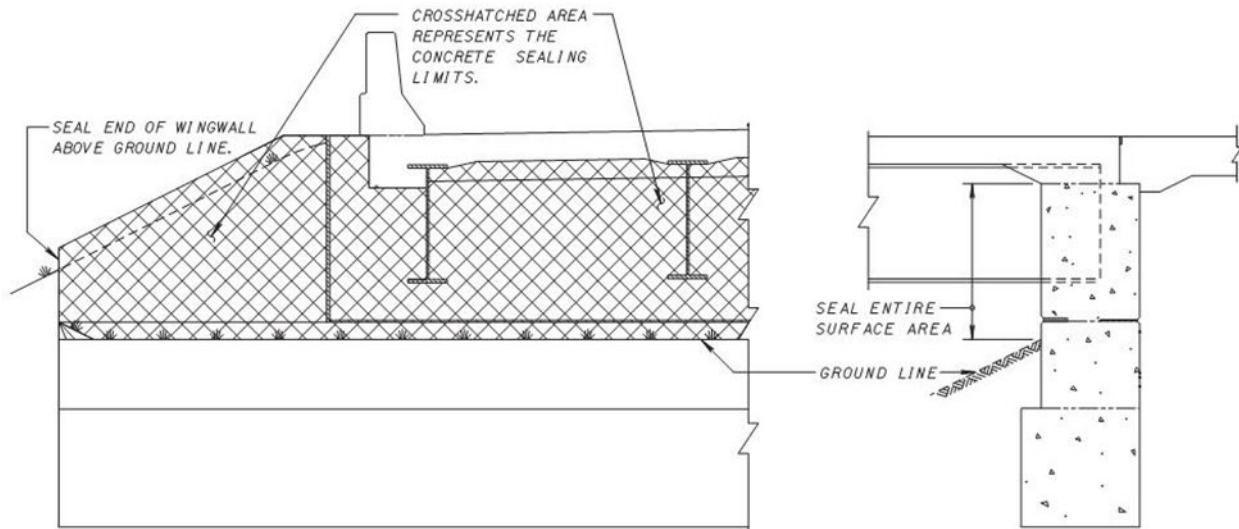


Figure 306-1

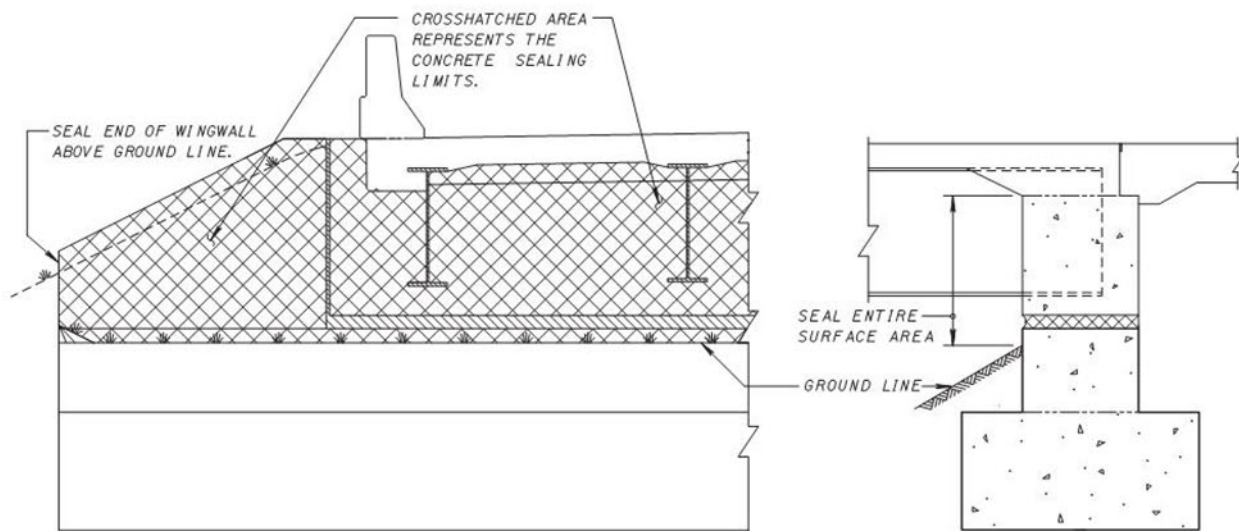


SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 306-2



ABUTMENT SEALING LIMITS
(FOR INTEGRAL ABUTMENT STEEL BEAM BRIDGE)



ABUTMENT SEALING LIMITS
(FOR SEMI-INTEGRAL ABUTMENT STEEL BEAM BRIDGE)

SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 306-3

306.2 ABUTMENTS**306.2.1 GENERAL**

For members designed to retain earth embankments and restrained from deflecting freely at their tops, determine the backfill pressure by using at-rest pressure.

Batter the front face of wall type abutments 1/16-in for each foot of abutment height. Measure height from bottom of footing to the roadway surface

306.2.1.1 BEARING SEAT WIDTH

Abutment seat widths shall meet the requirements of *LRFD 4.7.4.4*.

306.2.1.2 BEARING SEAT REINFORCEMENT

Provide supplementary reinforcement in the beam seat when the cover above the main seat reinforcement exceeds 4-in within the level portion of the beam seat

C306.2.1

Abutments should be provided with backwalls to protect the superstructure from contact with the approach fill and to assist in preventing water from reaching the bridge seat.

Examples include rigid frame bridges, abutment walls keyed to the superstructure, and some types of U-abutments.

The earth pressure to be used will vary between at-rest and passive depending on the amount of movement into the backfill. See BDM Section 306.2.2.6 for information on how to determine the appropriate earth pressure.

Battering the front face is to allow for slight tilting of wall type abutments after the backfill has been placed.

C306.2.1.1

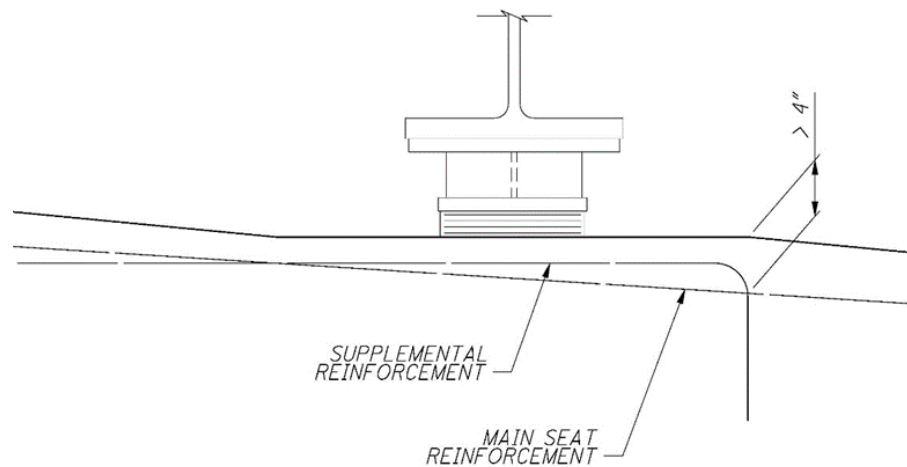
Numerous variables influence the abutment seat width. Refer to the applicable bridge standard drawing for bridge seat width requirements for the given superstructure and substructure combination.

For abutment bearing seat width requirements for prestressed box beams refer to BDM Figure 306-7 and Figure 306-8.

AASHTO seismic seat width requirements, based on length and height of structure, may require additional seat width. All defined abutment bearing seat widths can be affected due to special considerations for a specific structure or type of bearing. See BDM Section 303.1.4.

C306.2.1.2

Supplementary reinforcement is used to resist local compressive and shearing stresses in the beam seat.

**Figure 306-4**

Choose the location and spacing of all reinforcing in bridge seats so that adequate clearance is provided for bearing anchors whether cast-in or drilled-in place.

It should be noted that drilled-in place anchors use larger holes than the actual anchor and will require additional clearance.

Place a note on the substructure detail sheets cautioning the contractor to place the reinforcing to avoid interference with the anchor bolts. Also, provide a “Bearing Anchor Plan” to adequately show the location of the bearing anchors with respect to the main reinforcing bars and the edges of the bridge seats.

306.2.1.3 PHASED CONSTRUCTION JOINTS C306.2.1.3

Seal the vertical joint between construction phases on the back side of abutment backwalls and breastwalls from the top of the footing to the approach slab seat with ITEM 512, TYPE 2 WATERPROOFING, 3-ft wide centered on the joint.

306.2.1.4 EARTH BENCHES & SLOPES C306.2.1.4

Do not provide a rectangular or trapezoidal shaped bench in the soil as seen in the Plan View in front of the abutment face.

This makes the structure unnecessarily long. The Department will permit a triangular shaped bench formed where a level top of slope intersects at the nearest face of the abutment

Spill thru slopes shall be 2:1, except where soil analysis or existing slopes dictates flatter slopes. Measure the slope normal to the face of the abutment.

During scoping of the project consider right-of-way limits, wingwall lengths, future maintenance issues, and overall bridge aesthetics and if altering the slope from 2:1 may be beneficial.

For bridges over waterways where wildlife paths are included in the scope of services, benches & slopes shall be provided as shown in the Standard Bridge Drawing, [WU-1-26](#).

Wildlife paths/wildlife underpasses are a low-cost modification that effectively reduces the number of deer-vehicle collisions. Benching for wildlife paths can be installed under new and existing bridges without increasing the length of the bridge. For more information refer to the [ODOT User Guide: A Simple Solution to Reduce Deer-Vehicle Collisions: Wildlife Underpass](#) located on the OES website.

For superelevated bridges over waterways, the intersection of the top of slope with the face of abutment shall be on a level line.

For other superelevated structures the top of slope should generally be made approximately parallel to the bridge seat.

For structures over streets and roads having steep grades, the intersection of earth slope and face of abutment may be either level or sloping dependent upon which method fits local conditions and gives the most economical and aesthetically pleasing structure.

The spill-thru slope shall intersect the face of abutment a minimum of one foot, 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 shall be 1.5-ft.

306.2.2 TYPES OF ABUTMENTS

C306.2.2

Preference should be given to the use of spill-thru type abutments.

For stub abutments with spread footing on soil, refer to BDM Figure 305-2 for footing elevations.

Integral and semi-integral designs should be used where possible, see BDM Sections 306.2.2.5 and 306.2.2.6 respectively for additional information.

306.2.2.1 FULL HEIGHT ABUTMENTS

C306.2.2.1

In addition to the requirements in the ODOT [Aesthetic Design Guidelines](#) adhere to the following when considering an aesthetic treatment:

When aesthetic treatments are being considered the following criteria should be followed:

- A. Do not use aesthetic treatments when they will not be visible to the public.
- B. Meet minimum cover requirements for concrete reinforcement. Do not violate minimum concrete cover over patterns or indents of the formliner where a formliner is used. This will require additional concrete and, in some cases, dimensional changes.
- C. Specify generic formliner patterns. List an alternative available from at least three suppliers. The Department will not accept the listing of a formliner pattern available only from one supplier.
- A. The selected pattern of formliner should be easily visible from a distance. Small or ornate patterns not easily visible from a distance do not enhance the structure and are not cost effective.
- B. The cost of formliners selected should add only minimal additional cost to the overall cost of the concrete (1 to 3 percent per yard of the abutment, pier or wall)

306.2.2.1.a COUNTERFORTS FOR FULL HEIGHT ABUTMENTS

C306.2.2.1.a

For full height abutments exceeding 30-ft in height, the use of counterforts begins to become economical and should be investigated.

Place concrete reinforcement in the back, sloping, face of the counterfort in two rows with a 6-in clearance between rows. Stagger concrete reinforcement splices a minimum of 3-ft, by row.

Stagger splices for reinforcing extending from the footing of a counterforted wall into the highly reinforced areas of the counterforts.

In counterforted walls, provide drainage at each pocket formed by the intersection of the counterfort and wall.

306.2.2.1.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 shall be [C&MS 512](#) Type 2 Waterproofing, 3-ft wide, centered over, and extending the full length of the joint to the top of the footing. See BDM Section 306.2.5 on requirements for expansion joints in abutments.

306.2.2.2 ABUTMENTS SUPPORTED ON MSE WALLS

For a stub abutment supported on an MSE wall, support the abutment on deep foundations regardless of the proximity of bedrock below the MSE wall leveling pad. The Department will not permit the use of spread footing supported abutments on MSE walls. Piles shall have a minimum 15-ft embedment below the MSE wall. If rock exists within the minimum embedment depth, place the piles in pre-bored holes that extend a minimum of 5-ft into bedrock. Backfill the pre-bored holes with Class QC Misc. concrete up to the top of the leveling pad elevation after pile installation.

306.2.2.3 STUB ABUTMENTS

If a stub abutment is to support a bridge having provision for relative movement between the superstructure and the abutment, at least two rows of piles are required.

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

Where multiple rows of piles are used, the piles shall align in both directions.

306.2.2.4 CAPPED PILE ABUTMENTS

Use one row of vertical piles for integral capped pile abutments.

C306.2.2.2

When conditions are appropriate, the use of MSE walls to shorten bridge spans and eliminate embankment slopes is acceptable.

Refer to BDM Section 201.2.1.3.f for the staged review requirements for MSE walls.

Spread footing supported abutments on MSE walls are susceptible to loss of bearing caused by erosion during the service life of the structure

C306.2.2.3

The front row piles may be battered to resist lateral loads. However, refer BDM Section 305.3.5.8 regarding the use of battered piles.

Where two rows of piles are used, place the rear piles directly behind alternate front piles.

C306.2.2.4

Show the construction joint at the top of the footing for cap pile abutments as optional.

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

Support capped pile abutments on at least 5 piles. Refer to *LRFD 10.5.5.2.3* for more information

This requirement is to avoid non-redundant designs.

306.2.2.5 INTEGRAL ABUTMENTS

C306.2.2.5

Integral construction involves attaching the superstructure and substructure (abutment) together. The longitudinal movements are accommodated by the flexibility of the abutments (capped pile abutment on single row of piles regardless of pile type). The superstructure may be structural steel, cast-in-place concrete, prestressed concrete I-beams or prestressed concrete box beams.

Integral design should be considered where practical.

Do not use integral design at sites where there are concerns about settlement or differential settlement

Do not use integral design with curved primary members or primary members that have bend points in any stringer line (dog-legged).

Standard Bridge Drawing [ICD-1-20](#) shows details for integral abutments with a steel beam or girder superstructure and BDM Figure 306-5 shows the bridge length limits for [ICD-1-20](#). Standard Bridge Drawing [ICD-2-18](#) shows details for integral abutments with a prestressed concrete I-beam superstructure and BDM Figure 306-6 shows the bridge length limits for [ICD-2-18](#). The length limitations defined for integral prestressed I-beam bridges corresponds to the same superstructure movement demand for steel bridges with a maximum length of 400', considering a temperature change of 150° F for steel bridges and 80° F for concrete bridges, and a shrinkage coefficient of 0.0002 in/in for concrete bridges.

Refer to BDM Figure 306-7 for details for an integral abutment with a prestressed box beam superstructure. The bridge length limits for composite prestressed box beam bridges are the same as a prestressed concrete I-beam bridges.

If the design is outside of the constraints of the standard bridge drawing or bridge design data sheets, then an alternate abutment type should be chosen, or the integral design shall be designed to account for the given circumstances. The design shall consider important variables such as, structure length, skew, lateral earth pressure and fixity of the foundation and connecting elements.

Assume the expansion length, at the abutment, is two-thirds ($2/3$) of the total length of the structure.

Use expansion bearings at all piers for new structures. Design the pier expansion bearings proportionally (by distance) to the assumption that the $2/3$ movement could occur at one of the abutments.

Do not use cantilevered or turn-back wingwalls with integral abutments.

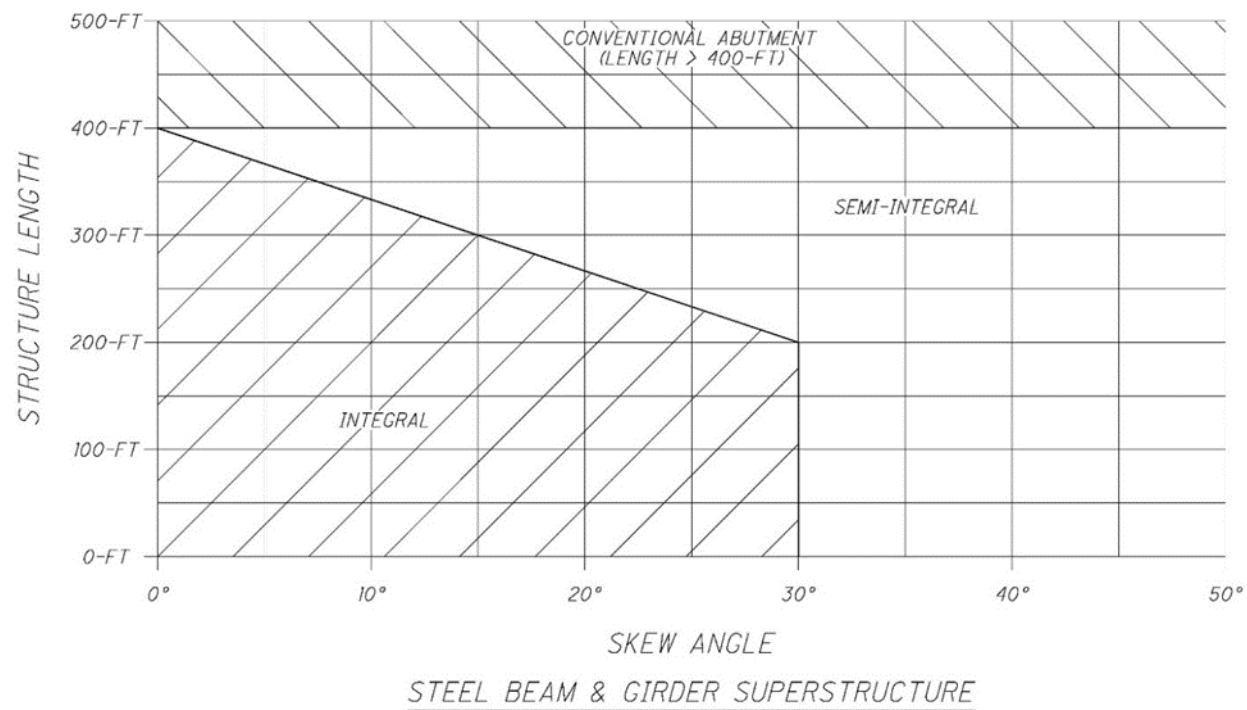
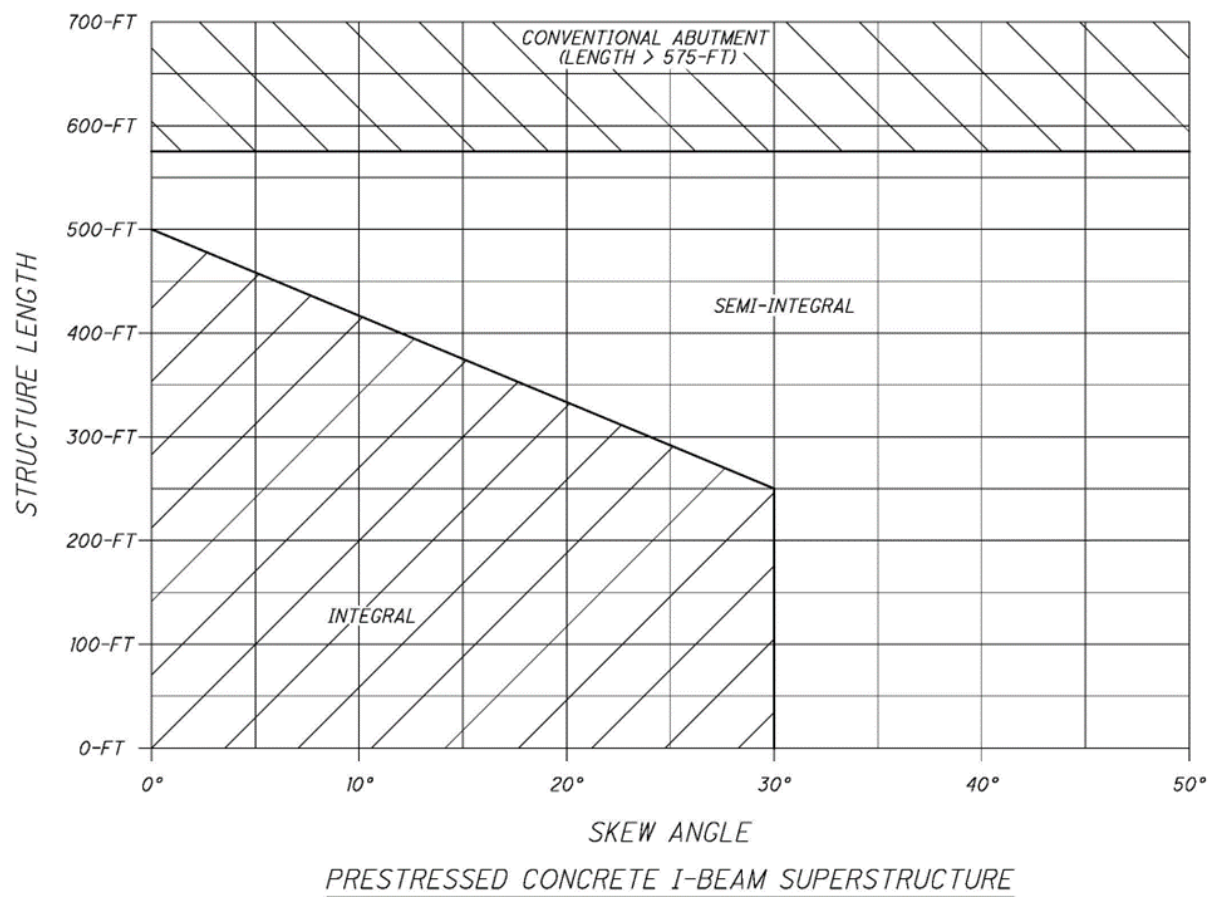
For an integral design to work properly, the geometry of the approach slab, the design of the wingwalls, (see BDM Section 306.2.4) and the transition to concrete bridge railings must be compatible with the freedom required for the integral (beams, deck, backwall, wingwalls and approach slab) connection to rotate and translate longitudinally. When this connection is not permitted to rotate or translate longitudinally an axial force will be induced onto the superstructure from the passive earth pressures that has not been accounted for in the design.

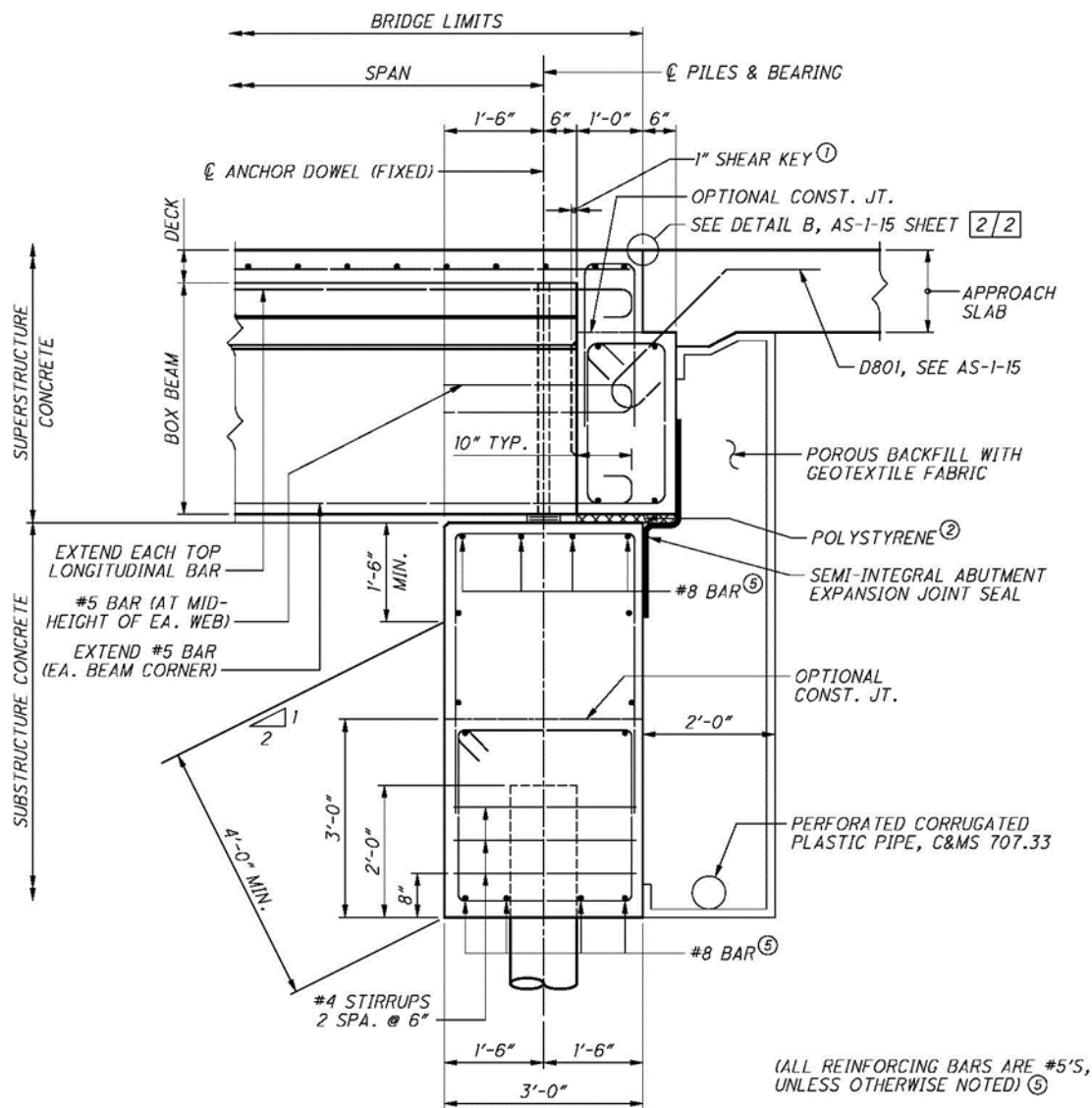
Support integral abutments on a single row of parallel piles. If an integral abutment design uses steel H-piles, orient the piles so the webs are parallel to the centerline of bearing.

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

For phased construction, see BDM Section 702.4.2 for concrete end diaphragm placement requirements.

Seal the horizontal and vertical joints at the back face of the backwall by use of a 3-ft wide sheet of nylon reinforced neoprene sheeting. Attach the sheeting on only one side of the joint to allow for the anticipated movement of the integral section. For more information refer to [C&MS](#) 516.

**Figure 306-5****Figure 306-6**



INTEGRAL BOX BEAM ABUTMENT DETAILS^④

(CAST-IN-PLACE FOUNDATION SHOWN, H-PILE SIMILAR)^③

- ① PROVIDE A 1" DEEP SHEAR KEY CENTERED IN THE BEAM END. THE SHEAR KEY HEIGHT SHALL BE ONE-HALF OF THE BOX BEAM HEIGHT AND THE WIDTH SHALL BE 26" FOR 3'-0" BOXES AND 38" FOR 4'-0" BOXES.
- ② WHEN DIMENSIONING THE TOTAL THICKNESS OF THE POLYSTYRENE, DESIGNERS SHOULD CONSIDER THAT THE MATERIAL IS AVAILABLE IN THE FOLLOWING THICKNESS: $\frac{3}{4}$ ", 1", $1\frac{1}{2}$ ", 2", $2\frac{1}{2}$ " AND 3". POLYSTYRENE SHALL BE PAID FOR UNDER ITEM 511 - CLASS QC2 CONCRETE, AS PER PLAN. INCLUDE THE FOLLOWING NOTE IN THE STRUCTURE GENERAL NOTES:
ITEM 511 - CLASS QC2 CONCRETE, AS PER PLAN; FURNISH MATERIAL MEETING THE REQUIREMENTS OF ASTM C578 TYPE IV. NEATLY CUT MATERIAL AS NECESSARY TO ALLOW FOR PROPER INSTALLATION. JOINTS AT ABUTTING PIECES SHALL BE SEALED WITH DUCT TAPE. ALLOWABLE TOLERANCE FOR THE TOTAL THICKNESS OF THE MATERIAL SHALL BE -0", +1/2". DO NOT PLACE MORE THAN TWO LAYERS OF POLYSTYRENE TO ACHIEVE TOTAL THICKNESS.
- ③ THIS DETAIL SHOWS CAST-IN-PLACE PILES. INTEGRAL ABUTMENT DETAILS FOR H-PILES SHALL BE SIMILAR. H-PILES SHALL BE ORIENTED WITH THE FLANGES PERPENDICULAR TO THE CENTERLINE OF ABUTMENT BEARINGS.
- ④ THE ASSUMED ORIENTATION OF THIS CROSS-SECTION IS PERPENDICULAR TO THE CENTERLINE OF ABUTMENT BEARINGS.
- ⑤ PLACE ALL VERTICAL BARS NORMAL TO THE CENTERLINE OF ABUTMENT BEARINGS. ALL REINFORCING SHOWN IS THE MINIMUM REQUIRED AND SHALL BE DESIGNED.

Figure 306-7

306.2.2.6 SEMI-INTEGRAL ABUTMENTS

Do not attach wingwalls to the superstructure. The vertical joints between the wingwalls and the superstructure shall be parallel with the centerline of the roadway. Fill the joints between superstructure and wingwalls with 2-in preformed expansion joint filler material, per [C&MS](#) 705.03.

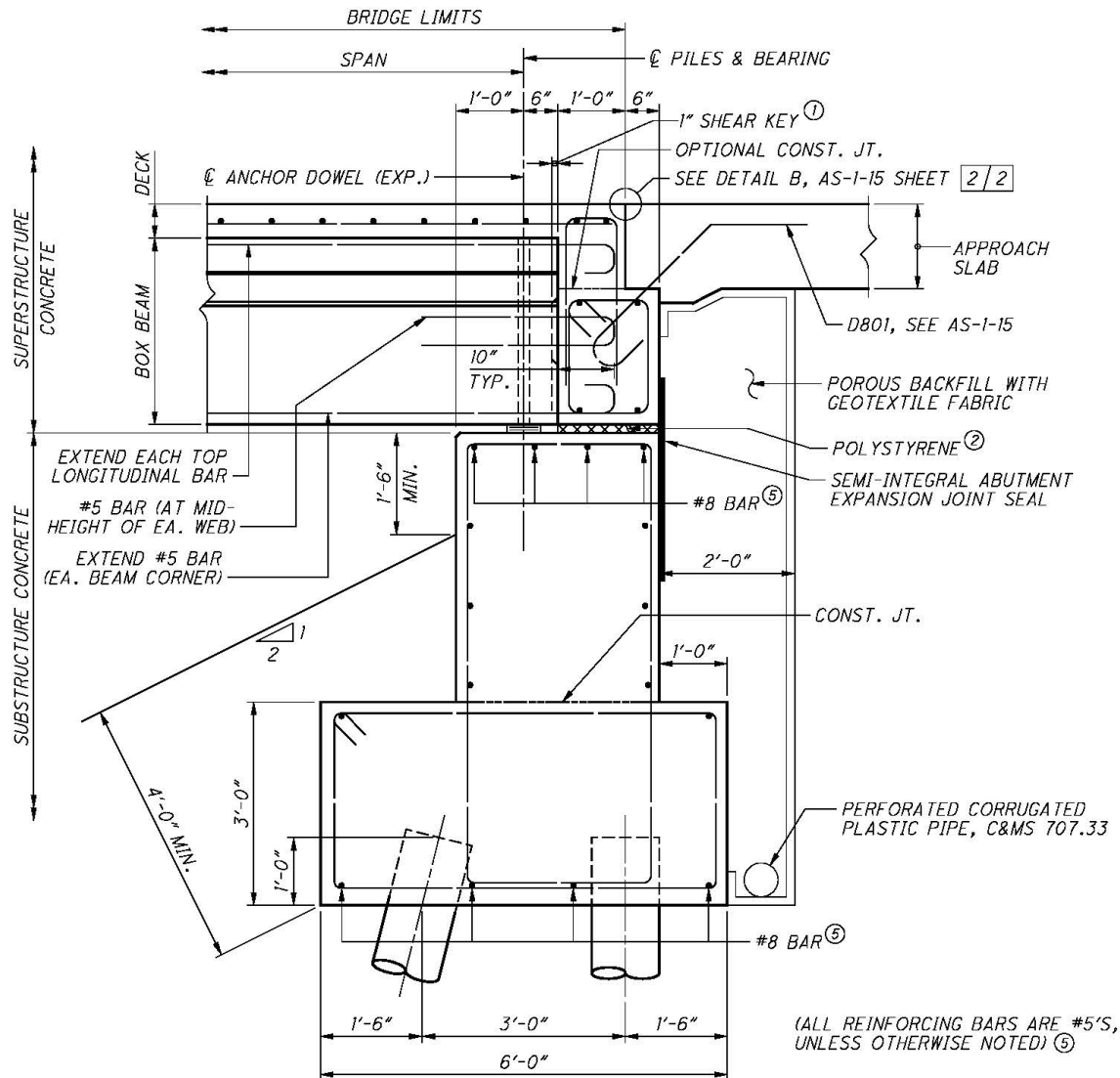
C306.2.2.6

Contrary to integral construction, semi-integral construction has no structural connection between the superstructure and the substructure. This design allows the superstructure and the approach slab to move together independent of the abutment. The longitudinal movements are absorbed between the end of the approach slab and the roadway. For details at the end of the approach slab refer the BDM Section 310.2 and standard bridge drawings [AS-1-15](#) and [AS-2-15](#). The superstructure may be structural steel, prestressed concrete I-beams or prestressed concrete box beams.

Semi-integral design should be considered and is preferred to abutments with a deck joint.

This design should be used for uncurved (straight beams) structures.

Generally, there are no skew limitations when using a semi-integral design. Refer to standard bridge drawing [SICD-1-21](#) for additional details for semi-integral abutment designs for steel structures. Use BDM Figure 306-5 for the bridge length limits for [SICD-1-21](#). Refer to standard bridge drawing [PSID-1-13](#) for additional details for semi-integral abutment designs for prestressed concrete I-beam structures. Use BDM Figure 306-6 for the bridge length limits for prestressed concrete I-beam structures. See BDM Figure 306-8 for semi-integral details for prestressed box beam structures.



SEMI-INTEGRAL BOX BEAM ABUTMENT DETAILS ④

(CAST-IN-PLACE FOUNDATION SHOWN, H-PILE SIMILAR) ③

- ① PROVIDE A 1" DEEP SHEAR KEY CENTERED IN THE BEAM END. THE SHEAR KEY HEIGHT SHALL BE ONE-HALF OF THE BOX BEAM HEIGHT AND THE WIDTH SHALL BE 26" FOR 3'-0" BOXES AND 38" FOR 4'-0" BOXES.
- ② WHEN DIMENSIONING THE TOTAL THICKNESS OF THE POLYSTYRENE, DESIGNERS SHOULD CONSIDER THAT THE MATERIAL IS AVAILABLE IN THE FOLLOWING THICKNESS: $\frac{3}{4}$ ", 1", 1½", 2", 2½" AND 3". POLYSTYRENE SHALL BE PAID FOR UNDER ITEM 511 – CLASS QC2 CONCRETE, AS PER PLAN. INCLUDE THE FOLLOWING NOTE IN THE STRUCTURE GENERAL NOTES:
ITEM 511 – CLASS QC2 CONCRETE, AS PER PLAN: FURNISH MATERIAL MEETING THE REQUIREMENTS OF ASTM C578 TYPE IV. NEATLY CUT MATERIAL AS NECESSARY TO ALLOW FOR PROPER INSTALLATION. JOINTS AT ABUTTING PIECES SHALL BE SEALED WITH DUCT TAPE. ALLOWABLE TOLERANCE FOR THE TOTAL THICKNESS OF THE MATERIAL SHALL BE -0", +1/2". DO NOT PLACE MORE THAN TWO LAYERS OF POLYSTYRENE TO ACHIEVE TOTAL THICKNESS.
- ③ THIS DETAIL SHOWS CAST-IN-PLACE PILES. SEMI-INTEGRAL ABUTMENT DETAILS FOR H-PILES SHALL BE SIMILAR. H-PILES SHALL BE ORIENTED WITH THE FLANGES PARALLEL TO THE CENTERLINE OF ABUTMENT BEARINGS.
FOR DRILLED SHAFT FOUNDATIONS, THE DRILLED SHAFT CAP SHALL EXTEND 6" BEYOND THE DIAMETER OF THE SHAFT TO ALLOW FOR MISALIGNMENT DURING CONSTRUCTION. THE CENTERLINE OF THE DRILLED SHAFT AND THE CENTERLINE OF ABUTMENT BEARINGS SHALL COINCIDE.
- ④ THE ASSUMED ORIENTATION OF THIS CROSS-SECTION IS PERPENDICULAR TO THE CENTERLINE OF ABUTMENT BEARINGS.
- ⑤ PLACE ALL VERTICAL BARS NORMAL TO THE CENTERLINE OF ABUTMENT BEARINGS. ALL REINFORCING SHOWN IS THE MINIMUM REQUIRED AND SHALL BE DESIGNED.

Figure 306-8

Do not use semi-integral design with a single row of piles.

The foundation for these designs must be stable and fixed in position for the design to function as intended.

Semi-integral details can be used on wall type abutments, spill-thru type abutments on two or more rows of piles, abutments on drilled shaft, or abutments with a spread footing may be appropriate for semi-integral abutments.

Do not use semi-integral designs at sites where there are concerns about settlement or differential settlement.

Assume the expansion length, at the abutment, is two-thirds ($2/3$) of the total length of the structure regardless of structural rigidity.

All pier bearings should be expansion bearings.

Always use expansion bearings for the abutment bearings. Design the pier expansion bearings proportionally (by distance) to the assumption that the $2/3$ movement could occur at one of the abutments.

If unsymmetrical spans (from a thermal neutral point viewpoint) are used, either all pier bearings are to be expansion or piers with fixed bearings are to be designed for the forces induced by unbalanced thermal movements.

The use of a fixed pier does not allow an increase in bridge length nor does it reduce the $2/3$ movement assumption.

If a fixed pier is used and the abutment diaphragm is subject to lateral earth pressures, consider the axial loads induced in the beams from the earth pressures in the pier design.

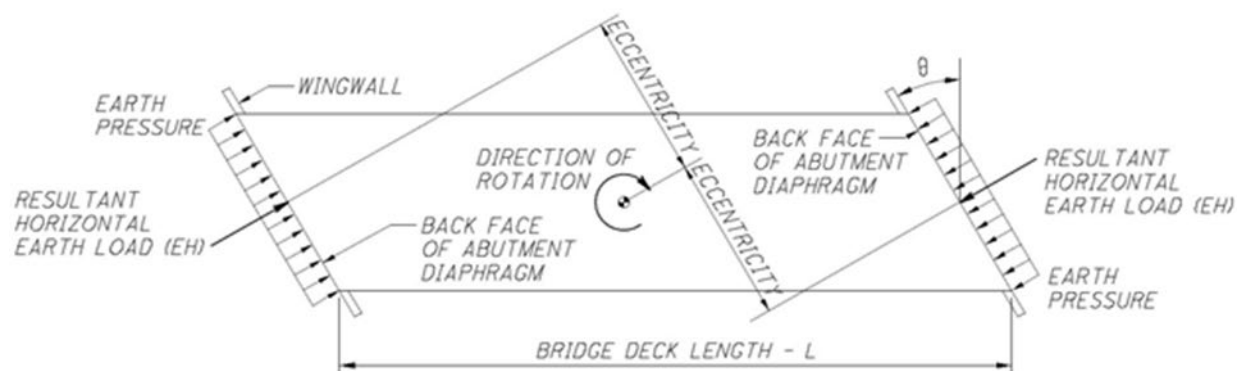
Fill the horizontal joint in the backwall created between the expansion section of the semi-integral abutment (diaphragm) and the beam seat with expanded polystyrene sheet or some equal material to act as form work for the placement of the semi-integral diaphragm concrete.

Seal both the horizontal and vertical joints at the back face of the backwall with a 3-ft wide sheet of nylon reinforced neoprene sheeting. Attach the sheeting on only one side of the joint to allow for the anticipated movement of the integral section. For more information refer to [C&MS 516](#).

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

For phased construction, see BDM Section 702.4.2 for concrete end diaphragm placement requirements.

When a semi-integral superstructure expands, earth pressures are generated on the back face of each diaphragm. These earth pressures act perpendicular to the diaphragm, and a skewed abutment will create an eccentricity about the center of the bridge. Without proper restraint, the bridge will rotate causing localized points of high stress at wingwalls and increased stresses in cross-frames, flanges, webs and bearings (See BDM Figure 306-9).



FORCE DIAGRAM DUE TO EXPANSION OF THE BRIDGE

Figure 306-9

For all semi-integral superstructures regardless of skew angle, place at least one diaphragm guide for each abutment as shown in the Standard Bridge Drawing, [SICD-2-14](#). For structures constructed in phases, construct the guide in the first phase of diaphragm construction.

The Department may consider alternative sources of restraint in lieu of diaphragm guides during the normal staged review process.

The amount of thermal movement, the height of the diaphragm, the length of the diaphragm, the amount of skew and soil data may warrant the need for an additional diaphragm guide at each abutment. A simple force analysis of a bridge superstructure with equivalent Resultant Horizontal Earth Loads (EH) at each abutment yields the following design loads on one diaphragm guide at each abutment:

$$DG_{\perp} = \gamma EH [\sin (\theta)]$$

$$DG_{\parallel} = 0.2 \gamma EH [\sin (\theta)]$$

Where:

$DG_{\perp}$ = Primary factored load acting on Diaphragm Guide in a direction perpendicular to the longitudinal axis of the bridge.

$DG_{\parallel}$ = Factored load acting on Diaphragm Guide in a direction parallel to the longitudinal axis of the bridge.

γ = Load factor, *LRFD Table 3.4.1-1*

θ = Skew angle

Depending upon the amount of thermal movement, the earth pressure acting on each diaphragm could be as little as the at-rest earth pressure or as much as passive earth pressure. As noted in *LRFD C3.11.5.4*, the movement required to mobilize full passive pressure is five percent of the height on which the passive pressure acts. When determining the earth pressure and its load factor, linear interpolation between at-rest and full passive pressure based on the amount of movement necessary to mobilize full passive pressure is acceptable.

Design Data Sheet, [SICDD-2-14](#), provides designers detailing guidance for diaphragm reinforcement around the diaphragm guide

Designing the Diaphragm Guides for a seismic horizontal connection force in accordance with *LRFD 3.10.9.2* is not required.

Install Diaphragm Guides on existing abutments being converted to semi-integral abutments as noted above. If the amount of abutment work will not accommodate the installation of the cast-in-place Diaphragm Guide reinforcement shown in [SICD-2-14](#), install the reinforcement in accordance with BDM Section 406.1. Fully detail the retrofit Diaphragm Guide in the plans, and show pay ITEM 511 SEMI-INTEGRAL DIAPHRAGM GUIDE as “As Per Plan”.

When all of the bridge bearings are expansion bearings, consider providing temporary fixity before the backfill is placed behind both abutment diaphragms. Sliding plate bearings especially those with PTFE surfaces have shown a tendency to allow the superstructure to move during construction.

306.2.2.6.a SEMI-INTEGRAL ABUTMENTS WITH REINFORCED EMBANKMENT

C306.2.2.6.a

In order to utilize semi-integral abutment details with reinforced embankment on a new structure with horizontally curved alignment, similar to what is shown in BDM Figure 404-6, satisfy **ALL** of the following criteria:

If the bridge does not meet the requirements listed, you may consult the Office of Structural Engineering to determine if semi-integral abutment details with reinforced embankment is an appropriate option on the structure.

1. Structure is dog-legged OR the curvature effects may be ignored in accordance to *AASHTO 4.6.1.2.4b*.
2. The design utilizes a fixed pier.
3. The superstructure shall also have an expansion length not greater than 265-ft. Determine the expansion length according to BDM Section 309.6.3.

Contact OSE for details on using these details on curved single span structures.

For structures that are not horizontally curved, semi-integral abutment details with reinforced embankment may be utilized on new structures that meet the requirements of BDM Section 306.2.2.6.

Do not specify diaphragm guides ([SICD-2-14](#)) when utilizing a semi-integral abutment with reinforced embankment.

Diaphragm guides are used to counteract non-uniform soil pressure on the backface of the semi-integral abutment diaphragm. This is not necessary when reinforced embankment is being used.

306.2.3 ABUTMENT DRAINAGE

306.2.3.1 BACKWALL DRAINAGE

306.2.3.1.a POROUS BACKFILL

For walls that will experience applied earth pressures exceeding active pressure, provide porous backfill immediately behind abutments and retaining walls according to [C&MS](#) 518. Drain the porous backfill using a corrosion resistant pipe system into which water can percolate. See BDM Section 306.2.3.3 for possible exceptions.

Wrap porous backfill with geotextile fabric per [C&MS](#) 712.09, Type A. Cover the vertical and bottom interface between the porous backfill and the excavation with the geotextile fabric. Include a 6-in vertical up turn between the porous backfill and the back face of the abutment. Extend the porous backfill excavation to the horizontal plane of the subgrade or 1-ft below the embankment surface. Extend the porous backfill to the bottom of the abutment footing except when the vertical backface of the abutment footing extends more than 1-ft out from the vertical backface of the abutment backwall. In this case, extend the porous backfill to the top of the abutment footing. Place the porous backfill so that it is 2-ft thick for its full height behind the abutment and wingwalls except where the vertical backface of the abutment footing extends out 1-ft or less.

Place a corrugated plastic pipe drainage system at the bottom of the porous backfill and slope pipe a minimum of 1% to allow drainage. Specify corrugated plastic pipe per [C&MS](#) 518 in the plans.

Use standard corrugated plastic pipe segments, tees and elbows (either 90-degree or adjustable) for the pipe drainage system design. Connect pipe segments with overlapping bands. Install end caps at the ends of runs, unless intended to function as outlets.

C306.2.3.1.a

Use porous backfill to provide drainage behind semi-integral and integral diaphragms.

While galvanized corrugated pipe was used for years, the inertness and life expectancy of smooth internal wall plastic corrugated pipe makes this the better material to specify.

While a single outlet for the pipe drainage systems in the porous backfill can be adequate, the designer should evaluate whether the length of the drainage run requires multiple outlets to supply the porous backfill with a positive drainage system.

306.2.3.1.b PREFABRICATED GEOCOMPOSITE DRAIN (PGD)

For walls that will not experience applied earth pressure exceeding active pressure, provide drainage for the retained earth using a Prefabricated Geocomposite Drainage (PGD) system in accordance with [C&MS 518](#). Place PGD strips to provide continuous coverage over the back face of the wall. Provide a detail in the Plans showing the bottom of the PGD terminating at least 6-in into a minimum 1-ft wide section of Porous Backfill with Geotextile Fabric in accordance with [C&MS 518](#). Provide collection pipes and pipe outlets in accordance with BDM Section 306.2.3.1.a to ensure positive drainage of the Porous Backfill.

C306.2.3.1.b

Do not use PGD to provide drainage behind semi-integral diaphragms or integral abutments. The drainage board cannot be placed behind abutment elements that are prone to movement. The movement may crush the PGD.

Refer to BDM Figure 306-10 and Figure 306-11 for more information.

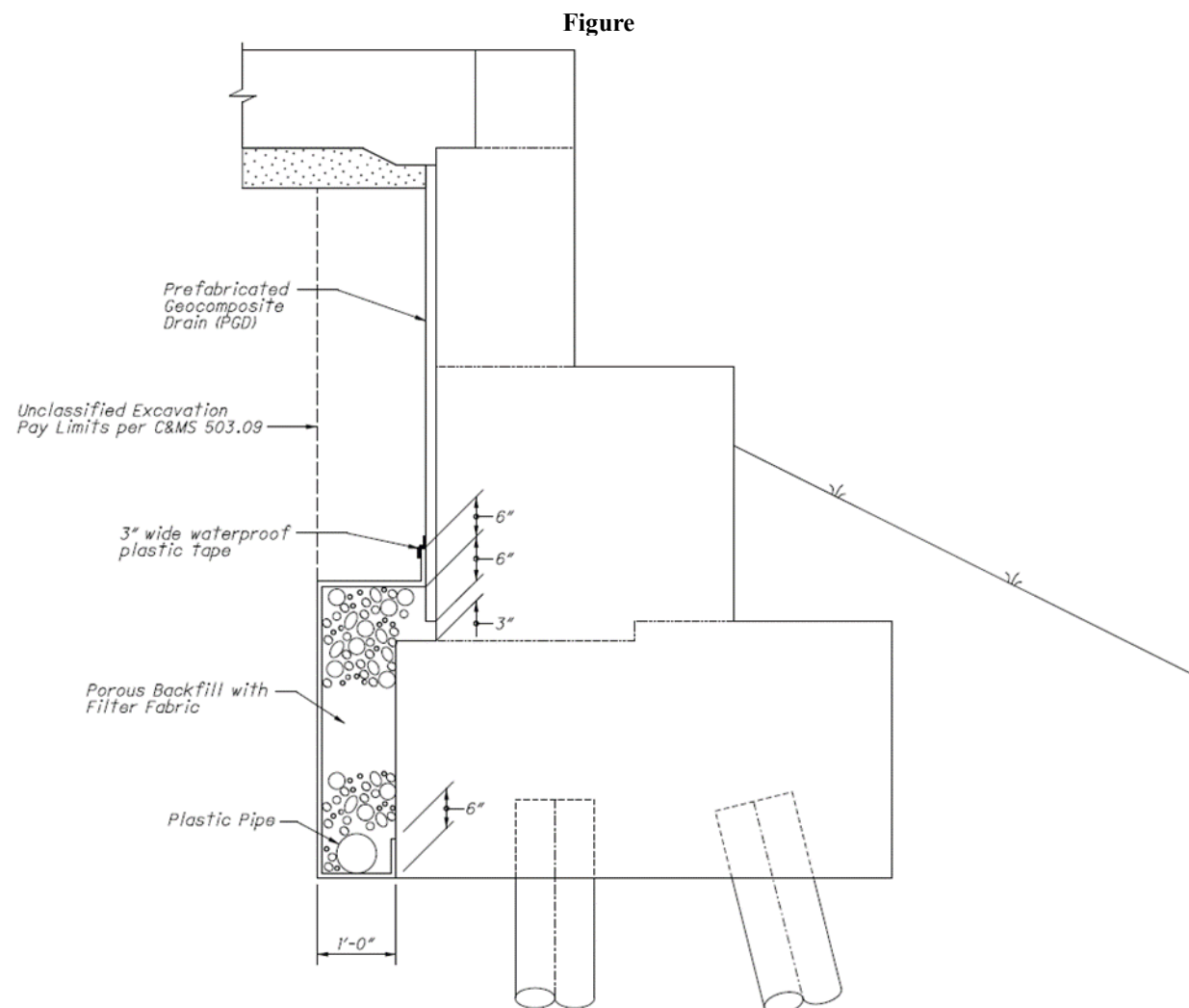


Figure 306-10

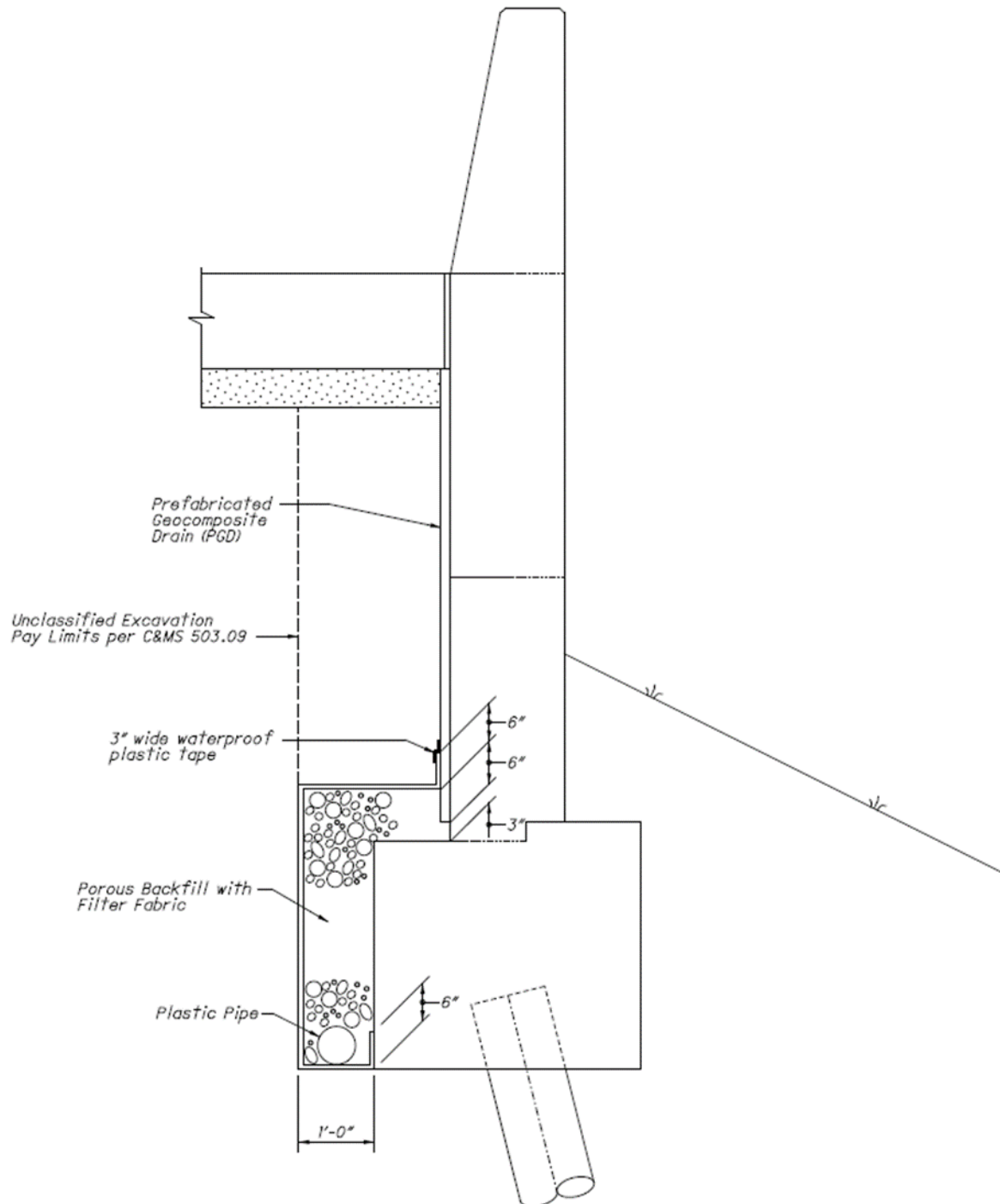


Figure 306-11

306.2.3.2 BRIDGE SEAT DRAINAGE

For abutments supporting steel beams, steel girders or prestressed I-beams with a deck joint, provide drainage of the bearing seat by sloping the bearing seat away from the backwall at 1/4-in/ft, except at the bearings.

306.2.3.3 WEEP HOLES

If a location demands the use of weep holes, place the weep holes through abutments and/or retaining walls 6-in to 12-in above the higher of OHWM or ground line. Show the porous backfill with geotextile fabric behind the walls to extend at least 6-in below the bottom of the weep holes.

Do not use weep hole type drainage systems with concrete slope protection

Where sidewalks are located immediately adjacent to wall type abutments or retaining walls, weep holes are not permitted. Instead use one of the options discussed in BDM Section 306.2.3.1.

306.2.4 WINGWALLS

Design wingwalls to have sufficient length to prevent the roadway embankment from encroaching on the stream channel or clear opening. The maximum slope of the fill behind the wingwall is 1 vertical to 2 horizontal, and compute wingwall lengths on this basis.

306.2.5 EXPANSION & CONTRACTION JOINTS

Do not place an expansion joint in an abutment breastwall beneath the superstructure. When the total length of wingwalls and breastwall exceeds 90-ft in length, place vertical expansion joints just beyond the limits the superstructure.

Fill expansion joints with preformed expansion joint material, [C&MS](#) 705.03.

Provide waterproofing for the expansion joints as described in BDM Section 306.2.2.1.b.

Detail concrete reinforcement so it does not project through expansion joints.

Do not use contraction joints on abutments or wingwalls.

306.3 PIERS**306.3.1 GENERAL**

Measure the height of the pier from the lowest of the footing elevation to the highest beam seat elevation.

C306.2.3.3

Positive drainage with a pipe system in porous backfill or a drainage board system are preferred.

The flow from the weep holes undermines the concrete protection, ultimately causing its failure.

C306.2.5

Expansion joints are required only when the total length of wingwalls and breastwall exceeds 90-ft.

C306.3.1

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, the Department will not permit individual free-standing columns without a cap.

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. Refer to ODOT's [Aesthetic Design Guidelines](#) for additional information.

306.3.2 FOOTING WIDTH

Where piling is used to support a free-standing pier, the minimum distance between centers of outside piles, measured across the footing, is one-fifth the height of the pier.

The minimum width of footing supported by a drilled shaft is the diameter of the shaft plus 6-in.

The minimum width of a spread footing for a free-standing pier is one-fourth the height of the pier where founded on soil and one-fifth the height of the pier where founded on bedrock.

306.3.3 TYPES OF PIERS

For waterway bridges use one of the following pier types:

- A. Capped Pile Pier
- B. Cap-and-Column Pier
- C. Solid Wall or T-type Pier

For bridges over railroads, use one of the following pier types:

- A. T-type Pier
- B. Solid Wall Pier
- C. Cap-and-Column Pier

306.3.3.1 CAP & COLUMN PIERS

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

C306.3.2

A “free-standing” pier is defined as one that does not depend upon its attachment to the superstructure for its ability to resist horizontal loads or forces.

C306.3.3

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.

Preference should be given to T-type piers.

C306.3.3.1

For highway grade separations, the preferred pier type is cap-and-column.

For grade separation structure, support cap-and-column piers within 30-ft of the edge of pavement on a minimum of three (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 only applies to the final pier configuration. Piers being phase constructed may be temporarily supported on less than three (3) columns.

Select cap dimensions that meet strength requirements and provide necessary bridge seat widths according to BDM Section 303.1.4.1.a.

Typically, the pier cap ends should be cantilevered and have squared ends. 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 concrete reinforcement. 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.

The minimum column diameter is 36-in.

Round columns are the preferred cross-section.

Design columns as compression members according to *LRFD 5.6.4* and *5.11*. The requirements of *LRFD 5.6.4.6* for minimum spiral reinforcement are not required. Design spiral and tie reinforcement using *LRFD Eqn. 5.11.4.1.4-1*.

Continuous spiral reinforcement is preferred over circular ties.

Refer to BDM Section 310.6 for crash wall requirements.

306.3.3.1.a CAP & COLUMN PIERS ON PILES

For piers supported on piles, place a separate footing under each column with a minimum of 4 piles per footing.

For grade separation structures, place the top of the pier's footings a minimum of 1-ft below the level of the bottom of the adjacent ditch. This applies even when the pier is located in a raised earth median barrier.

C306.3.3.1.b

306.3.3.1.b CAP & COLUMN PIERS ON DRILLED SHAFTS

Where columns are supported on a drilled shaft foundation, construct the drilled shaft at least 6-in larger in diameter than the column.

This is to allow for field location tolerances of the drilled shaft.

Locate the diameter change between the column and drilled shaft foundation 1-ft below ground level or 1-ft above the OHWM.

306.3.3.1.c CAP & COLUMN PIERS ON SPREAD FOOTINGS

Where cap-and-column piers are supported by spread footings on existing soils or on embankment fill, construct a continuous footing that extends beyond the center of the end column a minimum of 1/3 of the distance between the end column and the adjacent column.

Where cap-and-column piers are supported by spread footings on bedrock, construct a separate footing under each column.

For grade separation structures, locate the top of pier footings in accordance with BDM Section 305.2.1.2.a. Do not place the bottom of the footing in existing soil or on embankment fill above the frost line.

Build the footing for a free-standing pier with a minimum width equal to one-fourth the height of the pier where founded on soil and one-fifth the height of the pier where founded on bedrock.

306.3.3.2 CAPPED PILE PIERS

Steel H-piles shall be a minimum HP12x53. Show the piles on the plans with the flanges of the H-section perpendicular to the face of the pier cap.

For the determination of unsupported length, include the embedded depth to pile fixity and, if applicable, the scour depths

See BDM Section 305.3.5.3 for piling protection requirements and BDM Section 606.4 for a plan note to be added to design drawings when the Capped Pile Pier Standard Bridge Drawing is not referenced.

Place piles not less than 9-in from the edge of the concrete pier cap, measured along the length of the pier.

Design the exposed portions of cast-in-place reinforced concrete piles with a 16-in minimum diameter. If 16-in cast-in-place reinforced concrete piles are utilized, use the following reinforcing cage:

- A₁. #4 spiral, with a 12-in pitch and 12-in outside diameter
- B₁. 8 – #6 bars equally spaced around the interior circumference of the spiral reinforcing.

If a larger pile size is required, use the following reinforcing cage:

- A₂. #4 spiral with a 12-in pitch and an outside diameter that provides a minimum of 1.5-in of clear cover from the inside of the pipe pile

C306.3.3.1.c

This is done to provide approximately balanced moments in the footing.

Refer to BDM Section 305.2.1.2.a for additional information on the frost line.

C306.3.3.2

Capped pile type piers should be limited to an unsupported pile length of 20-ft.

The use of cast-in-place piles greater than 16-in in diameter will require an increase in the width of the cap of Standard Bridge Drawing [CPP-1-08](#).

B₂. Use the larger of 1% of the cross-sectional area of concrete or 8 – #6 bars for the longitudinal steel.

Extend the reinforcing cage from the finished top of the pile to 15-ft below the assumed point of fixity. Extend the cage a minimum of 15-ft below the ground level. Show the concrete reinforcement in the structure's reinforcing bar list and include in Item 507 for payment.

Provide pile protection for exposed H-piles and unreinforced concrete cast-in-place pipe piles.

Refer to the description in Standard Bridge Drawing [CPP-1-08](#). A plan note is also available. Also See BDM Section 606.4 for a description of pile protection.

For pile embedment requirements into concrete, see BDM Section 305.3.5.1.

Show an optional construction joint 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 to be supported on less than three (3) piles.

Support capped pile piers on at least 5 piles. Refer to *LRFD 10.5.5.2.3* for more information.

This provision is to avoid non-redundant designs.

Design the connection between the pile cap and the superstructure for the horizontal connection force specified in BDM Section 303.1.4.1.b. When using Standard Bridge Drawing, [CPP-1-08](#), verify the spacing of the P501 bars to resist this force.

No additional seismic analysis for capped pile piers is required.

306.3.3.3 T-TYPE PIERS

C306.3.3.3

In the cap of a T-type pier, place the top layer of reinforcing bars the full length of the cap and provide hooks at the face for development. Extend the second layer of concrete reinforcement into the stem of the pier at least the necessary development length plus the depth of the cantilever at its connection to the stem. Provide cap widths sufficient for bearing seat width according to BDM Section 303.1.4.1.a.

Design stems of T-type piers and wall type piers as compression members according to *LRFD 5.6.4*.

Wall type piers are characterized by the absence of clearly defined cap member.

Provide lateral ties according to *LRFD 5.10.4.3*. For bridges located in areas where $0.10 \leq S_{DI} < 0.15$, meet the requirements of *LRFD 5.11.4.1.4* and *5.11.4.1.5* for the transverse steel at the top and bottom of concrete compression members.

306.3.4 BEARING SEAT WIDTHS

Design the pier bearing seat width for reinforced concrete slab bridges according to Standard Bridge Drawing [CPP-1-08](#).

306.3.5 POST-TENSIONED CONCRETE PIER CAPS**306.3.6 STEEL CAP PIERS**

If a steel box girder is required as a pier cap, provide access to the interior.

Incorporate an access hatch at all access points. Bolt and seal access hatches with a neoprene gasket.

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

The Department will not permit box designs with cut away webs to allow for stringers to continue through the box.

Review all weld details for possible fatigue problems

C306.3.4

Pier caps on piles, drilled shafts or on columns are normally a minimum of 3-ft wide. While normally adequate, verify that bearing seat widths of 3-ft are adequate. Large bearings, skew angle, intermediate expansion devices, AASHTO earthquake seat requirements, etc. may require additional width.

C306.3.5

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 BDM Section 1001.2, these structure types require a concurrent detail design review to be performed by the Office of Structural Engineering.

C306.3.6

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 BDM Section 1001.2, these structure types require a concurrent detail design review to be performed by the Office of Structural Engineering.

The physical dimensions of the box need to be large enough to allow access to the interior for inspection, maintenance, and repair. The access hole should be a minimum of 24-in wide and 40-in tall.

Access hatches should be hinged on the side with a latch, handle and lock. It is recommended to have a vented hatch.

Situations that require stringers to be continuous through, and in the same plane with a steel pier cap or crossbeam should be avoided if possible.

306.3.7 PIERS ON NAVIGABLE WATERWAYS

Unless protected from collision by an adequate fendering system, design piers in the navigation channel of waterways, to resist collision forces based on *AASHTO Guide Specification for Vessel Collision Design of Highway Bridges*.

306.3.8 PIER CAP STIRRUP REINFORCEMENT

Detail stirrups for concrete beams of constant depth, such as pier caps, 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-degree bends at both ends of the rebar. Only select the single bar closed type stirrup when minimum required lap lengths cannot be provided with the “U” type stirrup. Place the corner with the 135-degree bends of the closed type stirrup in the compression zone of the concrete beam.

306.4 BEARINGS

306.4.1 GENERAL

When laminated elastomeric bearings are not used, provide justification, including design calculations showing why elastomeric bearings will not be adequate for the structure.

306.4.2 BEARING TYPES

306.4.2.1 ELASTOMERIC BEARINGS

Design elastomeric bearings to withstand load, expansion and rotation.

Design elastomeric bearings based on a selected durometer of either 50 or 60.

Design the laminated elastomeric bearings to fit the specific structure.

The minimum laminate thickness shall be 12 gage (0.1046-in).

Except for prestressed box beam superstructures, install a steel load plate on elastomeric bearings.

C306.4.1

The Department’s policy is to use laminated elastomeric bearings whenever possible. If additional load capacity and/or movement is required, consider using a high load multi rotational (HLMR) Bearing

C306.4.2.1

Refer to BDM Section 1014.8 for additional design requirements.

Non-laminated elastomeric bearings are acceptable only if actual design calculations support their use.

Elastomeric bearings should be limited to a 5-in maximum elastomeric height excluding internal laminates with a minimum total height of 1-in.

In July 2022, the minimum laminate thickness specified in [C&MS](#) 711.23 increased from 14 gage (0.0747-in) to 12 gage in order to stiffen the laminates making them less susceptible to warping during fabrication.

Bearing for prestressed box beams may also need load plates. See Standard Bridge Drawing [BD-1-11](#) for more information.

Control the field welding on the elastomeric bearing load plate so that the temperature of the elastomer does not exceed 300°F.

Bevel the elastomeric bearing assembly if the rotation and/or grade exceed the limitations of *LRFD Section 14*. If the elastomeric bearing assembly has a load plate and a column section, bevel the column. If the elastomeric bearing assembly only has a steel load plate, bevel the load plate. If the load plate is beveled, make smallest thickness of the load plate the design thickness.

Do not bear elastomeric bearings on unbonded steel surfaces. Vulcanize all steel plates in contact with an elastomeric bearing to the bearing.

Compensate for vertical deformations of the bearings greater than 1/8-in by adjusting the elevations of the bridge bearing seats. Place a note in the design plans.

Include the unfactored dead load, live load (without impact) and total load reactions for each elastomeric bearing design in the plans.

When the decks of twin structures without elastomeric bearings are being tied together resulting in a total structure width in excess of 60-ft, replace the existing bearings with elastomeric bearings.

For box beam bridges, place two elastomeric bearing pads at each end of each beam. Design the bearing for the required movement and rotation. At least 1-in minimum thickness is required.

On skewed box beam bridges, provide a 1/8-in thick preformed bearing shim material, [C&MS](#) 711.21, the same plan dimensions as the bearing, to accommodate any non-parallelism between bottom of beam and bridge seat. Include the preformed bearing pads in an item 516 in the Estimated Quantities.

306.4.2.2 HIGH LOAD MULTI ROTATIONAL (HLMR) BEARINGS

Where high load multi rotational (HLMR) bearings are required, provide the bearings per [SS869](#).

The column section is typically an HP shape. This assembly is used with semi-integral and integral abutment types.

It is considerably more expensive to bevel the steel load plate than the column section. For this reason, the column should be beveled if applicable. A nominal minimum thickness of 1.5-in is recommended but not mandatory.

Refer to Bridge Standard Drawing [PSID-1-13](#) for standard expansion bearing details for prestressed I-beam superstructures.

Assume a fixed bearing condition to be obtained by the use of 1-in thick laminated elastomeric bearing pads and the installation of anchor dowels with grout.

This non-parallelism between bottom of beam and bridge seat can result from camber and beam warpage due to skew and fabrication. Generally, half as many preformed bearing pads should be specified as the number of bearings.

C306.4.2.2

HLMR bearings are capable of sustaining high vertical loads and multi-directional rotations. Included with PTFE (Teflon) sliding surfaces, HLMR bearings are capable of large expansion movements.

HLMR bearings are not considered proprietary and, therefore, alternate bearing designs are not required.

For HLMR bearings, provide a plan note that requires the contractor to coordinate the substructure bearing seat elevations or dimensions with the selected bearing manufacturer.

If requested, provide the Department justification for the use of HLMR bearings. Include in the justification calculations showing why elastomeric bearings without sliding surfaces will not be adequate for the structure.

Show the design requirements for both vertical and horizontal loads, required movements, required rotations and maximum friction factor for the sliding surfaces in the plans. Clearly specify design requirements as factored/unfactored.

In the plans, require that the anchors for the HLMR bearings are to be set by use of a steel template with a minimum thickness of 1/4-in.

The minimum vertical load on a pot bearing shall not be less than 20% of total vertical design load.

306.4.3 GUIDELINES FOR FIXED ELASTOMERIC BEARINGS

306.5 SLOPE PROTECTION

For structures of the spill-thru type where pedestrian traffic adjacent to the toe of the slope is anticipated or the structure is located in an urban area within an incorporated city limit, pave the slope under the structure with concrete slope protection per [C&MS 601.07](#). Provide slope protection to all areas under freeway bridges over city streets not covered by pavement or sidewalk. Check drainage discharge from the bridge to ensure that discharge is not crossing sidewalks, etc.

On spill-thru slopes under grade separation structures, areas that are not protected by concrete slope protection, protect the slope with crushed aggregate material as provided in [C&MS 601.06](#).

AASHTO has both a design and construction section for HLMR bearings. Use both publications and this Manual as a guide in designing, selecting and specifying an HLMR bearing.

A note is available in BDM Section 702.14.

Elastomeric bearings with sliding surfaces are not economically superior to disc bearings and should not be considered.

To accommodate the required horizontal movements, these types of bearings utilizes PTFE (Teflon) to stainless steel sliding surfaces. The design should consider that the Teflon to stainless steel friction factors vary with the applied load (i.e. the lower the load, the higher the friction factor).

C306.4.3

Assume a fixed bearing condition to be obtained by the use of 1-in thick laminated elastomeric bearing pads and the installation of anchor dowels with grout

For additional information, see BDM Section 306.4.2.1 on elastomeric bearings.

C306.5

Drainage discharge that crosses sidewalks can cause ice, dirt and debris to build-up.

The slope protection, either concrete or rock, shall extend from the face of the abutment down to the toe of the slope and shall extend in width to 3-ft beyond the outer edges of the superstructure. At the acute corners of a skewed bridge, intersect the outside edge of the slope protection with the actual or projected face of the abutment 3-ft beyond the outer edge of the superstructure and extend down the slope, normal to the face of the abutment, to the toe of the slope. Toe in the base of the slope protection.

Note that the natural vegetation on the slopes when shaded by a new structure will die out. For this case consider placing additional slope protection.

307 RETAINING WALLS

307.1 GENERAL DISCUSSION

A retaining wall is a structure that supports a differential height of earth on either side or retains earth laterally.

Design bridge abutments, wing walls, and culvert headwalls as retaining walls in accordance with BDM Section 307 and *LRFD Section 11*. Design wall-type piers with differential heights of fill on either side as structure foundations in accordance with *LRFD Section 10*, and as retaining walls, in accordance with *LRFD Section 11* letting the critical conditions control the design.

The Department will only accept prefabricated retaining wall systems approved through the Prefabricated Retaining Wall System Approval Process. Select an approved wall system listed on the Department's Approved Products List. These systems include precast gravity and semigravity wall systems, prefabricated modular wall systems, and MSE wall systems, as discussed in BDM Sections 307.2, 307.3, and 307.4.

For retaining walls with cast-in-place concrete facing, provide expansion joints every 90-ft and contraction joints every 30-ft. Concrete reinforcement shall not project through expansion or contraction joints.

Place 1-in preformed expansion joint filler (PEJF), [C&MS 705.03](#), in all expansion joints.

C307.1

Refer to BDM Section 201.1.2.5 for information related to the Retaining Wall Justification.

The Prefabricated Retaining Wall System Approval Process and associated Approved List applies to all walls supporting roadway infrastructure (a roadway, roadway embankment, or other transportation-related structure, or used as a wingwall for a bridge or culvert) or any wall greater than 3 feet in exposed height. If the wall does not meet either of these criteria, then a prefabricated retaining wall system that is not on the Approved List may be used. An example would be a less than 3-foot tall landscaping wall supporting a lawn adjacent to the roadway.

Waterproof the earth-facing side of all expansion, contraction, and phased construction joints with a 3-ft width of Type 2 Waterproofing according to [C&MS 512](#), centered over the joint and extending from the top of the footing to the top of the wall. For wall types where the earth-facing side of the cast-in-place concrete is not accessible, install a 6-in PVC CRD-C 572-74 waterstop at the center of the wall thickness, centered over the joint and extending from the top of the footing to the top of the wall.

For retaining walls with full-height architectural precast concrete facing panels, set the bottom of the panels on a continuously reinforced concrete bearing grade beam. As a minimum, provide a grouted shear key connection between the panels and the grade beam designed to resist all applied lateral loads. Provide a structural attachment at the top of the panels to the primary retaining wall members (e.g. top of a drilled shaft or soldier pile) or to a reinforced concrete load distribution slab. Do not utilize an architectural concrete coping for the top structural attachment. Design all copings to accommodate thermal movements of the panels. Provide a minimum open gap of 6-in between the precast facing panels and the retaining wall. Provide drainage at the bottom of the gap in accordance with [C&MS 518](#).

For cast-in-place or precast retaining wall elements, provide a 2.0-in minimum concrete cover over the concrete reinforcement except as noted below:

- A. 2.5-in for surfaces exposed to air
- B. 3.0-in for surfaces cast against earth

For retaining walls other than culvert headwalls and wingwalls, ensure that surface drainage for the entire contributing drainage area does not overtop the coping or face of retaining walls for the 10-year storm event. Redirect surface drainage away from the wall or provide supplemental drainage consisting of a ditch above the wall with sufficient capacity to capture the drainage without overtopping. Design the ditch and line the ditch bottom in accordance with the [L&D, Vol. 2, Section 1102](#) and [Item 670 Ditch Erosion Protection](#). For specific requirements for MSE walls, see BDM Section 307.4. For guidance on collecting surface runoff at bridge abutments, see [L&D, Vol. 2, Section 1103.8.2, Erosion Control at Bridge Ends](#).

Examples of walls where the back side of the cast-in-place concrete facing is not accessible include soldier pile and lagging walls, other drilled shaft walls, soil nail walls, and sheet pile walls with permanent cast-in-place facings.

Architectural precast concrete facing panels are not a structural component of the retaining wall and are merely an aesthetic component. The concrete bearing grade beam may be a “freely floating” concrete footing bearing on the foundation soil, or may be structurally attached to the primary retaining wall members.

The requirement for concrete cover is greater for exposed surfaces, as these may come into contact with road salt. The 3-in concrete cover requirement for cast-in-place elements cast against earth is due to lesser quality control in the excavation and casting.

Provide drainage for the retained earth using a Prefabricated Geocomposite Drainage (PGD) system in accordance with [C&MS 518](#). Place PGD strips to provide continuous coverage over the back face of the wall. Provide a detail in the Plans showing the bottom of the PGD terminating at least 6-in into a 2-ft wide section of Porous Backfill with Geotextile Fabric in accordance with [C&MS 518](#). Provide collection pipes and pipe outlets to ensure positive drainage of the Porous Backfill.

Use BDM Table 307-1 for retaining wall fill soil design parameters for internal and external stability. Do not use these parameters for material acceptance. Determine soil parameters for the foundation soil or bedrock materials based on the in-situ conditions encountered by the soil borings.

PGD is not used for MSE walls or for most prefabricated modular walls. For specific drainage detailing requirements for MSE walls, see BDM Figure 201-2 through Figure 201-7. For prefabricated modular walls where PGD is impractical, provide granular drainage per [SS870](#).

This guidance is for fill soils that do not exist at the time of wall design. Walls supporting existing materials should be designed to support the existing conditions. Semigravity wall infill is the material placed behind the wall stem, and above the footing heel. Prefabricated Modular Wall Infill is the material placed within the prefabricated wall modules.

Table 307-1

Retaining Wall Fill Soil Design Parameters				
Typical Fill Zone	Type of Soil	Design Soil Unit Weight	Friction Angle	Cohesion
MSE Reinforced Soil	Select Granular Backfill (SGB), per SS840.03.E	120 lbs/ft ³	34°	0
Retained Soil (Soil behind the wall heel or behind the MSE Reinforced Soil Zone)	On-site soil varying from sandy lean clay to silty sand, per 703.16.A	120 lbs/ft ³	30°	0
CIP or Precast Semigravity Wall Infill	Granular Embankment, per 703.16.B	120 lbs/ft ³	32°	0
Prefabricated Modular Wall Infill	Modular Wall Infill, per SS870.03.C	120 lbs/ft ³	34°	0
Backfill behind abutments or headwall granular base, per C&MS 503.08 or 602.02	Granular Material Type B, per 703.16.C	120 lbs/ft ³	32°	0
MSE Foundation Preparation	Granular Material Type C, per 703.16.C	130 lbs/ft ³	34°	0

307.1.1 LOADING

Use the Coulomb equation to calculate the active earth pressure coefficient (k_a).

Use the at-rest earth pressure coefficient to compute the lateral earth loads for retaining walls that are restrained from deflecting freely.

Apply passive earth pressure in accordance with *LRFD C3.11.1* to structural members subjected to thermal movements into retained soils.

C307.1.1

Examples of retaining walls restrained from deflecting freely include rigid frame bridges, abutment walls keyed to the superstructure, and some types of U-abutments. Refer to *LRFD C3.11.1* for magnitude of tolerable movements to reach active earth pressure.

Incline the active and passive earth pressure loads on retaining walls at an angle from a perpendicular to the back face of the wall equal to the soil-structure interface angle (δ). Use the following values of δ for the following wall interfaces:

- $\delta = \frac{2}{3} \phi$ for soil-on-soil or for soil-on-cast-in-place concrete,
- $\delta = \frac{1}{2} \phi$ for soil-on-precast concrete,
- $\delta = \frac{1}{3} \phi$ for soil-on-steel, and
- $\delta = \frac{3}{4} \phi$ for soil-on-prefabricated modular stepped modules, where ϕ is the internal friction angle of the retained soil.

For analysis of earth pressure loading on non-gravity cantilever walls (e.g. drilled shaft walls and sheet pile walls) do not incline earth pressure loads at angle δ (assume $\delta = 0$).

In *LRFD Figures 3.11.5.6-1, 3.11.5.6-4, or 3.11.5.6-5* the equations for passive earth pressure do not apply if the discrete vertical wall elements are spaced at closer than three diameters ($3b$). If the foundation soil is very loose to loose granular or very soft to soft cohesive material, use the passive resistance only over a width of b . See *LRFD 3.11.5.6-1* for other restrictions.

When designing anchored walls under traffic loadings, do not exclude live load surcharge (LS). Include LS in addition to the apparent earth pressure (AEP) calculated according to *LRFD 3.11.5.7*.

If a broken back-slope is present within a horizontal distance of the height (H) of the wall behind the back pressure surface of a wall, utilize the notional slope of backfill (β') in place of the slope of the backfill surface behind the retaining wall (β) in the Coulomb equation for the calculation of the active earth pressure coefficient (k_a). Continue to use β to determine the notional height of earth pressure diagram (h). See BDM Figure 307-1 (a), (b), & (c) for examples of this procedure for a semigravity walls and MSE wall, or prefabricated wall, respectively. In place of this simplified approach, Generalized Limit Equilibrium (GLE) analysis and Coulomb trial wedge analysis, may also be used for determining the resultant horizontal earth pressure loading on retaining walls for “broken back” and more complex loading conditions.

See *LRFD Figure C3.11.5.9-2* for an example of prefabricated modular stepped modules. See *LRFD C3.11.5.3* and *Table C3.11.5.9-1* for the source of the specified values for δ . A soil-on-soil interface is typically assumed for semigravity walls regardless of whether the wall stem is composed of cast-in-place or precast concrete.

Applying vertical load components to these types of walls will tend to increase the stiffness response of the cantilever members, and is considered unconservative.

These equations assume the passive earth pressure to be distributed over a longitudinal distance of $3b$. For closer spaced discrete vertical wall elements, the load must be distributed over a longitudinal distance equal to the spacing between the discrete vertical wall elements (l). In soft or loose soils, full soil arching is assumed to not develop.

Add additional surcharge loads and water loads to the AEP, as applicable.

The “simplified approach,” is described in *LRFD C3.11.5.8.1* and is shown for the example of an MSE wall in *LRFD Figure C3.11.5.8.1-1*. Generalized Limit Equilibrium (GLE) analysis and Coulomb trial wedge analysis, are described in *LRFD 3.11.5.8.1*, and *Appendices A11.3.1* through *A11.3.3*.

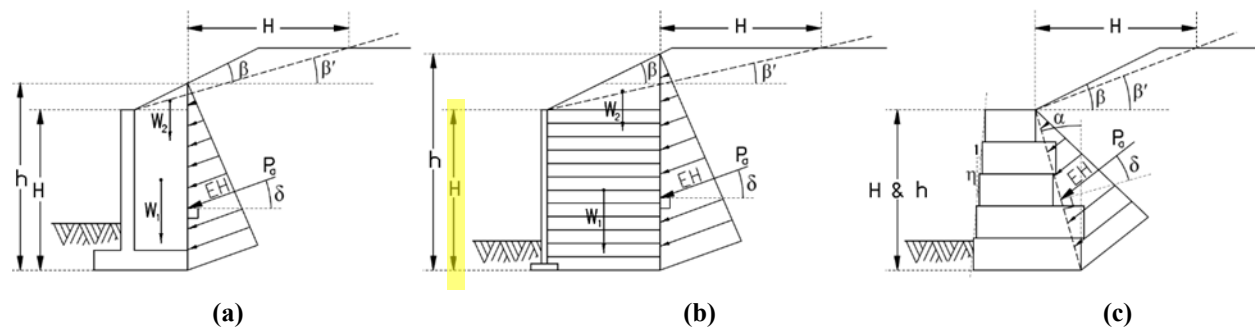


Figure 307-1

Use Earth Surcharge (ES) for earthen surcharge loads (such as a temporary earthen surcharge for consolidation) or for additional walls or earthen structures supported above the height of the wall (such as tiered walls). For tiered wall systems, the magnitude of the ES load shall be equal to the unit weight of the backfill soil times the height of the upper wall (or walls) above the top of the lower wall, as long as the upper wall is within a horizontal distance of one-half the height (H) of the lower wall behind the back face of the lower wall.

Do not treat a wedge of soil slope at the top of a wall as an earth surcharge (ES) load with load factor γ_{ES} if the wedge is located within the notional height of earth pressure diagram (h) for the wall (W_2 as shown in Figure 307 1). Instead, use vertical earth pressure (EV) for this load.

When calculating the Live Load Surcharge (LS) according to *LRFD 3.11.6.4*, the minimum total unit weight of soil shall be 125-pcf.

When factoring an inclined load into horizontal and vertical components for mathematical convenience in the calculations, do not use separate maximum and minimum load factors for the respective components of the loads. Treat each inclined load as a single load, which either serves to stabilize or destabilize the structure, then apply the same load factor (either maximum or minimum) to both the horizontal and vertical components of the load.

Include vehicular collision forces (CT) in foundation stability analyses in the Extreme Event II Limit State for walls with integral traffic barriers. Distribute vehicular collision forces along the length of the wall a minimum of the distance between two expansion joints.

When specifying retaining walls within the influence zone of a railroad track, consult with the applicable railroad company regarding their specific loading requirements.

The uncertainty attached to load factor γ_{ES} is higher than for γ_{EV} because the mechanism of load transfer to the wall is less certain.

The magnitude of the Live Load Surcharge shall be 125-pcf unless there is a special loading condition, such as heavy construction equipment or railroad loading.

Inclined loads do not have actual horizontal and vertical components; these are a mathematical convenience to make the stability computations easier to calculate. If it is uncertain whether a single, inclined load will either serve to stabilize or destabilize the structure, perform the calculations both ways.

For the Cooper E 80 load refer to Chapter 15, Part 1 of the AREMA Manual for Railway Engineering. A Cooper E 80 load on a single track equates to a Live Load Surcharge of an 1882-psf strip load, 8.5-ft wide.

Provide a crash wall as specified in the AREMA Manual for Railway Engineering for retaining walls located within 25-ft of the centerline of tracks, or other distance as specified by an individual railroad. If a retaining wall face constructed with cast-in-place or precast concrete has integral elements that meet the requirements of a crash wall, the retaining wall does not have to be protected by a separate crash wall. Prefabricated modular walls and MSE wall systems do not meet the definition of a crash wall.

Include wind-induced loads on noise barriers mounted on top of retaining walls in the design.

307.1.2 OVERALL STABILITY

Analyze overall (global) stability for each retaining wall at critical locations according to *LRFD 11.6.3.7*, with the following exceptions:

A. Do not use factored loads for the analyses.

B. Use the following resistance factors for the analyses:

Design retaining walls that support other structures for overall stability using a resistance factor of $\phi = 0.65$.

Design retaining walls that do not support other structures for overall stability using a resistance factor of $\phi = 0.75$.

When considering overall stability of a tiered wall system, analyze the stability of the tiered wall system collectively, and each tier separately.

307.1.3 RESISTANCE TO HORIZONTAL FORCES

For retaining wall types listed in BDM Section 307.2 through 307.4 and 307.7.2, perform a check for sliding resistance according to *LRFD 10.6.3.4*. If the sliding resistance of the foundation material is inadequate to withstand the horizontal forces:

- A. For retaining wall types listed in BDM Section 307.2, see BDM Section 305.2.1.3.
- B. For retaining wall types listed in BDM Section 307.3 and 307.7.2, increase the foundation width or depth, or specify internal fill with a higher unit weight.
- C. For retaining wall types listed in BDM Section 307.4, increase the soil reinforcement length.

C307.1.2

Factored loads are incompatible with limit equilibrium analysis.

Resistance factors as shown in *LRFD 11.6.2.3* should be taken as the inverse of the following factors of safety (FS) for the case of limit equilibrium slope stability analysis only:

$$\phi = 0.75 \text{ corresponds to FS} = 1.3$$

$$\phi = 0.65 \text{ corresponds to FS} = 1.5$$

Do not utilize passive resistance against sliding unless the wall penetrates below frost depth or soil disturbance. See *LRFD 11.6.3.5*. If utilities or pavement will exist in front of the wall, assume a minimum depth of soil disturbance of 3 feet.

For Reinforced Soil Slopes, refer to publications Geotechnical Engineering Circular 11 (GEC 11) Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Vol. I ([FHWA-NHI-10-024](#)) and II ([FHWA-NHI-10-025](#)) and [SS863](#).

For retaining wall types listed in BDM Section 307.6 and 307.7.1, refer to BDM Section 305.1.2 regarding *p-y* analyses.

The opposite is also the case: passive resistance can be used if the wall penetrates below frost depth or probable soil disturbance.

307.1.4 LIMITING ECCENTRICITY AND OVERTURNING RESISTANCE

For retaining wall types listed in BDM Section 307.2 through 307.4 and 307.7.2, perform a check for limiting eccentricity according to *LRFD 11.6.3.3*.

For retaining wall types listed in BDM Section 307.6 and 307.7.1, or for deep foundation elements, perform either:

- A. A *p-y* analysis at the Strength Limit State, checking that the vertical element does not have infinite or incalculable deflection
- B. A moment equilibrium analysis about the tip of the vertical elements using *LRFD Figures 3.11.5.6-1* through *3.11.5.6-7* with Strength Limit State Load Factors and Resistance Factors, in accordance with the commentary in *LRFD C11.8.4.1*.

307.1.5 BEARING RESISTANCE

For retaining wall types listed in BDM Sections 307.2 through 307.4 and 307.7.2, perform a check for bearing resistance in accordance with *LRFD 10.6.3.1* or *LRFD 10.6.3.2*, as applicable. See BDM Section 305.2 for additional guidance on bearing resistance analysis of shallow foundations.

For Reinforced Soil Slopes, refer to publications Geotechnical Engineering Circular 11 (GEC 11) Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Vol. I ([FHWA-NHI-10-024](#)) and II ([FHWA-NHI-10-025](#)) and [SS863](#).

For retaining wall types listed in BDM Sections 307.6 and 307.7.1, do not check bearing resistance unless the wall is being used to support a structure (e.g. as a bridge abutment).

C307.1.4

It is acceptable for the Strength Limit State analysis deflection to be quite large, as long as the Service Limit State analysis deflection meets the required serviceability limits.

For the determination of Service Limit State deflections using this method, it will be necessary to perform multiple integrations of the loading diagram, using the assumed stiffness, *EI*, of the vertical element.

For Anchored Wall systems, listed in BDM Section 307.8, consider bearing resistance for the vertical elements as deep foundations, assuming all vertical components of loads are transferred to the embedded portion, according to *LRFD 11.9.4.1*.

For Soil Nail Walls, refer to publication [FHWA-NHI-14-007](#), Geotechnical Engineering Circular 7 (GEC 7), Section 5.6.6, regarding Basal Heave.

307.1.6 VERTICAL AND HORIZONTAL MOVEMENTS

Limit total settlements to the serviceability requirements of the infrastructure supported by the wall. Evaluate settlement limits in the post construction condition.

Consider differential settlements for walls in the longitudinal direction.

Limit differential settlements for rigid gravity and semigravity walls, precast concrete panel walls, and all other walls with full-height concrete facings (whether cast-in-place or precast) to 1/500.

Limit differential settlements for prefabricated modular walls or modular block walls to 1/200.

Regardless of the size of facing elements, limit differential settlements for MSE walls to 1/100 without slip joints, and to 1/50 with slip joints. If differential settlements will exceed 1/50 or with total settlement of 6-in or more, design a two-stage wall system in accordance with BDM Section 307.4.1 or design a remediation to limit the settlement.

Limit differential settlements for gabion walls, Reinforced Soil Slopes (RSS), temporary walls, or for walls with facings applied after wall settlement has completed to 1/50.

For anchored walls, account for the relaxation or elongation of the anchors (and resultant unloading or loading of the anchors) due to settlement of the soil material behind the wall face or settlement of the vertical wall elements.

Additional limitations on total and differential settlement for retaining walls supporting bridge superstructures are provided in BDM Section 305.1.3. For settlement effects on bridge superstructures and settlement remediation, refer to BDM Section 305.1.3.

C307.1.6

Strength Limit State effects of settlement can properly be evaluated as soon as the affected structural element is constructed. Service Limit State effects should be evaluated post construction of the structure.

This limit will minimize cracking of the concrete.

Relaxation and unloading of anchors may result in unacceptable lateral deflection of the anchored wall face.

The Service Limit State horizontal deflection limit for all retaining walls is one percent (1%) of the retained height (H) of the wall.

Additional limits may control depending on other elements supported by or located behind the wall. For retaining walls that support bridge structures, superstructure movements shall not be impeded by horizontal deflection of the retaining structure.

307.1.7 SEISMIC DESIGN

A retaining wall does not need to be designed for seismic effects unless it supports a structure which is required to be designed for seismic loading.

For seismic design of retaining wall foundations, see Section 305.1.5.

C307.1.7

This is according to *LRFD 11.5.4.2*.

307.2 RIGID GRAVITY AND SEMIGRAVITY WALLS

Design rigid gravity and semigravity walls in accordance with *LRFD 11.6*.

C307.2

Refer to *LRFD Figure C11.6.1.1-1* for a graphical illustration of types of rigid gravity and semigravity walls.

Concrete used in rigid gravity and semigravity walls shall conform to Item 499, Class QC 1. Concrete reinforcement shall conform to [C&MS 509.02](#). Unless otherwise noted in the Scope of Services, all concrete reinforcement used in a cast-in-place concrete element shall be epoxy coated.

The minimum wall stem thickness shall be 10-in. The minimum thickness shall be 1.5-ft at the top of the stem when concrete bridge railings are cast directly on top of a retaining wall.

Refer to BDM Section 305.2.1.2 for minimum footing depth requirements.

Some traffic barrier shapes can support a differential height of earth on either side; traffic barriers used in this way are rigid gravity walls. Do not perform a stability analysis for standard traffic barrier Type C or C1 supporting a differential height of earth of 2-ft or less. If traffic barrier rigid gravity walls are bearing on pavement layers (including the aggregate base course) then do not embed the barrier further into the ground unless required for foundation stability analyses. Do not key traffic barrier rigid gravity walls bearing directly on bedrock that are found to have adequate resistance in foundation stability analyses, and the base of the wall is a minimum of 2-ft below the proposed ground surface, including pavement.

For deep foundation supported walls, analyze the external stability of the foundations according to BDM Section 305. Design other elements of the retaining wall in accordance with BDM Section 307.

307.2.1 RIGID GRAVITY WALLS

Reinforced and unreinforced cast-in-place concrete rigid gravity walls are acceptable.

307.2.2 CANTILEVER WALLS

For full height abutments and semigravity walls greater than 15-ft in height (H), batter the front face 1/16 in/ft of height of the wall stem. Refer to *LRFD Figure II.6.3.2-1* for a definition of (H). For semi-integral abutments, measure the height to the beam seat for this purpose.

307.2.3 COUNTERFORT WALLS

Provide counterforts for full height abutments and semigravity walls exceeding 30-ft in height (H).

Place concrete reinforcement in the back, sloping, face of the counterfort in two rows with a 6-in clearance between rows. Stagger the location of reinforcement splices of each row by a minimum of 3-ft. Do not post-tension counterforts.

Stagger splices of reinforcement extending from the footing of a counterfort wall into the highly reinforced areas of the counterforts.

Provide drainage in each pocket formed by the intersection of the counterfort and wall stem.

307.2.4 PRECAST GRAVITY AND SEMIGRAVITY WALLS

The Department will approve all precast gravity and semigravity wall systems through the Prefabricated Retaining Wall System Approval Process. Select a wall system listed on the Department's Approved Products List. Precast gravity and semigravity walls shall be rigid gravity, cantilever, or counterfort types, in accordance with the respective BDM Sections 307.2.1, 307.2.2, and 307.2.3.

If the wall includes a precast footing, place a bed of grout under the footing (between the foundation soil and the footing). If the wall does not include a precast footing, construct a cast-in-place footing, and place grout bedding between the cast-in-place footing and the bottom of the precast wall units. Specify the grout bedding as a minimum one inch thick. Grout shall conform to [C&MS 510.02](#).

C307.2.1

Unreinforced rigid gravity walls are often referred to as "concrete mass gravity walls."

C307.2.2

The specified batter allows for slight tilting of the wall after the backfill has been placed, in order to achieve an active earth pressure condition. For U-shaped structures, or in cases where considerable horizontal curvature increases the stability of a retaining wall, the front face shall be vertical.

C307.2.3

Counterforts may be used on shorter walls if found to be economically and structurally efficient.

This makes the minimum width of a counterfort 10-in plus the width of the two rows of concrete reinforcement. Refer to BDM Section 307.1 for minimum cover requirements in walls.

C307.2.4

These walls are often built with precast upright elements (stem and counterforts) but with cast-in-place footings; however, precast footings may also be used.

The bed of grout is to ensure an intimate contact on the bottom of the precast units and prevent rocking or differential settlement due to uneven contact with the ground.

Unless otherwise noted in the Scope of Services, all concrete reinforcement used in a precast concrete element shall be epoxy coated. The interface between two precast elements, where concrete reinforcement extends across the interface, shall be completely filled with non-shrink mortar meeting the requirements [C&MS 705.22](#).

307.3 PREFABRICATED MODULAR WALLS

Design prefabricated modular walls in accordance with *LRFD 11.11* and [SS870](#). Do not use walls of these types as bridge abutments or as bridge substructures. If precast concrete wall elements are reinforced, all concrete reinforcement shall conform to [C&MS 509.02](#), and shall be epoxy coated steel reinforcement. Concrete cover shall be in accordance with BDM Section 307.1. The interface between two precast elements, where concrete reinforcement extends across the interface, shall be completely filled with non-shrink grout meeting the requirements [C&MS 705.21](#).

The Department will approve all prefabricated modular wall systems through the Prefabricated Retaining Wall System Approval Process. Select a wall system listed on the Department's Approved Products List.

Foundation preparation shall consist of excavation to below the wall base elevation, and placement and compaction of a minimum 12-in of granular material leveling pad according to [C&MS 203.07](#). If the foundation of the wall requires more excavation, then increase the quantity of wall excavation and specify one of the following suitable materials: Modular Wall Infill according to [SS870.03.C](#); ITEM 203 GRANULAR EMBANKMENT; ITEM 203 GRANULAR MATERIAL TYPE B; or ITEM 203 GRANULAR MATERIAL TYPE C. Show the deeper excavation limits on the plans; include ITEM 203 – GRANULAR EMBANKMENT, AS PER PLAN; and a plan note permitting any of the four material types listed above. The granular material leveling pad shall extend over the entire wall foundation area, and 12-in horizontally in front of the wall face.

C307.3

These walls typically consist of square or rectangular prefabricated modular units. These units are stacked vertically at the face, stacked vertically with a setback (an effective batter), or placed battered.

The Department will not include modular wall systems with dry-cast units in the Approved Products List. Refer to BDM Section 307.3.1 for restrictions to the use of modular block walls with dry-cast units.

The thickness of the granular material leveling pad may be increased according to the requirements of the proprietary wall system manufacturer, or based on design necessity to remove unsuitable soils, provide increased bearing resistance, or reduce settlement.

For walls founded on bedrock, the foundation preparation may be eliminated.

Determine sliding resistance of prefabricated modular wall foundations both at the top and bottom of the granular material leveling pad. If the wall uses a concrete leveling pad over a granular material leveling pad, determine sliding both at the top of the concrete leveling pad and at the bottom of the granular material leveling pad. Use a sliding resistance factor of $\phi_r = 1.00$ in accordance with *LRFD Table 11.5.7-1* for gravity walls. Determine bearing resistance of prefabricated modular wall foundations at the bottom of the granular material leveling pad. Use a bearing resistance factor of $\phi_b = 0.55$ in accordance with *LRFD Table 11.5.7-1* for gravity walls.

If the proprietary wall system calls for a concrete leveling pad or for a prefabricated modular wall with a total wall height (H) greater than 6-ft, design and construct a concrete leveling pad on top of the granular material according to [SS870.06.D](#).

The minimum vertical distance from a point of the ground surface within 4-ft from the face of the wall to the top of the granular material leveling pad (or to the bottom of the concrete leveling pad, if provided) shall be 2-ft, defined as the foundation embedment. The design height of the wall (H) includes this foundation embedment as shown in BDM Figure 307-2. Measure the notional height of earth pressure diagram (h) from the lowest point at the heel of the wall foundation to the point where an imaginary line on the back face of the wall intersects the backfill slope above the wall, as shown in BDM Figure 307-2.

ODOT takes exception to the provisions of *LRFD Table 11.5.7-1* that state Article 10.5 applies for these checks. Prefabricated modular walls are not dissimilar to gravity walls with respect to external stability, and the use of lower resistance factors specified in Article 10.5 unnecessarily penalizes prefabricated modular walls. The resistance factors in *LRFD Table 10.5.5.2.2-1* are appropriate for bridge pier footings and are calibrated to previous practice factors of safety. The reason that the resistance factors in this table are in general less than for walls is due to the lower load factors specified in *LRFD Table 3.4.1-2* for the most significant sources of loading carried from a bridge superstructure.

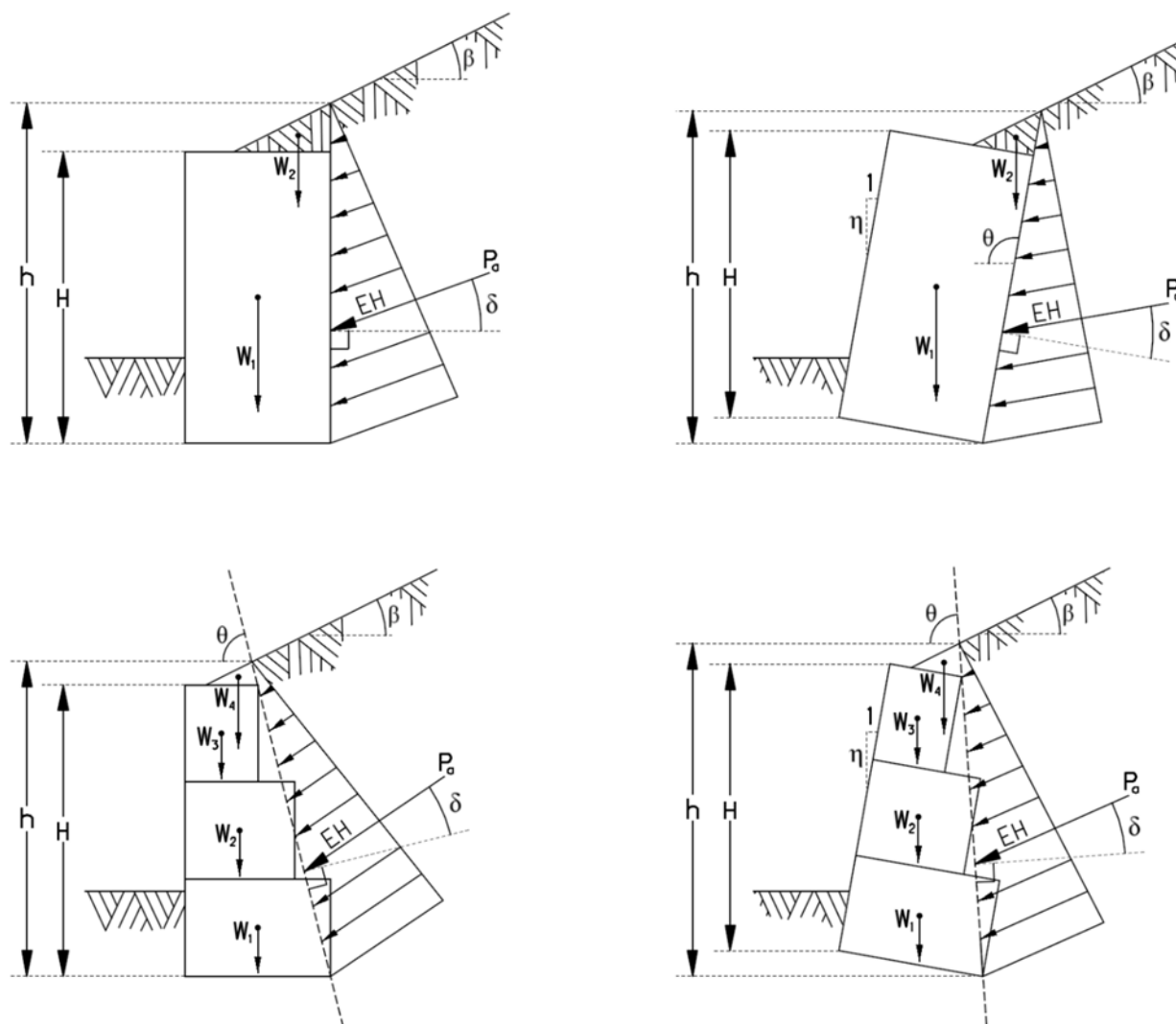


Figure 307-2

Where the alignment of prefabricated modular walls changes, use curves or corners with an interior angle of at least 90 degrees.

Check the stability of prefabricated modular walls at every module level according to *LRFD 11.11.4.1*. This includes overall stability, resistance to horizontal forces / sliding, and limiting eccentricity / overturning resistance. Also check the individual modules for structural resistance against earth pressure, according to *LRFD 11.11.5.1*.

307.3.1 MODULAR BLOCK WALLS

The alignment of prefabricated modular walls should be straight and with as few corners or curves as is practical.

C307.3.1

Modular block walls are made up of segmental concrete masonry units (blocks) of precast concrete (either dry-cast or wet-cast) laid in a running bond “brick” pattern, dry stacked (without mortar), in level courses (or layers) of blocks.

Design and construct geosynthetic reinforced modular block walls as MSE walls with block facings in accordance with BDM Section 307.4 and [SS840](#).

Do not use dry-cast block walls to support a roadway, roadway embankment, or other transportation-related structure, or as a wingwall for a bridge or culvert.

307.3.2 BIN WALLS

Design bin walls in accordance with the requirements of [SS870](#).

307.3.3 CRIB WALLS

Design crib walls in accordance with the requirements of [SS870](#).

Do not use timber crib walls for highway applications.

307.3.4 GABION WALLS

Gabion baskets used for walls shall have a minimum dimension of 36-in in all directions. Cover the back face of a gabion wall with Geotextile Fabric, Type A as a filter fabric between the retained soil and the gabions. Gabions shall meet the requirements of [SS838](#).

307.4 MSE WALLS

The Department will approve all MSE wall systems through the Prefabricated Retaining Wall System Approval Process. Select a wall system listed on the Department's Approved Products List.

Design MSE walls **for external stability** in accordance with this section, *LRFD 11.10*, and [SS840](#). The proprietary wall companies will be responsible for designing the internal stability of the wall in accordance with the project plans and [SS840](#).

Design MSE walls in accordance with the following requirements:

Modular Block Walls are also commonly known as Segmental Retaining Walls (SRW), Modular Segmental Retaining Walls, and Modular Concrete Block Retaining Walls.

Modular block walls may be designed as free-standing gravity walls depending upon the design and detailing requirements of the wall manufacturer/supplier.

Road salt and freeze-thaw cycle conditions have been shown to cause deterioration in dry-cast concrete, and severely limiting service life.

C307.3.2

A bin wall is generally a Metal Cellular Wall. However, some bin walls are made up of precast concrete bin units.

C307.3.3

A crib wall is a Concrete Cellular Wall.

C307.3.4

Gabion walls are composed of prefabricated metal baskets of welded or twisted wire mesh, backfilled with stone.

C307.4

For MSE wall submittal requirements refer to BDM Section 201.4.1.

[SS840](#) defines the requirements for construction and design for internal stability for Mechanically Stabilized Earth (MSE) walls.

- A. See BDM Figure 307-1(b) and GDM Section 1504 for a definition of how to measure the notional height of the back pressure surface of an MSE wall (h). See [SS840.04.A.2](#) for a definition of how to measure the wall height (H). The soil reinforcement length shall not be less than 70 percent of the wall height (H) or 8-ft, whichever is greater. Only increase this minimum soil reinforcement length as necessary to meet external stability requirements: bearing resistance, sliding resistance, limiting eccentricity (overturning), and overall (global) stability.
- B. The minimum vertical distance from the top of the leveling pad to any point on the ground surface within 4-ft of the face of the wall shall be 3-ft. Refer to BDM Figure 201-4 for more information. This location defines the Maximum Leveling Pad Elevation. Provide the Maximum Leveling Pad Elevation in the Plans along the wall alignment.
- C. The minimum step increment between leveling pad elevations is 1.25-ft, or one-quarter of a typical panel height. The maximum step between leveling pad elevations is 2.50-ft.
- D. To calculate the estimated quantity for the Select Granular Backfill, refer to [SS840](#) – Method of Measurement
- E. For Reinforced Soil and Retained Soil parameters, use [SS840](#) Table 840.04-1 in the design of MSE walls for internal and external stability. Determine soil parameters for the foundation soils based on the soils encountered by the soil borings. For MSE wall foundation stability, assume the groundwater level at the invert of the drainage pipe in accordance with [SS840.04.A.6](#).
- F. Foundation preparation shall consist of excavation to below the leveling pad elevation, and placement and compaction of a minimum of 12-in of Granular Material Type C according to the requirements of [C&MS](#) 204.07. If the foundation of the wall requires more excavation, then show the deeper excavation limits on the plans and include an “As per plan” item for Foundation Preparation. The foundation preparation shall extend over the entire wall foundation area, and 12-in horizontally in front of the leveling pad.
- G. Where the alignment of permanent MSE walls changes, use curves or corners with an interior angle of at least 90 degrees. For temporary MSE walls utilized for maintenance of traffic, do not use corners with interior angles of less than 45 degrees.

Typical wall panels are approximately 5-ft tall and the wall fabricators need the steps to be no less than one-quarter of their standard wall panel height. The more conservative step height would be 2.50-ft, which potentially overestimates select granular backfill quantity. Step heights less than 1.25-ft may lead to quantity underestimation for the select granular backfill.

The thickness of Type C may be increased according to the requirements of the proprietary wall system manufacturer, or based on design necessity to remove unsuitable soils, provide increased bearing resistance, or reduce settlement. For walls founded on bedrock, the foundation preparation may be eliminated.

- H. Abutments on MSE walls shall be supported on piles regardless of the proximity of bedrock to the MSE wall foundation. See BDM Section 305.3.5.7 for minimum pile embedment below the MSE wall. Abutment piles through MSE walls shall be vertical.
- I. For MSE walls containing abutments on piles:
1. The minimum distance between the back face of the MSE wall panels and the toe of the bridge abutment pile cap shall be 1-ft.
 2. The minimum distance between the back face of the MSE wall panels and the centerline of the closest row of piles shall be 3.5-ft.

For wire-faced MSE walls (temporary or permanent) or two-stage MSE walls, this minimum distance is measured to the back of the vertical face of the wire baskets. This offset is only necessary in the longitudinal direction of the bridge and is not required in the transverse direction for turn-back wingwalls or phase-line temporary walls.
 3. The minimum distance between the centerlines of adjacent rows of piles shall be 3.5-ft.

This provision allows for compaction of the fill between the pile sleeves.
- J. Do not place an integral abutment within an MSE wall.

The Department will allow the use of Semi-integral abutment designs within an MSE wall.
- K. All MSE soil reinforcements shall be connected to facing elements. The Department will not allow field cutting of MSE soil reinforcements to avoid piles or other obstacles.

Consider the ramifications of this provision when detailing the pile layout and the design of the abutments and/or wing walls.
- Do not place ITEM 204 GEOTEXTILE at the base of the Granular Material Type C foundation preparation for MSE walls.

Geotextile fabric would reduce sliding resistance of the wall foundation.
- Collect surface drainage before it reaches the face of the MSE wall. On structures with MSE walls at the abutments, provide a concrete bridge railing on the approach slab with a standard inlet, SCD I-3D to collect the drainage. Locate the inlet at least 25-ft beyond the limits of the MSE wall soil reinforcement. Continue the concrete barrier 10-ft past the catch basin. Refer to BDM Figure 309-29 for more information. When a vegetated slope is to be constructed above an MSE wall (see BDM Figure 201-8), provide a ditch in accordance with BDM Section 307.1.

Control of drainage is especially critical at MSE walls. MSE walls depend upon the integrity of the reinforced fill material for the stability of the wall. Infiltration of surface runoff into the wall can compromise the reinforced fill material by erosion or piping.
- Determine sliding resistance and bearing resistance of MSE wall foundations both at the top and bottom of the foundation preparation layer. Calculate settlement of MSE walls at the bottom of the foundation preparation layer.

307.4.1 TWO-STAGE MSE WALLS

Design wire-faced MSE walls in accordance with *LRFD 11.10* and [SS867](#). The proprietary wall companies will be responsible for designing the internal stability of the wall in accordance with the project plans and [SS867](#). All metal components of the wire-faced MSE wall shall be galvanized according to [SS867.03.A](#).

C307.4.1

This wall type consists of a wire-faced MSE wall, with permanent precast concrete facing panels applied post-construction to the wall. The precast panels can be either full-height, which are set on a concrete bearing grade beam and attached to a load distribution slab at the top of the wall, or can be conventional MSE wall panels, which are set on a leveling pad and individually attached to the wire-faced wall. The space between the full-height panels and the wire-faced wall can be left open or filled with MSE wall select granular fill; for walls with conventional MSE wall panels, the space must be filled with MSE wall select granular fill.

The typical sequence of construction involves construction of the temporary wire-faced MSE wall, a waiting period for settlement, and then attachment of the precast concrete panel facing.

[SS867](#) defines the requirements for construction and design for internal stability for wire-faced MSE walls.

307.4.2 GRS-IBS ABUTMENTS

See publications FHWA-HRT-11-026 and FHWA-HRT-11-027 for details of the system and a design methodology for GRS-IBS.

Do not use GRS-IBS to support bridges carrying Interstate, U.S. Federal Route, or State Route highways. Do not use GRS-IBS with dry-cast block wall facing elements.

C307.4.2

GRS-IBS stands for Geosynthetic Reinforced Soil Integrated Bridge System.

The maximum height for a GRS-IBS abutment is 30 feet.

307.5 REINFORCED SOIL SLOPES

Design Reinforced Soil Slope (RSS) systems in accordance with publications [FHWA-NHI-10-024](#) and [FHWA-NHI-10-025](#), Geotechnical Engineering Circular 11 (GEC 11); and [SS863](#).

Do not use RSS systems on spill-thru slope in front of abutments for water crossings.

C307.5

An RSS may be used to increase the steepness of a highway embankment slope, where a retaining wall might otherwise be used to avoid right-of-way or other impacts

Refer to BDM Section 404.6 for an acceptable application of RSS placed beneath an approach slab in combination with semi-integral abutments.

307.6 DRILLED SHAFT WALLS

Design drilled shaft walls in accordance with *LRFD 11.8*.

C307.6

The following definitions apply throughout BDM Section 307.6 and to *LRFD 3.11.5.6* and *LRFD Figures 3.11.5.6-1* through *3.11.5.6-7*:

- b = Drilled Shaft Diameter (width of a discrete vertical wall element)
- l = Drilled Shaft Center-to-center Spacing (center-to-center spacing of vertical wall elements)
- D = Foundation Depth (depth of drilled shaft embedment below design grade)
- H = Design Retained Height (height of the wall above design grade)
- L = Length of discrete vertical wall elements = D + H
- l/b = Spacing-to-diameter ratio
- D/L = Embedment-to-length ratio

Measure the design retained height (H) of drilled shaft walls from the top of the retained earth to the design grade, according to *LRFD Figures 3.11.5.6-1* through *3.11.5.6-7*. Service Limit State deflection criteria and p-y analyses control the drilled shaft embedment in accordance with BDM Sections 305.1.2 and 307.1.6. The minimum embedment (D) for drilled shaft walls founded entirely in soil shall be equal to the retained height (H) such that the embedment-to-length ratio (D/L) shall not be less than 0.5.

For drilled shaft walls with a cast-in-place concrete facing, provide a structural attachment between the facing and the exposed face of the discrete vertical wall elements. For drilled shaft walls with full-height precast concrete facing panels, see the requirements of BDM Section 307.1. For drilled shaft walls with a spacing-to-diameter ratio (l/b) greater than one, and for all soldier pile walls, design the facing to resist the lateral earth pressure.

For structural design of drilled shaft walls, in the Strength Limit State, check both the factored flexural resistance and factored shear resistance versus the maximum factored moment and shear in the discrete vertical wall elements. In the Service Limit State, check horizontal deflection at the top of the discrete vertical wall elements. Achieving fixity at the tip (bottom) of the discrete vertical wall elements is not required, provided the serviceability deflection limit is met at the top; see BDM Section 307.1.6 for details on serviceability deflection limits.

The design grade is typically taken as either the frost penetration depth, the depth to pavement subgrade located in front of the wall, or the assumed depth of future utility excavations which may occur immediately in front of the wall.

Typically, the attachment of the cast-in-place concrete facing is made with embedded dowels for drilled shafts and welded shear studs for soldier piles.

Do not analyze drilled shaft walls with the “Geotechnical Strength Limit State” analysis, specified in publication FHWA-NHI-10-016, Geotechnical Engineering Circular 10 ([GEC 10](#)) “Drilled Shafts: Construction Procedures and LRFD Design Methods,” Section 12.3.3.3.1.

When performing a p-y analysis for a drilled shaft wall, do not analyze the discrete vertical wall elements by cutting them off at the bottom of the retained height, and applying a resultant shear, moment, and axial load at the foundation elevation. Analyze the discrete vertical wall elements by using the theoretical distributed load over the entire retained height to determine the reactions above the foundation elevation.

For drilled shaft walls with an embedded steel section (typically a W-section) within a reinforcing bar cage, use composite behavior when analyzing for the shear and flexural resistance of the discrete vertical wall elements (the drilled shafts). For drilled shaft walls with an embedded steel section, and without a reinforcing bar cage, use the non-composite section (steel only) properties for the shear and flexural resistance of the discrete vertical wall elements. Assume the steel section to be continuously braced where it is embedded in the drilled shaft concrete.

See BDM Section 305.4.4.2 regarding the minimum concrete cover over concrete reinforcement for drilled shafts. For drilled shaft walls with an embedded steel section, the minimum concrete cover over concrete reinforcement also applies to the embedded steel section. When the vertical wall elements of drilled shaft walls contain both an embedded steel section and a reinforcing bar cage, the minimum clearance between the embedded steel section and the inside of the reinforcing bar cage shall be three times the maximum aggregate size.

Do not specify weathering steel for the vertical elements of a soldier pile and lagging wall or for a drilled shaft embedded steel section. Specify ASTM A709 Grade 50 steel.

Ignore the effects of self-weight of the drilled shaft foundation and of buoyancy for considerations of bearing resistance.

The “Geotechnical Strength Limit State” analysis is intended for analyses of bridge structure foundations and is not intended for retaining walls.

While “cutting off” the vertical wall elements may be sufficient for analyzing reactions below the foundation elevation, and produces realistic shear and moment distributions below the cut-off point, it does not include the shear and moment above the cut-off point and it does not adequately account for the deflection at the head of the vertical wall elements (the top of the wall).

Consideration may be given to embedding a steel section inside the reinforcing bar cage of a drilled shaft wall to increase flexural resistance and stiffness, decrease the amount of steel in the cage, and potentially decrease the size of the drilled shafts.

The equations for bearing resistance of deep foundations inherently already include self-weight of the foundation element and buoyancy, as they are based on load tests applied to the top of piles or shafts, and are measuring the net resistance of the system including the effects of self-weight and buoyancy

In *LRFD Figures 3.11.5.6-1, 3.11.5.6-4, and 3.11.5.6-5*, distribute the passive earth pressure resistance over a width of 3 times the width of a discrete vertical wall element (b). If the discrete vertical wall elements are placed closer together than 3 times b , replace the value of $3b$ with the center-to-center spacing of vertical wall elements (l) in the equations for the passive earth pressure.

In *LRFD Figures 3.11.5.6-4, 3.11.5.6-5, 3.11.5.6-6, and 3.11.5.6-7*, if $\gamma_s H < 2S_u$, use a value of zero (0) for the active earth pressure in the portion of the wall below the retained height (P_{a2}).

For tangent or secant drilled shaft walls that support a vertical load, calculate the axial side resistance for the group of drilled shafts by taking the perimeter of the group of drilled shafts, according to *LRFD 10.7.3.9 and 10.8.3.6* and publication FHWA-NHI-10-016, Geotechnical Engineering Circular 10 ([GEC 10](#)) “Drilled Shafts: Construction Procedures and LRFD Design Methods,” Section 14.4.

For drilled shaft walls embedded into bedrock, specify a single drilled shaft diameter for the full length of each vertical wall element; do not specify the drilled hole above bedrock as 6-in larger in diameter than the rock socket.

Tangent and secant drilled shaft walls with permanent cast-in-place facing, with a drilled shaft diameter of 30-in or greater, meet the requirements of an AREMA crash wall, specified in BDM Section 307.1. A soldier pile wall does not meet the requirements of an AREMA crash wall unless it has a permanent cast-in-place facing with a thickness of 30-in or greater.

For drilled shaft walls using reinforcing bar reinforcement in the shafts, specify Thermal Integrity Profiling (TIP) testing according to ASTM D7949 Method B for a minimum of 10% of the drilled shafts. For drilled shaft walls that do not use reinforcing bar reinforcement in the shafts (e.g. containing a W or HP steel section), and for soldier pile walls, do not specify TIP testing of the drilled shafts.

307.6.1 TANGENT DRILLED SHAFT WALLS

If the tangent drilled shaft wall has a permanent cast-in-place facing, place wall drainage between the permanent facing and the drilled shafts at the joints between the adjacent drilled shafts. Provide vertical drainage paths with a minimum width of 18-in. The vertical drainage may be composed of either free-draining granular material meeting the requirements of [C&MS 605.02](#), with a minimum thickness of 10-in measured perpendicular to the wall face, fully wrapped in

The zones of passive resistance for each drilled shaft discrete vertical wall element cannot overlap.

The active earth pressure in the portion of the wall below the retained height (P_{a2}) could result in a negative value if $\gamma_s H - 2S_u < 0$; however, it is not possible to have a negative active earth pressure.

This condition will typically occur where a drilled shaft wall is acting as a bridge abutment. Effective individual side resistance for each drilled shaft may be determined by dividing the group side resistance by the number of drilled shafts.

C307.6.1

Tangent drilled shaft walls have a spacing-to-diameter ratio (l/b) approximately equal to one. The wall is composed of a series of drilled shafts constructed adjacent to each other or with a small gap (less than 6-in) between the shafts.

Geotextile Fabric Type A; or may be composed of PGD strips. Where PGD is used, provide a detail in the Plans showing the bottom of the PGD terminating at least 6-in into a 2-ft wide section of Porous Backfill with Geotextile Fabric in accordance with [C&MS 518](#). Provide collection pipes and pipe outlets to ensure positive drainage of the Porous Backfill. Do not allow concrete from the facing to intrude into the joints between the adjacent drilled shafts.

If the wall is to have full-height precast concrete facing panels, see the requirements of BDM Section 307.1 regarding drainage.

307.6.2 SECANT DRILLED SHAFT WALLS C307.6.2

Design all flexural and shear resistance from horizontal loads to be provided by the reinforced primary drilled shafts alone.

Secant drilled shaft walls have a spacing-to-diameter ratio (l/b) less than one. The wall is composed of a series of reinforced primary drilled shafts, alternating and overlapping with unreinforced secondary drilled shafts. Generally, the secondary drilled shafts are constructed first, and then the primary drilled shafts are excavated through the secondary drilled shafts while the shaft concrete is still “green.” Sometimes a lower strength concrete is used for the secondary drilled shafts.

Do not provide drainage for secant drilled shaft walls. Design secant drilled shaft walls to resist static pressure from ground water.

Secant drilled shaft walls are impervious and will block ground water flow. This wall type is often used specifically for ground water control.

307.6.3 SOLDIER PILE WALLS C307.6.3

Design lagging to resist lateral earth pressure. Walls utilizing temporary timber lagging shall have a permanent cast-in-place facing.

Soldier pile walls have a spacing-to-diameter ratio (l/b) greater than one. The gaps between the discrete vertical wall elements (soldier pile beam sections) are filled with lagging, consisting of either permanent precast concrete panels or temporary lagging timbers. Timber lagging is assumed to deteriorate in the permanent condition, therefore the permanent cast-in-place facing is designed to resist the lateral earth pressure. Walls with permanent precast lagging may use full-height precast concrete facing panels.

Design precast concrete lagging panels as simply supported slabs according to *LRFD Section 5*. Precast concrete lagging panels shall be steel-reinforced, using epoxy-coated welded wire reinforcement conforming to [C&MS 709.14](#). Locate the center of the vertical wires at each end of the panel at a maximum 2.5-in from the end of the panel. Provide concrete cover over the concrete reinforcement in accordance with BDM Section 307.1. Precast concrete lagging panels shall have a minimum thickness of 5.5-in if a permanent cast-in-place facing is utilized, and 6-in otherwise. The minimum thickness of the permanent cast-in-place facing is 9-in. The ends of the panels shall overlap the flanges of the soldier pile beams on each end by a minimum of 1-in more than the concrete cover over the concrete reinforcement. If no other facing is added, the panels shall have a 1-in chamfer at the exposed top and bottom.

In some cases where the wall does not face a roadway or public space, the wall may be designed with permanent precast lagging without a facing.

BDM Figure 307-3 details the described wall dimensions.

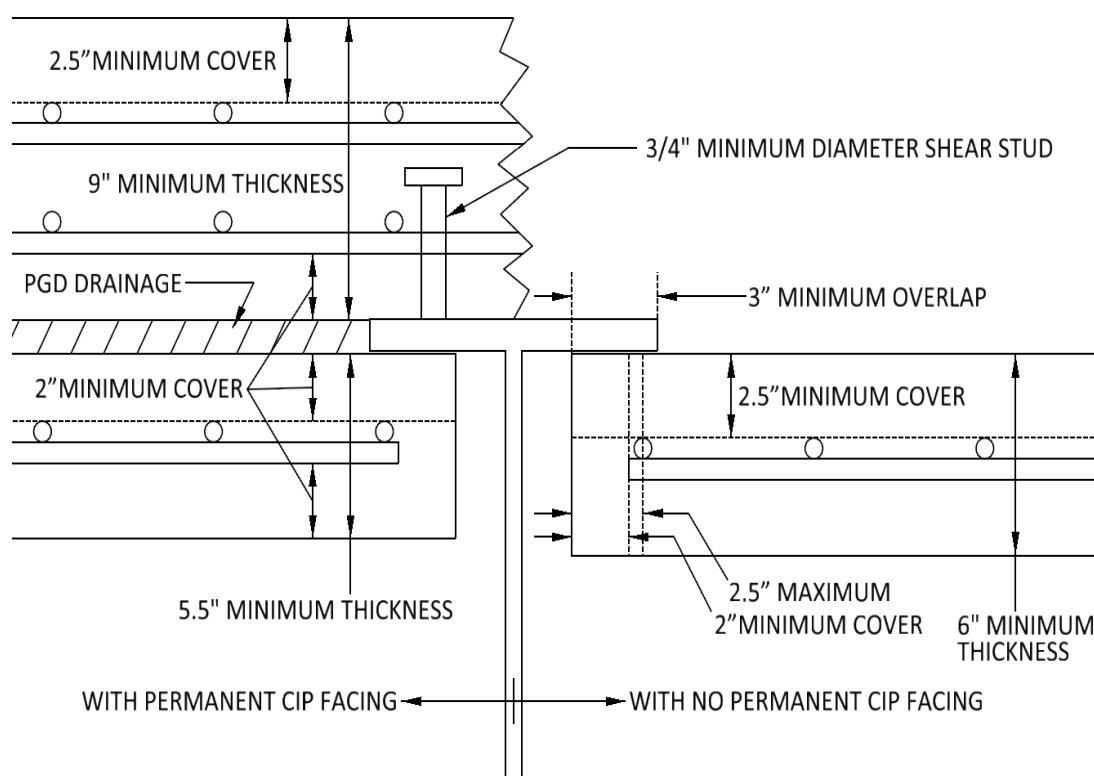


Figure 307-3

Design timber lagging according to *LRFD Section 8* or publication FHWA-RD-75-128, “Lateral Support Systems and Underpinning,” Section 9.32. Timber lagging shall have a minimum thickness of 3-in. The ends of the lagging timbers shall overlap the flanges of the soldier pile beams on each end by a minimum of 2-in.

Provide timber blocks between individual lagging timbers. Provide elastomeric bearing pads between individual precast concrete lagging panels. The timber blocks shall be at least 3-in long, 3-in wide and 3/8-in tall. The elastomeric bearing pads shall be at least 3/8-in tall with a length and width equal to the thickness of the precast concrete lagging panels minus 1-in. Place one timber block or elastomeric bearing pad at each end of each joint between the lagging timbers or precast concrete lagging panels.

If the soldier pile beam section is embedded in a drilled shaft, the width of a discrete vertical wall element (b) shall be equal to the nominal diameter of the drilled shaft. If an embedded soldier pile beam section does not extend to the bottom of the drilled shaft, use a concrete reinforcement cage in the drilled shaft. If the soldier piles are driven rather than placed in a drilled shaft, the width of a discrete vertical wall element (b) shall be equal to the flange width of the soldier pile beam section.

Assume the portion of the soldier pile beam section below the design grade to be continuously braced. Check flexural resistance for this portion in accordance with *LRFD 6.10.8.1.3*.

Assume the portion of the soldier pile beam section above the design grade to be unbraced, with an unbraced length equal to the retained height (H). Check flexural resistance for this portion according to *LRFD 6.10.8.2*. Compare the factored resistance to the maximum factored moment calculated within the unbraced length.

If soldier pile walls do not have a permanent cast-in-place facing, design the soldier pile beams with measures to resist corrosion and deterioration, in accordance with *LRFD 10.7.5* for a service life of 75 years. The corrosion protection shall extend 3-ft below the exposed portion of the beam.

The blocks and bearing pads promote drainage between the lagging timbers or panels. In the case of precast lagging panels, the elastomeric bearing pads also protect the panels from damage during installation and overstress at the corners.

If an embedded soldier pile beam section extends to the bottom of the drilled shaft, it is not necessary to use a concrete reinforcement cage in the drilled shaft; however, if a concrete reinforcement cage is used, see BDM Section 307.6 for additional considerations.

The requirements for corrosion protection do not apply to temporary soldier pile walls.

If the wall has a cast-in-place permanent facing, place wall drainage between the permanent facing and lagging. Provide continuous coverage over the width of the lagging, between the soldier pile beams. The vertical drainage may be composed of either free-draining granular material meeting the requirements of [C&MS 605.02](#), with a minimum thickness of 10-in measured perpendicular to the wall face, fully wrapped in Geotextile Fabric Type A; or may be composed of prefabricated geocomposite drain (PGD) strips. Where PGD is used, provide a detail in the Plans showing the bottom of the PGD terminating at least 6-in into a 2-ft wide section of Porous Backfill with Geotextile Fabric in accordance with [C&MS 518](#). Provide collection pipes and pipe outlets to ensure positive drainage of the Porous Backfill. Do not allow concrete from the facing to intrude into the joints between the lagging panels or timbers.

If the wall has full-height precast concrete facing panels, see the requirements of BDM Section 307.1 regarding drainage. Design the soldier pile beams with measures to resist corrosion and deterioration as above.

307.6.4 LANDSLIDE DRILLED SHAFTS

Refer to [GDM](#) Section 900 for the design methodology for drilled shaft structures for landslide stabilization.

307.7 STEEL SHEET PILE WALLS

C307.7

Design steel sheet pile with ASTM A572, Grade 50 steel, minimum. Specify the steel grade in the general notes. See BDM Section 602.3-1.

For steel sheet pile walls with a cast-in-place concrete facing, provide a structural attachment between the facing and the exposed face of the steel sheet piles. For steel sheet pile walls with full-height precast concrete facing panels, see the requirements of BDM Section 307.1.

Typically, the attachment of the cast-in-place concrete facing is made with welded shear studs.

Design permanent steel sheet pile walls with measures to resist corrosion and deterioration, in accordance with *LRFD 10.7.5* for a service life of 75 years. The corrosion protection shall extend 3-ft below the exposed portion of the beam.

If a steel sheet pile wall has a cast-in-place permanent facing, design drainage according to [C&MS 518](#).

If the wall has full-height precast concrete facing panels, the drainage requirements of BDM Section 307.1 apply.

A steel sheet pile wall does not meet the requirements of an AREMA crash wall unless it has a permanent cast-in-place facing with a thickness of 30-in or greater.

307.7.1 CANTILEVER SHEET PILE WALLS C307.7.1

Design cantilever sheet pile walls in accordance with *LRFD 11.8*. For cantilever sheet pile walls with permanent cast-in-place facing, designate a specific steel sheet pile section or equivalent. For all other cantilever sheet pile walls, specify a minimum section modulus and moment of inertia per unit width of the sheet pile.

Cantilever sheet pile walls are composed of PZ-Section elements, installed as a linear wall, perpendicular to the retained earth. The sheet pile sections in this case retain the soil in cantilever action, primarily through flexural resistance.

Cantilever sheet pile walls are designed on a per-unit-width basis of vertical face. For steel sheet pile walls with permanent cast-in-place facing, the connection details will depend on the width of the specific steel sheet pile section.

The following definitions apply throughout BDM Section 307.7.1 and to *LRFD 3.11.5.6* and *LRFD Figures 3.11.5.6-3, 3.11.5.6-6, and 3.11.5.6-7*:

D = Foundation Depth (depth of steel sheet pile embedment below design grade)

H = Design Retained Height (height of the wall above design grade)

L = Length of steel sheet pile elements = D + H

D/L = Embedment-to-length ratio

Measure the design retained height (H) of cantilever sheet pile walls from the top of the retained earth to the design grade according to *LRFD Figures 3.11.5.6-3, 3.11.5.6-6, and 3.11.5.6-7*. The minimum embedment (D) for cantilever sheet pile walls shall be equal to the retained height (H) such that, the embedment-to-length ratio (D/L) is not less than 0.5.

The design grade is typically taken as either the frost penetration depth, the depth to pavement subgrade located in front of the wall, or the assumed depth of future utility excavations which may occur immediately in front of the wall.

For structural design of cantilever sheet pile walls, in the Strength Limit State, check both the factored flexural resistance and factored shear resistance versus the maximum factored moment and shear per unit width of the wall. In the Service Limit State, check horizontal deflection at the top of the wall face. Achieving fixity at the tip (bottom) of the continuous vertical wall elements is not required, provided the serviceability deflection limit is met at the top; see BDM Section 307.1.6 for details on serviceability deflection limits.

When performing a p-y analysis for a cantilever sheet pile wall, do not analyze the continuous vertical wall elements by cutting them off at the bottom of the retained height, and applying a resultant shear, moment, and axial load at the foundation elevation. Analyze the continuous vertical wall elements by using the theoretical distributed load over the entire retained height to determine the reactions above the foundation elevation.

While “cutting off” the vertical wall elements may be sufficient for analyzing reactions below the foundation elevation, and produces realistic shear and moment distributions below the cut-off point, it does not include the shear and moment above the cut-off point and it does not adequately account for the deflection at the head of the vertical wall elements (the top of the wall).

In *LRFD Figures 3.11.5.6-6* and *3.11.5.6-7*, if $\gamma_s H < 2S_u$, use a value of zero (0) for the active earth pressure in the portion of the wall below the retained height (P_{a2}).

The active earth pressure in the portion of the wall below the retained height (P_{a2}) could result in a negative value if $\gamma_s H - 2S_u < 0$; however, it is not possible to have a negative active earth pressure.

307.7.2 CELLULAR SHEET PILE WALLS

The minimum foundation embedment of cellular sheet pile walls is the larger of 3-ft or the frost depth at the front (exposed) face. It is acceptable for the steel sheet pile PS-Sections to not be embedded below the minimum foundation embedment elevation of the wall. Check the external stability of the sheet pile cells in bearing resistance, sliding resistance, and limiting eccentricity (overturning) at the foundation elevation as gravity walls in accordance with *LRFD 11.6.3*.

For closed sheet pile cells, driving the rear sheet pile sections deeper to provide uplift resistance against overturning or passive resistance against sliding is acceptable. Check resistance versus uplift according to *LRFD 10.7.3.11* for a group of driven steel piles. If providing resistance to sliding, ensure that the steel sheet pile PS-Sections can resist the horizontal load in flexure and shear.

Check the connection in interlock tension between individual PS-Section elements versus the maximum interlock strength between the elements. For semi-circular cells (which are not closed at the back) also check pullout resistance of the cell wall versus the retained earth pressure. Assume the friction angle between the steel sheet pile and soil to be $\delta = 0.33 \phi$ for a soil-steel interface, in accordance with *LRFD C3.11.5.3*. Resistance factors for the failure modes of connection interlock tension and horizontal pullout shall be as follows:

A. Connection Interlock Tension: $\phi_{\text{interlock}} = 0.75$

B. Horizontal Pullout Resistance: $\phi_{\text{pullout}} = 1.00$

307.8 ANCHORED WALLS

Design anchored walls according to *LRFD 11.9*, [SS866](#), and publication [FHWA-IF-99-015](#), Geotechnical Engineering Circular 4 (GEC 4) “Ground Anchors and Anchored Systems.” Other than exceptions given in this section, these walls shall conform to the specifications of BDM Sections 307.6 or 307.7, as applicable.

C307.7.2

Cellular sheet pile walls are composed of rectangular, circular, or semi-circular sheet pile cells with PS-Section elements, containing retained earth, which together act as a gravity wall, similar to a bin wall.

The sheet pile sections in this case do not retain the soil in cantilever action, but merely in tension across the face of the cell. Publication EM 1110-2-2503 provides a useful design reference.

The resistance factors for the failure modes of connection interlock tension and horizontal pullout are calibrated to ASD factors of safety for interlock tension = 2.0 and for pullout resistance = 1.5, assuming the primary LRFD factored load to be EH, horizontal earth pressure, with load factor $\gamma_p = 1.50$.

C307.8

The terms “tieback anchor” and “ground anchor” are considered synonymous.

Tieback anchors are a possible solution to increase resistance to horizontal forces (sliding) for a gravity wall with spread footings, per BDM Section 305.2.1.3. Anchored walls may otherwise be of any of the non-gravity cantilever wall types listed under BDM Sections 307.6 or 307.7 (drilled shaft walls, or steel sheet pile walls), with the addition of ground anchors to limit deflection and resist overturning or structural failure of the vertical wall elements.

Anchored soldier pile walls utilizing laced-together, back-to-back MC or C channel steel sections for the discrete vertical wall elements are not preferred. Lacing two channel sections together requires a large amount of fabrication and is inefficient for weight of steel per flexural resistance for each soldier pile beam. The preferred approach is to use HP or W steel sections with shop-prefabricated “windows” for the placement of the tieback anchors.

Minimum Anchor Length is a consideration for external stability of anchored wall systems, in accordance with FHWA-IF-99-015, GEC 4, Section 5.8.3; in the event that it does not control, then Minimum Anchor Length shall be Minimum Unbonded Length + Minimum Bond Length.

Reference [SS866](#) for all walls utilizing ground anchors. Specify anchor locations, anchor inclination, minimum anchor lengths, minimum unbonded lengths, Factored Design Load (FDL), and lock-off load on the Project Plans. For permanent ground anchors, specify the number of proof, performance, creep, and investigative anchor pullout tests to be performed, and specify an appropriate level of corrosion protection for the anchors. Temporary ground anchors do not require specified testing or corrosion protection.

Ground anchors do not contribute to overall (global) stability unless the anchors develop their pullout resistance beyond the zone of potential soil shear failure.

When an anchored wall is subject to live load surcharge (LS) loading, the horizontal earth pressure load on the wall shall consist of a combination of an Apparent Earth Pressure (AEP) diagram load in accordance with *LRFD* 3.11.5.7 and a live load surcharge load in accordance with *LRFD* 3.11.6.4. For factored loading, separate these two components of load and have the appropriate load factors applied according to *LRFD Tables* 3.4.1-1 and 3.4.1-2.

For anchored walls, a p-y analysis of the foundation is not required. Design the embedment depth below design grade of anchored non-gravity walls according to [GEC 4](#), Section 5.5.

Unless the vertical wall elements are founded in bedrock, include the effect of settlement of vertical wall elements in design of anchored walls according to *LRFD* 11.9.3.1, and [GEC 4](#) Sections 5.6, and 5.11.1.

Both proof and performance tests in [SS866](#) are sometimes called “proof tests” in other publications; investigative anchor pullout tests are sometimes referred to as “verification tests” in other publications.

Overtipping and serviceability lateral deflection of the foundation are not considered valid failure modes for an anchored wall.

Settlement can cause reduction of anchor loads. There is additional risk associated in placing a bridge abutment on top of an anchored wall, as the imposition of the bridge loads after construction of the wall can cause vertical displacement and relaxation of the anchors.

307.9 SOIL NAIL WALLS**C307.9**

Design soil nail walls in accordance with *LRFD 11.12*.

Specify soil nail locations, soil nail inclination, minimum soil nail lengths, and Factored Design Load (FDL) on the Project Plans.

In which:

$$\text{FDL} = \gamma_p T_{\text{maxsn}} \leq R_{po} \phi_{po}$$

Where:

γ_p = maximum load factor for vertical earth pressure EV from *LRFD Table 3.4.1-2* = 1.35

T_{maxsn} = maximum tensile force in the soil nail (kips)

R_{po} = nominal pullout resistance of the soil nail (kips)

ϕ_{po} = resistance factor for soil nail pullout = 0.65

Specify a minimum of one verification test for each design soil nail length and design soil stratum in which the soil nails are to be installed. Show verification test locations in the plans.

The Verification Test Load (VTL) is equal to the nominal pullout resistance of the soil nail, per [GEC 7 Section 9.4.3](#).

Specify proof tests on a minimum of five percent of the production soil nails in each row, or a minimum of one per row.

The Proof Test Load (PTL) is equal to 75 percent of the nominal pullout resistance of the soil nail, per [GEC 7 Section 9.4.4](#).

Provide vertical drainage consisting of prefabricated geocomposite drain (PGD) strips, placed against the cut soil face, behind the initial (shotcrete) face. Provide continuous coverage over the width of the wall face, between the nail locations. Provide a detail in the Plans showing the bottom of the PGD terminating at least 6-in into a 2-ft wide section of Porous Backfill with Geotextile Fabric in accordance with [C&MS 518](#). Provide collection pipes and pipe outlets to ensure positive drainage of the Porous Backfill.

Include the Special Provision for soil nail walls in the plans.

The Special Provision for soil nail walls is available from the Office of Geotechnical Engineering.

307.10 TEMPORARY WALLS**C307.10**

Temporary walls shall have a design service life of no more than three years. If a retaining wall must serve more than three years, perform the design as for a permanent wall. Steel components of temporary walls do not require corrosion protection, including concrete reinforcement in reinforced concrete components. Otherwise, perform all aspects of the design for temporary walls the same as for permanent walls of the same type.

Temporary walls can be of the same type as any permanent retaining wall. Special retaining wall types that are only used for temporary applications include wire faced MSE walls, fabric wrapped walls, and temporary shoring (sheet pile temporary support of excavations).

307.10.1 WIRE FACED MSE WALLS

Design wire faced MSE walls in accordance with [SS867](#). Perform the design for external stability as an MSE wall according to BDM Section 307.4.

307.10.2 FABRIC WRAPPED WALLS

Design fabric wrapped walls as reinforced soil slopes in accordance with publications [FHWA-NHI-10-024](#) and [FHWA-NHI-10-025](#), Geotechnical Engineering Circular 11 (GEC 11); and [SS863](#) except that geosynthetics (geotextile sheets) comprise both the soil reinforcement and the face of the wall. Geotextiles used for fabric wrapped walls shall conform to Class 1 geotextile as specified in AASHTO M 288. Determine Ultimate Tensile Strength (strength per unit width) of the geotextile reinforcement from wide strip tests in accordance with ASTM D4595.

307.10.3 SUPPORT OF EXCAVATIONS**307.10.3.1 ESTIMATED QUANTITIES**

Provide a pay item for Cofferdams and Excavation Bracing when either of the following conditions exist:

- A. Excavation that extends below the ground water table or below an elevation defined as 3-ft above the OHWM (see BDM Section 201.3.1 for more information).
- B. Excavation of earth supporting:
 - 1. Structures/utilities when the structure/utility is located within 1-1/2 times the depth of excavation. Consider the location of bridge substructures supported on shallow foundations but not substructures supported on deep foundations unless the excavation will expose the deep foundation members.
 - 2. Railroads when the excavation encroaches on foundation material defined by each railroad.
 - 3. Roadways used to maintain traffic when the edge line is located within a distance of one-half times the depth of excavation.

When a pay item for Cofferdams and Excavation Bracing is required for (B) above, show the approximate locations of the Excavation Bracing in the Plans.

307.10.3.2 EXCAVATION BRACING PLAN WARRANTS

Provide a complete design for excavation bracing in the Plans for each of the following conditions:

C307.10.1

For a permanent application of this type of wall, galvanize the wire facing units in accordance with [SS867.03.A](#), and provide a permanent cast-in-place or precast concrete facing.

C307.10.2

Fabric wrapped walls are also known as geotextile faced MSE walls. The face of a fabric wrapped wall is typically steeper than 1H:1V.

- A. BDM Section 307.10.3.1.B.1
- B. BDM Section 307.10.3.1.B.2
- C. BDM Section 307.10.3.1.B.3
- D. The depth of any side of the excavation exceeds 8-ft

Consider the feasibility of this excavation bracing during the Structure Type Study.

307.10.3.3 DESIGN REQUIREMENTS

C307.10.3.3

When warranted according to BDM Section 307.10.3.2, design the excavation bracing in accordance with the latest *AASHTO Guide Design Specifications for Bridge Temporary Works* and the latest edition of either the *AASHTO LRFD Bridge Design Specifications* or the *AASHTO Standard Specifications for Highway Bridges*.

The design methodology may be in accordance with either Load and Resistance Factor Design or Allowable Stress Design.

For projects involving railroads, contact the responsible railroad and obtain the specific requirements for design and construction.

The requirements will be different as each railroad company has their own specific requirements.

Specify sheet piling per C&MS 711.03. If driving sheet piling within a distance equal to 1-½ times the depth of excavation from a wire faced MSE wall, then specify Item 504 Steel Sheet Piling Left In Place. If feasible with the project phasing, specify that the sheet piling be driven before the construction of the wire faced MSE wall.

It has been found that vibration from driving or extraction of sheet pile will unacceptably deform nearby wire faced MSE walls.

307.10.3.4 DESIGN CONSIDERATIONS

C307.10.3.4

For sheet pile temporary support of excavations, specify in the project plans the minimum length of the sheet pile sections and minimum section modulus per unit width of the sheet pile. Only specify a minimum moment of inertia per unit width of the sheet pile if the design is deflection critical.

Following are some conceptual ideas for the design of Excavation Bracing:

Provide a full detailed design in the project plans for all other aspects of a temporary support of excavation system, including walers, tiebacks, soldier piles, lagging, etc. Provide plan notes and detail drawings necessary to specify all aspects of the design; also provide Plan Note in BDM Section 611.3-1.

- A. A cantilever sheet pile wall should generally be used for excavation up to approximately 12-ft in height.
- B. For cuts greater than 12-ft in height, anchored or braced walls will generally be required.
- C. Braced walls using waler and struts can sometimes be braced against another rigid element on the excavated side.
- 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.

308 SUPERSTRUCTURE**308.1 SPAN ARRANGEMENTS****C308.1**

The length of a bridge will be determined by the requirements for horizontal clearance at grade (highway or railway) separations, by the requirements for waterway opening at stream crossings or the presence of existing utilities and/or infrastructure. 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).

Where permitted by the ROW limits, the substructure units should be placed outside of the clear zone. Refer to [L&D, Vol. 1](#) for the definition of the clear zone.

For waterway crossings, an effort should be made to keep the substructure footings outside of the limits of the OHWM.

The number and/or type of substructure units should be based on a cost analysis. 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. Refer to BDM Section 200 for additional cost analysis considerations.

308.2 SUPERSTRUCTURE TYPES**C308.2**

Select the type of superstructure used on the basis of safety, economy and appearance.

The types of superstructure generally used in Ohio consist of cast-in-place concrete slabs, prestressed concrete box or I-beams, and steel beams or welded plate girders. Since these superstructure types make up a large majority of the bridges in the inventory for the state of Ohio, these are the only structure types covered in this manual. For special conditions where other types of superstructures are being considered, consult the Office of Structural Engineering for recommendations and requirements prior to initiating the design.

For bridges with significant substructure costs, the difference in dead loads between the steel superstructure versus a concrete superstructure should be considered in the Structure Type Study, Cost Analysis for choosing the most economical structure type.

308.2.1 CONCRETE SLAB BRIDGES

Design continuous reinforced concrete slab bridges in accordance with *LRFD 4.6.2.3*.

For simple span reinforced concrete slab bridges cast in place directly on concrete substructures, consider the effective span length equal to the clear span plus 15-in.

Include a final deck surface elevation table in the plans. Show elevations for all profile grade lines, curblines, crown lines, and phased construction lines where they intersect bearing points, quarter-span points and mid-span points as well as any additional points required to meet a maximum spacing between points of 25-ft.

308.2.1.1 CS-1-24

For a continuous concrete slab bridge, per [CS-1-24](#), 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. Design the bearing system for the temperature movements described in BDM Section 1003.21 and bearing design requirements of *LRFD 14.6*.

If the slab is rigidly connected to rigid piers, custom design the slab/substructure. The details in [CS-1-24](#) are not applicable.

308.2.2 STRUCTURAL STEEL

308.2.2.1 PRELIMINARY CONSIDERATIONS & GENERAL INFORMATION

Use continuous spans for multiple span steel beam and girder bridges.

C308.2.1.1

The details shown in bridge standard drawing [CS-1-24](#) were designed to function with details shown in bridge standard drawings [CPA-1-08](#) and [CPP-1-08](#). These details were setup to allow the slab to behave as a beam rather than a frame.

When the details provided in [CS-1-24](#) are being used in conjunction with a substructure unit(s) that differ from what is shown in bridge standard drawings [CPA-1-08](#) and [CPP-1-08](#), the design will need to provide suitable bearing condition to allow the beam type behavior.

This modification creates a semi-integral type abutment for the slab bridge and maintains the beam type behavior with the slab.

C308.2.2.1

For spans greater than 60-ft, rolled beams, up to and including the 40-in depth, or welded plate girders should be considered.

The ratio of the length of the end spans to the intermediate spans usually should be 0.7 to 0.8. The latter ratio is preferred because it nearly equalizes the maximum positive moment of all spans. Integrally designed structures may have end span ratios of as low as 0.6 if prevention of uplift is considered. For multi-span, composite designed, rolled beams, the maximum intermediate span is generally around 120-ft. For single span, composite designed, rolled beams, the maximum span is generally around 100-ft.

While constant depth plate girders can be used in the same range as rolled beams, they are generally not as cost effective as rolled beams for the same span lengths. Consider haunching girders over the intermediate substructure units for spans greater than 350-ft. For additional information on optimizing steel structure types refer to the NSBA Steel Design Handbook.

Stringer type	Span length
Rolled beam	up to 100-ft (Simple) 120-ft (Continuous)
Constant Depth Girder	120-ft – 350-ft
Haunched Girder	> 350-ft

The minimum economical beam spacing for rolled beams is approximately 8-ft. For plate girders a minimum spacing of 9-ft is generally recommended. Girder spacings between 11-ft and 14-ft typically yield the most efficient design for the section.

Steel rolled beam or girder highway bridges shall have a minimum of 4 stringer lines.

Utilize a composite section when designing structural steel.

Do not design deck slab overhangs to exceed 4-ft measured from the centerline of beam/girder to deck edge.

On over-the-side drainage structures, the minimum overhang shall be lesser of 75% of the beam/girder depth or 4-ft.

For designs that assume the unbraced length of the top flange to be zero, investigate the strength of the non-composite section during steel erection, deck slab construction, etc. using laterally unsupported lengths that reflect actual bracing conditions.

308.2.2.1.a ATTACHMENTS

Do not weld attachments, either permanent or temporary in tension areas. Limit welded attachments to compression areas.

Show the extent of the compression and tension areas in the detail plans.

Show in the detail plans of steel beam and girder bridges where welded attachments are permitted for construction purposes.

Do not weld scuppers, down spouts or drainage supports in tension areas of primary members.

Refer to *LRFD 6.10.10.1* and BDM Section 1006 for more information.

This limit is dependent on the maximum capacity of the overhang brackets contractors use to form the deck slab overhang.

This requirement helps to reduce corrosion resulting from the surface drainage.

C308.2.2.1.a

Base the extents of the compression and tension areas on the Strength I load case.

**308.2.2.1.b STEEL FABRICATION
QUALIFICATION**

Specify the required steel fabricator classification in the pay item for the structural steel.

C308.2.2.1.b

The Department's requirements for steel fabricators are defined in [C&MS](#) 513 and S1078. Steel fabricators are classified according to their capabilities into eight levels (1 thru 6, SF & UF). Levels 1 thru 6 require certification according to the American Institute of Steel Construction (AISC). No AISC certification is required for Levels SF and UF.

The AISC categories of certification are listed here for information:

AISC Category: Simple (SBR) – Qualified for unspliced rolled sections.

AISC Category: Intermediate (IBR) – Qualified for typical bridges that do not require extraordinary measures.

AISC Category: Advanced (ABR) – Qualified for bridges requiring an additional standard of care in fabrication or erection.

AISC has also established three optional endorsement categories for Complex Coatings (CCE), as well as an optional Fracture Control Endorsement (FCE).

**308.2.2.1.c MAXIMUM AVAILABLE LENGTH
OF STEEL MEMBER**

Consider reducing cost of structural steel members by providing field splices and allowing for optional field splices.

C308.2.2.1.c

Length of a girder is generally limited by the ability to transport the member from the fabricator's shop to the job site. A length of 120-ft is generally the maximum trucking length between splices, but girder lengths of 160-ft and greater have been transported to project sites. When the trucking lengths exceed 110-ft contact local fabricators for NSBA for maximum shipping lengths.

Mills can supply lengths up to maximum shipping limits, but extra charges may be added for lengths over 80-ft.

The National Steel Bridge Alliance (NSBA) and the American Iron and Steel Institute (AISI) are available to provide assistance with material sizes.

308.2.2.1.d STRUCTURAL STEEL COATINGS**C308.2.2.1.d**

ASTM A709 50W/70W should be selected wherever applicable as it eliminates the need for a coating system on the entire beam and the maintenance associated with a coating system. See BDM Section 308.2.2.1.d.1 for additional information.

Specify an alternative coating system per BDM Section 308.2.2.1.d.2 if a structure requires the steel to be ASTM A709 Grade 50.

For more information on steel materials, see BDM Section 304.1.

308.2.2.1.d.1 PRIMARY COATING SYSTEMS

C308.2.2.1.d.1

The Department's primary system is un-coated weathering steel. Weathering steel reduces the initial cost of the structure by approximately the cost of a three-coat paint system on the structure. It may also eliminate future maintenance coatings.

In suitable environments, uncoated weathering steel develops a thin oxidation layer, also called a surface patina, that is small grained (dense) and tightly adherent to the base metal. Once this oxidation layer builds to a sufficient thickness and density the exposure of the base metal to moisture and oxygen is blocked and further corrosion is greatly reduced.

Do not use uncoated weathering steel for bridges over roadways that meet **ALL** of the following criteria:

- A. Vertical clearance is 20-ft or less
- B. AADT is 50,000 or larger
- C. ADTT is 20% or more

Do not use uncoated weathering steel when the low chord is 10-ft or less above the OHWM.

Proximity to industrial areas where concentrated chemical fumes may drift directly onto the bridge should be considered in the site-specific study.

Specify an alternative coating system when a site-specific study finds that un-coated weathering steel is not suitable.

Tunnel-like conditions should be considered in the site-specific study. These conditions can result in the protective patina not developing correctly due to increased moisture or deicing salt exposure and poor air circulation resulting in the steel deteriorating at a faster rate.

When weathering steel is specified for a site meeting the requirements for tunnel like conditions, apply a protective coating, per BDM 308.2.2.1.d.2, to the fascia beams/girders for the entire length of the structure. Refer to BDM Figure 308-1 for the section coating limits.

Tunnel-like conditions are defined as bridges over roadways that meet **ALL** of the following criteria:

- A. Vertical clearance is 20-ft or less
- B. Bridges over interstates or four lane divided highways
- C. ADTT = 10% or more under the bridge
- D. Posted speed limit under the bridge is 55 mph or greater

Additional resources available to aid in the site studies include:

- A. FHWA Technical Advisory T5140.22, October 1989.
- B. FHWA Maintenance Coating of Weathering Steel FHWA-RD-92-055, March 1992.

- C. NCHRP 314, 1989
- D. AISC Uncoated Weathering Steel Bridges, Vol. 1, Chap. 9
- E. Ohio Department of Transportation, unpublished internal study, September 2000.
- F. Texas Department of Transportation, Research report 1818-1, May 2000.
- G. Missouri Department of Transportation, Task Force Report on Weathering Steel

If the requirements of tunnel-like conditions are not met, aesthetics may still dictate that a protective coating system be applied to the entire length of fascia beams over traffic.

Apply a protective coating, per BDM 308.2.2.1.d.2, to all exposed weathering steel surfaces within 10-ft of the beam/girder end at abutments with expansion joints and 10-ft on both sides (20-ft total) at intermediate expansion joints. Also coat all cross-frames, end frames or other steel in these 10-ft sections.

Coat all surfaces at the ends of un-coated weathering steel bridges on integral or semi-integral abutments with an inorganic zinc prime coat to 1-ft past the end diaphragm. Show limits of zinc prime coat in the plans.

Apply a protective coating, per BDM 308.2.2.1.d.2, to weathering steel fascia beams/girders along their entire length for bridges with over-the-side drainage. Refer to BDM Figure 308-1 for the section coating limits.

Apply a protective coating, per BDM Section 308.2.2.1.d.2, to weathering steel fascia beams/girders along their entire length when the width of the gap between adjacent structures is less than 30-ft. Refer to BDM Figure 308-1 for the section coating limits.

When a three-coat paint system is applied to the splice location, as shown in BDM Figure 308-1, use galvanized bolts at these splice locations.

When applying a protective coating, per BDM 308.2.2.1.d.2, to only a portion of the beam, tint the top coat to a color closely matching AMS Standard 595A 20045 or 20059, the color of weathering steel.

De-icing salts are able to get between the structures and due to inadequate sun exposure and air circulation the fascia beams/girders stay wet for prolonged periods and are prone inadequate protective patina development and more rapid corrosion.

In lieu of the three-coat system limits shown in BDM Figure 308-1 at the splice location, the owner may elect to apply a three-coat system at the same limits as the rest of the beam/girder. In this case, apply a shop applied zinc prime coat to all surfaces of the splice plates, filler plates, and faying surface of the beam/girder.

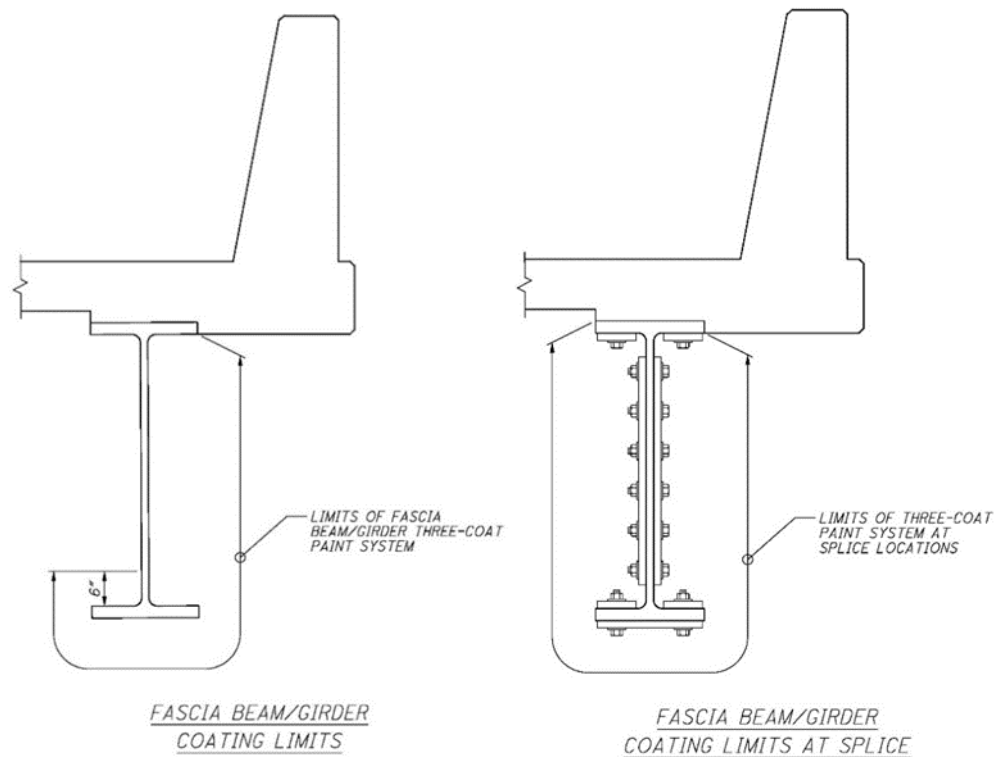


Figure 308-1

308.2.2.1.d.2 ALTERNATIVE COATING SYSTEMS

Where the primary coating system is not applicable, coat the members of new steel structures with one of the following alternative coating systems:

- A. Three-coat paint system
- B. Hot-dipped galvanized coating
- C. Shop or field metallizing

The three-coat paint system shall consist of an inorganic zinc prime coat, an epoxy intermediate coat and a urethane finish coat (IZEU) in accordance with [C&MS 514](#).

C308.2.2.1.d.2

This system has proven to have a life span of up to 30 years.

Typically, the inorganic prime coat is shop applied while the intermediate and top coats are field applied.

An IZEU three-coat shop applied system with field touch up may be the system of choice where:

- A. There is limited access to the superstructure in the field. Examples may include stream crossings with shallow clearances.
- B. The environment is especially sensitive to possible construction debris.
- C. The bridge is located in a highly urbanized area that may have limited access for future coatings.

When a galvanized coating system is specified, contact the American Galvanizers Association to confirm the structural members detailed can be galvanized in a regional plant.

Refer for BDM Section 308.2.2.1.j for bolt hole sizes for bolted connections of galvanized members.

Do not field weld end cross-frames and intermediate cross-frames on galvanized or metallized structures.

When welding on galvanized member is required, repair the galvanizing per [C&MS](#) 711.02.

Shop or field metallizing shall be in accordance with [SS845](#).

The shop applied IZEU three-coat system may provide a better protective coating than the standard field applied system due to the fabricator's automated blasting processes, environmental controls and better coating application access. However, the total cost of the shop-applied system is higher than the standard field applied system. Extra costs include special care needed during shipping and erection, field painting of field splices, field touch up, final cleaning and possible time delays to the project due to the additional shop work. Additionally, all field connections are to be bolted to minimize the damage to the coating by field welding.

If a shop applied three-coat system is selected, refer to [C&MS](#) 514.16 for additional information.

A galvanized coating system is an alternative for new steel structures. Galvanized systems are proven durable up to 40 years.

Galvanizing tanks are relatively shallow and normally not longer than 45-ft in length. Therefore, beam lengths should not be longer than 60-ft. Additional information on regional galvanizer size restrictions is available from the American Galvanizers Association.

Bolted cross-frames will be required. Bolted cross-frames as detailed in the Standard Bridge Drawing [GSD-1-19](#) may be specified.

Welding onto galvanizing causes damage to the coating and should be avoided wherever possible.

Required plan or proposal notes for the galvanized coating system are available upon request with the Office of Structural Engineering.

A shop applied metallizing system is another alternative coating system, but the costs are relatively high compared to the standard IZEU system.

Metallized systems have an expected life similar to galvanizing. Metallized coating systems should have field bolted connections rather than field welded connections, but oversized holes are not required. Metallizing, as compared to galvanizing, has no limit on the size of members being coated and causes no additional distortion from heat.

When un-coated weathering steel is not suitable for a bridge of a railroad, galvanized or metalized coating systems should be considered. Any structural alternative with any type of paint system, including coated weathering steel, should be avoided in these locations because it may not be possible to get access the structure for future maintenance.

308.2.2.1.e OUTSIDE MEMBER CONSIDERATIONS

Evaluate the actual loads for outside primary members.

C308.2.2.1.e

Heavy sidewalks, large overhangs of the concrete deck slab and/or live loads may cause higher loads on an outside member than loads on an internal member.

This analysis requirement does not alleviate the designer from conforming to *LRFD 4.6.2.2.1* (i.e. "...exterior girders of multibeam bridges shall not have less resistance than an interior beam.).

308.2.2.1.f CAMBER & DEFLECTIONS

Do not include the loading for future wearing surfaces in determining required shop camber.

C308.2.2.1.f

When establishing dead load deflection for determining the required shop camber of composite beam or girder bridges with concrete deck slabs and determining deck screed elevations, the weight of curbs, metal railings, concrete railings and separate wearing surface may be equally distributed to all beams.

In the deflection and camber table in the design plans, detail all points for each beam or girder line for the full length of the bridge. Detail bearing points, quarter-span points, mid-span points and splice points and any additional points required to meet a maximum spacing between points of 25-ft.

In cases of special geometry (e.g. spirals, horizontal or vertical curves, superelevation transitions, etc.), provide additional points in the deflection and camber table if the normally required points do not adequately define the required curvature of a beam or girder.

In all cases, calculate the required shop camber as the algebraic sum of the computed deflections, vertical curve adjustment and horizontal curve adjustment. Measure camber to a chord between adjacent bearing points.

Provide a camber diagram showing the location of the points defined above and giving vertical offset dimensions at the bearing points from a "Base" or "Work" line between abutment bearings.

308.2.2.1.g FATIGUE DETAIL CATEGORY

In order to allow for future rehabilitation involving welded attachments less than 2.0-in., the fatigue limit state loading for steel members in the tensile stress and /or stress reversal regions shall not exceed the nominal fatigue resistance for Detail Category C.

308.2.2.1.h TOUGHNESS TESTS

On steel structures, main load carrying members (primary members such as beams, moment plates, bolted joint splice plates excluding fill plates, etc.) require Charpy V Notch Testing. Identify these components on the detail plans by placing “(CVN)” after the component's description.

Example: W36x150 (CVN)

Designate the web and all flanges of plate girders as a CVN material.

Cross-frame members, cross-frame connection stiffeners and gusset plates on horizontally curved beam or girder structures are primary members. Identify these items as CVN on the detail plans.

C308.2.2.1.h

Bracing members are considered primary members in curved bridges since they transmit forces necessary to provide equilibrium.

308.2.2.1.i STANDARD END CROSS-FRAMES

Support expansion devices with end cross-frames. If end cross-frame details shown in the standards for the expansion device are not used, design the end cross-frame to limit the maximum deflection in the expansion device to $L/2000$.

C308.2.2.1.i

End cross-frames are needed for support and reduction of deflection of expansion device. Standard expansion joints have designs already established as part of the standard drawings.

308.2.2.1.j BOLTED SPLICES

For bolted connections of galvanized structures using bolts 7/8" diameter or less, detail the diameter of the bolt holes to be 1/8-in larger than the bolt diameter. For bolted connections of galvanized structures using bolts 1" diameter or greater, detail the diameter of the bolt holes to be 3/16-in larger than the bolt diameter.

C308.2.2.1.j

Since standard holes may become partially filled with galvanizing, bolted splice designs will require a non-standard hole size. This will also decrease the splice capacity.

Design allowable bolt stresses for painted surfaces based on AASHTO 's values for Class A, Contact Surface, Standard Hole Type.

Design allowable bolt stresses for unpainted weathering steel surfaces based on AASHTO 's values for Class B, Contact Surface, Standard Hole Type.

Design allowable bolt stresses for metallized surfaces based on AASHTO's values for Class B, Contact Surface, Standard Hole Type.

Design allowable bolt stresses for galvanized surfaces based on AASHTO's values for Class C, Contact Surface, Oversized Hole Type.

For splices at bend points, place the lines of holes in the beam or girder flanges parallel to the centerline of the web. If the bend angle is small enough use rectangular splice plates (splice plates shall not overhang flange by more than 1/2-in and inside splice plates shall not have to be trimmed to clear web or web to flange radius). When the angle is too large to allow rectangular splice plates, trim the plates to align with the flange edges. Meet the minimum edge distances defined in BDM Section 308.2.2.1.j.2 for both cases.

For bearing type compression splices, such as in a column, mill the ends of the spliced members to be in full bearing. For compression splice members with milled ends, meet the requirements of *LRFD 6.13.6.1.2*. Specify the fit requirements of the mating surfaces in the plans.

The designer should recognize that “FULL BEARING” of beams and girders is not defined by AASHTO. “FULL BEARING” is defined for this application as 75 percent of the bearing surface in contact and the other 25 percent with no gap greater than 1/32-in.

Alternatively, a slip-critical type connection can be used to avoid milling the beam ends.

Refer to BDM Figure 308-2 for additional bolted splice details.

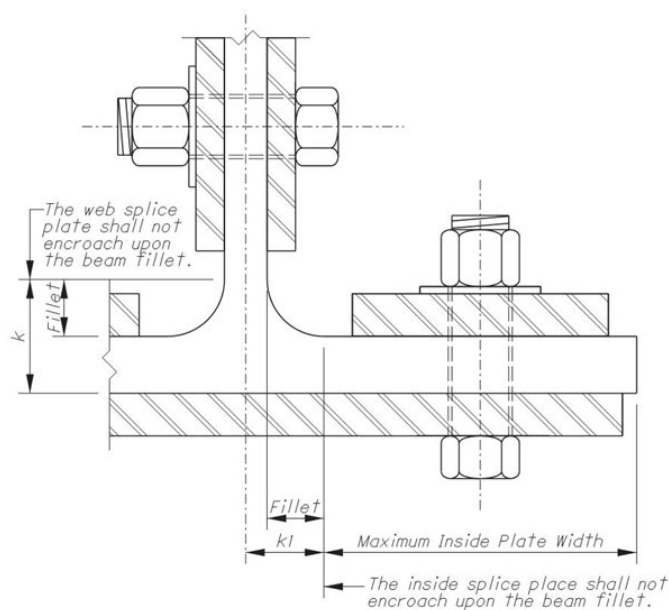


Figure 308-2

308.2.2.1.j.1 BOLTS

Bolt field splices in beams and girders using high strength bolts, ASTM F3125, Grade A325.

Specify the diameter of the bolts and provide the ASTM F3125, Grade A325 bolt type in the coating notes or bolt material specifications.

Use Type I bolts for structures with coating systems that are zinc based, such as OZEU, IZEU, galvanizing or metallizing.

Use Grade A325, Type III bolts for un-coated weathering steel. If the faying surfaces under both the head and nut of every bolt of a weathering steel member are coated, specify galvanized Grade A325 Type I bolts. Otherwise, specify Grade A325, Type III bolts.

308.2.2.1.j.2 EDGE DISTANCE

For design and detailing purposes, increase the minimum edge distances listed in *LRFD Table 6.13.2.6.6-1* by 1/4-in.

C308.2.2.1.j.2

The edge distances provided in *LRFD 6.13.2.6.6* are absolute minimums allowed during fabrication. This increase will allow for fabrication tolerances when drilling bolt holes in splice plates, especially the inside flange splice plates.

308.2.2.1.j.3 LOCATION OF FIELD SPLICES

C308.2.2.1.j.3

Bolted splices should be located at points of dead load contraflexure on a continuous span structure. Splices may also be supplied to help meet shipping and handling limitations. Plans should show optional field splice locations.

308.2.2.1.k INTERMEDIATE EXPANSION DEVICES

When required, locate intermediate expansion devices for a structure, over a pier and design the structural members to be discontinuous at that pier.

308.2.2.1.l SHEAR CONNECTORS

C308.2.2.1.l

Design shear connectors in accordance with *LRFD 6.10.10*.

Use automatic end-welded type shear connectors. Do not use channel sections for shear connectors.

The Department recommends 7/8-in diameter studs as a standard diameter. Check for availability with manufacturers regarding the length of stud specified in the plans.

Detail the shear studs so that the requirements of *LRFD 6.10.10.1.4* are met.

The Department's policy of using a 2-in deep haunch over the top flange will have an effect on the length of shear studs. For deeper haunches refer to BDM Section 403.5.2 for haunch reinforcement requirements.

Field install shear studs. In the case of galvanized structures, install the shear studs per [C&MS 513.22](#).

308.2.2.2 ROLLED BEAMS

For structures with curved rolled beams that do not meet the criteria of *LRFD 4.6.1.2.4b*, design the beams in accordance with BDM Section 308.2.2.3.g.

308.2.2.2.a INTERMEDIATE STIFFENERS & CONNECTION PLATES

Do not use intermediate stiffeners for rolled beams. Size connection plates to be a minimum thickness of 3/8-in and wide enough to make an adequate cross-frame connection.

Clip corners of connection plates in contact with both web and flange. Clip the stiffener 1-in horizontally and 2.5-in vertically.

Fillet weld both sides of the connection plate to the beam web and both flanges.

308.2.2.2.b INTERMEDIATE CROSS-FRAMES & DIAPHRAGMS

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

- A₁. Do not exceed 25-ft for cross-frame spacings between points of dead load contraflexure in the positive moment regions.
- B₁. Do not exceed 15-ft for cross-frame spacings between points of dead load contraflexure in the negative moment regions.
- C₁. Connect cross-frames for rolled beams directly to either connection plates or gusset plates that are bolted to connection plates.
- D₁. When the nominal depth of the beam is less than 36-in, use structural steel channel diaphragms rather than cross-frames.
- E₁. Place cross-frames perpendicular to stringers. This does not apply to skewed pier diaphragms.

For structures with flared stringers, the following exceptions apply:

- A₂. If the differential angle between individual stringers is 5-degrees or less, place the cross-frames perpendicular to one stringer and in line across the total width of the structure.
- B₂. If the differential angle between individual stringers is greater than 5-degrees, divide the differential angle evenly between connections to both stringers.

Show the following in the design plans:

- A₃. The cross-frame spacing for each region along the length of the stringer.

C308.2.2.2.a

Intermediate transverse stiffeners are defined as plates used to stiffening the web of the beam and are not connected to the cross-frames. Connection plates are defined as plates used to connect cross-frames to beams.

When determining the width of the connection plates, consider the ease of access for the cross-frame connections. Connection plates should not extend beyond the edge of flange.

Connection plate clip details are shown on the Standard Bridge Drawing [GSD-1-19](#).

C308.2.2.2.b

See the General Steel Details Standard Bridge Drawing, [GSD-1-19](#), for standard cross-frame configurations.

For additional information on cross-frame layout refer to *LRFD 6.7.4.2* and Skewed and Curved Steel I-Girder Bridge Fit by NSBA.

When the nominal depth of the rolled beam is 36-in, either channel diaphragms or cross-frames may be used.

B₃. The typical cross-frame details or reference to the General Steel Details Standard Bridge Drawing, [GSD-1-19](#), for standard cross-frame configurations.

Ensure cross-frame locations do not conflict with bolted splices or provide appropriate details to attach cross-frames at bolted splice locations.

In accordance with [C&MS](#) 513.26, the fabricator shall detail all cross-frames to fit with the webs plumb at the completion of superstructure erection. This condition applies to all cross-frame/diaphragm bays without closure pours.

For steel superstructures constructed under phased construction, do not permanently attach cross-frames/diaphragms between phases until the deck placement on each side of the closure pour is complete.

In bays with closure pours, provide a plan note requiring the fabricator to detail cross-frames to fit at the completion of deck placement on each side of the closure pour.

This condition is referred to as Steel Dead Load Fit (SDLF) in the National Steel Bridge Alliance (NSBA) document, Skewed and Curved Steel I-Girder Bridge Fit.

Refer to BDM Section 309.3.8.5 closure pour requirements.

Standard Bridge Drawing, [GSD-1-19](#), provides oversized holes in the cross-frame/diaphragm members and standard sized holes in the connection plates. This combination provides the following center hole alignment tolerance for bolt fit between the cross-frame/diaphragm member and connection plates:

A₁. Intermediate X-frame: $\pm 1/8$ -in

B₁. Diaphragm: $\pm 3/16$ -in

If the cross-frames are sized to fit prior to deck placement, and the deflections due to deck placement on each side of the closure pour differ by more than these alignment tolerances, then the connection bolts will not fit. The use of standard sized holes commonly specified in non-standard cross-frame configurations reduces these fit tolerances even more.

This differing fit condition for closure bay cross-frames will require the plans to include the following deflection values in the camber and deflection table:

A₂. Deflection due to weight of steel

B₂. Deflection due to staged deck placement including weight of portable concrete barrier

C₂. Deflection due to remaining dead load

Fabricating the bracing members to fit after the deck placements on each side of the closure pour will help with bolt alignment in the field, but the deflection values specified may not be sufficiently accurate to avoid conflicts. One of the following conditions is also recommended:

- A₃. For bracing systems utilizing gusset plates, specify all cross-frame-to-gusset plate connections to be field welded on one of the beam/girder lines and shop welded on the other.
- B₃. For bracing systems without gusset plates, specify all bolt holes in the connection plate on one of the beam/girder lines to be field drilled and shop drilled on the other.

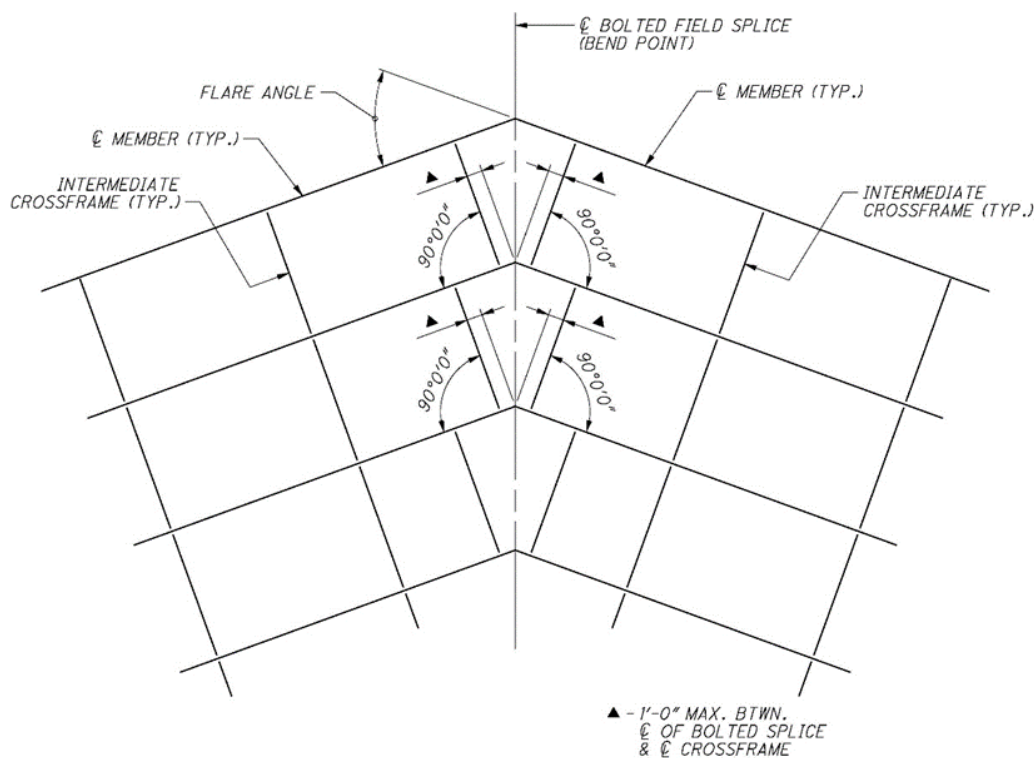
For bridges with “dog-legged” stringers, install cross-frames no greater than 12-in from the bend points. Place cross-frame normal to the stringer used to set the 1-ft clearance dimension and connect to the adjacent stringer only on the same side of the centerline of the splice. Place an additional horizontal angle near the top flange of the stringers.

The cross-frame units should be similar to standard cross-frames.

See BDM Figure 308-3 for plan view layout of cross-frames for dog-legged stringers.

When standard cross-frames with a top strut are used for curved stringers, confirm that the cross-frames and their connections meet the additional loading developed in a curved member design. If specially designed cross-frames are used, bolt the cross-frames to stiffeners with oversized holes. Include the reduction in allowable capacity associated with standard bolt holes in the design. Refer to BDM Section 308.2.2.1.h for material toughness requirements.

Cross-frames for curved stringers may be one of the types shown on the Standard Bridge Drawing, GSD -1-19, with an additional top strut. However, the use of oversized holes as specified in GSD-1-19 are not permitted for curved structures.



PLAN VIEW
CROSSFRAMES FOR
DOG-LEGGED SPLICES

Figure 308-3

308.2.2.2.c WELDS

Specify fillet weld leg size required, in the case of fillet welds, or CP (complete joint penetration) in the case of full penetration groove welds. Do not select the joint configuration to be used for a full penetration weld. This is left to the fabricator and the welding code.

308.2.2.2.c.1 MINIMUM SIZE OF FILLET WELD

Design fillet welds for required stresses but also meet the following size requirements:

- A. Minimum size of fillet weld is based on the thickness of the thicker steel section in the weld joint. AWS D1.5 defines the minimum size of fillet weld.
- B. 1/4-in leg for up to 3/4-in thick material.
- C. 5/16-in leg for greater than 3/4-in material.

308.2.2.2.c.2 NON-DESTRUCTIVE INSPECTION OF WELDS

Perform nondestructive testing of welds in accordance with [C&MS](#) 513.

308.2.2.2.d MOMENT PLATES

Do not terminate fully welded moment plates where the calculated total stresses are tensile.

308.2.2.3 GIRDERS**308.2.2.3.a GENERAL**

Investigate multiple designs to determine the most economical option.

Do not use longitudinal stiffeners.

C308.2.2.2.c

[C&MS](#) 513 permits welding by the following processes:

- A. Shielded Metal Arc Welding (SMAW)
- B. Flux Cored Arc Welding (FCAW)
- C. Submerged Arc Welding (SAW)

Fabricators may choose to use one or more of these processes and each process has its advantages. Therefore, the designer should not specify the process.

C308.2.2.2.c.2

For any special NDT inspection of unique or special welded joints, contact the office of structural engineering for additional requirements.

C308.2.2.2.d

End bolted cover plates, as defined in AASHTO, are acceptable for use in zones of tensile stress if cost effective. Welded moment plates may be economical in the compression flange areas over the piers of continuous span structures and may be investigated by the designer.

C308.2.2.3.a

Often a design with an unstiffened web, eliminating transverse stiffeners, is the most economical. In this context, the term “unstiffened web” refers to a web without stiffeners except for the stiffeners used to connect cross-frames. A design with a thicker unstiffened web is also desirable from a maintenance standpoint because field and shop painting of stiffeners is a problem and is often a localized point of failure for the coating system. The NSBA and AISI are available to evaluate your options.

For haunched girders, detail the corner between the flat bottom flange bearing seat area and the curved section of the bottom flange as two plates with a full penetration weld. Give the fabricator the option of hot bending this flange per *AASHTO LRFD Bridge Construction Specifications Section 11.4.3.3.3*. A detail note is provided in BDM Section 702.15.

308.2.2.3.b **NONREDUNDANT STEEL TENSION MEMBERS (NSTM)**

Structures with fracture critical details require a concurrent detail design review to be performed by the Office of Structural Engineering.

If a bridge design includes members or their components that are **nonredundant**, clearly identify those members and components as **NONREDUNDANT STEEL TENSION MEMBERS (NSTM)** in the plans. Designate fracture critical welds as **NSTM** in the plans. Include the detail note provided in BDM Section 702.16 that references the appropriate sections of the *AASHTO/AWS Bridge Welding Code*.

If a girder is non-redundant, include the entire girder in the pay quantity for ITEM 513 - STRUCTURAL STEEL MEMBERS, LEVEL 6. Designate the tension and compression zones in the **nonredundant steel tension** members.

308.2.2.3.c **WIDTH & THICKNESS REQUIREMENTS**

308.2.2.3.c.1 **FLANGES**

C308.2.2.3.b

This section is not intended to recommend **NSTM** designs. The designer should make all efforts to not develop a structure design that requires **nonredundant** members.

Nonredundant steel tension members are defined in Section 2, Definitions, of the *AASHTO/AWS D1.5, Chapter 12 Fracture Control Plan (FCP) For Nonredundant Members*.

C308.2.2.3.c.1

In the design of welded steel girders, the thickness of the flange plates is varied along the length of the girder in accordance with the bending moment. Each change in plate thickness requires a complete penetration butt-weld in the flange plate. These butt-welds are an expensive shop operation requiring considerable labor. In determining the points where changes in plate thickness occur, the designer should weigh the cost of butt-welded splices against extra plate thickness. In many cases it may be advantageous to continue the thicker plate beyond the theoretical stepdown point to avoid the cost of the butt-welded splice.

In order to help make this decision, guidelines proposed by United States Steel in their pamphlet "Fabrication - Its Relation to Design, Shop Practices, Delivery and Costs" may be used. The amount of steel that must be saved to justify providing a welded splice should be as follows:

- A. For A709 grade 36 steel:

300-lb + (25-lb X cross sectional area, in in², of the lighter flange plate)

- B. For A709 grade 50 & 50W steel, the cutoff point shall be 85 percent of the value for grade 36 material.

In addition to design limitations of width to thickness, design flanges to be wide enough such that the girder will have the necessary lateral strength for handling and erection. Employ the following two empirical rules:

$$A_1. \quad b_f = d_w/6 + 2.5 \geq 12\text{-in}$$

$$B_1. \quad b_{fc} \geq L/85$$

Where:

b_f = full width of either girder flange rounded up to the nearest inch (in)

b_{fc} = full width of the compression flange (in)

L = length of the girder shipping piece (in)

d_w = web depth (in)

Design the flange to have a minimum thickness of 7/8-in. Use the following for selection of flange thicknesses:

Whenever possible, use constant flange widths throughout the length of the girder.

- A₂. For material 7/8-in to 3-in thick, specify thickness in 1/8-in increments.

- B₂. For material greater than 3-in thick, specify thickness in 1/4-in increments.

308.2.2.3.c.2 WEBS

Use a minimum web thickness of 3/8-in.

See BDM Section 308.2.2.3.a for recommendations on use of unstiffened web designs

308.2.2.3.d INTERMEDIATE STIFFENERS & CONNECTION PLATES

C308.2.2.3.d

Intermediate transverse stiffeners are defined as plates used to stiffening the web of the girder and are not connected to the cross-frames. Connection plates are defined as plates used to connect cross-frames to beams.

Design intermediate web stiffeners with a minimum thickness of 3/8-in. Clip the corners of the stiffeners that extend beyond the edge of flange at a 45-degree angle. Make all intermediate stiffeners the same size.

Use single stiffeners on alternate sides of the web of interior girders and only the inside of the web for fascia girders. Weld these stiffeners to the web and the compression flange. Make the top tension flange a tight fit.

Weld connection plates to the web and both flanges to help eliminate cracking of the web due to out of plane

bending. Investigate that the fatigue criteria is met in these areas.

Do not stitch weld or single side weld stiffeners and/or connection plates.

Clip corners of stiffeners and connection plates in contact with both web and flange. Clip the stiffener 1-in horizontally and 2.5-in vertically.

For details of stiffeners refer to the Standard Bridge Drawing [GSD-1-19](#).

308.2.2.3.e INTERMEDIATE CROSS-FRAMES

C308.2.2.3.e

Connect cross-frames for girders to intermediate web stiffeners as shown in the Standard Bridge Drawing, [GSD-1-19](#).

For plate girder bridges, provide erection bolts for the connections of cross-frames to girder stiffeners.

Erection bolts are normally 5/8-in diameter. Bolt holes should generally be oversized. See the General Steel Details standard bridge drawing for typical details.

For additional intermediate cross-frame information, refer to BDM Section 308.2.2.2.b.

308.2.2.3.f WELDS

C308.2.2.3.f

The removal of reinforcement improves fatigue characteristics and makes NDT interpretation easier.

308.2.2.3.f.1 WELD TYPES

C308.2.2.3.f.1

There are generally two (2) types of welds acceptable for bridge fabrication, fillet and complete penetration welds.

308.2.2.3.f.2 MINIMUM SIZE OF FILLET & COMPLETE PENETRATION WELDS, PLAN REQUIREMENTS

C308.2.2.3.f.2

Refer to BDM Section 308.2.2.2.c.1 for additional information on minimum fillet weld sizes.

Do not use partial penetration welds, except for secondary members not subject to tension or reversal stresses.

Complete or full Penetration welds are by definition welded through the full section of the plates to be joined.

Specify either fillet weld leg size, in the case of fillet welds, or CP (complete penetration) for complete joint penetration groove welds. Do not detail actual complete penetration welded joints symbols but only show the requirement that the welded joint be Complete Joint Penetration, CP.

Inspection and acceptance of a complete penetration weld is based on whether the weld will be loaded in tension or compression. In order to utilize this permissible quality difference between welds subjected to only compression or tension stresses, designate in the detail plans for steel girders all flange butt welds that are subjected to compressive stresses only. Make this designation by placing the letters “CS” next to full penetration welds shown on detail drawings. Place the following explanatory legend on the same detail sheet:

CS - indicates butt weld subject to compressive stresses only.

308.2.2.3.f.3 INSPECTION OF WELDS, WHAT TO SHOW ON PLANS

C308.2.2.3.f.3

Refer to BDM Section 308.2.2.2.c.2 for additional information on NDT requirements.

For bridges carrying railroads, when full penetration web to flange welds are specified, add a note requiring 10 percent ultrasonic inspection. Check the AREMA specifications and with the actual railroad to confirm the individual railroad's requirements for NDT of welds.

308.2.2.3.g CURVED GIRDER DESIGN REQUIREMENTS

C308.2.2.3.g

When designing curved girder structures, investigate all temporary and permanent loading conditions including loading from wet concrete in the deck pour for all stages of construction. Consider future re-decking as a separate loading condition. Design diaphragms as full load carrying members and according to BDM Section 308.2.2.2.b. Perform a three-dimensional analysis representing the structure as a whole and as it will exist during all intermediate stages and under all construction loadings.

A three-dimensional analysis is essential to accurately predict stresses and deflections in all girders and diaphragms and to ensure that the structure is stable during all construction stages and loading conditions.

Include basic erection data on the contract plans. As a minimum, include the following information:

- A. If temporary supports are required, provide the location of the assumed temporary support points, reactions and deflections for each construction stage and loading condition.
- B. Instructions to the Contractor as to when and how to fasten connections for cross-frames or diaphragms to assure stability during all temporary conditions.

Further design information for curved structures is contained in the “Guide Specifications for Horizontally Curved Highway Bridges”, published by the American Association of State Highway and Transportation Officials.

Use standard bolt holes for intermediate and end cross-frame connections and lateral bracing

Over-sized bolt holes and slotted holes can lead to excessive unforeseen movements during the erection process.

308.2.3 PRESTRESSED CONCRETE BEAMS C308.2.3

The Bridge Design Manual and Standard Bridge Drawings [PSBD-1-25](#) and [PSID-1-13](#) represent standard practice for the prestressed industry in Ohio.

When considering design deviations from these publications, contact the Ohio Prestressers Association and/or the PCI Central Region to request documentation that the deviation is acceptable and can be produced within the project timeframe by at least two independent producers prequalified under S1079.

Model multi-span, non-composite members as simple-span for all loading conditions. Define the live load and future wearing surface as stated in BDM Section 303.1.

Model multi-span, composite members using the two loading conditions that follow:

- A. Design the beams as simple-span for all loading conditions. Define the live load and future wearing surface as stated in BDM Section 303.1.
- B. Design the deck reinforcing for beams per BDM Section 309.3.4

308.2.3.1 TRANSPORTATION & HANDLING C308.2.3.1
CONSIDERATIONS

Consider the site limitations for beam hauling.

For beams 100-ft long or more, contact at least two approved fabricators of precast bridge members to obtain a written agreement stating that the member can be shipped to the project site. Include the agreements in the Structure Type Study, Narrative of Bridge Alternatives.

ODOT may request verification of this documentation during project reviews. These industry organizations are also valuable resources for preliminary pricing and practical hauling limitations to specific project locations.

While weight of a precast bridge member is not typically a limiting factor, its length and ability to reach the jobsite may be a restriction. Maximum lengths are normally dictated by the smallest turning radius enroute to the project site.

In order to prevent damaging the beams during transit and erection, fabricators may require additional strands to be placed in the top flange. These shipping strands keep the top flange in compression until the beams are set into final position. Once set, the shipping strands are cut to release their prestressing force and allow the beams to reach their design ultimate capacity.

308.2.3.2 LENGTH ADJUSTMENTS

In order to prevent fabrication mistakes for beam length, address the effect that the longitudinal grade has on dimensions measured along a beam's length in the plans. When the beam length measured along the grade differs from the beam length measured horizontally by more than 3/8-in, clearly label all affected dimensions measured along the length of the beam so that the fabricator can make the necessary allowances in the Shop Drawings. A Typical Detail note is available in BDM Section 702.6.

308.2.3.3 BOX BEAMS

Refer to BDM Section 304 for concrete and reinforcing material requirements for box beams and concrete wearing surfaces.

Do not use box beams with a span to depth ratio exceeding $L/36$. Do not include the deck in the depth for the ratio.

Do not specify box beams that exceed 42-in in depth.

Refer to BDM Section 308.2.3 when considering a deviation from standard prestressed practice in Ohio.

Do not use non-composite prestressed box beam superstructures on four lane divided highways or where the total ADTT is greater than 250. Do not use composite box beam superstructures with a single mat of deck reinforcement on four lane divided highways or where the total ADTT exceeds 1000. For locations not meeting acceptable requirements for box beams using the previous criteria, a composite deck slab reinforced with two mats of reinforcement and a thickness in accordance with BDM Section 309.3.1 is required.

Box beams are acceptable on curved alignment where the mid-ordinate is 6-in or less and the required bridge width is provided.

Do not use 36-in wide box beams for full bridge replacements.

The maximum asphalt wearing surface thickness for a non-composite designed box beam bridge is 8-in.

C308.2.3.3

The span limits for prestressed, side by side, concrete box beams generally range from 15-ft to 100-ft. 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.

Shear keys have proven to perform poorly under high ADTT loading.

36-in wide box beams may be used for replacing in-kind beams on existing bridges. Due to current fabrication practice, 36-in wide box beams are typically less economical to use than 48-in wide box beams.

Beam section details, section properties, and reinforcing for 36-in wide box beams are provided in the [PSBDD-3](#) design data sheet. Mixing 48-in wide box beams with 36-in wide box beams in the same span is not permitted unless replacing in-kind.

For multiple span bridges, use the same beam depth for all spans.

Do not change the physical dimensions and/or section properties of box beam cross sections from what is shown in standard bridge drawing [PSBD-1-25](#).

Box beam ends are limited to a maximum skew of 30-degrees.

Join multiple span box beam bridges over the piers with a T-joint as shown in standard bridge drawing [PSBD-1-25](#).

Accommodate expansion at the piers by elastomeric expansion bearings or by flexibility of the piers for integral designs.

Design the length of abutment and pier seats of prestressed concrete box beam bridges long enough to accommodate the total width out-to-out of all beams including a fit-up allowance of 1/2-in per joint between beams

Support each prestressed box beam member with two bearings at each support.

For box beam bridges that have skew combined with grade or which have variable superelevation, design and dimension beam seats to provide support for the full width of the box beams.

If a bridge structure's geometry causes a bridge deck in an individual span to have a different cross slope at one bearing than at the other bearing, evenly divide the difference so that the box beam seat cross slopes at the bearings for both beam ends are the same. Accommodate elevation differences created by this beam seat adjustment in the overlay, whether asphaltic or concrete.

Do not cast abutment wingwalls above the bridge seat and backwalls until after box beams have been erected. Separate the cast in place wingwall and box beams by 1-in joint filler per [C&MS](#) 705.03. Show both requirements in the plans.

For box beam bridges with steel railing, detail the post spacing and position of post anchorage on the plans. Reference the dimensioning for the post to each prestressed beam end. Ensure the post anchor spacing does not interfere with tierod locations or the "T" joint over the pier. Confirm that post anchors at the ends of skewed box beams have both adequate concrete cover and do not interfere with the tierods.

Individual span lengths may vary.

Refer to BDM Section 404.4.2 for additional information and possible alternatives.

This adjustment gives the box beam full support at the seat without creating any twist or torsion on the box beam.

Casting the backwall and wingwalls after the box beams are erected eliminates installation problems associated with the actual physical dimensions of the box beam and the joint filler. Cracking and spalling of backwall and wingwall concrete due to movements of the elastomeric bearings is also alleviated.

If the designer finds that no post spacing option can comply with these requirements, the option of relocating the tie rods may be chosen. See bridge standard drawing [PSBD-1-25](#) for maximum allowable spacing of tie rods.

Encase all box beam ends in concrete. See BDM Figure 306-7 and BDM Figure 306-8 for additional information.

308.2.3.3.a DEBONDED STRANDS

Straight pretensioned strands debonded at the ends of beams are acceptable subject to the following restrictions:

- A. Do not debond more than 45% of the strands in a given row.
- B. Do not debond more than six strands to the same length. When a total of ten or fewer strands are debonded, do not debond more than four strands to the same length.
- C. Longitudinally space debonding termination locations at least 60 times the strand diameter apart.
- D. Distribute debonded strands symmetrically about the vertical centerline of the cross-section of the member. Terminate debonding symmetrically at the same longitudinal location.
- E. Alternate bonded and debonded strand locations both horizontally and vertically.
- F. Where a portion or portions of a prestressing strand are debonded and where tension exists in the precompressed tensile zone, determine the development lengths, measured from the end of the debonded zone, using *LRFD Eq. 5.9.4.3.2-1* with a value of $\kappa = 2.0$.
- G. For simple span precast, prestressed beams, the maximum debonding length from the beam end is the smaller of 20 percent of the span length or one half the span length minus the development length.
- H. Uniformly distribute debonded strands between webs.
- I. Bond the outer-most strands within the section.

308.2.3.3.b STRAND SPACING

Space strands at increments or multiples of 2-in.

Place the centerline of the first row of strands 2-in from the bottom of the beam. Completely enclose all strands with the #4 stirrup bars. Place strands near the top flange below all transverse and longitudinal concrete reinforcement and to the left and right of the void.

308.2.3.3.c COMPOSITE DESIGN

The minimum thickness of composite reinforced deck slabs on prestressed box beams is 6-in and reinforced with #6 bars. Space the longitudinal bars at 18-in maximum and the transverse bars spaced at 9-in maximum.

On multiple span composite box beam bridges, provide additional longitudinal reinforcing over the piers. Use #6 bars for the additional reinforcement. Space the additional bars alternately between the standard longitudinal reinforcement. The length of the additional reinforcement over the pier is the larger of: 40 percent of the length of the longer adjacent stringer span or a length that meets the requirements of *LRFD 5.10.8.1.2c*. Place the pier bars longitudinally and approximately centered on the pier but with a 3-ft stagger.

Incorporate fit-up tolerances into the finished deck width for composite box beam structures with concrete railings or sidewalks. See BDM Figure 308-4 for a sketch of the cross-section of the composite deck superstructure.

C308.2.3.3.c

For ease of placement on skewed structures, the transverse bars may be placed parallel to the substructure units with spacing measured parallel to the longitudinal axis of the structure.

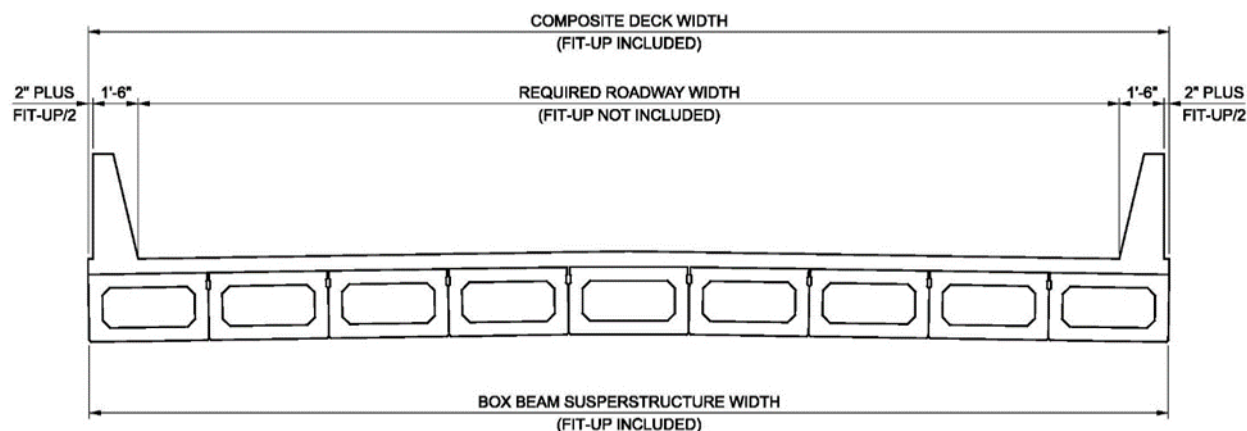


Figure 308-4

If the required roadway width is wider than the box beam superstructure width, the composite deck may be designed to overhang the box beams a maximum of 8-in each side.

308.2.3.3.d NON-COMPOSITE DESIGN

Specify Type 3 Waterproofing to be used for non-composite box beam bridges with asphalt overlays per [C&MS 512](#). Place the waterproofing on the boxes before the first layer of asphaltic concrete is applied. See BDM Sections 309.1.B and 309.2.

C308.2.3.3.d

Provide over-the-side drainage when non-composite design is used.

Only use non-composite box beams with integral or semi-integral abutments.

Do not use non-composite box beams for structure with a total length exceeding 475-ft.

Limit non-composite box beam bridges with asphalt overlays to a 4 percent combined grade. Combined grade (C_g) includes both the longitudinal and transverse structure grades calculated as follows:

$$C_g = ([\text{deck slope}]^2 + [\text{transverse grade}]^2)^{1/2}$$

Use a composite deck as described in BDM Section 308.2.3.3.c when the combined deck grade is greater than 4 percent.

308.2.3.3.e CAMBER

As shown in BDM Figure 308-5, determine the topping thickness (T_x) at any point, x , along the length of a prestressed box beam superstructure by the following:

$$T_x = A + B_x + C + D_{t,x} - E$$

Where:

- A = Design deck thickness
- B_x = Vertical grade adjustment
- C = Sacrificial haunch depth
- $D_{t,x}$ = Beam camber adjustment at member age equal to Day t
- E = Haunch adjustment

For a normal transverse deck grade horizontal to vertical of 3/16-in per foot (1.56 percent), the maximum roadway grade would be 3.68 percent or less for non-composite design.

C308.2.3.3.e

The topping thickness on prestressed box beam superstructures will vary along the length of the beams to account for beam camber and other vertical elevation adjustments. Proper determination of the topping thickness is crucial in order to properly establish beam seat elevations.

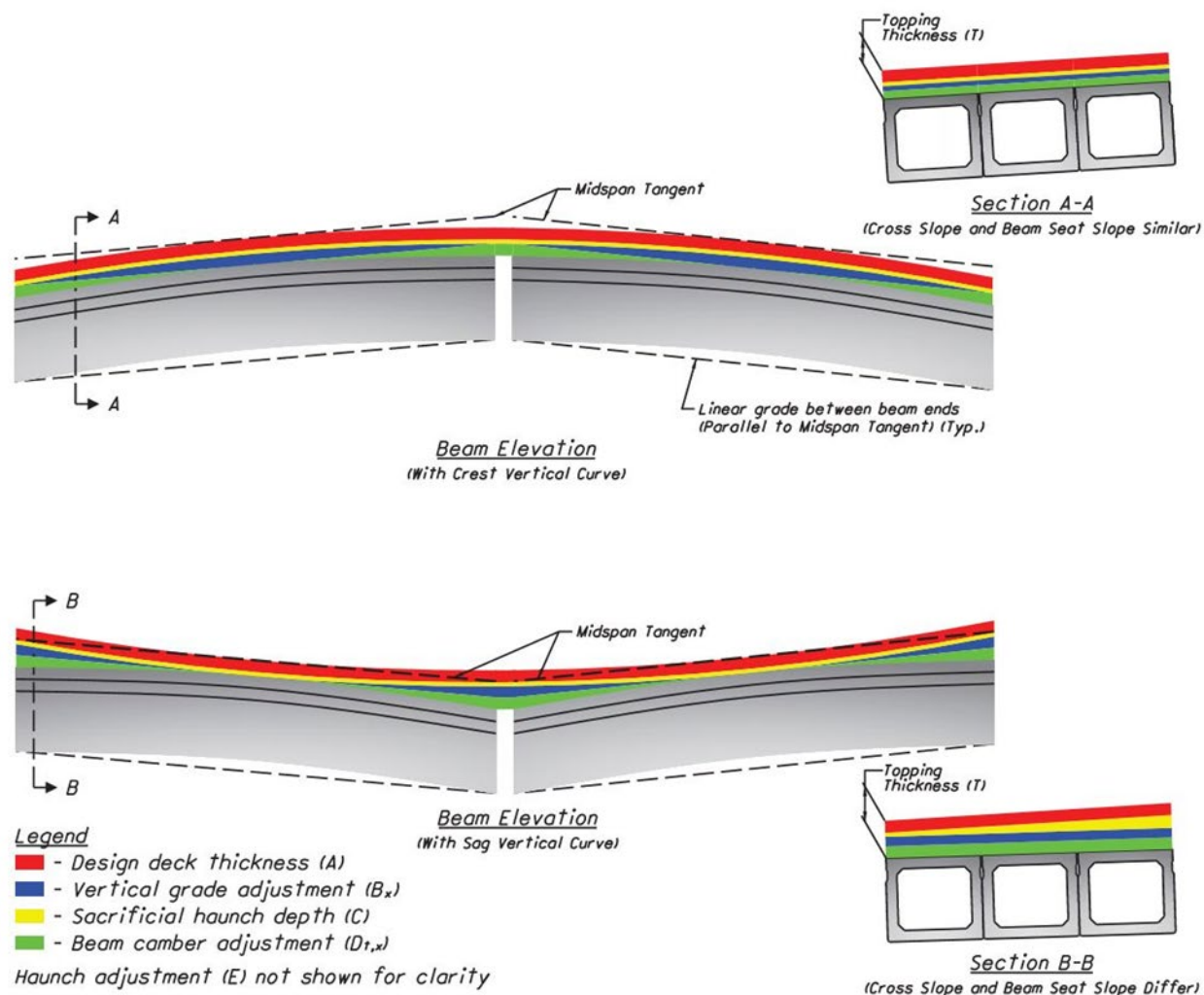


Figure 308-5

308.2.3.3.e.1 VERTICAL GRADE ADJUSTMENT (B_x)

Minimize the vertical grade adjustment along the length of the bridge by setting the linear grade between the beam ends parallel to the tangent of the vertical grade at the midpoint of the beam span (see BDM Figure 308-5).

C308.2.3.3.e.1

The Vertical Grade Adjustment accounts for any elevation differences between a non-linear profile grade and the linear grade connecting the centerline of beam supports. The value of the Vertical Grade Adjustment depends on many geometric factors such as vertical curvature, skew, cross-slope transitions, etc.

308.2.3.3.e.2 SACRIFICIAL HAUNCH DEPTH (C)

C308.2.3.3.e.2

The purpose of the Sacrificial Haunch Depth is to account for camber in excess of that calculated in the Beam Camber Adjustment above and account for the roadway cross-slope.

For multiple span box beam bridges with design speeds exceeding 45-mph, the minimum thickness of the Sacrificial Haunch Depth (C) is 2-in. For all other box beam bridges, the minimum thickness of Sacrificial Haunch Depth (C) is 0-in.

If the wearing surface cross slope is different than the beam seat at the beam bearings as noted in BDM Section 308.2.3.3, the difference between the cross slope and the beam seat slope will be accommodated by the sacrificial haunch depth.

Because box beams are set on sloping seats that approximate the cross-slope, the sacrificial haunch depth is typically constant.

The minimum thickness, C, will occur at the outside edge of the fascia beam on the low side of the cross slope.

308.2.3.3.e.3 BEAM CAMBER ADJUSTMENT $(D_{t,x})$ C308.2.3.3.e.3

As prestressed concrete beams age, beam camber will increase due to concrete creep under the constant loading from the prestressing force. Although designers cannot accurately predict the girder age when the deck is placed, general assumptions can be made to prevent camber growth from becoming an issue during construction.

Show two values for camber at midspan in the design plans which the contractor can use to establish seat elevations according to [C&MS](#) 511.07 and tolerance according to [C&MS](#) 515.17: camber at Day 0 (D_0) and camber at Day 30 (D_{30}). These values represent the midspan camber in the beams before application of any dead load other than self-weight.

To determine these camber values, calculate the creep coefficient, $\psi(t, t_i)$, according to *LRFD* 5.4.2.3.2 with humidity (H) equal to 70%; age of concrete at release (t_i) equal to 0.75 days; and V/S and f'_{ci} according to the project requirements. To calculate the creep coefficient at Day 0 and Day 30, use a maturity of concrete (t) equal to 0 days and 30 days respectively

The respective camber values are found by multiplying the net midspan camber at the time of release by the appropriate creep coefficient as follows:

$$D_{30} = [1 + \psi(t, t_i)] D_0$$

The net camber at the time of release ($\delta_{net,x}$) is the difference between the initial beam camber due to the prestressing force ($\delta_{o,x}$) and beam deflection due to self-weight ($\delta_{sw,x}$) [i.e. $\delta_{net,x} = \delta_{o,x} - \delta_{sw,x}$].

For the purposes of determining the topping thickness, calculate the final Beam Camber Adjustment ($D_{t,x}$) at any point, x, along the length of the beam as follows:

$$D_{t,x} = [(\delta_{\text{net,mid}} - \delta_{\text{net,x}})(1 + \psi(t, t_i))] - [(\delta_{\text{NC,mid}} + \delta_{\text{C,mid}}) - (\delta_{\text{NC,x}} + \delta_{\text{C,x}})]$$

Where:

$\delta_{\text{net,mid}}$ = Net camber at mid-span

$\delta_{\text{net,x}}$ = Net camber at point x

$\delta_{\text{NC,mid}}$ = Deflection due to non-composite loading at mid-span

$\delta_{\text{C,mid}}$ = Deflection due to composite loading at mid-span

$\delta_{\text{NC,x}}$ = Deflection at point x due to non-composite loading

$\delta_{\text{C,x}}$ = Deflection at point x due to composite loading

$\psi(t, t_i)$ = Creep coefficient

(Note: The equation shown for $D_{t,x}$ assumes camber is upward and deflections are downward.)

To establish beam seat elevations, calculate the Beam Camber Adjustment using the creep coefficient at t equal to 30 days.

308.2.3.3.e.4 HAUNCH ADJUSTMENT (E)

C308.2.3.3.e.4

The Haunch Adjustment is the portion of the Beam Camber Adjustment that can be utilized for Vertical Grade Adjustment in crest vertical curve profiles. This adjustment prevents the haunch from becoming excessive along the full length of the beam.

Calculate the Haunch Adjustment (E) as follows:

$$E = B_{\text{mid}} \leq \text{Max } D_{t,x}$$

The Haunch Adjustment is equal to the Vertical Grade Adjustment at the beam midspan (B_{mid}) but cannot exceed the maximum value for $D_{t,x}$.

308.2.3.3.f ANCHORAGE

In a box beam design, anchor all beams at abutments and piers. Place the anchor at the center of the cross section of the box beam and conform to details presented in the Standard Bridge Drawing [PSBD-1-25](#).

Install fixed end anchor dowels with a non-shrinking grout (mortar). Fill expansion end anchor dowel holes with joint sealer per [C&MS](#) 705.04.

Install polystyrene, semi-rigid closed-cell polypropylene foam, or sponge rubber per [C&MS](#) 705.03, the same thickness as the elastomeric bearing, under the box beam and around the anchor dowel to halt the grout or sealer from leaking through to the beam seat.

308.2.3.3.g TIE RODS

Provide and install tie rods according to the Bridge Standard Drawing [PSBD-1-25](#).

Provide diaphragms and transverse tie rods for prestressed concrete box beam spans at mid-span for spans up to 50-ft, at third points for spans from 50-ft to 75-ft and at quarter points for spans greater than 75-ft.

308.2.3.4 I-BEAMS

Use AASHTO standard prestressed I-beam shapes, type II through type IV; modified type IV and WF36-49 through WF72-49 as shown in the standard bridge drawing [PSID-1-13](#).

Refer to BDM Section 304 for concrete and reinforcing material requirements for I-beam and the reinforced concrete deck.

Use a minimum of 4 prestressed I-beam stringer lines on bridge carrying vehicular traffic.

In designing prestressed I-beams, use the non-composite section for computing stresses due to the beam and deck slab. Use the composite section for computing stresses due to the superimposed dead, railing and live loads.

When designing the deck reinforcement for a multiple span structure, model as a continuous beam on a single support centered on the pier.

Aside from access for shipping strands and clipped flanges as shown in the Bridge Standard Drawing [PSID-1-13](#), do not place abrupt changes or discontinuities in the beam cross-section. If abrupt changes are not avoidable, a refined analysis that accounts for section loss, resulting stress concentrations, and time dependent loading is required.

When detailing beam elevations, dimension the locations of all inserts, hold-downs, etc. to the ends of the beam rather than the centerlines of bearing.

C308.2.3.4

Refer to BDM Figure 308-6 for approximate maximum spans for each beam shape. The maximum spans provided may be used during the preliminary design of the structure to determine the approximate beam shape required.

This provision is to avoid non-redundant structures.

Prestressed I-beam bridges that meet the vertical clearance specified in BDM Section 302.1 are acceptable over highway crossings.

For additional information see BDM Section 309.3.

These dimensions are primarily for the prestressed beam fabricator and dimensioning from the end of the beam is a more practical work point for the fabricator.

BEAM SHAPE	BEAM SPACING	APPROX. MAXIMUM SPAN
AASHTO TYPE 2	6'-0"	61-ft.
	7'-6"	57-ft
	9'-0"	50-ft
AASHTO TYPE 3	6'-0"	83-ft
	7'-6"	78-ft
	9'-0"	72-ft
AASHTO TYPE 4	6'-0"	107-ft
	7'-6"	100-ft
	9'-0"	93-ft
MODIFIED AASHTO TYPE 4 (60")	7'-0"	115-ft
	8'-6"	109-ft
	10'-0"	101-ft
MODIFIED AASHTO TYPE 4 (66")	7'-0"	124-ft
	8'-6"	117-ft
	10'-0"	110-ft
MODIFIED AASHTO TYPE 4 (72")	7'-0"	133-ft
	8'-6"	127-ft
	10'-0"	119-ft
WF36-49	7'-0"	85-ft
	8'-6"	80-ft
	10'-0"	76-ft
WF42-49	7'-0"	97-ft
	8'-6"	90-ft
	10'-0"	85-ft
WF48-49	7'-0"	108-ft
	8'-6"	100-ft
	10'-0"	94-ft
WF54-49	8'-0"	113-ft
	9'-6"	104-ft
	11'-0"	99-ft
WF60-49	8'-0"	125-ft
	9'-6"	117-ft
	11'-0"	110-ft
WF66-49	8'-0"	134-ft
	10'-0"	124-ft
	12'-0"	116-ft
WF72-49	8'-0"	143-ft
	10'-0"	134-ft
	12'-0"	126-ft

Notes:

1. Designs in accordance with AASHTO LRFD Bridge Design Specifications, the ODOT Bridge Design Manual and Bridge Standard Drawing PSID-1-13
2. Design Assumptions:
 - a. Five parallel beam lines
 - b. No skew
 - c. 3-in average haunch thickness
 - d. Beam concrete strengths: $f'_c = 5000\text{psi}$, $f'_e = 7000\text{psi}$
 - e. 3'-0" Overhang
 - f. 42-in single slope concrete barrier
 - g. Steel intermediate crossframes for 60", 66" & 72" beams

Figure 308-6

308.2.3.4.a STRANDS

Do not transform the strand area in order to increase the section properties.

308.2.3.4.a.1 SPACING

Space strands at increments of 2-in.

Maintain a minimum of 2-in dimension from bottom of beam to centerline of the first row of strands and any exterior beam surface.

308.2.3.4.a.2 DEBONDED STRANDS

Straight pretensioned strands may be debonded at the ends of beams subject to the following restrictions:

- A. Do not debond more than 45% of the strands in a given row.
- B. Do not debond more than six strands to the same length. When a total of ten or fewer strands are debonded, do not debond more than four strands to the same length.
- C. Longitudinally space debonding termination locations at least 60 times the strand diameter apart.
- D. Distribute debonded strands symmetrically about the vertical centerline of the cross-section of the member. Terminate debonding symmetrically at the same longitudinal location.
- E. Alternate bonded and debonded strand locations both horizontally and vertically.
- F. Where a portion or portions of a prestressing strand are debonded and where tension exists in the precompressed tensile zone, determine the development lengths, measured from the end of the debonded zone, using *LRFD Eq. 5.9.4.3.2-1* with a value of $\kappa = 2.0$.

C308.2.3.4.a

The preferred strand pattern is straight, parallel strands with no debonding. However, excessive tensile stresses may develop in the beam ends during the release of the prestressing force. To relieve these excessive stresses, the following strand patterns are allowed: (listed in order of preference)

- A. Partially debonded bottom flange strands up to the limits specified in BDM Section 308.2.3.4.a.2.
- B. Combination of the maximum partially debonded bottom flange strands permitted by BDM Section 308.2.3.4.a.2. and draped strands.

- G. For simple span precast, prestressed beams, the maximum debonding length from the beam end is the smaller of 20 percent of the span length or one half the span length minus the development length.
- H. Bond all strands within the horizontal limits of the web when the total number of debonded strands exceeds 25 percent.
- I. Bond all strands within the horizontal limits of the web when the bottom flange to web width ratio exceeds 4.
- J. Bond the outer-most strands in all rows located within the full-width section of the flange.
- K. Position debonded strands furthest from the vertical centerline.

308.2.3.4.a.3 DRAPING**C308.2.3.4.a.3**

Draping or harping of the strands may be done to relieve excessive stresses at the beam ends.

Locate the hold down point at least 5-ft on each side of the midspan of the beam using increments of 6-in. Calculate the vertical uplift force, P_U , at each hold-down location and ensure the limits of BDM Table 308-1 are not exceeded.

Table 308-1

No. of Draped Strands per Row	P_U/Strand (lb)	P_U/Unit (lb)
1	6000	48,000
2	4000	48,000
3	4000	48,000

Where:

$$P_U/\text{Strand} = (1.05)0.75 f'_s A_{PS} \tan \psi$$

$$P_U/\text{Strand} = (1.05)0.00075 f'_s A_{PS} \tan \psi$$

$$P_U/\text{Unit} = \sum_{i=1}^n P_U/\text{Strand}$$

And:

$$f'_s = 270,000\text{-psi}$$

$$A_{PS} = \text{Area of single strand (in}^2\text{)}$$

$$n = \text{no. of strands}$$

$$\psi = \text{Angle of strand inclination measured from horizontal}$$

To minimize the uplift force, locate the hold down point as close as allowed to the midspan and limit the height of the draped strands at the beam ends to only the height required to control stresses.

Detail the height of draped strands at beam ends and at midspan at multiples of 2-in.

Do not place straight strands above draped strands in the same vertical column.

308.2.3.4.b CAMBER

As shown in BDM Figure 308-7, determine the concrete topping thickness (T_x) at any point, x , along the length of a prestressed I-beam superstructure by the following:

$$T_x = A + B_x + C + D_{t,x} - E$$

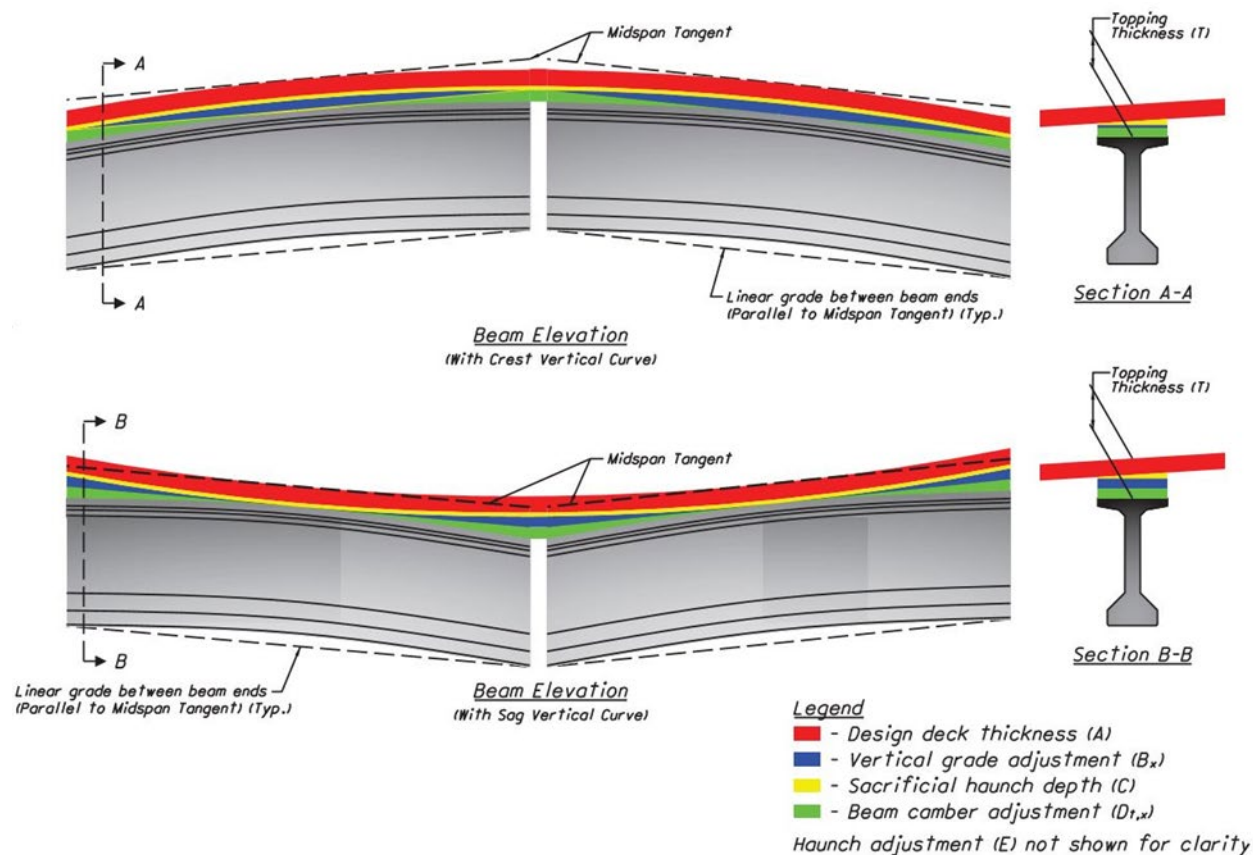
Where:

- A = Design deck thickness
- B_x = Vertical grade adjustment
- C = Sacrificial haunch depth
- $D_{t,x}$ = Beam camber adjustment at member age equal to Day t
- E = Haunch adjustment

It is not necessary for the angle of inclination for each row of draped strands to be the same.

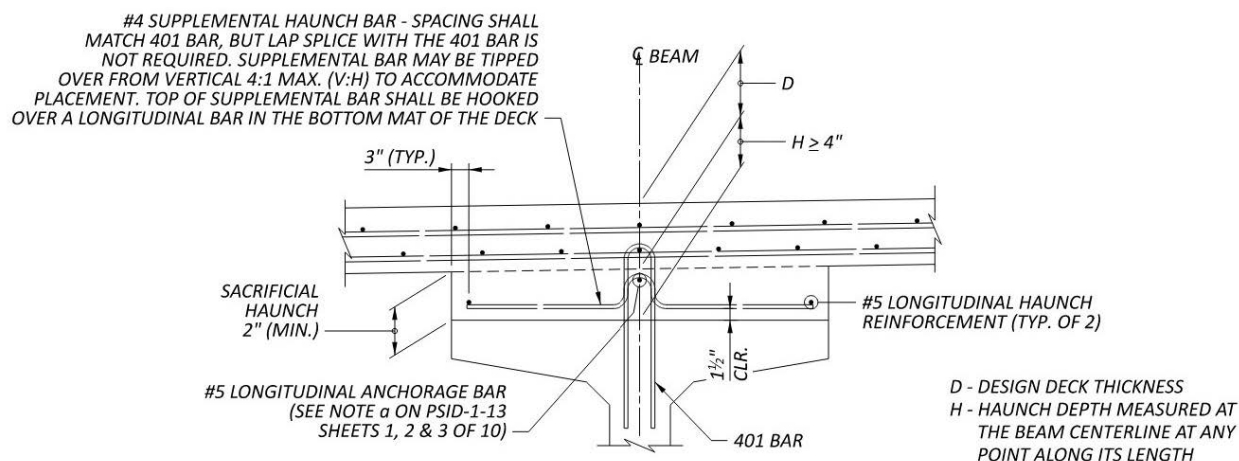
C308.2.3.4.b

The concrete topping thickness on prestressed I-beam superstructures will vary along the length of the beams to account for beam camber and other vertical elevation adjustments. Proper determination of the topping thickness is crucial in order to properly establish beam seat elevations.

**Figure 308-7**

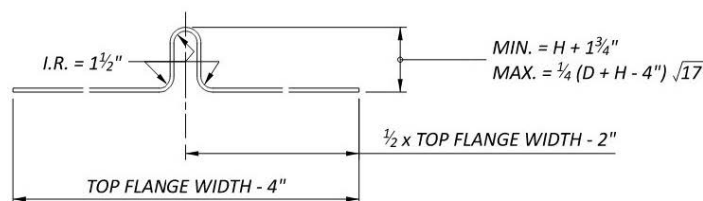
Include additional haunch reinforcement when the haunch depth is greater than or equal to 4-in. (i.e., $B_x + C + D_{t,x} - E \geq 4\text{-in}$) for the sections shown in Bridge Standard Drawing [PSID-1-13](#). Detail additional haunch reinforcement as shown in BDM Figure 308-8. Show the locations of the additional haunch reinforcement in the plans.

In order to ensure the entire haunch depth acts compositely with the beam and deck, additional reinforcement may be necessary to extend the beam's composite reinforcement at least 2-in into the deck thickness.



HAUNCH REINFORCEMENT

NOTE - THE SUPPLEMENTAL HAUNCH BAR IS NOT REQUIRED WHERE $H < 4"$



#4 SUPPLEMENTAL HAUNCH BAR

Figure 308-8

308.2.3.4.b.1 VERTICAL GRADE ADJUSTMENT (B_v)

Minimize the vertical grade adjustment along the length of the bridge by setting the linear grade between the beam ends parallel to the tangent of the vertical grade at the midpoint of the beam span (see BDM Figure 308-7).

C308.2.3.4.b.1

The Vertical Grade Adjustment accounts for any elevation differences between a non-linear profile grade and the linear grade connecting the centerline of beam supports. The value of the Vertical Grade Adjustment depends on many geometric factors such as vertical curvature, skew, cross-slope transitions, etc.

308.2.3.4.b.2 SACRIFICIAL HAUNCH DEPTH (C)

C308.2.3.4.b.2

The purpose of the Sacrificial Haunch Depth is to account for camber in excess of that calculated in the Beam Camber Adjustment above and account for the roadway cross-slope.

Design the Sacrificial Haunch Depth (C) to be a minimum of 2-in.

Detail the haunch so the 2-in minimum thickness, C, will occur at a flange edge. Show this requirement in the plans.

Because I-beams are set on level seats, the beams will be plumb after erection. The difference between the grades of the top flange and the cross-slope will be accommodated by the sacrificial haunch depth.

308.2.3.4.b.3 BEAM CAMBER ADJUSTMENT ($D_{t,x}$)

C308.2.3.4.b.3

As prestressed concrete beams age, beam camber will increase due to concrete creep under the constant loading from the prestressing force. Although designers cannot accurately predict the girder age when the deck is placed, general assumptions can be made to prevent camber growth from becoming an issue during construction.

Show two values for camber at midspan in the design plans which the contractor can use to establish seat elevations according to [C&MS](#) 511.07 and tolerance according to [C&MS](#) 515.17: camber at Day 0 (D_0) and camber at Day 30 (D_{30}). These values represent the midspan camber in the beams before application of any dead load other than self-weight.

To determine these camber values, calculate the creep coefficient, $\psi(t, t_i)$, according to *LRFD* 5.4.2.3.2 with humidity (H) equal to 70%; age of concrete at release (t_i) equal to 0.75 days; and V/S and f'_{ci} according to the project requirements. To calculate the creep coefficient at Day 0 and Day 30, use a maturity of concrete (t) equal to 0 days and 30 days respectively. The respective camber values are found by multiplying the net midspan camber at the time of release by the appropriate creep coefficient as follows:

$$D_{30} = [1 + \psi(t, t_i)] D_0$$

The net camber at the time of release ($\delta_{net,x}$) is the difference between the initial beam camber due to the prestressing force ($\delta_{o,x}$) and beam deflection due to self-weight ($\delta_{sw,x}$) [i.e. $\delta_{net,x} = \delta_{o,x} - \delta_{sw,x}$].

The gross moment of inertia for the non-composite I-beam may be used to determine $\delta_{o,x}$ and $\delta_{sw,x}$.

For the purposes of determining the topping thickness, calculate the final Beam Camber Adjustment ($D_{t,x}$) at any point, x, along the length of the beam as follows:

$$D_{t,x} = [(\delta_{net,mid} - \delta_{net,x})(1 + \psi(t, t_i))] - [(\delta_{NC,mid} + \delta_{C,mid}) - (\delta_{NC,x} + \delta_{C,x})]$$

Where:

- $\delta_{net,mid}$ = Net camber at mid-span
- $\delta_{net,x}$ = Net camber at point x
- $\delta_{NC,mid}$ = Deflection due to non-composite loading at mid-span
- $\delta_{C,mid}$ = Deflection due to composite loading at mid-span

$\delta_{NC,x}$ = Deflection at point x due to non-composite loading

$\delta_{C,x}$ = Deflection at point x due to composite loading

$\psi(t,t_i)$ = Creep coefficient

(Note: The equation shown for $D_{t,x}$ assumes camber is upward and deflections are downward.)

To establish beam seat elevations, calculate the Beam Camber Adjustment using the creep coefficient at t equal to 30 days.

308.2.3.4.b.4 HAUNCH ADJUSTMENT (E)

Calculate the Haunch Adjustment (E) as follows:

$$E = B_{mid} \leq \text{Max } D_{t,x}$$

308.2.3.4.c ANCHORAGE

Provide 1-in. diameter anchors at each fixed pier as shown on bridge standard drawing [PSID-1-13](#).

Determine the required number of anchors by analysis. Design the anchors to transfer superstructure loads to the substructure at the Strength Limit States and resist seismic loads at Extreme Event Limit State.

Use anchors with a minimum length of 2-ft. Embed the anchors a minimum of 1-ft into the pier cap. Post install (drill and grout) the anchors at the centerline of at the pier. Confirm the pier cap has concrete reinforcement clearance to accept these anchors.

308.2.3.4.d DIAPHRAGMS

Space intermediate diaphragms a maximum of 40-ft.

Intermediate diaphragms for 60-in and deeper beams shall be either cast-in-place concrete or galvanized steel. The designer shall choose the type. Intermediate diaphragms for less than 60-in deep beams shall be cast-in-place concrete. Details for each type are provided in the standard bridge drawing. The design plans shall show the centerline location of each intermediate diaphragm. The Department will pay for the intermediate diaphragms at the contract price for ITEM 515 - EACH, INTERMEDIATE DIAPHRAGMS.

C308.2.3.4.b.4

The Haunch Adjustment is the portion of the Beam Camber Adjustment that can be utilized for Vertical Grade Adjustment in crest vertical curve profiles only. This adjustment prevents the haunch from becoming excessive along the full length of the beam.

The Haunch Adjustment is equal to the Vertical Grade Adjustment at the beam midspan (B_{mid}) but cannot exceed the maximum value for $D_{t,x}$.

C308.2.3.4.d

The designer shall specify in the plans which type of intermediate diaphragms are to be used.

Detail cast-in-place intermediate diaphragms as to not contact the underside of the deck. Show the top of the cast-in-place intermediate diaphragm at the bottom vertical edge of the top flange and the bottom of the cast-in-place intermediate diaphragm at the top of the vertical edge of the bottom flange.

Contact with the intermediate diaphragm could act as a support to the deck, causing cracking and possible over stressing of the deck.

Refer to Bridge Standard Drawing [PSID-1-13](#) for additional details on cast-in-place intermediate diaphragms.

If the Standard Bridge Drawing for I-beams is not referenced by the contract plans, add a note to the plans for prestressed I-beam designs requiring cast-in-place intermediate diaphragms to be placed and cured at least 48 hours before deck placement.

Use threaded inserts to connect the concrete reinforcement for the cast-in-place diaphragm to the I-beam. Galvanize the threaded inserts and the threaded rods according to [C&MS](#) 711.02.

Provide cast-in-place end diaphragms. Detail the end diaphragm to make complete contact with the deck. End the bottom limits of the end diaphragm at the top of the elastomeric bearing.

Refer to the Standard Bridge Drawing [PSID-1-13](#) for typical diaphragm details.

309 DECK

Do not use precast deck panels.

309.1 WEARING SURFACE TYPES

Use only one of the following wearing surfaces unless directed otherwise in the scope of services.

- A. 1-in monolithic concrete wearing surface:
Do not include the top 1-in thickness in the structural design of the deck slab or as part of the composite section.
- B. 3-in asphalt concrete:
Use the 3-in minimum asphalt concrete wearing surface only on non-composite prestressed box beams.

C309.1

1-in monolithic concrete wearing surface is defined as the top 1-in of a concrete deck slab.

3-in asphalt concrete is defined as the minimum asphaltic concrete wearing surface on non-composite prestressed box beams.

Construct the asphalt concrete wearing surface to a minimum thickness of 3-in placed in the follow manner:

1. Place two separate 1.5-in minimum lifts of ITEM 441 – ASPHALT CONCRETE SURFACE COURSE, TYPE 1, PG70-22M. Place the first lift at a variable thickness to accommodate beam camber and vertical grade. Place the second lift in a uniform 1.5-in thickness.
2. Apply two applications of ITEM 407 – TACK COAT - one prior to placement of the first lift of surface course and one prior to placement of the second lift of surface course. Refer to the [C&MS Table 407.06-1](#) for application rates.

C. 6-in cast-in-place composite deck:

Use the 6-in cast-in-place composite deck only on composite prestressed box beams.

Consider the top 1-in to be a monolithic wearing surface as defined above. Also see BDM Section 308.2.3.3.c.

6-in cast-in-place composite deck is defined as the minimum thickness of concrete slab for composite prestressed box beams.

309.2 CONCRETE DECK PROTECTION

Unless otherwise noted in the Scope of Services, use epoxy coated steel reinforcement.

Use class QC2 concrete for bridge deck.

Provide a minimum of 2.5-in top cover with all cast-in-place concrete decks.

Use a drip strip on decks with over-the-side drainage.

Use Type 3 waterproofing per [C&MS](#) 512 for non-composite box beam bridges with an asphalt concrete overlay. Minimum thickness of overlay is 3-in. See BDM Section 309.1.B.

Refer to BDM Section 403.3 for deck sealing methods.

309.2.1 SEALING OF CONCRETE SURFACES, SUPERSTRUCTURE

Specifications for sealing material are defined in [C&MS](#) 512. Seal concrete surfaces with an approved concrete sealer as follows: (See BDM Figure 309-1 & Figure 309-2)

C309.2.1

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

In areas where concrete surfaces have a history of graffiti vandalism, the owner may add a permanent graffiti coating in the Scope of Services meeting the requirements of S1083 on top of the epoxy-urethane or non-epoxy sealer. A plan note is available in BDM Section 602.6. The scope should limit the concrete surfaces that are treated with permanent graffiti coatings to those reachable by easy climbing and visible to the traveling public.

A. Concrete slabs or concrete decks on steel superstructures with over-the-side drainage:

Seal the deck fascia and a 6-in (minimum) width under the deck with either an epoxy-urethane or non-epoxy sealer.

- B. Concrete slabs, composite prestressed box beam superstructures or concrete decks on steel superstructures with sidewalks:

Seal the vertical face of curb; the top of the curb/sidewalk; the inside face, top and outside face of the concrete bridge railing; the deck fascia; and a 6-in (minimum) width under the deck with either an epoxy-urethane or non-epoxy sealer.

- C. Concrete slabs, composite prestressed box beam superstructures or concrete decks on steel superstructures with concrete bridge railing:

Seal the inside face, top and outside face of concrete bridge railing; the deck fascia; and a 6-in (minimum) width under the deck with either an epoxy-urethane, or non-epoxy sealer.

- D. Non-composite prestressed concrete box beam decks with over-the-side drainage:

Seal the fascia of the outside beams and a minimum 6-in width under the beam with an epoxy-urethane or a non-epoxy sealer.

- E. Concrete decks on prestressed I-beam superstructures with over-the-side drainage:

Seal the deck fascia; the underside of the deck to the edge of the top flange; the exterior fascia of the beam; the underside of the bottom flange; and the inside face of the bottom flange with an epoxy-urethane sealer.

- F. Concrete decks on prestressed I-beam superstructures with sidewalks:

Seal the vertical face of curb; the top of the curb/sidewalk; the inside face, top and outside face of the concrete bridge railing; the deck fascia; the underside of the deck to the edge of the top flange; the exterior fascia of the beam; the underside of the bottom flange; and the inside face of the bottom flange with an epoxy-urethane sealer.

- G. Concrete decks on prestressed I-beam superstructures with concrete bridge railing:

Seal the inside face, top and outside face of concrete bridge railing; the deck fascia; the underside of the deck to the edge of the top flange; the exterior fascia of the beam; the underside of the bottom flange; and the inside face of the bottom flange with either an epoxy-urethane sealer.

Seal concrete surfaces that include patches with an epoxy-urethane sealer.

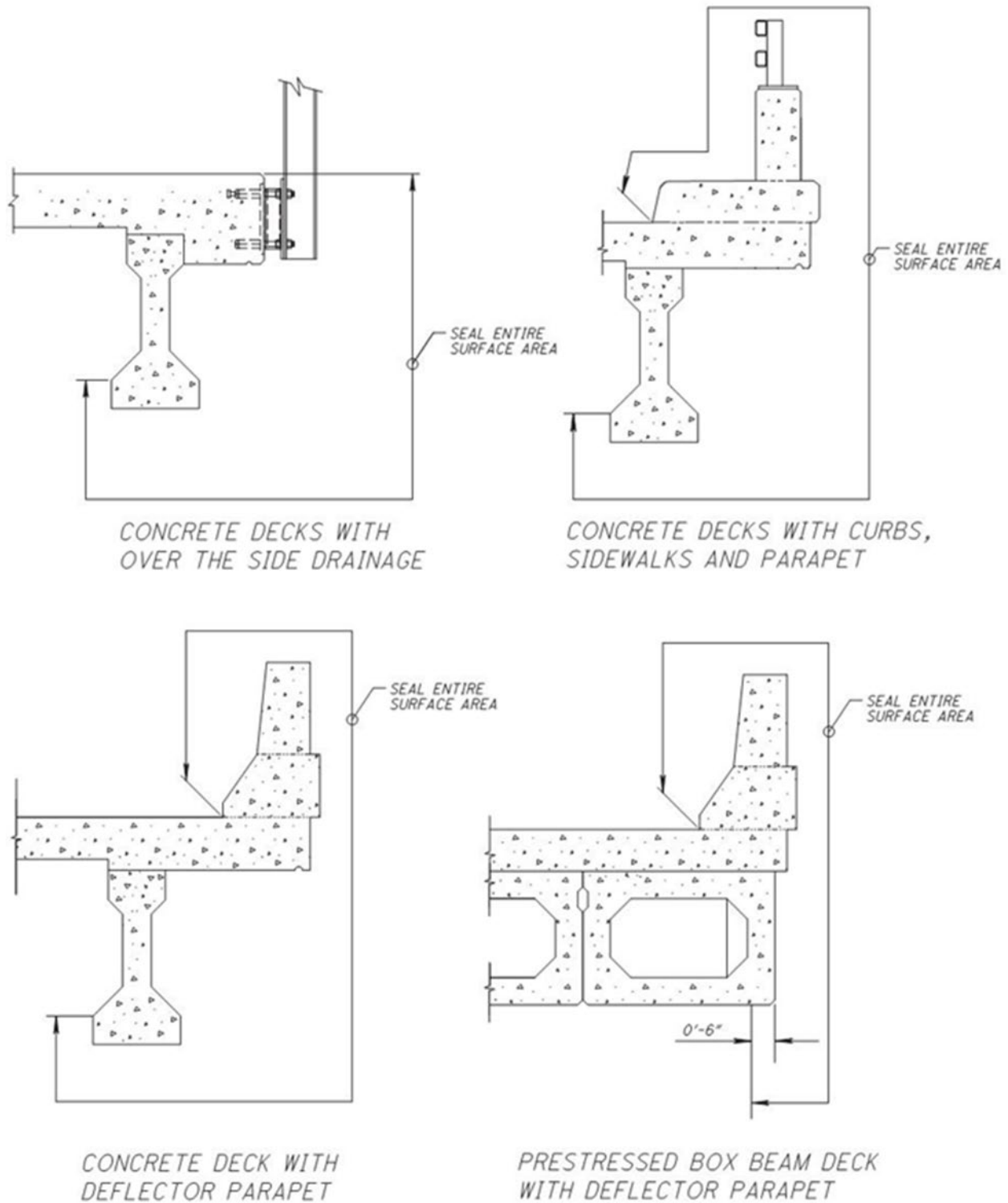
This is required so the concrete color will remain uniform.

Include details in the plans showing the position, location and area required to be sealed. Do not use a plan note to describe the location.

This can lead to both description and interpretation problems.

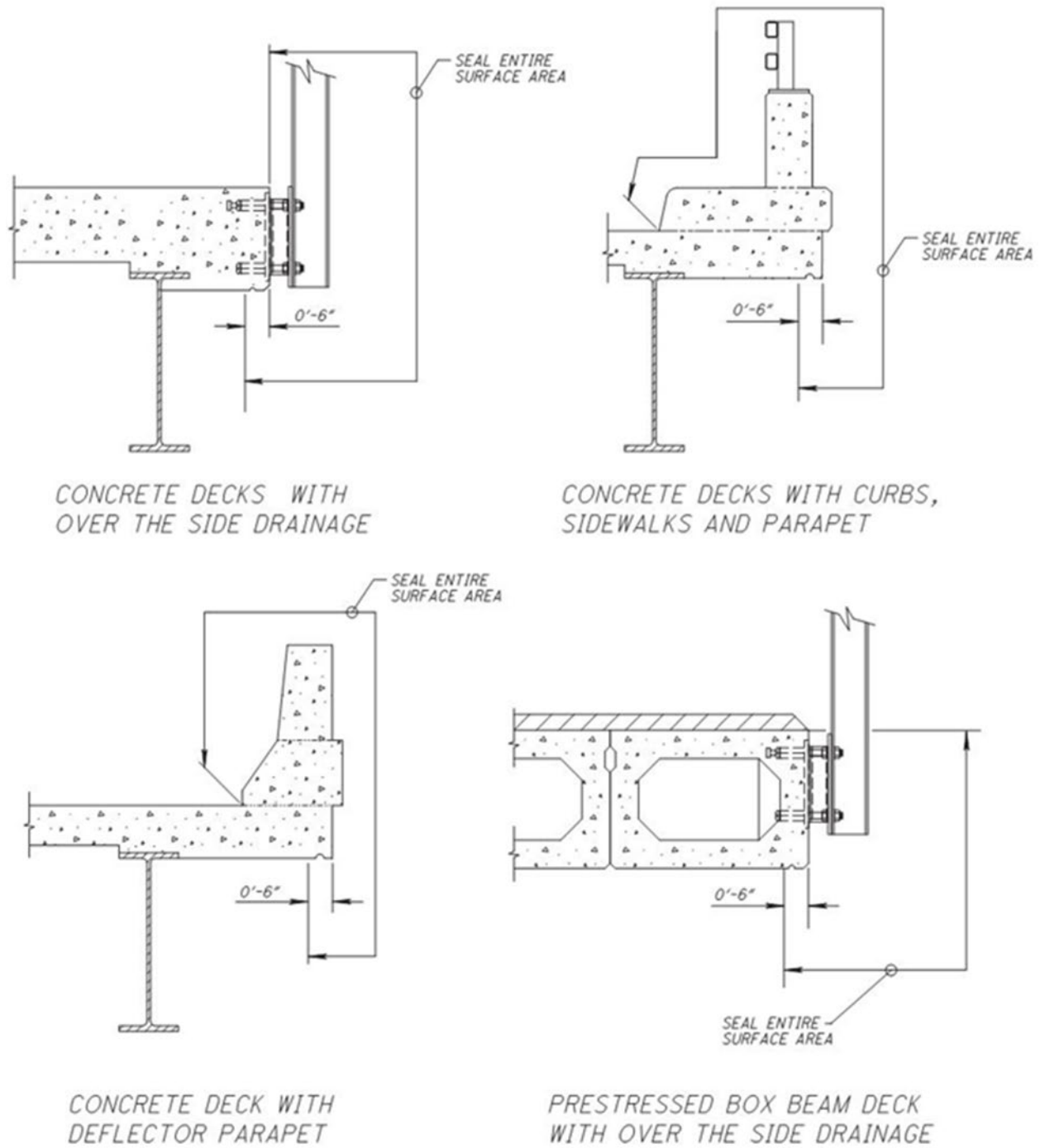
Do not use epoxy-only sealers.

Epoxy-only sealers have proven to perform poorly.



SEALING OF CONCRETE SURFACES, SUPERSTRUCTURE

Figure 309-1



SEALING OF CONCRETE SURFACES, SUPERSTRUCTURE

Figure 309-2

309.3 REINFORCED CONCRETE DECK ON LONGITUDINAL MEMBERS

309.3.1 DECK THICKNESS

For reinforced concrete decks on steel or concrete longitudinal members, compute the deck thickness with the following formula:

$$T_{\min} (\text{inches}) = (S + 17)(12) \div 36 \geq 8.5\text{-in}$$

Where S is the effective span length in feet determined according to *LRFD* 9.7.2.3.

Round up $T_{\min}$ to the nearest 1/4-in.

The 1-in monolithic wearing surface, BDM Section 309.1A, is included in the minimum concrete deck thickness but excluded in the calculations for structural design of the deck slab.

309.3.2 CONCRETE DECK DESIGN

Design the concrete deck design in conformance with the approximate elastic methods of analysis specified in *LRFD* 9.7.3, and the additional requirements specified in this Manual.

Do not use refined methods of analysis or the empirical design method, *LRFD* 9.7.2.

Use HL-93 for the design live load include an allowance for a future wearing surface equal to 0.06-ksf in the design dead load.

Place reinforcement in the underside of the deck overhang as noted in BDM Figure 309-4 and Figure 309-5. Use 1.5-in clear cover measured to the transverse steel.

Meet BDM Section 309.3.4.2. for the transverse spacing of the top and bottom reinforcing in a deck design.

C309.3.2

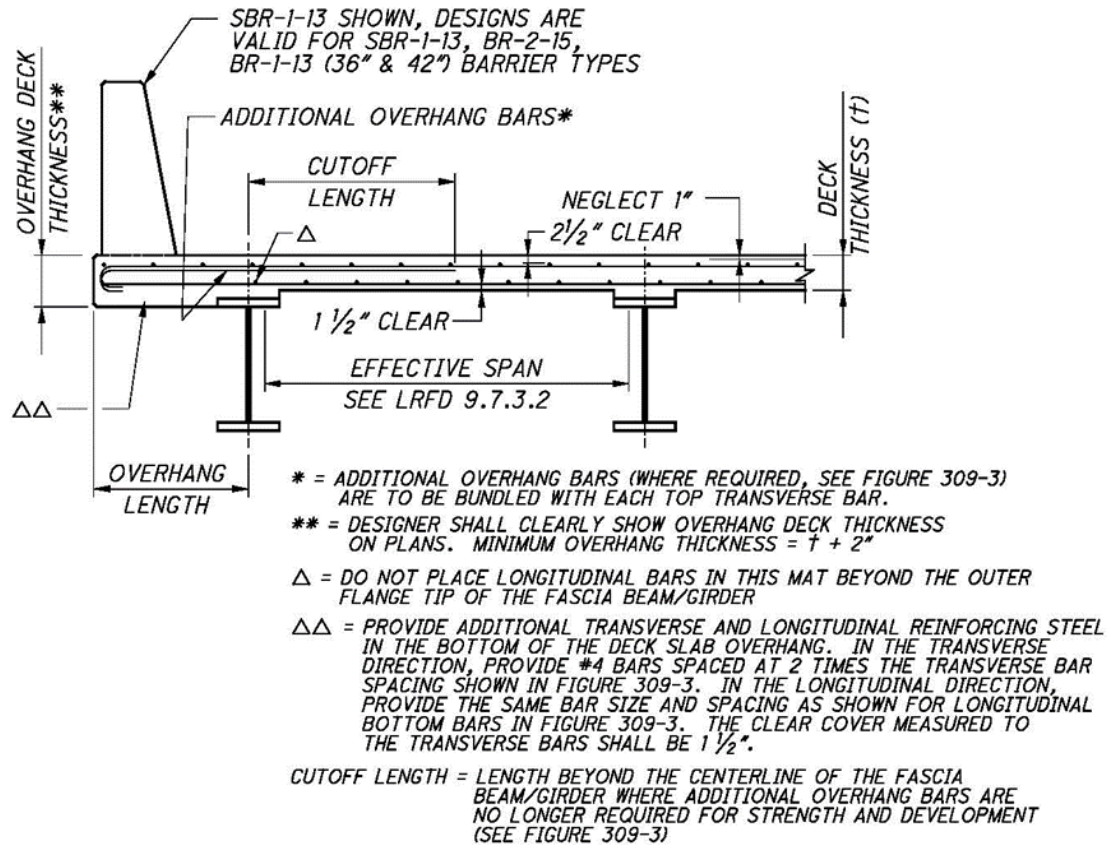
Deck designs for superstructures with effective span lengths ranging from 7.0-ft. to 14.0-ft. in 0.5-ft. increments are provided in BDM Figure 309-3, Figure 309-4 and Figure 309-5. These designs apply for the full length of the bridge and preclude the need for additional transverse reinforcement at supported deck ends. The design of overhang reinforcement is valid for [BR-1-13](#), [SBR-1-20](#), [SBR-2-20](#), [SBR-3-20](#), [BR-2-15](#), [TST-1-99](#) and [TST-2-21](#) railing systems. A complete list of design assumptions is provided with BDM Figure 309-3.

Eff. Span Length (ft.)	Deck Thick (in.)	Overhang Deck Thick. (in.) (see Note 2k)	Transverse Steel						Longitudinal Steel			
			Top Bars				Bottom Bars		Top Bars		Bottom Bars	
			Size	Spa. (in.)	Additional Overhang Bar Size	Cutoff Length (in.)	Size	Spa.	Size	Spa.	Size	Spa.
7.0	8.50	10.50	#5	6.00	#5	54.0	#5	6.00	#4	12.50	#5	10.75
7.5	8.50	10.50	#5	6.00	#5	54.0	#5	6.00	#4	12.00	#5	10.25
8.0	8.50	10.50	#5	6.00	#5	54.0	#5	6.00	#4	11.50	#5	9.75
8.5	8.50	10.50	#5	5.75	#4	54.0	#5	5.75	#4	11.00	#5	9.25
9.0	8.75	10.75	#5	5.75	#4	54.0	#5	5.75	#4	11.00	#5	9.25
9.5	9.00	11.00	#5	5.75	#4	54.0	#5	5.75	#4	11.00	#5	9.25
10.0	9.00	11.00	#5	5.25	#4	48.0	#5	5.25	#4	10.00	#5	8.75
10.5	9.25	11.25	#5	5.25	#4	48.0	#5	5.25	#4	10.00	#5	8.75
11.0	9.50	11.50	#5	5.00	#4	48.0	#5	5.00	#4	9.50	#5	8.75
11.5	9.50	11.50	#6	5.75	#4	28.0	#5	5.75	#4	7.75	#5	8.75
12.0	9.75	11.75	#6	5.75	#4	28.0	#5	5.75	#4	7.75	#5	8.75
12.5	10.00	12.00	#6	5.75	--	--	#5	5.75	#4	7.75	#5	8.75
13.0	10.00	12.00	#6	5.75	--	--	#5	5.75	#4	7.75	#5	8.75
13.5	10.25	12.25	#6	5.75	--	--	#5	5.75	#4	7.75	#5	8.75
14.0	10.50	12.50	#6	5.75	--	--	#5	5.75	#4	7.75	#5	8.75

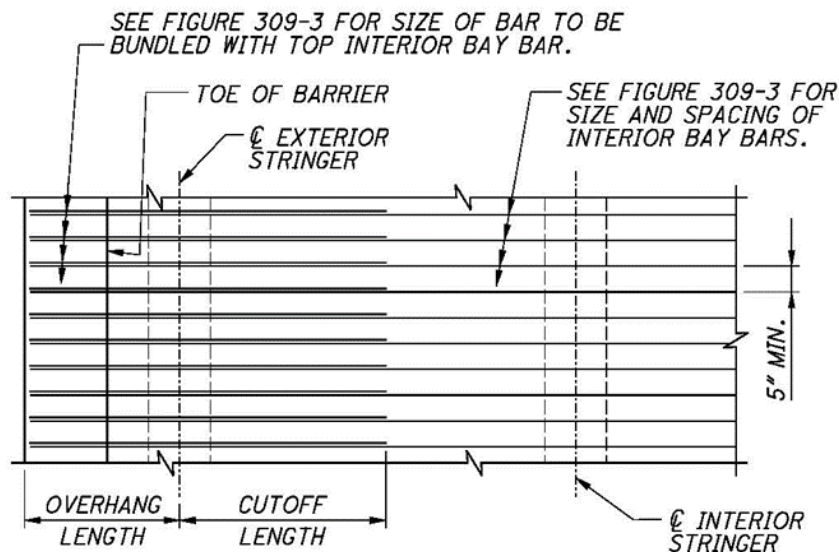
Notes:

1. Designs in accordance with AASHTO *LRFD Bridge Design Specifications* and the ODOT Bridge Design Manual
2. Design Assumptions:
 - a. Four or more beam/girder lines
 - b. Maximum beam/girder spacing = 15-ft. c/c
 - c. Transverse steel is placed perpendicular to beam/girder lines
 - d. Normal weight concrete with $f'_c = 4.5$ -ksi
 - e. Reinforcing steel with $f_y = 60$ -ksi
 - f. Monolithic Wearing Surface = 1.0-in.
 - g. Future Wearing Surface = 0.06-ksf
 - h. *LRFD* 5.6.7 - Exposure Factor (γ_e) = 0.75
 - i. Top cover = 2.50-in.; Bottom cover = 1.50-in.
 - j. Maximum overhang width = 4.0-ft. (measured from cl. of fascia beam/girder to deck edge)
 - k. Valid for [BR-1-13](#) (36" & 42"), [SBR-1-20](#), [SBR-2-20](#), [SBR-3-20](#), [BR-2-15](#), [TST-1-99](#) and [TST-2-21](#) railing systems. The minimum overhang deck thickness for the [TST-1-99](#) and [TST-2-21](#) railing systems are 18" and 20" respectively. Calculate Effective Span Length according to *LRFD* 9.7.3.2 and round up to the nearest 0.5-ft. increment
4. Minimum Deck Thickness in accordance with BDM 309.3.1
5. Cutoff Length = length beyond the centerline of the fascia beam/girder where additional overhang bars are no longer required for strength and development.
6. Longitudinal bar spacing does not include additional reinforcing required for negative moments in accordance with *LRFD* 5.6.3.2 (for prestressed beams) and *LRFD* 6.10.1.7 (for steel beams/girders & prestressed beams)
7. Refer to BDM Figure 309-4 & Figure 309-5 for more information

Figure 309-3



DECK SECTION



OVERHANG PLAN VIEW

TYPICAL DECK DETAILS ODOT LRFD STANDARD DECK DESIGN

Figure 309-4

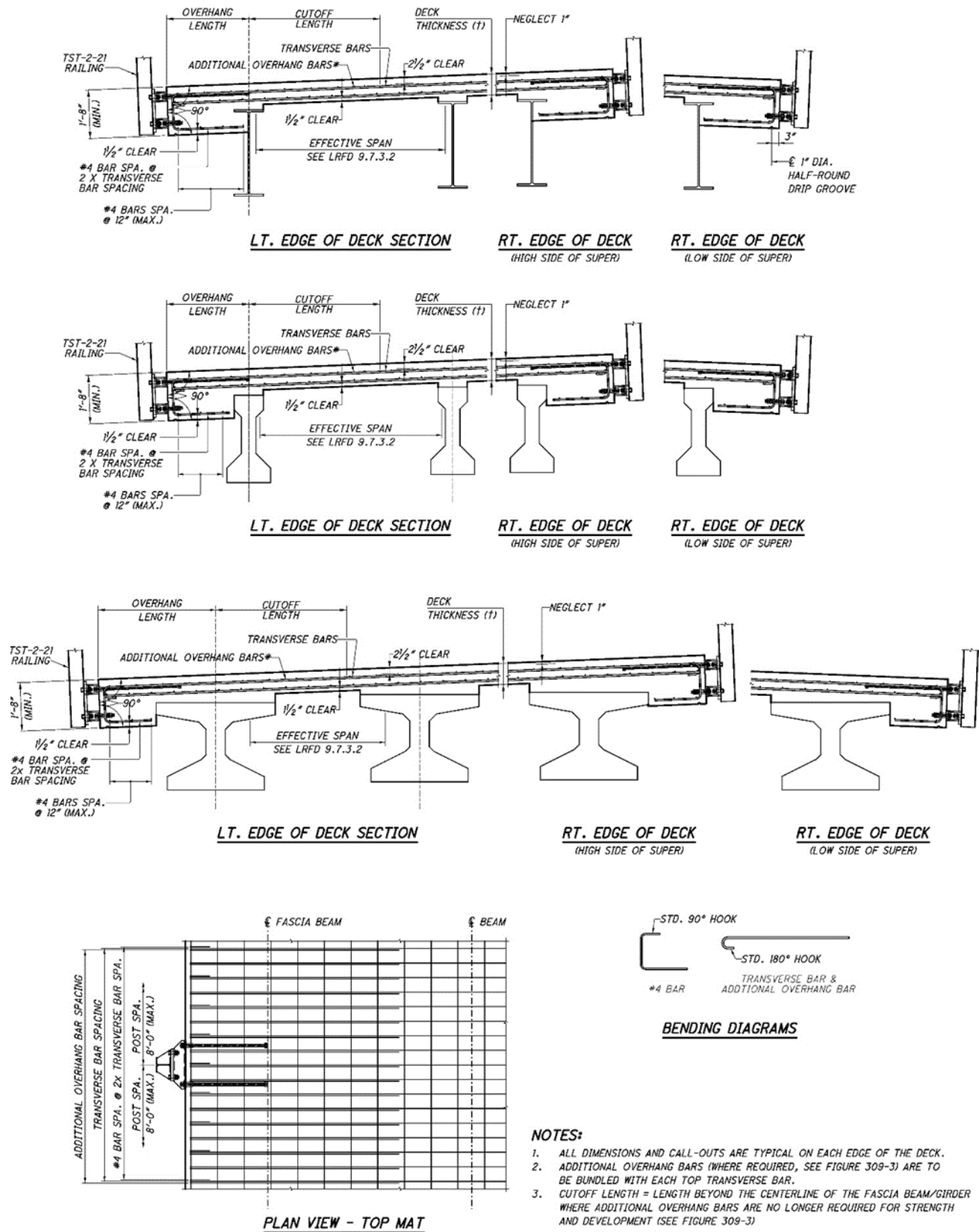


Figure 309-5

309.3.3 DECK ELEVATION REQUIREMENTS

309.3.3.1 SCREED ELEVATIONS

Do not include adjustment for deflections due to the future wearing surface loading in screed elevations. Assume a completed placement sequence to calculate deflections caused by the weight of the deck concrete. Use deflection data from girder lines closest to each screed line to determine elevations. Refer to BDM Figure 309-6.

If the deflections are determined through a line girder analysis method, distribute the deck load 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.

Include a screed elevations table in the bridge plans. Identify the locations of all screed elevations in the table on a transverse section and plan view. Provide elevations 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. Detail bearing points, quarter-span points, mid-span points and splice points as well as any additional points required to meet a maximum spacing between points of 25-ft.

For bridges with a separate wearing course, give the elevations at the top of the portland cement concrete deck. Provide a plan note stating at what surface the elevations are given.

When calculating screed elevations for composite box beam bridges, provide the same requirements as steel beam, girder and prestressed I-beam bridges.

309.3.3.2 TOP OF HAUNCH ELEVATIONS

Provide top of haunch elevations at the centerline of each girder at bearing points, quarter points, mid-span points, splice points and additional points to meet a maximum spacing between points of 25-ft. Identify the top of haunch elevation locations in a plan view and on the transverse section. Provide a plan note for a definition and description of the purpose for the top of haunch elevations (see BDM Section 702.12.2). Refer to BDM Figure 309-6.

C309.3.3.1

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 are not required for non-composite box beam bridges or slab bridges.

C309.3.3.2

Top of haunch elevations represent the theoretical bottom of deck elevation before the concrete deck is placed.

Top of haunch elevations are not required for composite box beam bridges.

309.3.3.3 FINAL DECK SURFACE ELEVATIONS

Provide final deck surface elevations at bearing points, quarter points, mid-span points, splice points and additional points to meet a maximum spacing between points of 25-ft for each: girder centerline; curbline or deck edge; transverse grade-break line; and phased construction line. Identify the final deck surface elevation locations in a plan view. Refer to BDM Figure 309-6.

C309.3.3.3

Final deck surface elevations represent the position of the deck after all dead loads except future wearing surface have been applied.

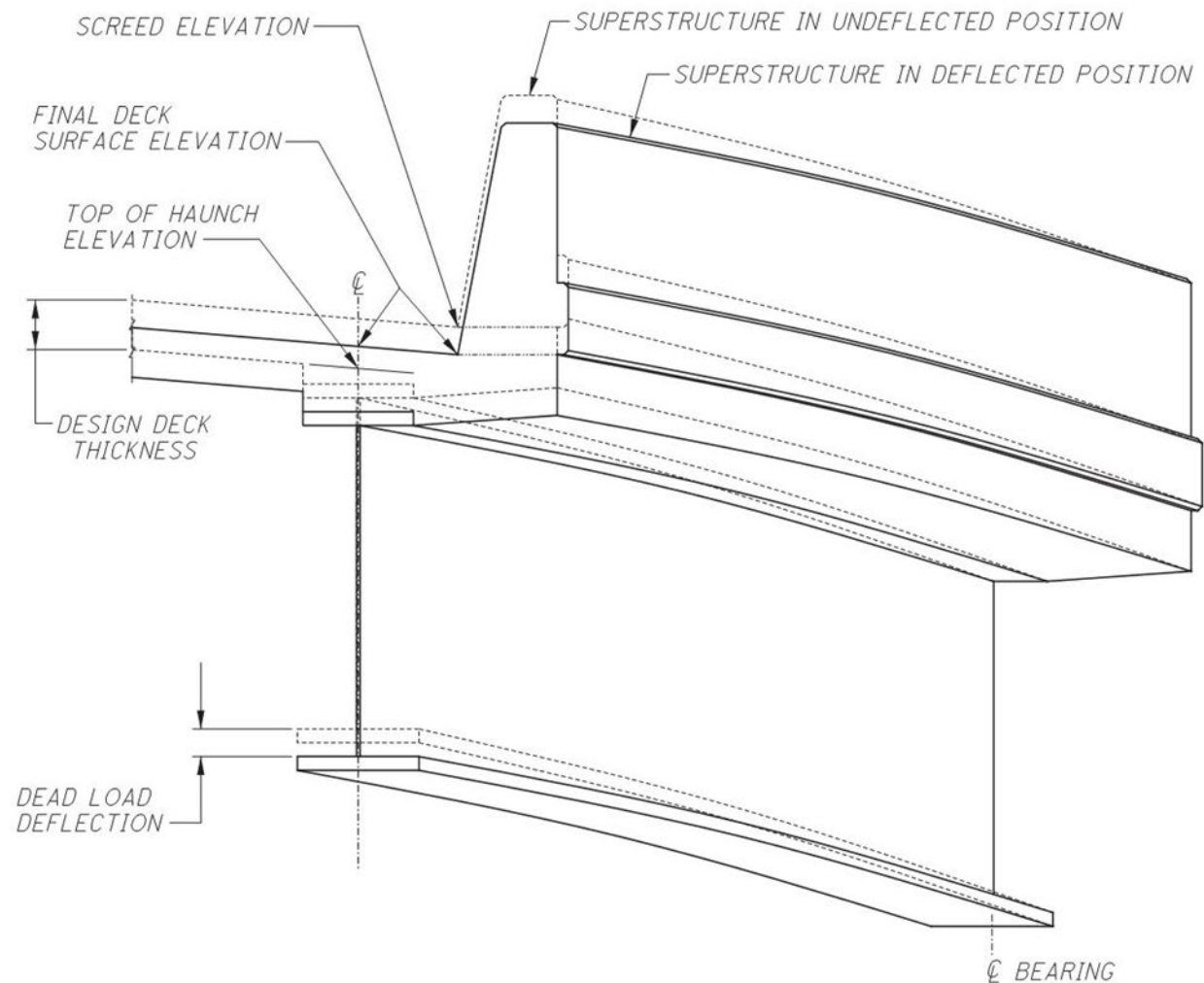


Figure 309-6

309.3.4 REINFORCEMENT**309.3.4.1 LONGITUDINAL**

On reinforced concrete decks for steel or concrete stringers, design the secondary reinforcement in the top-reinforcing layer to be a minimum of 1/3 the main reinforcement, spaced uniformly.

Detail the secondary bar size as a #4. The only exception to this requirement is if the bar spacing becomes less than 3-in.

For stringer type bridges with reinforced concrete decks, place the secondary bars above the top of deck primary bars.

For steel or concrete stringers, meet the requirements of *LRFD 6.10.1.7* for the longitudinal reinforcement over a pier. Stagger every other reinforcing bar over the pier 3-ft longitudinally.

309.3.4.2 TRANSVERSE

Place the top and bottom main reinforcement in transversely reinforced deck slabs at equal spaces to coincide in a vertical plane.

Bridges with a skew equal to or greater than 15-degrees or where the transverse reinforcing will interfere with the shear studs, place the transverse reinforcement perpendicular to the centerline of the bridge. Refer to Standard Bridge Drawing [CS-1-24](#) or [SB-1-24](#) for the requirements on slab bridges.

For prestressed I-beams, place transverse reinforcing perpendicular to the centerline of the bridge.

For composite box beam decks, place the transverse concrete reinforcement parallel to the abutment.

For steel beam or girder bridges, check the clearance of the bottom transverse bars over the top of bolted beam splice plates or moment plates.

309.3.5 HAUNCHED DECK REQUIREMENTS

Use a concrete haunch on concrete decks on steel beam, girder or prestressed I-beam structures. The minimum design haunch is 2-in. Detail the sides of the haunch as vertical and aligned with the edges of the top flange. See BDM Figure 309-7 & Figure 309-8.

C309.3.4.1

Research has shown that secondary bars in the top mat of reinforced concrete bridge decks on stringers should be small bars at close spacing.

This helps in reducing shrinkage cracking and adds additional cover over the primary bars.

C309.3.4.2

This requirement is to help facilitate the placement of concrete reinforcement and concrete.

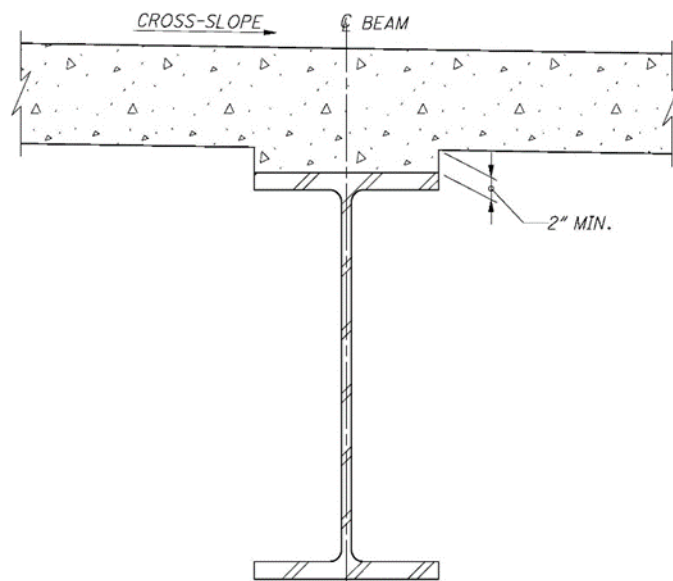
For steel beam or girder bridges with a skew of less than 15-degrees the transverse reinforcing may be shown placed parallel to the abutments.

For phased constructed bridges where mechanical connectors will be required placing the bars parallel to the centerline of bearing should be avoided.

Reinforcing bars at a skew generally cannot be placed between bolt heads.

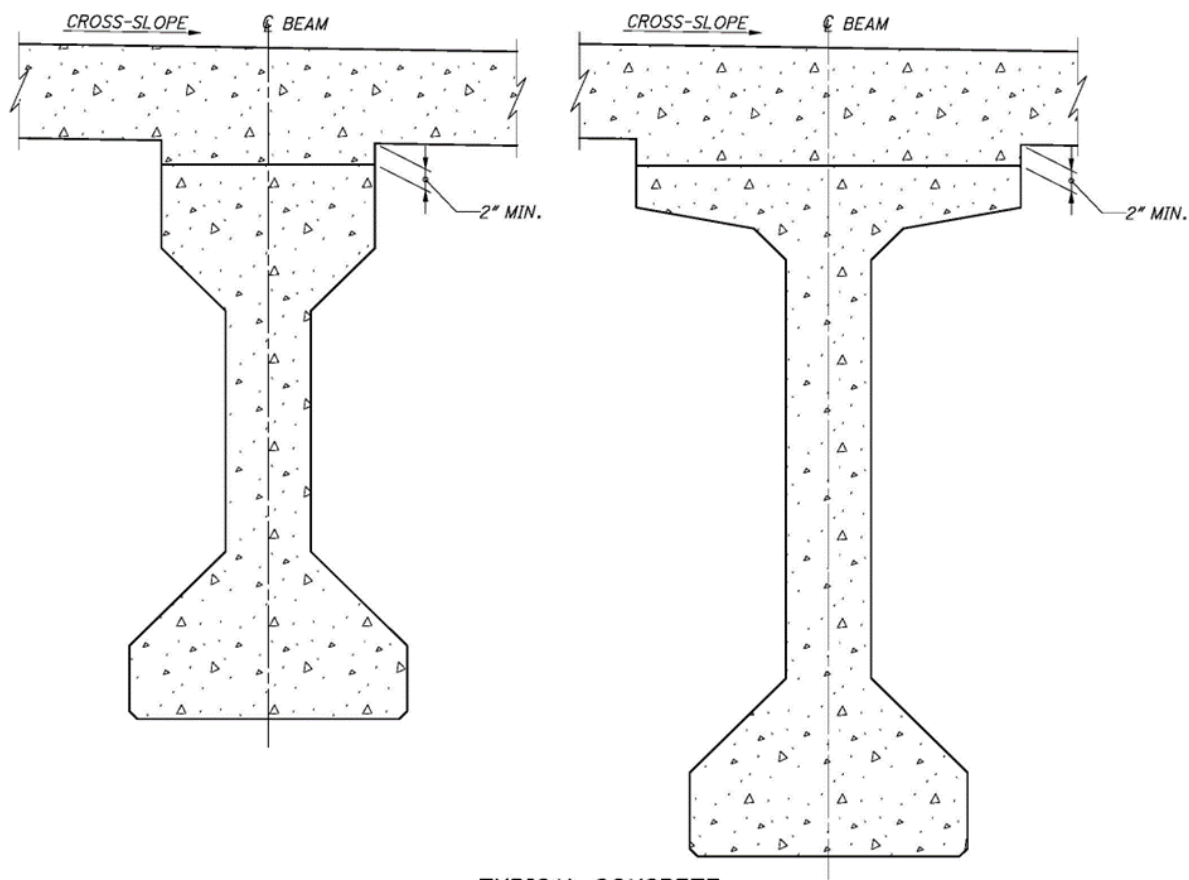
C309.3.5

Haunches are used to prevent a thinning of the deck slab as a result of unforeseen variations in beam camber.



TYPICAL CONCRETE
DECK HAUNCH DETAIL

Figure 309-7



TYPICAL CONCRETE
DECK HAUNCH DETAILS

Figure 309-8

309.3.6 TRANSVERSE DECK SECTION

Include the following information on the transverse deck section: deck thickness, crown location, cross slope, railing type, construction joints, out-to-out of the deck dimension, face/face railing dimension, reinforcing labels, reinforcing cover, and reinforcing lap location and type.

309.3.6.1 SUPERELEVATION**C309.3.6.1**

When the change in cross slope at the superelevation break point is greater than 7 percent, detail the bridge deck surface profile as follows:

- A. When the roadway break point is located between roadway lanes (not at the edge of pavement), extend the bridge cross slope to extend to the toe of the concrete bridge railing. See “CASE a” in BDM Figure 309-9.
- B. When the roadway break point is located at the edge of pavement (adjacent shoulder width is less than 4-ft), continue the bridge cross slope past the break point to the toe of the concrete bridge railing. See “CASE b” in BDM Figure 309-9.
- C. When the roadway break point is located at the edge of pavement (adjacent shoulder width is equal to or greater than 4-ft and less than 8-ft), use a 4-ft rounding distance from the edge of lane onto the shoulder to transition from the bridge cross slope to the 1/2-in. per ft. shoulder cross slope. See “CASE c” in BDM Figure 309-10.
- D. When the roadway break point is located at the edge of pavement (adjacent shoulder width is equal to or greater than 8-ft), use a 5-ft rounding distance from the edge of lane onto the shoulder to transition from the bridge cross slope to the 1/2-in. per ft. shoulder cross slope. See “CASE d” in BDM Figure 309-10.

If the change in cross slope at the superelevation break point is less than or equal to 7 percent, then no rounding is required.

Transition from the roadway approach transverse section to the bridge deck transverse section according to [L&D, Vol. 1](#), Section 301.2.4.

For decks with over-the-side drainage, the treatment of the deck and the shoulder slopes shall be as described in subsections a through d above except that the slope shall continue to the edge of the deck.

TRANSVERSE DECK SECTIONS
FOR SUPERELEVATED SITES

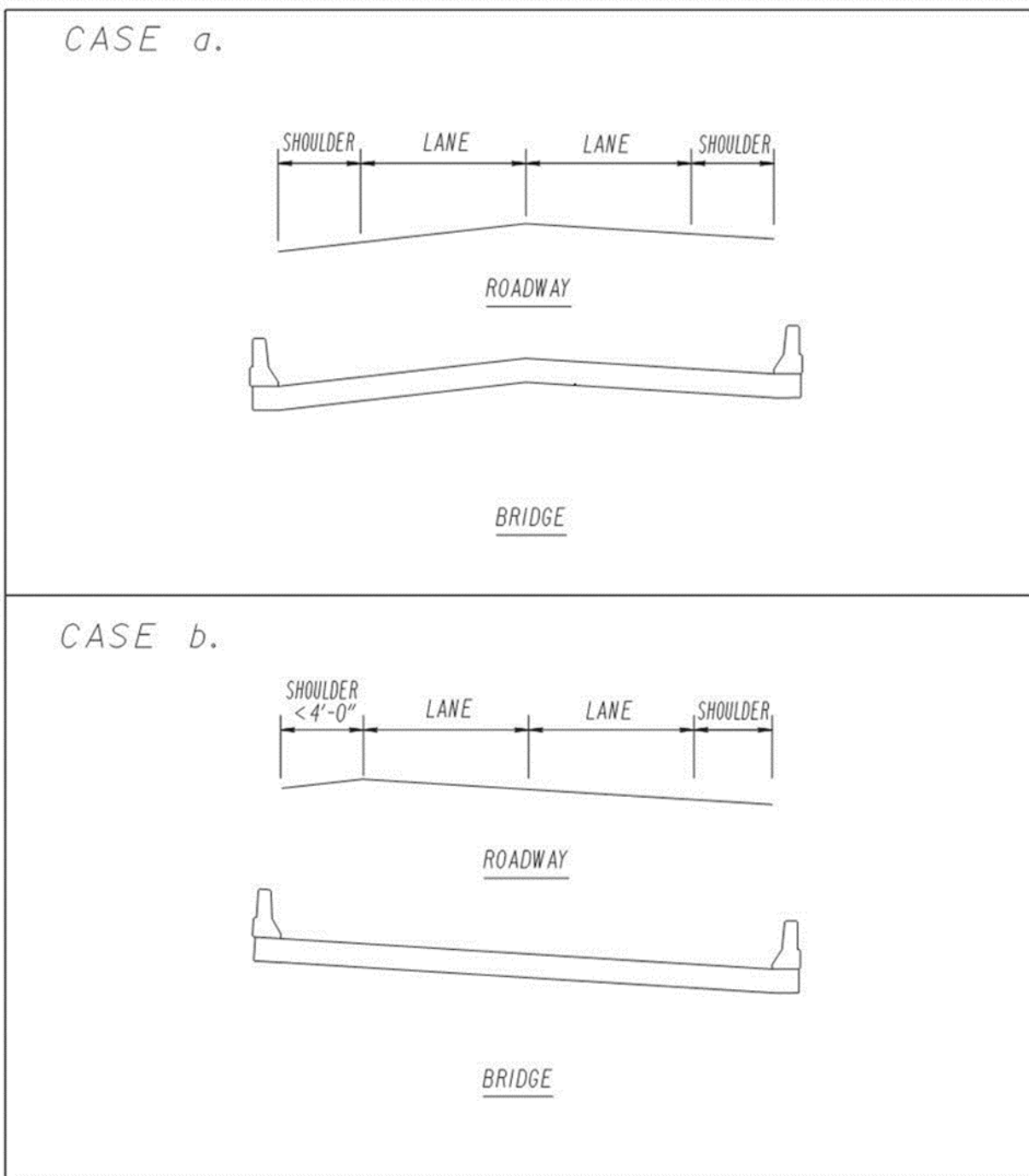


Figure 309-9

TRANSVERSE DECK SECTIONS
FOR SUPERELEVATED SITES

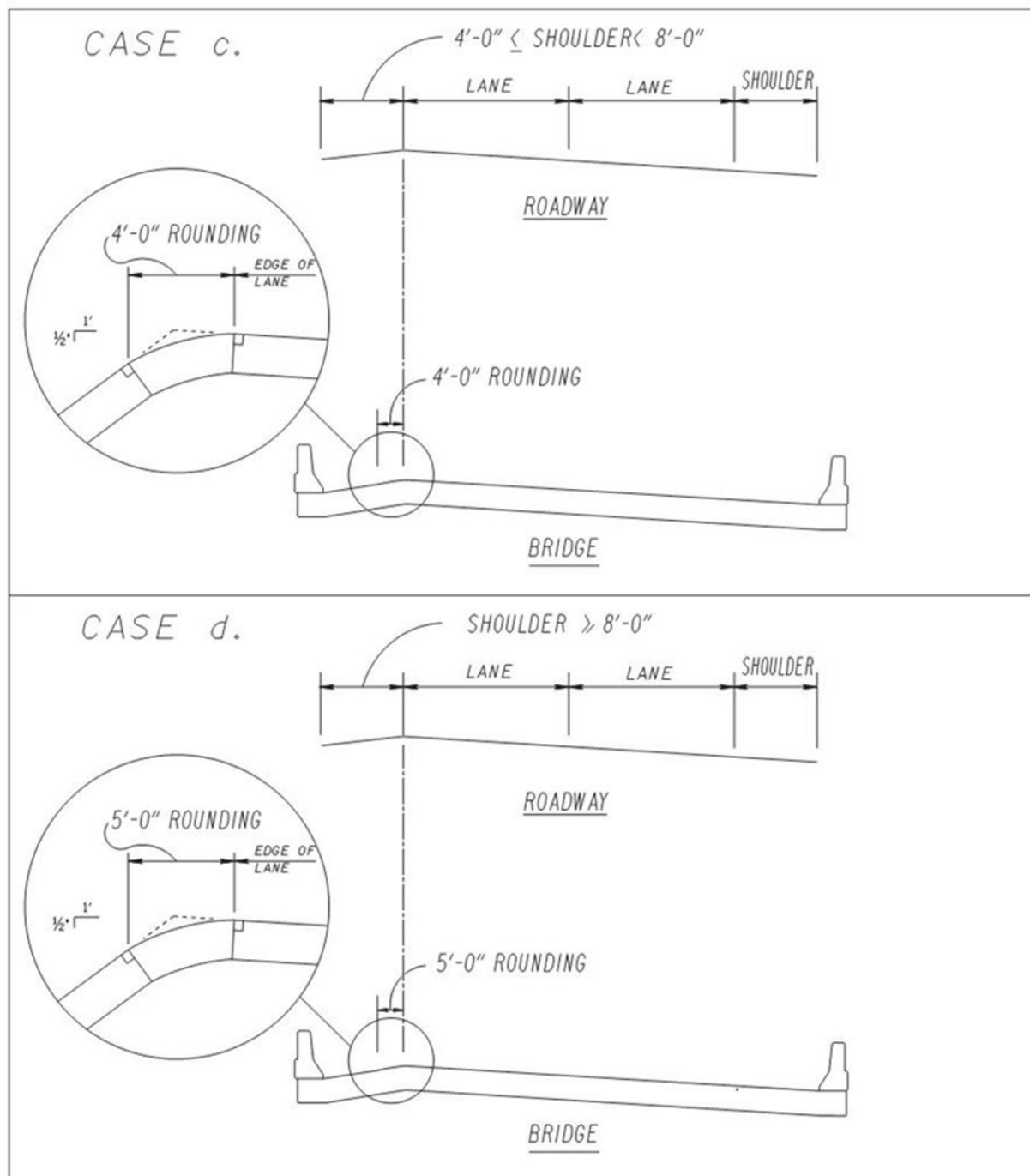


Figure 309-10

309.3.6.1.a SUPERELEVATION TRANSITIONS

Provide a transition diagram in the plans, similar to what is shown in BDM Figure 309-11. In lieu of the diagram, a table with the transition information shown in BDM Figure 309-11 is also acceptable.

C309.3.6.1.a

Because of the complexities associated with superelevation transitions on bridge superstructures (e.g. beam and girder cambering, cross-frame fabrication, deck form construction, slip forming of concrete bridge railing, etc.) all reasonable attempts should be made to keep such transitions off of bridge decks. Where transitions are located on bridge decks, the transitions should be straight. Where this is not possible, the transition's discontinuities should be smoothed by inserting 50-ft roundings at each discontinuity.

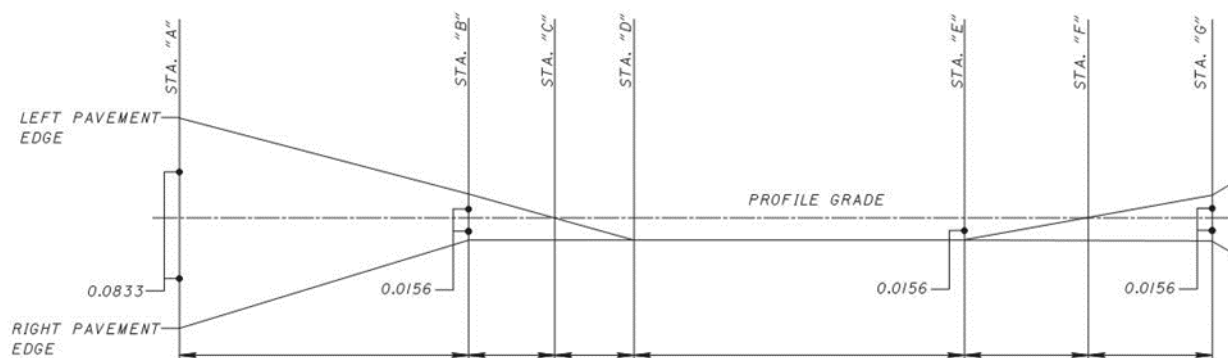


Figure 309-11

309.3.7 STAY IN PLACE FORMS

Do not use stay in place forms.

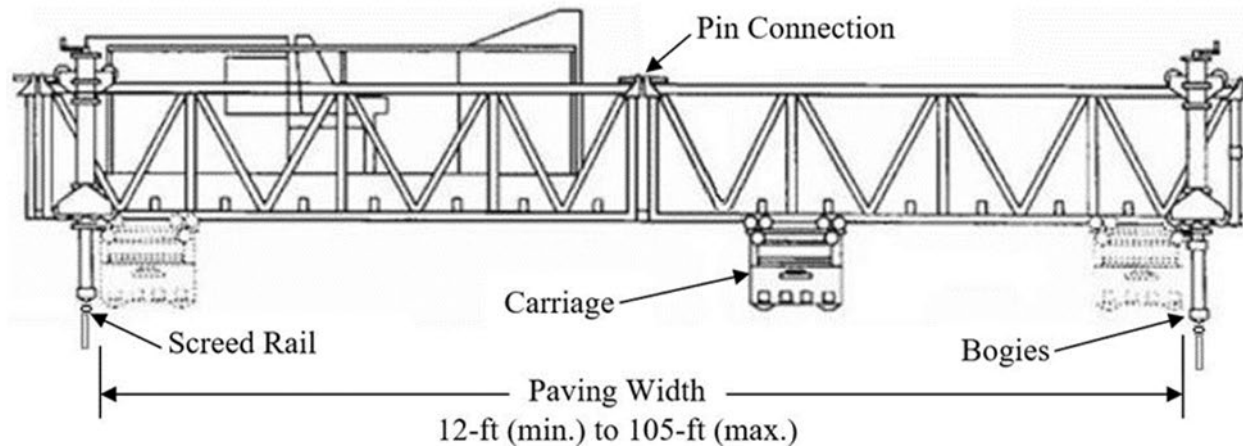
309.3.8 CONCRETE DECK PLACEMENT CONSIDERATIONS**C309.3.8**

Mechanized finishing machines are preferred to hand finishing methods for both consistency of surface finish and economics. Designers should be aware of finishing machine limitations in order to avoid deck designs that require hand finishing methods.

The placement of deck concrete using mechanized finishing machines alone does not ensure a smooth riding surface. Achieving a smooth riding surface as well as ensuring the proper geometry of the concrete deck is further complicated by deflections of the concrete falsework and of the main structural support members during the placement operation. The Contractor is responsible for designing falsework and finishing machine support to minimize deflection during placement, but the Designer is responsible for deflections induced by deck placement on the superstructure. Many complications due to deflection during placement can be avoided with proper design considerations.

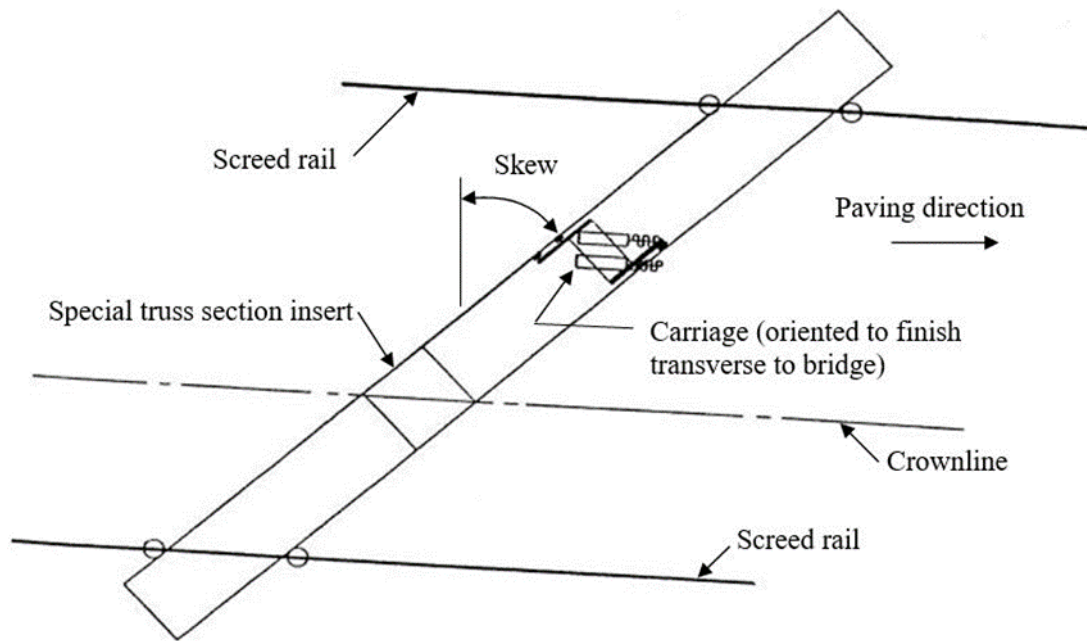
309.3.8.1 FINISHING MACHINES**C309.3.8.1**

Mechanized finishing machines are comprised of fabricated truss sections pinned together to span the bridge deck width to be paved. The truss spans are supported at each end on a set of wheels, called “bogies,” which ride along the length of the bridge on screed rails. Suspended below the truss is a finishing head, called a “carriage,” which levels, compacts, vibrates and finishes the concrete. See BDM Figure 309-12.

**Figure 309-12**

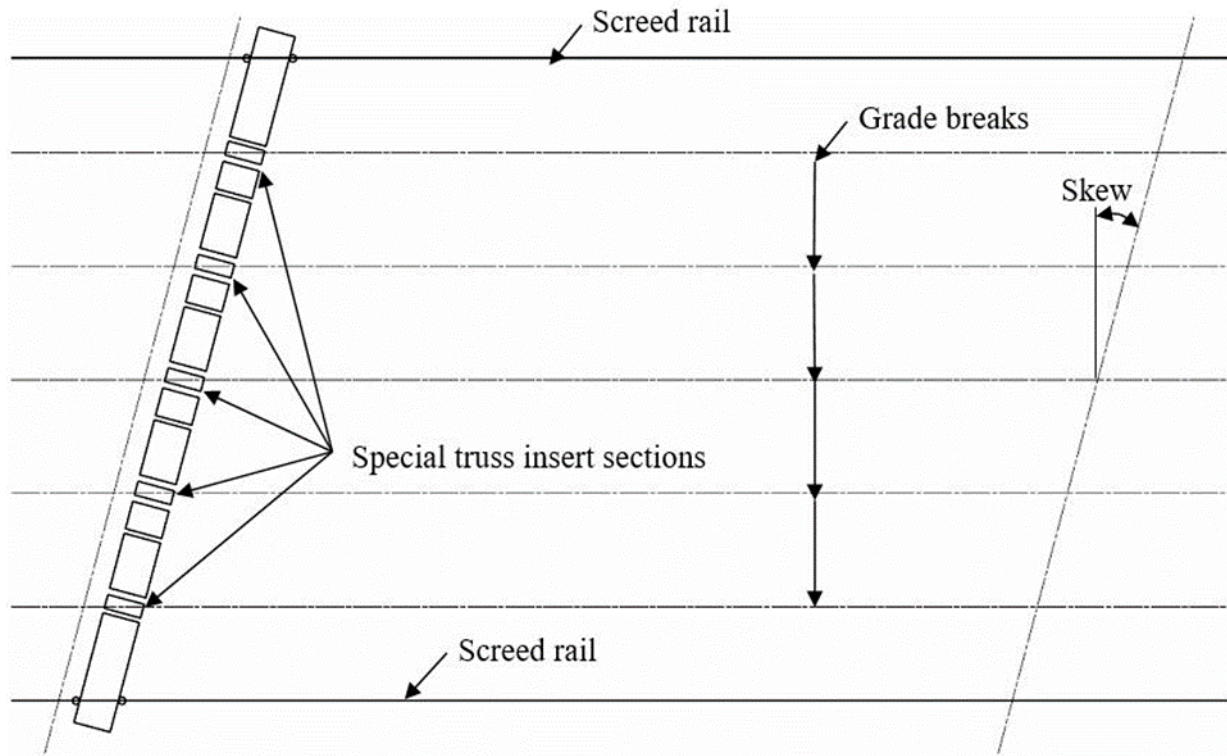
Finishing machines can be placed such that the truss sections are skewed with respect to the screed rails. This orientation allows for concrete placement parallel to the substructure skew as required by the [C&MS 511](#). For skew angles of 15-degrees and greater, the finishing machine can be skewed to within 5-degrees of the plan specified skew angle.

The carriage can also be skewed with respect to the truss sections. This feature allows the carriage to finish the concrete transverse to the bridge when the truss sections are placed at some other orientation (e.g. parallel to the substructure skew). To ensure a proper finish at transverse grade breaks (e.g. crown points), the carriage should always be oriented to finish the concrete transverse to the bridge. A special length truss section insert is required above the grade break locations such that the grade break line lies directly below opposite corners of the section. For skewed bridges without transverse grade breaks, skewing the carriage with respect to the truss sections is not required. (See BDM Figure 309-13)

**Figure 309-13**

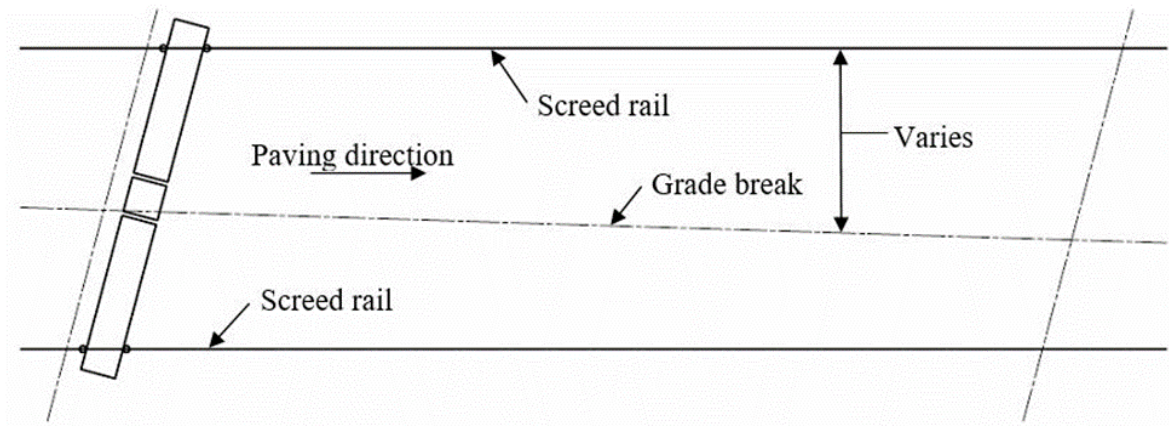
Most finishing machines do not easily accommodate non-parallel rails. The distance between the screed rails should be a fixed width. Designs that require tapered paving widths should be avoided.

The finishing machines can be hinged at the pin connections between truss sections to provide transverse grade breaks (e.g. crown points). In theory, multiple transverse grade breaks can be accommodated, but the grade breaks must remain at a fixed spacing to line up with a pin connection. The BDM Figure 309-14 illustrates the complexity of the machine set-up to accommodate multiple grade breaks in a transverse section placed on a skew. Note that the length of truss sections required between grade breaks must fit the standard truss section lengths.

**Figure 309-14**

Avoid grade break locations that move laterally along the length of the bridge.

This arrangement cannot be paved in a single operation using a mechanized finishing machine. The only way this is possible is with the use of a phase construction deck. Note that as the machine progresses forward, the truss hinge locations and the grade break locations no longer coincide. See BDM Figure 309-15.

**Figure 309-15**

309.3.8.2 SOURCES OF GIRDER TWIST

C309.3.8.2

Fully install cross-frames/diaphragms prior to deck placement.

The interconnectivity between girders, intermediate cross-frames/diaphragms and end cross-frames/diaphragms is essential to a structure's stability throughout the construction process. Therefore, it is of

utmost importance to ensure that all cross-frames/diaphragms are fully installed prior to deck placement. Failure to do so may lead to construction disputes, expensive repairs and lengthy construction delays or even impact project safety. One major drawback to this interconnectivity is that the deflection caused by the placement of the concrete deck will result in girder twisting.

There are primarily three independent sources of girder twist resulting from deck placement. This manual will refer to these sources as: global superstructure distortion, oil-canning and girder warping.

309.3.8.2.a GLOBAL SUPERSTRUCTURE DISTORTION

C309.3.8.2.a

Global superstructure distortion is distortion of the bridge transverse section primarily caused by differential deflections between adjacent girders. As a girder deflects downward with respect to an adjacent girder, the rigidity of the cross framing between the two girders causes the deflecting girder to rotate as it deflects. This distortion may occur with both steel and prestressed concrete superstructures. The most common differential deflections occur between the exterior girders and adjacent interior girders for a given construction phase when the loaded tributary areas over the girders differ.

Transverse sections with more heavily loaded exterior girders distort in a convex shape. Refer to BDM Figure 309-16.

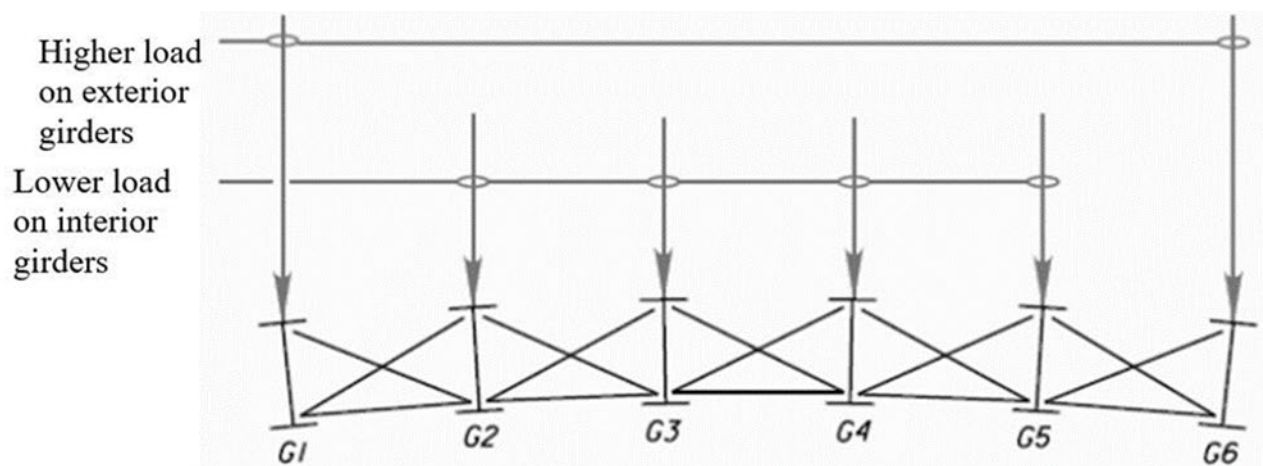
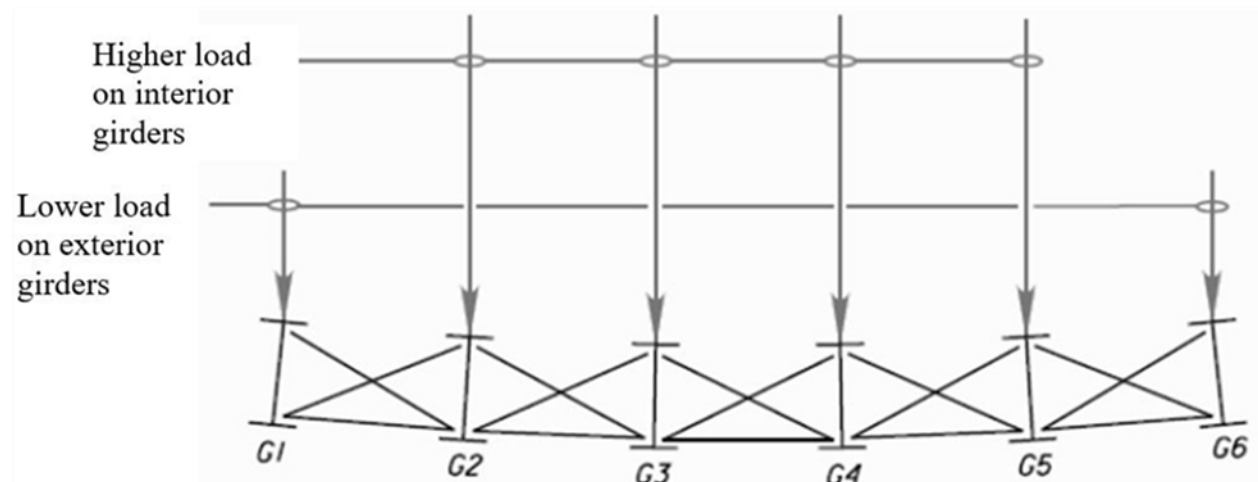
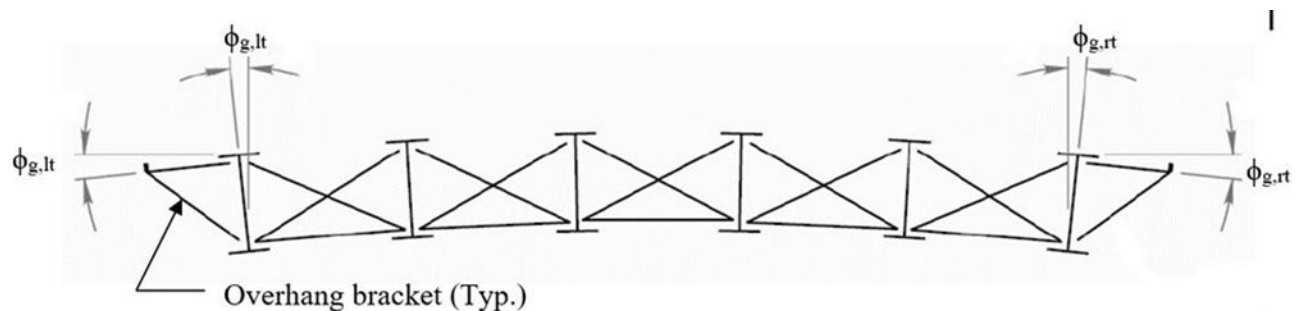


Figure 309-16

Transverse sections with more heavily loaded interior girders distort in a concave shape. Refer to BDM Figure 309-17.

**Figure 309-17**

Twisting of the exterior girders can result in deck thickness and cover loss if the screed rails are supported on cantilevered falsework. The magnitude of girder twist (measured as ϕ_g) will vary over the length of the bridge and will be different for the left and right sides if loading or geometry is not symmetrical. See BDM Figure 309-18.

**Figure 309-18**

For a new superstructure, calculate the amount of girder twist due to global superstructure deformation when the tributary deck load carried by the fascia girder exceeds 110% of the average of the tributary deck load carried by the interior members for a given construction phase.

For an existing superstructure, calculate the amount of girder twist due to global superstructure deformation when the tributary deck load carried by the fascia girder exceeds 115 % of the average of the tributary deck load carried by the interior members for a given construction phase. See BDM Figure 309-19.

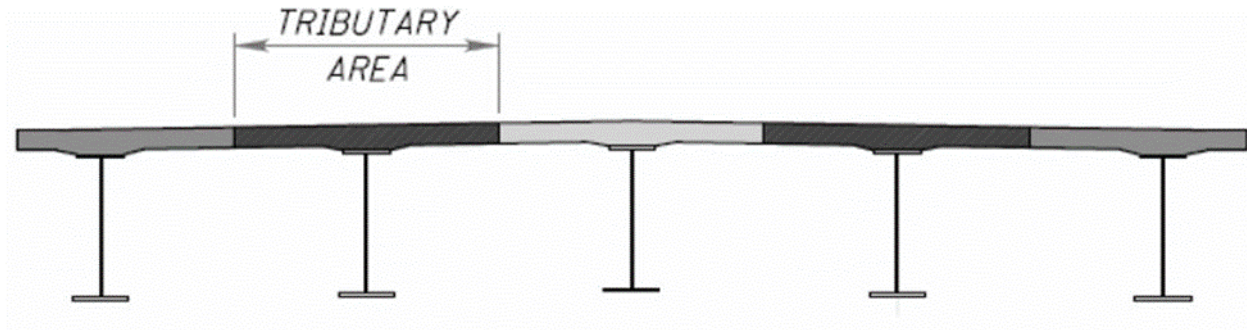


Figure 309-19

When the aforementioned tributary deck loading requirements of the fascia members are met, perform a refined analysis of the superstructure system to determine the magnitude of fascia girder twist (ϕ_g) due to deck concrete placement.

To properly calculate the effect of the twist angle on deck thickness, the analysis should be based on the deflection occurring due to the concrete present at the time that the finishing machine passes over the point under consideration. This degree of precision requires a separate refined analysis for each point of consideration. It is generally sufficient to calculate ϕ_g based on the full wet concrete load placed over the entire structure. However, on complex structures with variable skews and/or curved girders, a higher degree of precision may be warranted to ensure proper deck thickness.

Additional measures to reduce global deformation include adding or stiffening the cross-frames/diaphragms; and increasing the stiffness of the girders. An increase in the cross-frame stiffness results in better load distribution across the width of the structure and less distortion. An increase in the stiffness of the girders reduces the magnitude of vertical deflection resulting in less distortion of the transverse section.

309.3.8.2.b OIL-CANNING**C309.3.8.2.b**

Distortion due to oil-canning occurs when large lateral loads from the cantilevered deck slab falsework bracket deform the girder web. See BDM Figure 309-20.

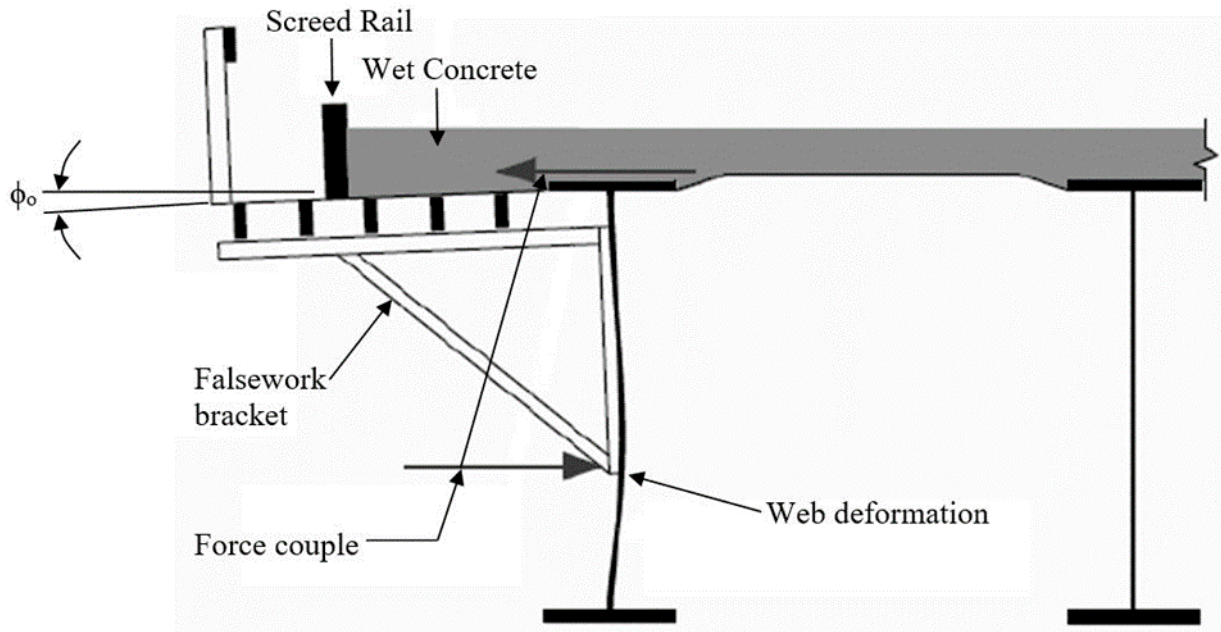


Figure 309-20

Locating the falsework bracket near the bottom flange will reduce the amount of web deformation. [C&MS](#) Item 508 requires the lower point of contact to be within 8-in of the top of the bottom flange. Given this requirement and the geometric capabilities of the falsework brackets, the magnitude of girder twist (ϕ_o) resulting from oil-canning may be neglected for girder webs 84-in deep or less.

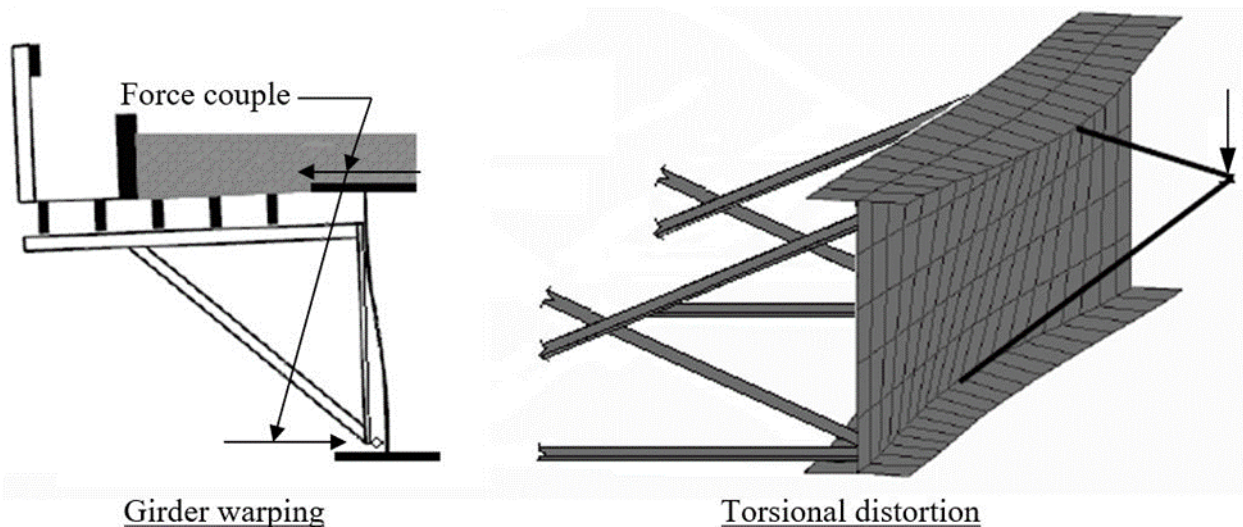
For web depths greater than 84-in, provide the location of the falsework bracket in the plans. Provide a General Note that removes the lower point of contact requirement of [C&MS](#) Item 508 (see BDM Section 611.4.1 for an example). Specify the pay item for deck concrete as “as per plan”. Using the plan bracket location, determine ϕ_o . Detail any temporary bracing in the plans.

Designers may assume the lowest location of the falsework bracket to be 76-in measured below the bottom of the top flange. The magnitude of twist can be predicted using finite element analysis of the web or by various approximate methods. If the magnitude results in excessive deck thickness loss, reducing the transverse stiffener spacing or adding temporary bracing on the inside of the web may be necessary.

The magnitude of girder twist resulting from oil-canning may be neglected for prestressed I-beam superstructures.

309.3.8.2.c GIRDER WARPING**C309.3.8.2.c**

Distortion due to girder warping occurs as a result of deck slab overhang falsework loading on the fascia girder between points of lateral bracing (e.g. cross-frames). The bracket loads produce twist between the cross-frames due to a combination of girder warping and pure torsional distortion (See BDM Figure 309-21). The girder is restrained from warping at the cross-frame locations. Due to the inherent torsional stiffness of prestressed I-beams, the distortion due to girder warping may be neglected. Other design considerations for I-beams due to the overhang bracket loadings are presented at the end of this section.

**Figure 309-21**

For steel superstructures, calculate the magnitude of twist (ϕ_w) due girder warping.

For design-build projects and value engineering change proposals (VECP's), represent the actual falsework and equipment to be used by the contractor in the data input for girder warping analysis.

The TAEG ("Torsional Analysis of Exterior Girders") software developed by the Kansas Department of Transportation. TAEG is available at no cost and can be downloaded at: <https://kart.ksdot.org/>.

Since most of the data input in the girder warping analysis is dependent upon the contractor's equipment and falsework design, designers should use conservative assumptions to accommodate most contractor resources.

Designers may use the following assumptions in lieu of actual contractor supplied information:

Detail any temporary tie rods and timber blocks in the contract plans.

A. Girder Data:

For bridges with constant web depths, designers may select the cross section with the least torsional resistance to represent the entire structure. For bridges with variable depth webs, designers may disregard the effect of girder warping in the web depth transition sections.

B. Bridge Lateral Data:

Designers may select the largest cross-frame spacing to represent the entire structure. For structures with variable beam spacings (i.e. flared girders) designers may select the largest spacing dimension to represent the entire structure. Designers should generally avoid utilizing temporary tie rods and timber blocks.

C. Permanent Lateral Support Data:

The default cross-frame type assumed by the TAEG software consists of a stiffener and diagonal x-bracing with top and bottom horizontal chords. In order to analyze the structure with a standard ODOT cross-frame, designers should input stiffener dimensions and select the “Diaphragms (Inputted I_x)” option. For ODOT standard cross-frames, designers should assume a fictitious stiffener of dimensions: 5-in x 3/8-in. Determine the diaphragm moment of inertia for all standard ODOT cross-frames as shown in BDM Figure 309-22.

$$I_x = \frac{h^2 s}{4L_d^3 \left(\frac{1}{A_d L_h^2} + \frac{L_h}{A_h L_d^3 + A_d L_h^3} \right)}$$

Where:

A_d = Area of the diagonal member (in²)

A_h = Area of the horizontal member (in²)

$$L_d = \sqrt{L_h^2 + h^2}$$

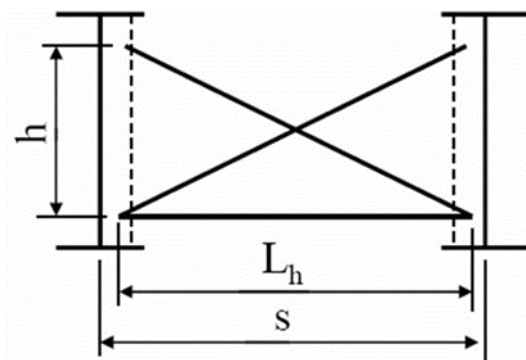


Figure 309-22

D. Temporary Lateral Support Data:

Designers should generally avoid utilizing temporary tie rods and timber blocks; however, if required these should be detailed in the contract plans.

Do not exceed 120-ft for the total machine length. If greater lengths are required, consult the Office of Structural Engineering for recommendations.

E. Load Data:

1. Live Load on Walkway 50-lb/ft²
2. Live Load on Slab 50-lb/ft²
3. Dead Load of Formwork 10-lb/ft²

4. Dead Load of Concrete $150(t_{avg})\text{lb/ft}^2$

(t_{avg} = Average thickness [ft] of deck slab overhang)

5. Wheel Spacing [1-2-3] 36-in – 31-in – 36-in

6. Maximum Wheel Load:

To estimate the total finishing machine length required for placement along the skew, add the rail-to-rail length and the extra end length from the following table using the plan specified skew rounded to the nearest 5-degrees. W is the rail-to-rail length as measured perpendicular to the centerline of the bridge. See BDM Table 309-1

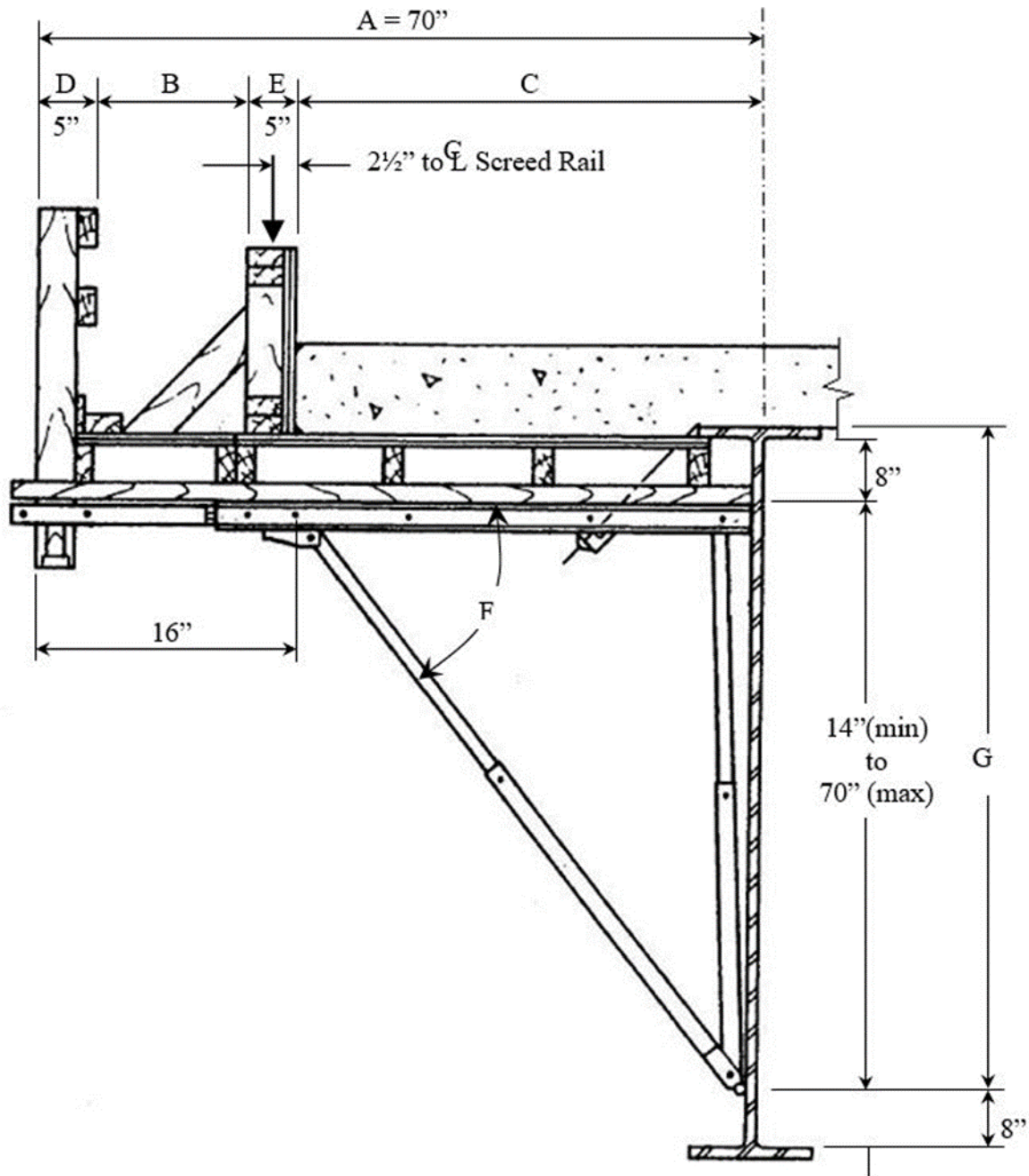
Table 309-1

Skew Angle	Rail-to-Rail Length, ft	Extra End Length, ft
0	1.00 W	0.0
15	1.04 W	5.0
20	1.06 W	5.5
25	1.10 W	6.5
30	1.15 W	7.0
35	1.22 W	8.0
40	1.31 W	9.0
45	1.41 W	10.5
50	1.56 W	11.5
55	1.74 W	13.5

For total machine lengths of 36-ft and less, assume a maximum wheel load of 2.2-kip. Add 0.012-kip for each additional foot of machine length required above 36-ft.

F. Bracket Data:

1. Refer to BDM Figure 309-23 to determine TAEG dimensions A, B, C, D, E, F and G.
2. Designers may assume a center-to-center bracket spacing of 48.0-in.
3. Designers may assume a bracket weight of 50-lbs.



Assumptions for TAEG Bracket Data Input

Figure 309-23

For prestressed I-beam superstructures, verify that the intermediate cross-frames/diaphragms in the exterior bay are capable of resisting the torsion caused by the cantilevered falsework. See BDM Figure 309-24

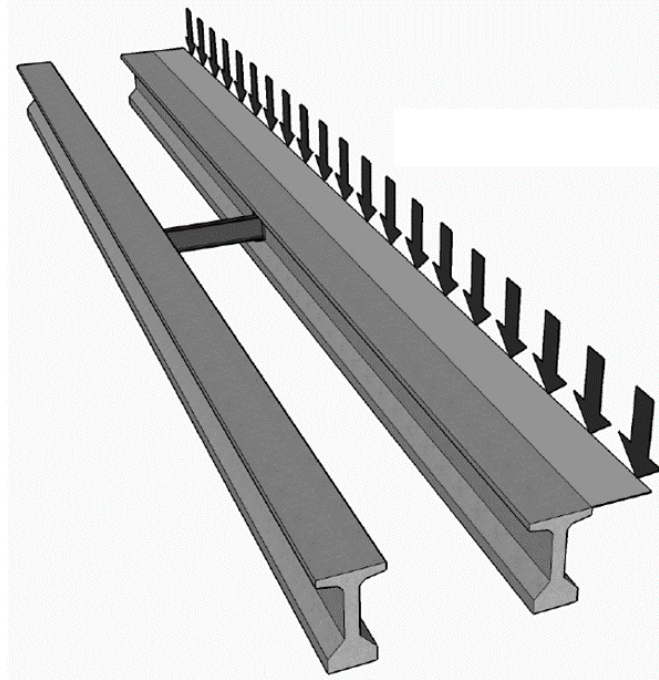


Figure 309-24

309.3.8.3 DETERMINING EFFECT OF GIRDER TWIST

C309.3.8.3

Once all sources of girder twist are quantified, determine the total effect that girder twist has on the finished deck surface.

Do not exceed 1/2-in cover loss due to girder twist.

The greatest concern in girder twist is the loss of concrete cover over the top mat of deck reinforcement and the subsequent loss of deck thickness.

Determine the total amount of girder twist at both the left and right screed rails as follows:

$$\phi_{\text{left}} = (\phi_g + \phi_o + \phi_w)_{\text{left}} \quad \text{AND}$$

$$\phi_{\text{right}} = (\phi_g + \phi_o + \phi_w)_{\text{right}}$$

Where:

ϕ_g = Girder twist due to global superstructure distortion (See BDM Section 309.3.8.2.a)

ϕ_o = Girder twist due to “oil-canning” (See BDM Section 309.3.8.2.b)

ϕ_w = Girder twist due to girder warping (See BDM Section 309.3.8.2.c)

Determine the total amount of screed rail deflection at both the left and right screed rail as follows: (See BDM Figure 309-25)

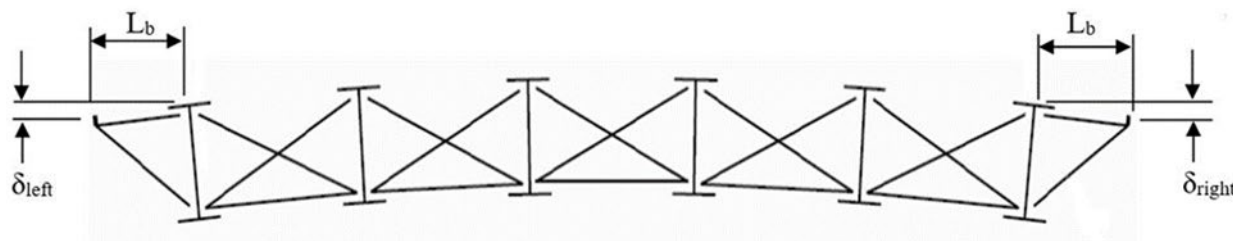


Figure 309-25

$$\delta_{\text{left}} = \tan(\phi_{\text{left}}) \times L_b \quad \text{AND}$$

$$\delta_{\text{right}} = \tan(\phi_{\text{right}}) \times L_b$$

Where:

$\delta_{\text{left}}, \delta_{\text{right}} =$ Deflection of the screed rail due to total girder twist (in). Upward deflection is positive and downward deflection is negative.

$L_b =$ Lateral distance from center of screed rail to centerline of fascia girder (in)

Determine the total loss of deck thickness as follows:

$$\delta_{\text{Total}} = (\delta_{\text{left}} + \delta_{\text{right}})/2$$

309.3.8.4 SLAB DEPTH OF CURVED BRIDGES

For a curved deck on straight steel beams, steel girders or prestressed I-beams, the distance from the top of the slab to the top of the beams or girders will vary from end to end. Show this variation in the slab depth dimension by giving the maximum and minimum depth dimensions with their respective location, over the piers, center of span, etc.

309.3.8.5 STAGED CONSTRUCTION

For bridges with a cast-in-place concrete deck on steel beam/girders or prestressed I-beams, where the differential dead load deflection between adjacent phases at any point along the span is greater than 1/4-in, a full-length deck closure pour is required.

C309.3.8.4

An alternate is to accommodate the differential depth by including it in the Camber Table as geometric camber.

C309.3.8.5

For the purposes of determining differential deflection, consider the amount of relative vertical movement due to the weight of the deck placement between any point along the beam centerline nearest the phase construction line and its relative point along a radial line at the edge of the previous stage edge of deck. This movement can conservatively be estimated as the deflection of the beam nearest the phase line due to the weight of the deck concrete. For example, if the deck dead load deflection of the beam nearest the phase line is 1-in at midspan, then the differential dead load deflection between phases at the midpoint is 1-in. For a more refined analysis, this movement can be determined as the remaining deck dead load deflection of the beam nearest the phase line at the point of concrete placement. For example, if the total deck deflection at the first quarter point is 1-in, but when the deck concrete is loaded to the first quarter point, the beam deflection is 0.25-in., then the remaining

For slab bridges built with phased construction, a deck closure pour is required regardless of the differential deflection magnitude. Remove the falsework at the completion of each phase of construction and before completing the closure pour.

For composite box beam bridges with phased construction, a deck closure pour is required regardless of the differential deflection magnitude. Do not place the grout for the shear key in the closure pour until the deck on both sides of the closure pour are completed.

The minimum closure pour width between the stages is 30-in. When the transverse reinforcing is being lap spliced, detail the width of the closure pour to accommodate the required concrete reinforcement lap splices.

When mechanical connectors are utilized, use a mechanical connector system in accordance with [C&MS 509](#).

Do not permanently attach intermediate cross-frames and diaphragms attached in the closure pour location until the concrete pours on both sides of the closure pour location have been completed.

Provide plan notes on the stage construction details sheet that detail the sequence of construction.

309.4 RAILING

309.4.1 GENERAL

The provisions in this section apply to new bridge railing. For requirements applied to existing bridge railings, refer to BDM Section 403.10.

differential deflection between phases at the first quarter point is 0.75-in.

This closure pour is required to avoid over-stressing the top transverse concrete reinforcement and also avoid increasing the width of the construction joint when the falsework is removed from the latter phase of construction. Avoid a solution to this issue that requires falsework for all phases to be left in place; removed at the completion of slab construction and designed for live load.

Refer to Standard Drawing PSBD-1-25 for an example composite box beam deck transverse section showing staged construction.

The preferred location for staged construction joints is the positive moment regions of the cast-in-place concrete deck slab.

C309.4.1

Bridge railing refers to the railing on the bridge designed for vehicular impacts. Other railing on the bridge, for pedestrian or bicycle purposes, that is protected from vehicular impacts by bridge railing shall be in accordance with *LRFD 13.8* and *LRFD 13.9* and need not be in compliance with MASH or NCHRP Report 350 criteria.

- A₁. For bridges where ODOT is responsible for Major Maintenance or bridges carrying NHS routes regardless of Major Maintenance responsibility: All bridge railing, transitions, roadway railing and railing end terminals shall meet acceptance criteria contained in the AASHTO “Manual for Assessing Safety Hardware” (MASH). For high-speed routes (i.e. Design Speed > 45-mph) with more than two lanes, the minimum acceptance level for railing at the deck edge shall be TL-5. Otherwise, the minimum acceptance level shall be TL-3.
- Maintenance responsibility is provided in AssetWise as National Bridge Inventory Item #21. As noted in the Ohio Bridge Inventory Coding Guide: “For bridges in Ohio, the agency legally responsible for the Major Maintenance of the bridges should be coded. The agency legally responsible should be coded in accordance with the Ohio Revised Code, Roadway Infrastructure Maintenance Responsibility Manual or by legal agreement.”
- Refer to BDM Section 309.4.2 for a listing of MASH accepted ODOT Standard Bridge Railing Systems.
- Be aware when identifying non-standard railing systems for use on a project that the entire length of need including the bridge railing, transition, roadway railing and end terminal need to be MASH compliant.
- Useful on-line resources for MASH acceptable railing systems include:
- A₁. The Texas Transportation Institute (TTI) Roadside Safety Pooled Fund website which provides a database of hardware tested under the MASH criteria.
- B₁. The TxDOT Bridge Standards website and the TxDOT Roadway Standards website.
- C₁. The FHWA Office of Safety website which provides eligibility letters for MASH accepted longitudinal barriers.
- B₁. For bridges carrying non-NHS routes where Major Maintenance is not ODOT’s responsibility: All bridge railing, transitions, roadway railing and railing end terminals shall meet acceptance criteria contained in either NCHRP Report 350 or MASH. For high-speed routes (i.e. Design Speed > 45-mph) with more than two lanes, the minimum acceptance level for railing at the deck edge shall be TL-5. Otherwise, the minimum acceptance level shall be TL-3.
- Refer to BDM Section 309.4.2 for a listing of MASH and NCHRP Report 350 accepted ODOT Standard Bridge Railing Systems.
- The FHWA Office of Safety website provides a listing NCHRP Report 350 accepted hardware.

For projects that include non-standard railing systems, submit the following information to the Office of Structural Engineering for review and acceptance with the Stage 1 submission:

The Department will consider standard ODOT railing systems utilizing aesthetically formed surfaces on the traffic face to be non-standard. The aesthetic treatments shall be in accordance with the Final Design Guidelines defined in NCHRP Report 554.

The Department will consider structural modifications to standard ODOT railing systems to be non-standard. Examples of structural modifications include exceeding railing post length and railing span maximum dimensions, changes to specified structural shapes, etc.

The Department may consider lower acceptance level railing systems for project locations analyzed in accordance with the selection guidelines presented in the Final Report for NCHRP Project 22-12(03).

- A₂. Final Structure Site Plan in accordance with BDM Section 201.2.1.1.
- B₂. Transverse Section of the bridge
- C₂. Plan and elevation view of the non-standard bridge railing; transition; roadway railing; and end terminal
- D₂. Cross-sections of the non-standard bridge railing at every section change. Include size and spacing of concrete reinforcement and structural steel member sizes.
- E₂. Traffic data according to [L&D, Vol. 1](#), Section 102.2 and Design Speed
- F₂. Cost comparison between proposed non-standard bridge railing design and applicable standard ODOT bridge railing systems.
- G₂. Accident history within 1000-ft of the bridge limits
- H₂. Proof of MASH/NCHRP Report 350 acceptability

309.4.2 STANDARD RAILING TYPES**Table 309-2**

Drawing No.	Description	NCHRP 350 Level	MASH Level
BR-1-13	36-in New Jersey Shape Concrete Bridge Railing	TL-4	TL-3 ⁽¹⁾⁽²⁾
	42-in New Jersey Shape Concrete Bridge Railing	TL-5	n/a
BR-2-15	Bridge Sidewalk Railing with Concrete Parapet	TL-4	TL-4 ⁽³⁾
DBR-2-73	Deep Beam Bridge Guardrail	TL-2	n/a
DBR-3-11	Deep Beam Bridge Retrofit Railing	TL-3	n/a
PCB-91	Portable Concrete Barrier (Fully Anchored)	TL-4	n/a
	Portable Concrete Barrier (Unanchored)	TL-3	n/a
SBR-1-20	42-in Single Slope Concrete Bridge Railing	TL-5	TL-5 ⁽³⁾
SBR-2-20	57-in Single Slope Concrete Median (Unreinforced)	TL-3	TL-3 ⁽⁴⁾
	57-in Single Slope Concrete Median (Reinforced)	TL-5	TL-5 ⁽³⁾
SBR-3-20	36-in Single Slope Concrete Bridge Railing	TL-4	TL-4 ⁽⁵⁾
TBR-1-11	Thrie Beam Retrofit Railing	TL-4	n/a
TST-1-99	Twin Steel Tube Bridge Railing	TL-4	n/a
TST-2-21	Three Steel Tube Bridge Railing	n/a	TL-4 ⁽⁶⁾

n/a – Indicates railing was neither tested nor evaluated under the listed acceptance criteria

- (1) – Bullard, D.L., Bligh, R.P., Menges, W.L., Haug, and R.R. 2010. Evaluation of Existing Roadside Safety Hardware Using Manual for Assessing Safety Hardware (MASH), Research Results Digest 349, Transportation Research Board, National Research Council, Washington, DC.
- (2) – Polivka, K.A., Faller, R.K., Sicking, D.L., Rohde, J.R., Bielenberg, B.W., Reid, J.D., Coon, B.A. 2006. Performance Evaluation of The Permanent New Jersey Safety Shape Barrier – Update to NCHRP 350, TRP-03-177-06, Midwest Roadside Safety Facility, Nebraska Transportation Center, University of Nebraska-Lincoln, Lincoln, Nebraska.
- (3) – Silvestri-Dobrovolsky, C., Schulz, N., Moran, S., Bligh, R.P. and Williams, W.F. 2017. Final Report NCHRP Project 20-07/Task 395 MASH Equivalency of NCHRP Report 350-Approved Bridge Railings, Transportation Research Board, National Research Council, Washington, DC.
- (4) – Bielenberg, R.W., Faller, R.K., and Ronspies, K.B. 2018. MASH TL-3 Evaluation of the Unreinforced, Single-Slope Concrete Median Barrier, TRP-03-388-18, Midwest Roadside Safety Facility, Nebraska Transportation Center, University of Nebraska-Lincoln, Lincoln, Nebraska.
- (5) – Sheikh, N.M., Bligh, R.P., and Menges, W.L. 2011. Determination of Minimum Height and Lateral Design Load for Mash Test Level 4 Bridge Rails, Test Report 9-1002-5, Texas Transportation Institute, Texas A&M University System College Station, Texas
- (6) – Pena, O., Faller, R.K., Rasmussen, J.D., Steelman, J.S., Rosenbaugh, S.K., Bielenberg, R.W., Mauricio, P., and Duren, J.T. 2020. Development of a MASH Test Level 4 Steel, Side-Mounted, Beam-and-Post, Bridge Rail, TRP-03-410-20, Midwest Roadside Safety Facility, Nebraska Transportation Center, University of Nebraska-Lincoln, Lincoln, Nebraska.

309.4.3 WHEN TO USE**309.4.3.1 NEW JERSEY SHAPE CONCRETE BRIDGE RAILING (36-in BR-1-13)**

This railing system is acceptable for all bridges regardless of Major Maintenance responsibility except as noted below.

Do not specify 36-in New Jersey shaped bridge railing at the deck edge of a high-speed route with more than two lanes.

Do not specify the 36-in New Jersey shaped bridge railing on a new bridge deck where the 42-in Single Slope Concrete Bridge Railing is acceptable.

Include the following information in the project plans: plan views, elevation views, cross-sections, deflection joint spacing, deflection joint details, reinforcing marks, reinforcing bending diagrams, steel reinforcement weights and GFRP reinforcement lengths.

Include provisions to collect and carry deck drainage off the ends of the bridge in the project plans

309.4.3.2 NEW JERSEY SHAPE CONCRETE BRIDGE RAILING (42-in BR-1-13)

This railing system is acceptable only for bridges carrying non-NHS routes where Major Maintenance is not ODOT's responsibility.

C309.4.3.1

The Department prefers the use of the 36-in Single Slope Concrete Bridge Railing ([SBR-3-20](#)) to the 36-in New Jersey Shaped Concrete Bridge Railing where possible.

This provision is consistent with BDM Section 309.4.1.

Even though the 36-in New Jersey shaped bridge railing has been tested and evaluated as a TL-3 MASH acceptable railing system, the Department would prefer to utilize the higher TL systems where they are applicable.

The 36-in tall NJ shape should only be considered under the following circumstances:

- A. Locations where the 42-in Single Slope system causes a stopping sight distance issue.
- B. Locations where the structural capacity of existing superstructure will not accommodate the weight of the 42-in Single Slope system.

For projects where both 36-in and 42-in railings are considered as noted above, the lowest cost alternative is acceptable.

Reference in the plans to the Standard Bridge Drawings is necessary for historical purposes only.

Refer to the [L&D, Vol. 2](#), Section 1113.1 for more information.

C309.4.3.2

The 42-in New Jersey shaped bridge railing has not been tested or evaluated to the MASH acceptance criteria. Use the 42-in Single Slope Concrete Bridge Railing in lieu of the 42-in NJ shape railing on bridges carrying NHS routes and where ODOT has Major Maintenance responsibility.

Include the following information in the project plans: plan views, elevation views, cross-sections, deflection joint spacing, deflection joint details, reinforcing marks, reinforcing bending diagrams, steel reinforcement weights and GFRP reinforcement lengths.

Include provisions to collect and carry deck drainage off the ends of the bridge in the project plans.

309.4.3.3 BRIDGE SIDEWALK RAILING (BR-2-15)

This railing system is acceptable for all bridges regardless of Major Maintenance responsibility except as noted below.

Do not specify this railing system at the deck edge of a high-speed route with more than two lanes.

Include the following information in the project plans: plan views, elevation views, cross-sections, deflection joint spacing, deflection joint details, reinforcing marks, reinforcing bending diagrams and steel reinforcement weights.

Include provisions to collect and carry deck drainage off the ends of the bridge in the project plans.

Refer to the [L&D, Vol. 2](#), Section 1113.1 for more information.

C309.4.3.3

This railing system is for use on bridges with sidewalks at least 5-ft wide and a curb height of 8-in. 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.

For projects with sidewalk railing and Vandal Protection Fencing, [BR-2-15](#) provides a modified base plate that includes a welded post sleeve for any [VPF-1-24](#) post section.

This provision is consistent with BDM Section 309.4.1.

Refer to the [L&D, Vol. 2](#), Section 1103.8 for more information.

309.4.3.4 DEEP BEAM BRIDGE GUARDRAIL (DBR-2-73)

This railing is acceptable only for bridges over waterways carrying non-NHS routes where Major Maintenance is not ODOT's responsibility except as noted below.

Do not specify the [DBR-2-73](#) railing system on bridges carrying High-speed routes.

C309.4.3.4

The original [DBR-2-73](#) railing was tested under the NCHRP Report 230 acceptance criteria using a 2000-lb car at a speed of 60-mph with a 20° impact angle and a 4500-lb car at a speed of 60-mph with a 25° impact angle⁽¹⁾. That acceptance level was grandfathered to a TL-2 acceptance level under NCHRP Report 350 because the performance of the TL-3 vehicle is unknown.

The use of the Deep Beam Bridge Guardrail with the Deep Beam Retrofit Railing ([DBR-3-11](#)) is acceptable in accordance with BDM Section 304.3.5.

⁽¹⁾ Bronstad, M.E., Michie, J.D. and Mayer, J.D. 1987. *Performance of Longitudinal Traffic Barriers*, NCHRP Report 289, Transportation Research Board, National Research Council, Washington, DC.

Variable post lengths may be required along the length of a structure due to beam camber. Retired design data sheet, DBP-1-92, is available from the Office of Structural Engineering Archived Standard Drawings website to address these concerns.

309.4.3.5 DEEP BEAM BRIDGE RETROFIT RAILING (DBR-3-11)

This railing system when used in combination with the Deep Beam Bridge Railing, [DBR-2-73](#), on new bridge decks is acceptable only for bridges carrying non-NHS routes over waterways where Major Maintenance is not ODOT's responsibility. Refer to BDM Section 403.10.1.3.e for applications on existing bridges.

To transition the Deep Beam Retrofit Railing to Type MGS approach roadway guardrail, use an ODOT Type 4 Bridge Terminal Assembly as detailed on Plan Insert Sheet GR-3.4 with a revision date of July 20, 2018. At the end of the Type 4 BTA, transition to MGS using SCD MGS-4.3 followed by SCD MGS-2.1.

C309.4.3.5

The Deep Beam Bridge Retrofit Railing system was evaluated under the NCHRP Report 350 acceptance criteria⁽¹⁾. There is no design speed restriction associated with the use of the Deep Beam Bridge Retrofit Railing system.

⁽¹⁾ Abu-Odeh, A.Y., Williams, W.F., Buth, C.E., Kim, K.M. 2010. *Development of a TL-3 Deep Beam Tubular Backup Bridge Rail*, Texas Transportation Institute, Texas A&M University, Bryan, Texas.

The top of w-beam element for the Deep Beam Bridge Railing and the Type 4 BTA is located at 27.75-in above the pavement surface. For standard Type MGS guardrail, the top of the w-beam element is located at 31-in above the pavement surface. Detail this height transition to occur in the shortest horizontal distance possible beyond the BTA Type 4 and within the first four panels of Type MGS guardrail.

309.4.3.6 PORTABLE CONCRETE BARRIER (PCB-91 & RM-4.2)

The project plans shall include the following information: plan view showing the location of the portable barrier along the entire length of the bridge; typical section showing lane widths and deck width;

C309.4.3.6

RM-4.2 was evaluated under MASH acceptance criteria and the [PCB-91](#) barrier system was evaluated under NCHRP Report 350 acceptance criteria. A joint implementation agreement between AASHTO and FHWA established a deadline for the use of non-MASH accepted portable barrier systems.⁽¹⁾ Under the

anchoring requirements for both [PCB-91](#) and RM-4.2 barrier sections; and the appropriate pay item.

agreement, the Department cannot allow the use of [PCB-91](#) segments cast after December 31, 2019. [PCB-91](#) segments cast prior to that date may be utilized until December 31, 2029.

Since contractors are permitted to use existing [PCB-91](#) or RM-4.2 segments, the plans need to address anchoring requirements for both systems.

⁽¹⁾ Everett, T., Griffith, M.S., January 7, 2016. *Joint Implementation Agreement for Manual for Assessing Safety Hardware (MASH)*, AASHTO/FHWA, Office of Safety, Federal Highway Administration, Washington D.C.

Placement of unanchored [PCB-91](#) and RM-4.2 on bridge decks and the anchoring requirements for [PCB-91](#) are defined in Design Data Sheet, [PCB-DD](#) available at the OSE website. The anchoring requirements for RM-4.2 are provided on sheet 4 of the drawing.

The Department will pay for portable concrete barrier as follows:

ITEM 622 – PORTABLE BARRIER, UNANCHORED

ITEM 622 – PORTABLE BARRIER, ANCHORED

When anchoring to the bridge deck is required for any temporary railing system, note in the plans that railing sections that span an expansion joint shall only be anchored on one side of the expansion joint.

[C&MS](#) 622 does allow the Contractor to substitute an ODOT approved alternative portable barrier product listed on the Office of Roadway Engineering's website. Designers should become familiar with these approved products to ensure their potential use is not detrimental for a specific project. Provide Plan notes restricting these products where required.

309.4.3.7 SINGLE SLOPE CONCRETE BRIDGE RAILING (SBR-1-20)

The [SBR-1-20](#) railing system is acceptable for all bridges regardless of Major Maintenance responsibility

Include the following information in the project plans: plan views, elevation views, cross-sections, deflection joint spacing, deflection joint details, reinforcing marks, reinforcing bending diagrams, steel reinforcement weights and GFRP reinforcement lengths.

Include provisions to collect and carry deck drainage off the ends of the bridge in the project plans.

C309.4.3.7

309.4.3.8 57-in SINGLE SLOPE CONCRETE MEDIAN BRIDGE RAILING (SBR-2- 20)

The [SBR-2-20](#) railing system is acceptable for all bridges regardless of Major Maintenance responsibility

Include the following information in the project plans: plan views, elevation views, cross-sections, deflection joint spacing, deflection joint details, reinforcing marks, reinforcing bending diagrams, steel reinforcement weights and GFRP reinforcement lengths.

Include provisions to collect and carry deck drainage off the ends of the bridge in the project plans.

309.4.3.9 TWIN STEEL TUBE BRIDGE RAILING (TST-1-99)

This railing system is acceptable **only** for bridges over waterways carrying non-NHS routes where Major Maintenance is not ODOT's responsibility.

For box beam bridge types, reference post spacing dimensions to each box beam end.

The site plan shall show the station of the center of the first inlet-mounted post on each corner of the bridge.

C309.4.3.8

The unreinforced 57-in median bridge railing is based on the barrier detailed in SCD RM-4.3.

The acceptance of the reinforced 57-in median bridge railing is based on the Final Report for NCHRP Project No. 20-07/Task 395.

List GFRP reinforcement in the Reinforcement Schedule. The Department will pay for GFRP by total length in feet for each bar size under Item 509.

C309.4.3.9

The Twin Steel Tube Bridge Railing is a modified version of the Illinois DOT Type SM system which was tested according to the AASHTO Guide Specifications for Bridge Railing using an 1800-lb car at 60-mph with a 20° impact angle; a 5400-lb pickup truck at 60-mph with a 20° impact angle; and an 18000-lb single unit truck at 50-mph with a 15° impact angle.⁽¹⁾

⁽¹⁾ Buth, C.E., Hirsch, T.J. and Menges, W.L. 1997. *Testing of New Bridge Rail and Transition Designs Volume I; Technical Report*, Texas Transportation Institute, College Station, Texas

This rail type does not meet the minimum requirements specified by AASHTO for pedestrian and bicycle railings and shall not be used where pedestrian or bicycle traffic is expected.

The typical post spacing is 6.25-ft. The standard drawing allows a reduced span for the first, last and one additional post spacing per span on each side of the bridge to account for construction clearances. Carefully review the position of the posts that are near the corner of a structure for possible interference with wingwalls, tie rods, etc.

**309.4.3.10 SINGLE SLOPE CONCRETE
BRIDGE RAILING (SBR-3-20)**

The [SBR-3-20](#) railing system is acceptable for all bridges regardless of Major Maintenance responsibility except as noted below.

Do not use the [SBR-3-20](#) railing system at the deck edge of a bridge carrying a High-speed route of more than two lanes.

Include the following information in the project plans: plan views, elevation views, cross-sections, deflection joint spacing, deflection joint details, reinforcing marks, reinforcing bending diagrams, reinforcing steel weights and GFRP reinforcement lengths.

Include provisions to collect and carry deck drainage off the ends of the bridge in the project plans.

**309.4.3.11 THREE BEAM RETROFIT RAILING
(TBR-1-11)**

This railing system is acceptable for retrofitting Safety Curb railing systems on all existing bridges regardless of Major Maintenance responsibility.

This railing system is not acceptable for use on bridges receiving a new: deck, deck edge or superstructure.

Refer to BDM Section 403.10.1.3.g for installation and maintenance requirements for [TBR-1-11](#) bridge railing.

**309.4.3.12 THREE STEEL TUBE BRIDGE
RAILING (TST-2-21)**

This railing system is acceptable for all bridges over waterways regardless of Major Maintenance responsibility except as noted below.

Do not use the [TST-2-21](#) railing system on bridges carrying a High-speed route of more than two lanes.

C309.4.3.10

A Low-speed route is a route with a Design Speed ≤ 45 mph. A High-speed route is defined in BDM Section 309.4.1. The [SBR-3-20](#) railing system may be considered for bridges where the taller 42-in [SBR-2-20](#) railing system presents sight distance issues. The lighter weight [SBR-3-20](#) railing system may also be useful on rehabilitation projects where the existing deck is to remain.

C309.4.3.11

Refer to BDM Section 403.10.1.3.g for additional description of the [TBR-1-11](#) railing system.

C309.4.3.12

The Three Steel Tube Bridge Railing system was developed as a joint research project between ODOT and the Illinois Department of Transportation at the Midwest Roadside Safety Facility⁽¹⁾⁽²⁾⁽³⁾⁽⁴⁾

(1) Mauricio, P., Rasmussen, J.D., Faller, R.K., Rosenbaugh, S.K., Bielenberg, R.W., Peña, O., Steelman, J.S., and Stolle, C.S. 2020. *Development of a Post-to-Deck Connection for a TL-4 Steel-Tube Bridge Rail*. TRP-03-409-20. Midwest Roadside Safety Facility, Nebraska Transportation Center, University of Nebraska-Lincoln, Lincoln, Nebraska.

(2) Pena, O., Faller, R. K., Rasmussen, J.D., Steelman, J.S., Rosenbaugh, S.K., Bielenberg, R.W., Mauricio, P., and Duren, J.T. 2020. *Development of a MASH Test Level 4 Steel, Side-Mounted, Beam-and-Post, Bridge Rail*. TRP-03-410-20. Midwest Roadside Safety Facility, Nebraska Transportation Center, University of Nebraska-Lincoln, Lincoln, Nebraska.

(3) Rasmussen, J.D., Rosenbaugh, S.K., Faller, R.K., Alvarado, R., Mauricio, P., Bielenberg, R.W., and

Steelman, J.S. 2020. *Development of a MASH TL-3 Approach Guardrail Transition to a MASH TL-4 Steel Bridge Rail*. TRP-03-411-20. Midwest Roadside Safety Facility, Nebraska Transportation Center, University of Nebraska-Lincoln, Lincoln, Nebraska.

⁽⁴⁾ Rosenbaugh, S.K., Rasmussen, J.D., Faller, R.K., Bielenberg, R.W., and Steelman, J.S. 2021. *Steel Railing, Type IL-OH: MASH Evaluation and Implementation Guidance*. TRP-03-438-21. Midwest Roadside Safety Facility, Nebraska Transportation Center, University of Nebraska-Lincoln, Lincoln, Nebraska.

Reference Standard Construction Drawing, MGS-3.3, for the proper bridge terminal assembly.

The Site Plan shall show the station of the center of the first post located off the bridge at each corner of the structure.

For box beam superstructures, reference post spacing dimensions to each box beam end.

The maximum post spacing is 8.0-ft which can be reduced to fit the span layout provided for the project. Carefully review the position of the posts that are near the corner of a structure for possible interference with wingwalls, tie rods, etc. Refer to [TST-2-21](#) for specific clearance requirements.

309.4.4 SEALING OF CONCRETE SURFACES, RAILING

In addition to the requirements of BDM Section 309.2.1, seal all exposed surfaces of continuously reinforced concrete railing with an approved concrete sealer per [C&MS](#) 512.

309.5 FENCING

309.5.1 GENERAL

C309.5.1

The primary purposes of protective fencing are to provide security for pedestrians and to discourage the throwing or dropping of objects from bridges onto traffic below.

[VPF-1-24](#) provides standard details for fencing attached to bridges. The designer may need to enhance this standard to deal with requirements for the specific structure.

309.5.2 WHEN TO USE

C309.5.2

Install fencing on all bridges over vehicular and pedestrian traffic except as noted herein. Do not install fencing on bridges that carry freeways as defined in the ORC 4511.01 where pedestrians are prohibited per ORC 4511.051, unless otherwise specified in the Scope of Services. For facilities not defined as freeways by the ORC, use BDM Table 309-3.

Install fencing on bridges over rail traffic if required in an agreement with the affected railroad.

For existing bridges meeting the fencing requirements of this section, install the fencing when new concrete bridge railings are installed.

At locations where fencing will adversely affect public safety (e.g. reduced sight distance), submit a written request for exemption to the Administrator of the Office of Structural Engineering. Include supporting documentation with the request for exemption.

Fencing should not be installed on existing concrete railings that are not in good condition.

An exemption request form is available as a Design Data Sheet on the Office of Structural Engineering web page.

Table 309-3

Under Bridge Feature	Fence Required
Interstate	Yes
US Route	Yes
State Route	Yes
County\Township Route	Yes
City Route	Yes
Railroad	Yes/No (Based on RR agreement)
Waterway	No
Bike/Walking path	No

309.5.3 FENCING CONFIGURATIONS

For structures without sidewalks, the minimum fence height is 8-ft above the pavement surface. For structures with sidewalks, the minimum fence height is 8-ft above the sidewalk.

For curved fence, extend the post vertically for at least 8-ft above the sidewalk before curving inward over the sidewalk. Construct the overhang at least 1-ft less than the width of the sidewalk. See BDM Figure 309-26 & Figure 309-27.

For pedestrian bridges, use bent pipe frames with pipe bend radii of 24-in at the upper corners and start the radii at least 8-ft above the sidewalk surface. Start the fabric at the deck line, top of curb or concrete bridge railing and extend to at least the top of the bent portion of the frame.

Install the fence a maximum of 1-in above the top of the concrete bridge railing.

Locate baseplates so that the adhesive anchors are a minimum of 1.5 times the post-installed anchor embedment from the deflection joint of the railing

C309.5.3

For a greater degree of protection against objects being thrown from the bridge, the fence may be curved to overhang the sidewalk.

Omitting fabric on the top horizontal area of the frame may prevent an individual from walking on the top of the enclosure. See BDM Figure 309-28 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.

Install posts and frames either plumb or perpendicular to the longitudinal grade of the bridge, subject to considerations of aesthetics or practicality of construction. For non-standard systems, show the posts and frames orientation with respect to the longitudinal grade in the plans. Provide complete details of base plates, pipe inserts or other types of base anchorage in the plans. [VPF-1-24](#) is shown as plumb to the longitudinal grade.

Extend the fencing between end posts placed at the locations selected from the following list that creates the shortest length:

- A. 30-ft $\pm$ 2.5-ft beyond the route under bridge 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 railing (-2.5-ft, +0-ft)

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.

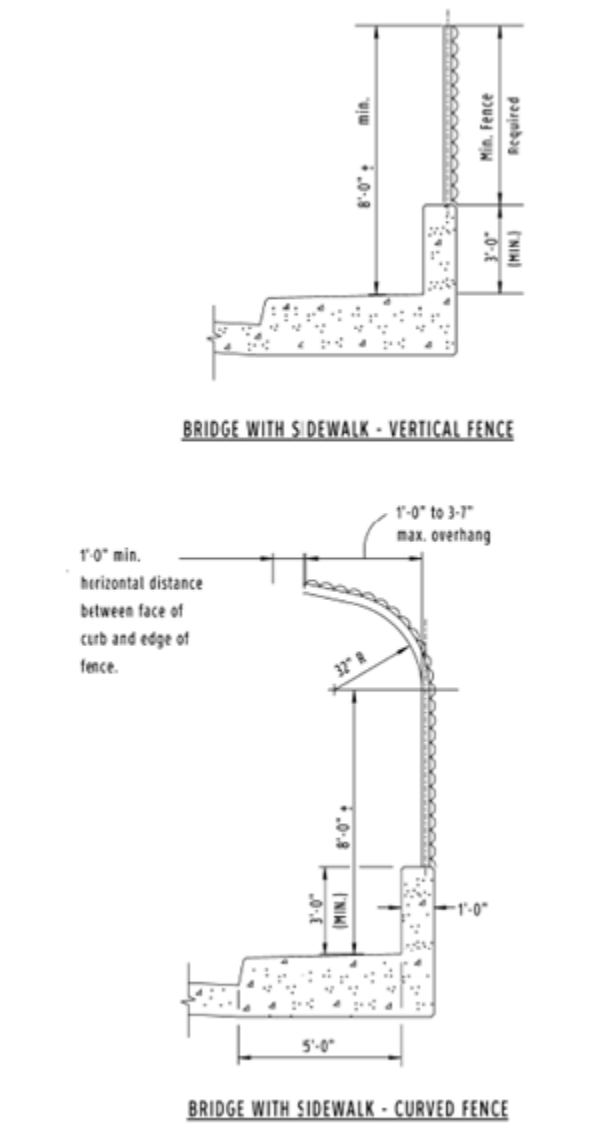
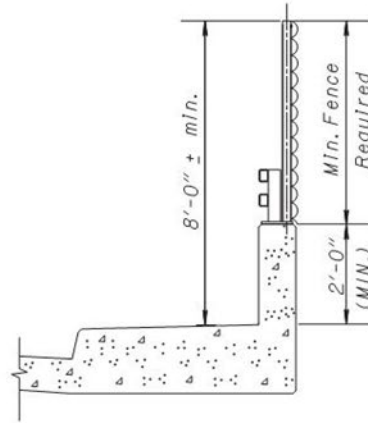
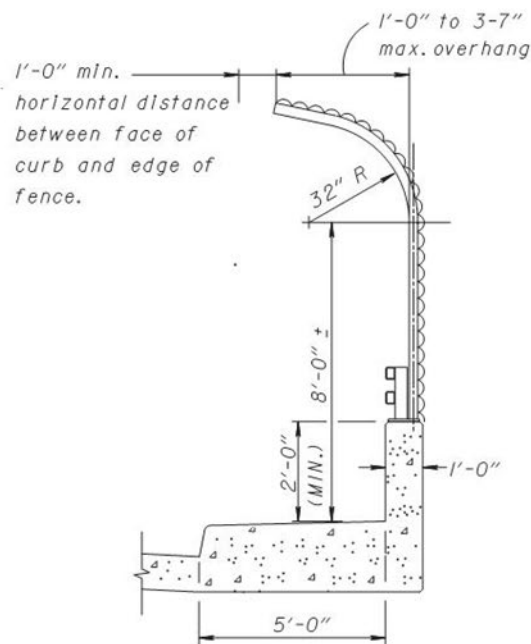


Figure 309-26



BRIDGE WITH SIDEWALK - VERTICAL FENCE



BRIDGE WITH SIDEWALK - CURVED FENCE

Figure 309-27

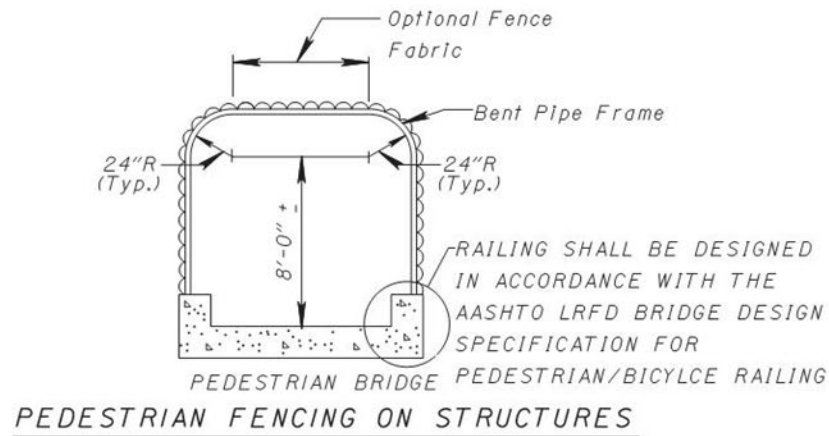
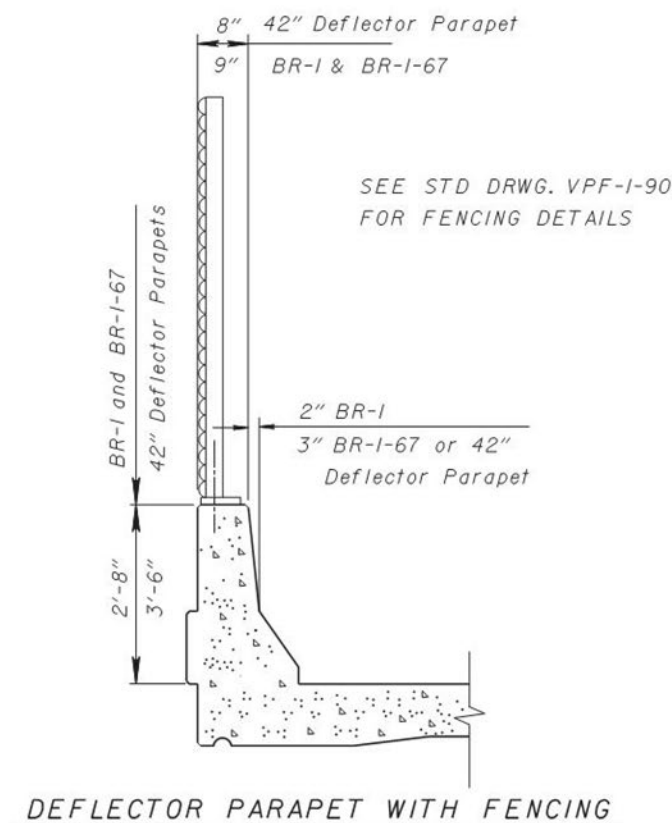


Figure 309-28

309.5.4 SPECIAL DESIGNS**C309.5.4**

For fence installation projects on new structures, install a traffic railing (steel tubing) when the top concrete bridge railing or concrete wall is less than 36-in above roadway for structures without sidewalks or 36-in above the top of sidewalk for structures with sidewalks. See BDM Figure 309-27.

Utilize a two coat shop applied epoxy/urethane system in accordance with [C&MS 708.02](#) for color coating of posts and rails. Plan notes for this coating system are available from OSE upon request.

For special fence designs, provide plan notes to define non-standard color, materials, traffic maintenance, construction procedures and other requirements.

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.

The designer should follow the example of standard bridge drawing for development of required notes.

309.5.5 FENCE DESIGN REQUIREMENTS

309.5.5.1 GENERAL

Furnish fencing mesh with either of the following materials:

- A. Chain-link wire mesh with 1-in diamonds. The core wire shall be 11 gage with a Polyvinyl chloride coating. ([C&MS 710.03](#))
- B. Welded wire fabric with 1/2-in x 3-in opening size. The core wire shall be 10.5 gage; galvanized after welding (1.2-oz zinc/ft²), and PVC coated (10-mil).

Clamp brace and bottom rails to posts or post frames.

Detail the top rail, if any, of a free-standing fence as continuous over two or more posts and provide suitable cap fittings.

Bent pipe frames for narrow pedestrian bridges are permitted. Fabricate bent pipe frames for narrow pedestrian bridges 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, leave both ends of pipe open. Therefore, put holes in the base plates equal to the pipes' inside diameter.

309.5.5.2 WIND & PEDESTRIAN LOADS

Design non-standard chain link fence designs with loading in accordance with *LRFD 13.8.2*.

For decorative railing that includes lettering and/or other appurtenances, determine the appropriate wind pressure per *LRFD 3.8* and apply the loading to the wind contact area for each appurtenance including the area for the posts and rails.

309.5.5.3 ADHESIVE ANCHOR DESIGN

When the fencing design is not according to [VPF-1-24](#), design the post installed anchors using an adhesive anchor system. Mechanical anchors are not permitted.

C309.5.5.2

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

The loading provisions in *LRFD 13.8.2* for chain link fencing should not be used in addition to the wind loading per *LRFD 3.8*.

For additional information in the design of adhesive anchor systems, refer to BDM Section. 406.1

Design the adhesive anchor system in accordance with *LRFD 5.13*. Provide an adhesive anchor system that has been evaluated by ICC-ES.

Design anchors assuming cracked concrete.

309.5.6 TEMPORARY VANDAL PROTECTION FENCING

Use the Design Data Sheet, [TVPFDD-1-18](#), Temporary Vandal Protection Fencing to determine the fencing requirements on bridges during construction.

Detail the length of the temporary vandal protection fencing installation in accordance with the minimum lengths defined in BDM Section 309.5.3.

The Department will utilize permanent vandal protection fencing where possible to minimize costs for temporary fencing. Therefore, place new permanent fence installations prior to shifting traffic onto that phase of construction. Leave temporary vandal protection fencing installed on existing concrete railing in place until the concrete railing is removed from the bridge. Leave all other types of temporary vandal protection fencing in place until the Engineer determines that it is no longer necessary. Once removed, reinstallation of temporary vandal protection fencing is not required.

In the plans provide pay item(s) in the Estimated Quantities; show the limits and location of each temporary vandal protection fencing installation; and show the appropriate temporary vandal protection fencing type on the Maintenance of Traffic Transverse Sections. If necessary, provide all non-standard connection and fence details in the plans; address special installation sequencing; and include all other special notes. Provide details for temporary vandal protection fencing across intermediate expansion joints.

C309.5.6

The intent of temporary vandal protection fencing is to discourage pedestrians from dropping or throwing heavy objects off the side of the bridge onto traffic below during construction.

Standard Bridge Drawing, [TVPF-1-18](#), Temporary Vandal Protection Fencing, provides details for:

- A. Type A – Fencing installed on existing railing systems and on the existing deck surface near the phased construction joint
- B. Type B – Fencing installed on the back side of the PCB. Refer to BDM Section 309.4.3.6 for PCB anchoring requirements. As a minimum, provide at least one anchor per PCB segment on the traffic side of the barrier.
- C. Type C – Fencing installed along the phased construction joint side of a newly constructed deck.

309.6 EXPANSION DEVICES**309.6.1 GENERAL**

Use an expansion device that provides a total seal against penetration and moisture.

For fabricated steel expansion devices, specify the type of steel required. Include the type of steel in a plan note if the plan requirements are not covered by a selected standard bridge drawing.

To protect steel expansion devices, metallize the exposed surfaces per [C&MS](#) 516.

309.6.1.1 PAY ITEM

Pay for expansion devices, except as specifically listed in this section, with an Item 516 pay item.

For sealed expansion devices, pay for the elastomeric seal, either strip or compression with an Item 516 pay item.

Clearly show what components are included with the expansion devices in the plans to be paid for with the Item 516 pay item. Consider the seal as part of the expansion device and included in the Item 516 pay item.

309.6.1.2 EXPANSION DEVICES WITH SIDEWALKS

On structures with sidewalks, furnish the same expansion device on the side walk as utilized for main bridge deck expansion joint.

For non-standard devices, use a curb plate and sidewalk cover plate. Separate the curb and sidewalk plates at the interface of the sidewalk and curb.

309.6.1.3 EXPANSION DEVICES WITH STAGE CONSTRUCTION

On projects involving stage construction, locate and show joints in the seal armor in the plans. At the stage construction lines, use complete penetration welded butt joints to connect adjoining expansion devices. If butt welds will be in contact with a sealing gland, grind the butt-weld flush at the contact area.

C309.6.1

Standard bridge drawings are available for expansion devices for typical bridge superstructure types.

C309.6.1.1

As an example, cross-frames, which are field welded to both the superstructure girders and the expansion devices, are part of the Item 513 structural steel pay item.

C309.6.1.2

Sidewalk details for standard expansion devices (strip seals) are shown on the standards. See details on Standard Bridge Drawings: [EXJ-2-81](#), [EXJ-3-82](#), [EXJ-4-87](#), [EXJ-5-93](#) and [EXJ-6-17](#) for sidewalk plates.

309.6.2 EXPANSION DEVICE TYPES**309.6.2.1 EXPANSION JOINTS USING POLYMER MODIFIED ASPHALT BINDER**

Do not use polymer modified asphalt binder on structures with movement greater than 1-in or structures with sidewalks.

Construct the polymer-modified joint to be between 2-in and 5-in thick. Show a plan view and cross-section of each polymer modified asphalt expansion joint location on the bridge in the plans. Provide the station of the joint centerline at the centerline of construction, skew angle and dimension its length as measured along the centerline of the joint in the plan view.

Dimension the width and thickness of the joint, width of the expansion gap and other significant joint details in the cross-section.

C309.6.2.1

This device is for use on structures with concrete or asphalt overlays and where total expected movement is 0-in to 1.0-in. See [SS846](#) for additional information.

309.6.2.2 STRIP SEAL EXPANSION DEVICES

The seal size is limited to a 5-in maximum. Do not use unpainted A709 50W weathering steel in the manufacture of this type expansion device. Ensure that all details are covered in the plans because the standard drawing is not inclusive for all structure types.

Construct the strip seal in one piece across the total width of the structure. The Department will not permit splices.

C309.6.2.2

A709 50W does not perform well in the atmospheric conditions an expansion device is subjected. Standard Bridge Drawings, [EXJ-4-87](#), [EXJ-5-93](#) and [EXJ-6-17](#), are available.

309.6.2.3 COMPRESSION SEAL EXPANSION DEVICES

The maximum allowable seal size is 4-in. Do not use a 5-in wide seal. Do not use compression seal expansion devices on structures with a skew greater than 15-degrees. Limit the movement so that the seal is not compressed greater than 60 percent or less than 20 percent.

Construct the compression seal in one piece across the total width of the structure. The Department will not permit splices.

C309.6.2.3

Installation problems have been encountered with the 5-in wide seal.

Standard Bridge Drawings [EXJ-2-81](#) & [EXJ-3-82](#) give generally used details

309.6.2.4 MODULAR EXPANSION DEVICES**C309.6.2.4**

Modular expansion devices may be required for structures when total required movements exceed movement capacity of a strip or compression seal. Modular expansion joint notes are available upon request.

Design the modular devices main load bearing beams, support beams and welds for fatigue.

Design of support for the modular device and deck thickness should allow for multiple styles or designs of modular devices. Contact suppliers and become familiar with the modular devices available.

309.6.2.5 TOOTH TYPE, FINGER TYPE OR NON-STANDARD SLIDING PLATE EXPANSION DEVICES

C309.6.2.5

Finger or sliding plate joints are another alternative type of expansion device where movements exceed the capacity of either strip or compression seal devices. This type of expansion device generally competes against Modular joints. Their advantage is their simplicity of design. Their disadvantage is their inability to seal against intrusion of water and debris.

Use of a tooth type expansion device also requires neoprene drainage troughs and a suitable drainage system to carry away the water. Completely detail both the neoprene trough and downspout to drainage trough connection. Pay special attention to developing a complete seal at the downspout to trough connection. Vulcanize the down spout to the trough.

Design finger devices for fatigue.

Do not design finger devices with fracture critical components.

Use pre-qualified 513 Level UF fabricators to construct finger or sliding plate devices.

Refer BDM Section 308.2.2.1.b for additional information.

309.6.3 EXPANSION DEVICE USES – BRIDGE OR ABUTMENT TYPE

C309.6.3

Expansion length is defined as the total length if no fixed bearing exists, or length from fixed bearing to proposed expansion device location, if one exists.

The expansion lengths shown in the table are approximate.

Table 309-4

Bridge Type	Expansion Length (ft)	Bridge Joint Required	Approach Slab/Structure Joint Type	Approach Slab/Roadway Joint Type
Reinforced Concrete Slab	0 - 160	None	AS-1-15 detail B or C	AS-2-15 Type A or B
	160+			AS-2-15 Type C
Steel Stringer	0 – 125*	IA or EXJ-4-87	AS-1-15 detail B or C	AS-2-15 Type A or B
	125 – 400*	IA		AS-2-15 Type C
		EXJ-4-87		AS-2-15 Type A or B
	400+	TTED or MED		
Prestressed Concrete I-Beam	0 – 160*	IA or EXJ-6-17	AS-1-15 detail B or C	AS-2-15 Type A or B
	160 – 575*	IA	AS-1-15 detail B or C	AS-2-15 Type C
		EXJ-6-17		AS-2-15 Type A or B
	575+	TTED or MED	AS-1-15 detail B or C	AS-2-15 Type A or B
Non-Composite Prestressed Concrete Box Beam	0 – 160*	IA	AS-1-15 detail E	AS-2-15 Type A or B
	160 – 475*			AS-2-15 Type C
Composite Prestressed Concrete Box Beam	0 – 160*	IA or EXJ-5-93	AS-1-15 detail B or C	AS-2-15 Type A or B
	160 – 475*	IA		AS-2-15 Type C
		EXJ-5-93		AS-2-15 Type A or B
	475 – 575*	IA		Non-standard Strip Seal
		TTED or MED		AS-2-15 Type A or B
	575+	TTED or MED		AS-2-15 Type A or B

IA = Integral/Semi-Integral Abutment

TTED = Tooth Type expansion device

PM = Polymer Modified Asphalt Joint

MED = Modular Expansion Device

* - “Expansion length” shown for integral and semi-integral superstructures refer to the total structure length to coincide with BDM Figure 306-5 and Figure 306-6. Otherwise, the expansion lengths shown refer to expansion lengths as define in BDM Section 309.6.3. Determine the expansion lengths for integral and semi-integral superstructures in accordance with BDM Sections 306.2.2.5 & 306.2.2.6, respectively.

309.7 BRIDGE DECK DRAINAGE**C309.7**

When using concrete bridge railing, provide a minimum longitudinal grade of 0.3% across the entire length of the bridge deck surface.

Minimize or eliminate scuppers used for collecting the deck surface drainage.

Compute the allowable spread of flow using the procedures described in Section 1103 of the [L&D, Vol. 2](#) to determine if scuppers are required.

Slope drainage collection systems as steeply as practical, and not less than 15-degrees from horizontal. The minimum bend radius for the system is 18-in, and no 90-degree bends are permitted. Provide adequate pipe supports and cleanouts at the low ends of runs.

Scupper should be eliminated on existing structure when feasible.

Scuppers, when provided, should preferably be located inside the fascia beam.

The cleanout plugs should be easily and safely accessible.

Place scuppers with drainage collection as close as possible to the substructure unit which drains them. Locate and detail the drainage collection system in the plans.

Do not place uncollected scuppers over roadways, multimodal facilities or sidewalks.

Place uncollected scupper downspouts as far away as possible from the substructure.

When the deck drainage is to flow off the ends of the bridge, make provisions to collect and carry away this run-off. Refer to the [L&D, Vol. 2](#), Section 1103.8 for more information.

Control of drainage is especially critical at abutments with MSE walls. On structures with MSE walls at the abutments, provide a concrete bridge railing on the approach slab with a standard inlet, SCD I-3D to collect the drainage. Locate the inlet at least 25-ft beyond the limits of the MSE wall soil reinforcement and continue concrete barrier 10-ft past the catch basin. Refer to BDM Figure 309-29 for more information.

For bridges with over-the-side drainage, provide a stainless steel drip strip to protect the deck edge and beam fascia from the deck surface run-off.

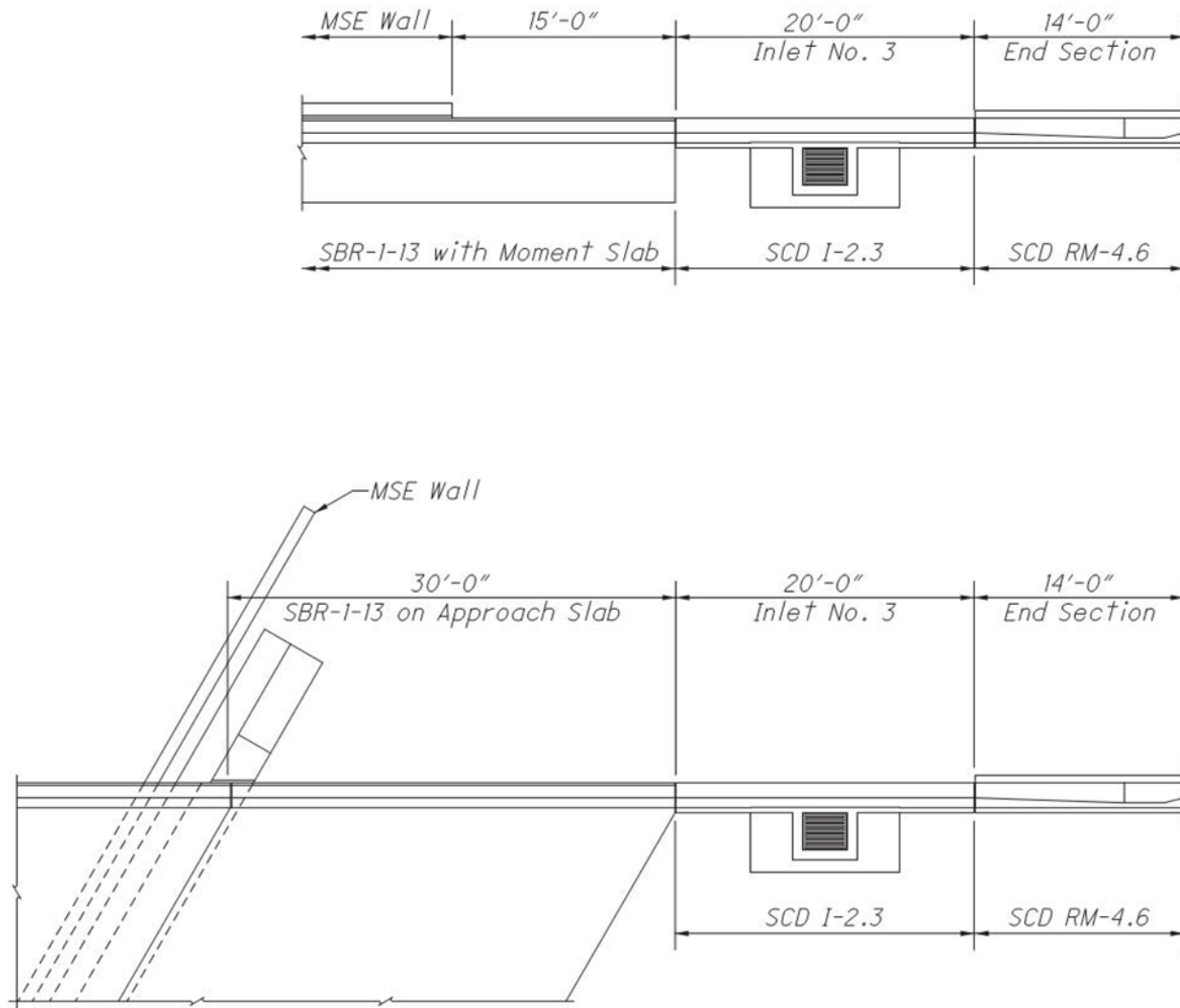


Figure 309-29

309.8 SIDEWALKS & SHARED USE FACILITY C309.8

Provide a sidewalk when required by the [L&D, Vol. 1](#), Section 306.4. The minimum width of the bridge sidewalk is the wider of: 5-ft or the approach sidewalk plus 12-in.

Provide a 1.56 percent cross slope on the sidewalk towards the curbline. Provide an 8-in tall sidewalk on the bridge.

Prepare a detail of the standard curb (height, face slope, and corner rounding) in the plans.

Tapering the sidewalk curb down to the approach curb height may or may not be performed with the limits of the approach slab.

310 MISCELLANEOUS**310.1 TEMPORARY SHORING****310.1.1 SUPPORT OF EXISTING STRUCTURE**

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

Include the following information in the plans: permissible locations of temporary support; temporary support loads; construction sequences; construction limitations not otherwise provided in [C&MS](#) 501.05; and all remaining plan notes. As a minimum, address method of measurement and basis of payment for temporary support in the plan notes.

310.2 APPROACH SLABS

Provide approach slabs for all ODOT bridges.

The approach slab width shall extend to the back side of the concrete sidewalk, barrier, and curb.

310.2.1 STANDARD BRIDGE DRAWING – AS-1-15 C310.2.1

Standard Bridge Drawing, [AS-1-15](#), provides contract ready designs and details of skewed and non-skewed approach slabs on tangent alignments with lengths of 15-ft, 20-ft, 25-ft and 30-ft under phased or non-phased construction.

Determine the length of the approach slab using the following formula:

$$L = [1.5(H + h + 1.5)] \div \cos \theta \leq 30\text{-ft}$$

Where:

- L = Length of the approach slab measured along the centerline of the roadway rounded up to the nearest 5-ft
- H = Height of the embankment measured from the bottom of the footing to the bottom of the approach slab (ft)
- h = Width of the footing heel (ft)
- θ = Skew angle

For four lane divided highways on new embankment, provide an approach slab with a minimum length of 25-ft (measured along the roadway centerline). For structures with MSE walls at the abutments, provide an approach slab with a minimum length of 30-ft. For all other structures, provide an approach slab with a minimum length of 15-ft.

310.2.1.1 PLAN REQUIREMENTS FOR STANDARD APPROACH SLABS

When [AS-1-15](#) is specified, provide the following information in the plans:

- A. Reference Standard Bridge Drawings: [AS-1-15](#) and [AS-2-15](#)
- B. Provide a plan view of the approach slab that includes: all width and length dimensions; skew angle; curb and concrete railing locations; and the final approach slab surface elevations at each transverse grade break line; phased construction line; and curbline/slab edge at each end of the approach slab.
- C. Include the D801 or D802 bars in the concrete reinforcement bar list for payment under Item 509.
- D. Include the appropriate pay item: ITEM 526 – REINFORCED CONCRETE APPROACH SLAB (T=__) or ITEM 526 – REINFORCED CONCRETE APPROACH SLAB WITH QC/QA (T=__)
- E. Include pay ITEM 526 – TYPE __ INSTALLATION
- F. Include, as necessary, pay ITEM 846 – POLYMER MODIFIER ASPHALT EXPANSION JOINT SYSTEM or ITEM 516 – ARMORLESS PREFORMED JOINT SEAL

For bridge replacement projects, when the existing approach slab is to be removed, include ITEM 202 – APPROACH SLAB REMOVED in the structures estimated quantities.

310.2.1.2 PLAN REQUIREMENTS FOR NON-STANDARD APPROACH SLABS

In addition to the plan requirements listed in BDM Section 310.2.1.1, provide all geometry and the reinforcement layout for the non-standard approach slab. Include these detail drawings in the structure plans for review during the detail design review stage. Include the appropriate pay item: ITEM 526 – REINFORCED CONCRETE APPROACH SLAB (T=__), AS PER PLAN or ITEM 526 – REINFORCED CONCRETE APPROACH SLAB WITH QC/QA (T=__), AS PER PLAN.

310.2.2 STANDARD BRIDGE DRAWING – AS-2-15**310.3 PRESSURE RELIEF JOINTS**

When the approach roadway pavement is rigid concrete and the approach slab Installation is Type C, specify a Type B pressure relief joints at a location 50-ft from the end of the sleeper slab.

310.4 UTILITIES

Make the request to allow utilities on the bridge through the ODOT District Utilities Coordinator. Install utilities in substantial ducts or enclosures adequate to protect the lines from future bridge repair and maintenance operations.

Do not place utilities inside of prestressed concrete box beams.

When it is necessary to place a utility, storm sewer trunk line, or culvert through, beneath or within a 10-ft radius of the limits of the select granular backfill of an MSE wall, encase it in steel casing pipe according to [C&MS 748.06](#).

C310.2.1.2

Examples of non-standard approach slabs include approach slabs that are: a non-standard length; tapered; curved; a non-uniform width or other such variation.

C310.2.2

Standard Bridge Drawing, [AS-2-15](#), provides contract ready installation details for standard approach slabs. Refer to the Designer Supplement to [AS-2-15](#) for additional information.

C310.3

Alternatively, a 25-ft length of full depth asphalt pavement may be specified between the sleeper slab and the rigid pavement. The pressure relief joints are detailed on Standard Construction Drawing BP-2.4, Pressure Relief Joint Types B, C, & D.

C310.4

The type of superstructure selected for a site may be dependent upon the number of utilities supported on the bridge.

If the bridge design is a composite deck on prestressed box beams, the design may either eliminate an interior box beam or provide a space between two interior box beams to provide utility access in this space. This alternative will require a special design for both the box beams and the deck.

Placing utilities, storm sewers and culverts through, beneath or within a 10-ft radius of the select granular backfill of an MSE wall should be avoided.

When it is necessary to place a storm sewer within the limits of the select granular backfill of an MSE wall, provide reinforced concrete pipe according to [C&MS 706.02](#). Provide premium joints according to [C&MS 706.11](#) and wrap the pipe joints in geotextile fabric, Type A according to [C&MS 712.09](#).

Show and dimension utility conduits embedded in concrete to clear construction joints by a minimum of 1-in and other conduits by a minimum of 2-in.

Do not embed utilities in the actual vehicular traffic carrying section of a concrete deck.

Do not suspend utilities below the bottom of the bridge superstructure or attach directly to the deck.

Reference the [Real Estate Manual](#) for the permitting process for accommodating utilities within the right-of-way. See section 8108 for additional information for utility installations on highway bridges.

310.4.1 UTILITIES ATTACHED TO BEAMS AND GIRDERS

Do not place utility lines between the stringers in the exterior bays of grade separation structures.

Locate critical utility lines (gas, etc.) that could contribute to the severity of a collision well above the bottom of the superstructure or be otherwise protected.

C310.4.1

This is to protect the lines from collisions.

310.5 AESTHETICS

310.5.1 GENERAL

C310.5.1

Refer to the ODOT [Aesthetic Design Guidelines](#) for aesthetics treatments on structures.

Each structure should be evaluated for aesthetics. Normally it is not practical to provide cost premium aesthetic treatments without a specific demand; however careful attention to the details of the structure lines and forms will generally result in a pleasing structure appearance.

Some basic guidelines that should be considered are as follows:

- A. Avoid mixing structural elements, for example concrete slab and steel beam superstructures or cap and column piers with wall type piers.
- B. In general, continuous superstructures shall be provided for multiple span bridges. Where intermediate joints cannot be avoided, the depth of spans adjacent to the joints preferably should be the same. Avoid the use of very slender superstructures over massive piers.

- C. Abrupt changes in beam depth should be avoided when possible. Whenever sudden changes in the depth of the beams in adjacent spans are required, care should be taken in the development of details at the pier.
- D. The lines of the structure should be simple and without excessive curves and abrupt changes.
- E. All structures should blend in with their surroundings.

The most significant design factors contributing to the aesthetic quality of the structure is unity, consistency, and continuity. These qualities will give the structure an appearance of a design process that was carefully thought out.

The aesthetics of the structure can generally be accomplished within the guidelines of design requiring only minimum special designs and minor project cost increase.

If formliners are being considered, the depth of the projections should be as deep as possible in order to have the desired visual effect. Using shallow depths, such as 2-in to 3-in, provides very little, if any, visual effect (relief) when viewed from a distance. The depth of the formliner shall not be included in the measurement of the concrete clear cover.

The use of colored concrete, where the color is integral with the concrete mix, should generally not be used since the final visual appearance of the concrete is not uniform. The color varies greatly due to the aggregate, cement type, cement content and the curing of the concrete. None of these items are reasonably controlled in the field to a sufficient enough degree to ensure a uniform final appearance. If color is required, a concrete coating should be used which will not only produce the required color but will also provide the necessary sealing of the concrete as required in BDM Sections 306.1.2 and 309.2.1.

The use of formliners and/or coloring of the concrete should be evaluated on a cost basis and submitted as part of the Structure Type Study, Cost Analysis.

310.5.2 LETTERING AND LOGO POLICY ON ODOT FACILITIES

Obtain approval from the Office of Structural Engineering for all lettering and logos to be placed on a bridge. Obtain approval from the Office of Environmental Services for all lettering and logos to be placed on noise walls. Obtain approval from the Division of Innovative Delivery for all lettering and logos to be placed on ODOT facilities through sponsorship naming proposals.

C310.5.2

Office of Structural Engineering approval of a lettering and logo submission does not constitute acceptance of the structural design. Structural design acceptance will occur through the review processes defined in BDM Section 200.

Aesthetic lighting shall be in accordance with the ODOT [TEM](#) note 1142-26.

The criteria for permitting lettering and logos on Bridges and Noise Walls is as follows:

- A. City, village, township, or county names and logos are acceptable provided the bridge or noise wall is within the territorial jurisdiction of that administrative division.
- B. Names and logos that promote tourism are acceptable provided the bridge or noise wall is within a 5 mile radius of the destination or area being promoted.
- C. Street names and Path names are acceptable provided the bridge carries that public street or public path. Private street names or private path names are not permissible.
- D. Obtain FHWA acceptance, if necessary.
- E. Lettering or logos shall not extend above the top of bridge railing or fence. Lettering or logos shall not extend below the normal lines of the bridge superstructure.
- F. Lettering or logos on bridge substructure units shall be within the normal limits of those units. Do not add extraneous elements for the sole purpose of displaying lettering or logos.
- G. Lettering or logos shall follow all applicable design and placement guidelines listed in the [L&D, Vol. 1 Section 904.2.1](#).
- H. Provide 5 copies of rendering(s) of the proposal with the request.

The owner of the bridge should consider obtaining a written agreement regarding operating costs and maintenance responsibilities with local agency requesting the lettering or logo enhancements.

The local agency requesting the lettering or logo may be required to fund the additional cost over what ODOT would normally install on the bridge or noise wall.

- A. ODOT traditionally uses standard concrete form liners on bridges and noise walls. If the lettering or logo will require additional or custom concrete form liners, the cost over a standard concrete form liner may be required to be secured/funded by the local agency.
- B. ODOT traditionally places vandal protection fencing on bridges. If the lettering or logo will require non-standard fence or supports, the cost over the standard fence may be required to be secured/funded by the local agency.

310.6 RAILWAY BRIDGES

Contact the Ohio Rail Development Commission (ORDC) for assistance with railroad-roadway coordination including overpass design requirements.

When temporary shoring details are required for construction of substructure units adjacent to railway tracks, include details in the plans. When considering excavation for substructure units, address whether sheet piling can be driven (avoid existing footing, clear any battered piles, elevation of bedrock, etc.) and whether the proper lengths can be provided to retain the railway tracks. Do not allow settlement of the tracks to occur.

310.7 BICYCLE BRIDGES

For new structures, the minimum bridge width shall be the same as the width of the paved bicycle path and approach shoulders. Use a minimum transverse slope of 1/4 in per foot sloped in one direction. Provide bicycle railings at a minimum of 3.5-ft high. For the design of the railing refer to *AASHTO LRFD Article 13.9*. If an occasional maintenance vehicle is going to use the bridge, design the railing as only a bicycle railing. Use bicycle safe bridge deck joints.

If a timber deck is used, apply a 1-1/2-in minimum thickness of ITEM 441 - ASPHALT CONCRETE SURFACE COURSE, TYPE 1, PG64-22, in order to provide an abrasive skid resistant surface.

C310.6

Ohio's largest railroads, CSX and Norfolk Southern, publish Public Project Manuals to assist public entities in interacting with their organizations. Regional and short-line railroads often publish information on their websites to identify appropriate processes and contacts. Links to CSX and Norfolk Southern Public Project Manuals are available on the ORDC website.

For railway overpasses the specific requirements of the railway company involved need to be addressed. The design and operational requirements of the railway companies will vary from railway line to railway line and between companies. Some of the common railway concerns are as follows:

- A. Horizontal and vertical clearances for both the final proposed design and during construction,
- B. The constructability of the substructure units adjacent to their tracks,
- C. Allowing adequate clearances for drainage ditches and access roads that are parallel to their tracks,
- D. Location of railway utilities, and
- E. Provisions for crash walls on piers.

Drainage from the bridge should be collected in drain pipes and drained away from the railway right of way

Interlocking sheet piling of cantilever design is preferred. It may be appropriate to leave the temporary shoring in place after construction.

C310.7

Refer to the ODOT [Multimodal Design Guide](#), for more information and resource references.

Other alternative surfaces may be used if approved by the Department.

310.8 PEDESTRIAN BRIDGES

Meet the grade and cross slope requirements specified in [L&D, Vol. 1](#), Section 306.2.5 for pedestrian facilities. For pedestrian bridges over highways, provide an additional one foot of vertical clearance.

If a timber deck is used, apply a 1.5-in minimum thickness of ITEM 441 - ASPHALT CONCRETE SURFACE COURSE, TYPE 1, PG64-22, in order to provide an abrasive skid resistant surface.

C310.8

Other alternative surfaces may be used if approved by the Department.

310.9 MAINTENANCE AND INSPECTION ACCESS**C310.9**

Maintenance and inspection access requirements should be included in the Structure Type Study, Narrative of Bridge Alternatives. For multiple span bridges with 8-ft or deeper girders, an inspection handrail located on the girders should be provided. Catwalks should also be considered. Safety cables and other fall arrest systems should be considered in addition to handrails and catwalks. Provisions for maintenance and inspection access should be provided for **NSTM** girders, cross girders and bents that cannot be inspected from a snooper. The use of **nonredundant steel tension** members is strongly discouraged. For these types of structures, consult the Office of Structural Engineering for details and recommendations.

310.10 SIGN SUPPORTS**C310.10**

Make every effort to locate overhead sign supports off bridge structures. When this is not possible, only two locations on the structure are acceptable and are listed below in order of preference:

Research has shown that overhead sign supports located on bridges are highly susceptible to fatigue damage.

- A. Mounted directly to the substructure unit.
- B. Mounted to the superstructure directly over a substructure unit.

Underpass sign supports attached to the fascia of overpass bridges, as shown on Plan Insert Sheets: PIS 201824 and PIS 201826, should also be avoided. Consult with the District Bridge Engineer before specifying their use.

310.11 STRUCTURAL GROUNDING & LIGHTING**C310.11**

Provide structural grounding for the following structures:

See Sections 1140 and 1150 of the [TEM](#) for additional information.

- A. Structures with metal railing
- B. Structures with fencing
- C. Structures carrying electrical utilities or have overhead electrical utilities.

Provide underpass or tunnel lighting as required by Section 1103-6.9 of the [TEM](#).

311 BURIED STRUCTURES

For hydraulic design, foundation and scour requirements, refer to the L&D Vol 2.

The structural design of buried structure components shall be in accordance with the latest edition of the *AASHTO LRFD Bridge Design Specifications*.

311.1 HALF-HEIGHT HEADWALLS

For buried structures that utilize half-height headwalls, provide a pay item for ITEM 602 – CONCRETE MASONRY.

For corrugated metal pipe and plastic pipe, provide a reference to Standard Bridge Drawing HW-2.1. For concrete pipe, provide a reference to Standard Bridge Drawing HW-2.2. For non-standard half-height headwalls, provide the complete details in the Plans.

311.2 FULL-HEIGHT HEADWALLS

311.2.1 CIRCULAR, ELLIPTICAL AND PIPE ARCH STRUCTURES

For circular, elliptical and pipe arch buried structures utilizing full-height headwalls, provide a pay item for ITEM 602 – CONCRETE MASONRY.

Provide a reference to Standard Bridge Drawing HW-1.1 when applicable. For non-standard full-height headwalls, provide the complete details in the Plans.

311.2.2 PRECAST CONCRETE BOX, FLAT TOPPED, ARCH & ROUND SECTIONS

Use one of the following methods of Plan preparation for projects with precast concrete box, [C&MS](#) 706.05; three-sided flat topped, [C&MS](#) 706.051; arch, [C&MS](#) 706.052; and round, [C&MS](#) 706.053:

A. Cast-in-place full-height headwalls

C311.1

[C&MS](#) 602.03.D permits the Contractor to use precast concrete for circular conduits up to a maximum of 78-in and elliptical and pipe arch conduits up to a maximum 78-in round equivalent.

C311.2.1

The Department will not permit the use of precast concrete structures for these types of full-height headwalls.

C311.2.2

The selection of a Plan preparation strategy for full-height headwalls for these structure types should occur during project scoping. Projects with time-sensitive closure periods in the construction schedule should consider precast full-height headwalls.

Refer to BDM Figure 311-1 for an illustration of headwall component terminology.

The Contractor will not have an option in the [C&MS](#) to provide precast structures for cast-in-place full-height headwalls. These would need to be considered according to [C&MS](#) 105.19 Value Engineering Change Proposals.

1. Provide cubic yard pay items for ITEM 511 – CLASS QC1 CONCRETE, HEADWALL; ITEM 511 – CLASS QC1 CONCRETE, RETAINING/WINGWALL NOT INCLUDING FOOTING; and ITEM 511 – CLASS QC1 CONCRETE, FOOTING. For precast concrete box structures, [C&MS 706.05](#), refer to the Design Data Sheet for Concrete Headwalls for Precast Box Culverts for complete designs; to the Plan Insert Sheets for Precast Concrete Box Culverts for pre-detailed Plan sheets; and to Design Data Sheet HWDD_Rebar for concrete reinforcement bar list development.
 2. Provide Plan and Elevation views of the headwalls showing wingwalls, foreslope walls, and footings including cutoff walls at each end of the conduit detailing all wall dimensions, angles and elevations. For additional information regarding Plan content for drainage structures, refer to the Sample Plans and the [L&D, Vol. 3](#) provided on the Office of CADD and Mapping Services webpage.
 3. Provide Plan, Elevation and Section views of the headwalls showing wingwalls, foreslope walls, and footings including cutoff walls detailing all reinforcement locations and bar marks.
 4. Provide a schedule of concrete reinforcement for all bars in the wingwalls, foreslope walls, and footings including cutoff walls.
- B. Precast full-height headwalls
- Do not use precast headwalls where the angle between the centerline of the waterway and the exposed face of the wall is less than 30-degrees.
1. Provide pay items for:
 - a. ITEM 851 (SQUARE FOOT) – PRECAST GRAVITY AND SEMIGRAVITY RETAINING WALL
 - b. ITEM 851 (DAYS) – ON-SITE ASSISTANCE
 - c. ITEM 851 (LUMP SUM) – PGSRW INSPECTION AND COMPACTION TESTING

Contractors are required to select precast gravity or semigravity retaining wall systems preapproved by the Office of Geotechnical Engineering for full-height headwalls paid for with [SS851](#).

The pay items listed here are the intended as the minimal items necessary to install the headwall structures. Additional items (e.g. Structure Removed, Cofferdams and Excavation Bracing, Sealing of Concrete Surfaces, etc.) should be included as necessary for each project. The Department will consider all PEJF required for the precast wall installation to be included in the payment for the wall.

Refer to the Designer Note for [SS851](#) for more information regarding the use of the pay items listed. There are additional pay items provided in [SS851](#) not listed herein because they may not be applicable for all buried structure applications or may be paid elsewhere under separate pay items (e.g. Embankment, Using Natural Soils, 703.16A, 6" drainage pipe, etc.).
 - d. ITEM 851 (LUMP SUM) – WALL DRAINAGE SYSTEM
 - e. ITEM 503 (CUBIC YARD) – UNCLASSIFIED EXCAVATION
 - f. ITEM 509 (POUND) – EPOXY COATED STEEL REINFORCEMENT
- This pay item includes all backfill, filter fabric, prefabricated geocomposite drains, pipes and pipe outlets required to drain the precast walls.
- This pay item is intended for excavation to install the conduit footings and wingwalls.
- Include the reinforcement detailed in the footings, cutoff walls, closure pour and foreslope walls only.

- | | | |
|----|---|--|
| g. | ITEM 511 (CUBIC YARD) – CLASS QC1
CONCRETE, FOOTING | Include concrete in the footings, cutoff walls and closure pours only. |
| h. | ITEM 511 (CUBIC YARD) – CLASS QC1
CONCRETE, HEADWALL | Provide a plan notes clarifying that this quantity represents the foreslope wall located atop each end of the conduit. |
| i. | ITEM 518 (CUBIC YARD) – POROUS
BACKFILL WITH FILTER FABRIC | |
2. Provide Plan, Elevation and Section views of the headwalls showing wingwalls, foreslope walls and footings including cutoff walls at each end of the conduit detailing all wall dimensions, angles and elevations. In the Plan view, show the location of the outside face of the wall.
- For wall designs under [SS851](#), the Contractor will be responsible for providing the structural design and detailing of all precast walls including modifications to the footings necessary to accommodate the precast walls. The Designer is responsible for providing a fully detailed footing and assumed wall face location that will satisfy foundation design requirements of BDM Section 305 and fit within the construction limits and other constraints required at the site. For the foundation stability analyses, assume a wall stem thickness equal to that used in Standard Bridge Drawings HW-1.1 or HWDD-1, as applicable, for the type of culvert being constructed. For additional information regarding Plan content for drainage structures, refer to the Sample Plans and the [L&D, Vol. 3](#) provided on the Office of CADD and Mapping Services webpage.
- For precast concrete box structures, [C&MS](#) 706.05, refer to the Design Data Sheet for Concrete Headwalls for Precast Box Culverts to detail wall and footing geometry.

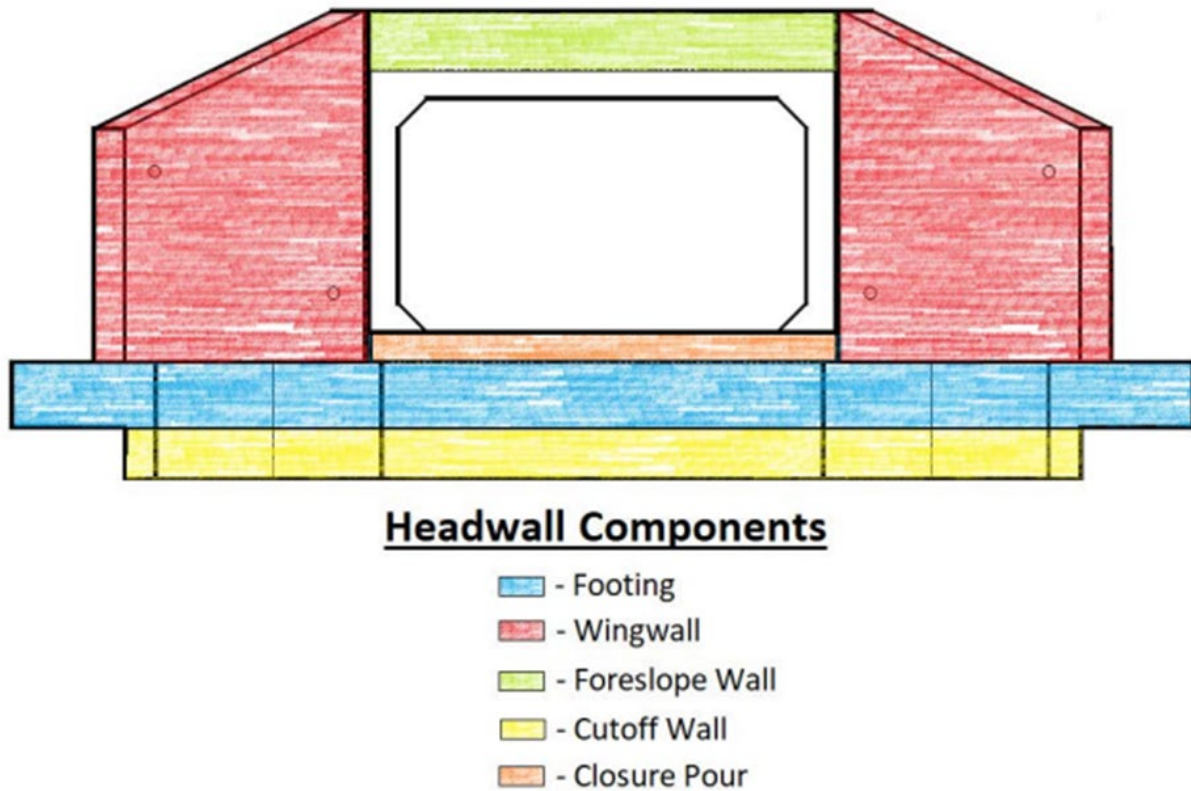


Figure 311-1

312 REDUNDANCY**C312**

The purpose of this section is to provide policy and commentary for the classification of members as redundant or non-redundant. When members are deemed to not have load path redundancy, the sections concerning system redundancy and internal redundancy offer options on how to redesignate members previously classified as an NSTM. These provisions may be utilized to analyze and designate of both existing and proposed members.

For additional information regarding the inspection requirements of these members, refer to the Ohio Manual of Bridge Inspection (OMBI).

Historically, only load path redundancy has been considered when determining a member's redundancy and subsequent design, fabrication and inspection requirements. However, two additional forms of redundancy were introduced in the National Bridge Inspection Standards (NBIS) in 2022. These new classifications for redundancy afford designers the ability to characterize steel tension members that would have previously been considered nonredundant as redundant if it can be proven, through analysis, that the member is redundant. This section will cover the necessary resources and analysis procedures required to

determine a member's redundancy in terms of load path redundant, system redundant, internally redundant, or nonredundant

The NBIS defines Load Path, System, and Internal Redundancy as follows:

Load Path Redundancy: A redundancy exists based on the number of primary load-carrying members such that fracture of the cross section at one location of a member will not cause a portion of or the entire bridge to collapse.

System Redundancy: A redundancy that exists in a bridge system without load path redundancy, such that fracture of the cross section at one location of a primary member will not cause a portion of or the entire bridge to collapse.

Internal Redundancy: A redundancy that exists within a primary member cross-section without load path redundancy, such that fracture of one component will not propagate through the entire member, is discoverable by the applicable inspection procedures, and will not cause a portion of or the entire bridge to collapse.

The NBIS also defines Nonredundant Steel Tension Member (NSTM) as follows:

NSTM: A primary steel member fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse.

312.1 LOAD PATH REDUNDANCY

For the strength limit state only, apply a redundancy load modifier (η_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 (η_R) equal to 1.00 for all limit states.

For superstructure types (a) and (k), as defined in *LRFD Table 4.6.2.2.1-1*, consider the primary members as nonredundant when the superstructure consists of less than three girder lines.

For superstructure types (a) and (k), as defined in *LRFD Table 4.6.2.2.1-1*, consider the primary members as nonredundant when the superstructure consists of three longitudinal girder lines spaced greater than 16-ft.

For superstructure types (a) and (k), as defined in *LRFD Table 4.6.2.2.1-1*, consider the primary members as redundant when the superstructure consists of three longitudinal girder lines spaced at 16-ft or less or four or more longitudinal girder lines regardless of spacing.

312.2 FRACTURE CONTROL FOR STEEL MEMBERS C312.2

All steel members without load path redundancy, as defined in BDM Section 312.1, shall be subjected to fabrication Fracture Control regardless of the member designation. Identify these members in the plans as “FC”.

If a member has been designated as system redundant member (SRM) or an internally redundant member (IRM), fracture control is still required for fabrication. Refer to *LRFD 6.6.2.2* for additional information on the designation requirements for the plans.

Until a steel member without load path redundancy, as defined in BDM Section 312.2, has been redesignated to be a system redundant member (SRM) or an internally redundant member (IRM), follow the provisions of BDM Section 312.2 for the load modifier in the design.

The redundancy load modifier (η_R) may be taken as 1.0 for members that have redesignated as SRMs or IRMs.

312.3 MEMBER REDESIGNATION

When evaluating an NSTM element for redesignation as either a SRM or IRM, use one of the following guide specifications:

1. *AASHTO Guide Specification for Analysis and Identification of Fracture Critical Members and System Redundant Members (GSFCM-1-UL)*
2. *AASHTO Guide Specification for Internal Redundancy of Mechanically-Fastened Built-Up Members (GSBSM-1-UL)*

Do not consider any of the following NSTM elements in a redundancy analysis for SRM or IRM redesignation.

- Pins
- Hangers
- Eyebars

When investigating an existing NSTM element for a redundancy analysis, all the following conditions need to be met to for the NSTM element to be eligible.

- Existing structure has undergone a NSTM inspection within the past 24 months.
- No plug welds or discontinuous back-up bar splices
- No fatigue cracks
- No impact damage
- No load posting
- No element level condition state 4 on the NSTM member
- The member under consideration does not warrant an NSTM component condition rating of 6 or less.

This criterion is only applicable on bridges that receive element level inspections.

The objective of this provision is to allow an analysis of an individual NSTM member in good condition when the structure may have an overall NSTM component condition rating of fair or poor.

- Not susceptible to constraint induced fracture

Refer to Publication No. FHWA-HIF-21-046: Evaluation of Steel Bridge Details for Susceptibility to Constraint-Induced Fracture for additional information.

312.3.1 SYSTEM REDUNDANCY

C312.3.1

Use a nationally recognized method to evaluate a member as a SRM in lieu of NSTM. If required, conduct a three-dimensional finite element system analysis (FEA) to classify a member as a SRM in lieu of NSTM. The software used to perform the FEA shall meet the minimum requirements in Section 3.1.1 of *GSFCM-I-UL*.

If a method other than FEA is chosen for the redesignation analysis, this method shall be approved by the Office of Structural Engineering prior to the analysis.

Meet all applicable requirements presented in Sections 4, 5, 6, and 7 of *GSFCM-I-UL* in the FEA.

Guidance on Material Models, Meshing Requirements, Interactions, Constraints, Application of Loads, and Boundary Conditions are given in Sections 4, 5, 6, and 7 of *GSFCM-I-UL*.

Consider all applicable failure scenarios outlined in Section 3.1.3 of *GSFCM-I-UL* in the FEA.

312.3.1.1 LOADS & LOAD COMBINATIONS

Use the provisions of *GSFCM-I-UL* to investigate two distinct redundancy load combinations. For each redundancy load combination, apply the total load, Q_u , calculated using the following equation:

$$Q_u = (1 + DA_R)[\gamma_{DC}DC + \gamma_{DW}DW + \gamma_{LL}(LL + IM)]$$

Where:

γ = Load factor

DC = Dead load of structural components and nonstructural attachments

DW = Dead load of wearing surfaces and utilities

LL = Vehicular live load

DA_R = Dynamic amplification factor

IM = Vehicular dynamic allowance

If bridge members were fabricated using the AASHTO/AWS D1.5 FCP use the load factors presented in BDM Table 312-1.

Table 312-1: Bridge Members Fabricated using AASHTO/AWS D1.5 FCP

Load Combination	γ_{DC}	γ_{DW}	γ_{LL}	DA_R	IM
Redundancy I	1.05	1.05	0.85	0.40	0.00
Redundancy II	1.05	1.05	1.30	0.00	0.15

If bridge members were not fabricated using the AASHTO/AWS D1.5 FCP use the load factors presented in BDM Table 312-2.

Table 312-2: Bridge Members Not Fabricated using AASHTO/AWS D1.5 FCP

Load Combination	γ_{DC}	γ_{DW}	γ_{LL}	DA_R	IM
Redundancy I	1.15	1.25	1.00	0.40	0.00
Redundancy II	1.15	1.25	1.50	0.00	0.15

Dead loads for proposed structures shall consist of all gravity loads, including future loads. Apply dead loads as body forces.

For existing structures, apply only the dead loads present on the structure at the time of analysis. A new analysis will be required when the loading on the bridge changes. Refer to BDM Section 914 for a list of loading changes that warrants a new analysis.

Live Loads shall consist of the HL-93 vehicular live load as defined in *LRFD Section 3.6.1.2*. Apply the live loads in each load combination as described below.

Redundancy I: This is the load combination relating to a point-in-time live load applied at the instant when the assumed failure of the member occurs. The intent of this loading scenario is to capture the effects of dynamic amplification immediately following the member failure.

- The number of lanes is the number of striped travel lanes only. Center the live load at the center of the striped lanes.

Redundancy II: This is the load combination relating to normal vehicular use of the bridge without wind after the failure of a primary member. The intent of this load combination is to capture the loading after the fracture has occurred and the structure has reached a steady state.

- Use *LRFD Section 3.6.1.1.1* to determine the number of lanes. Position the lanes transversely to produce the largest demand on the remaining unfractured components.

312.3.2 INTERNAL REDUNDANCY**C312.3.2**

The *GSBSM-I-UL* groups similar types of built-up members into flexural, two-component axial, and multi-component axial. Flexural members are bolted girders or beams with at least one tension cover plate. Two-component axial members are composed of only two steel sections bolted together so if one section fails, the other will take the full load. Multi-component axial members are composed of 3 or more steel sections bolted together and providing multiple load paths if one section fails.

Evaluate members at the worst-case stress locations for each individual cross section configuration in the faulted state for strength in accordance with the guide specifications. Any location or cross section of a member failing the strength check in the faulted state is not eligible to be classified as internally redundant.

For members passing the strength checks in the faulted state, evaluate fatigue for both unfaulted and faulted states to gain an understanding of the potential life of the member before a fatigue crack may form.

Calculate inspection intervals based on the fatigue life in accordance with the Guide Specifications based on the analysis results.

Inspection intervals are likely to differ along the bridge at each cross section analyzed. For each structure, the owner will determine if the critical locations will all be inspected at the lowest inspection interval allowed or if inspection intervals at each location will be based on what is allowed per the analysis.

Fatigue may be evaluated using field measured effective stress ranges at the desired cross sections in the structure to potentially obtain more favorable fatigue life calculations.

312.3.3 PERFORMANCE IN THE FAULTED STATE

To determine if an existing or new steel tension member without load path redundancy can be designated as a SRM or an IRM in lieu of NSTM can be redesignated as SRM or IRM in lieu of NSTM, the member shall meet performance criteria developed to establish the minimum acceptable performance level in the faulted state.

These elements shall meet all applicable Strength, Service, and Extreme Event requirements detailed in *AASHTO LRFD Bridge Design Specification* and this manual.

For designation as SRM, meet the Performance Criteria detailed in Section 8 of *GSFCM-I-UL*, except as noted in this manual.

Section 8.2.1 of the *GSFCM-I-UL* requires an analysis with the live load at a 15% increase, this shall apply only to existing members being investigated for redesignated. Proposed members being designated as SRM shall be designed to remain linearly elastic at all load combinations.

For designation of existing members as IRM, designs shall meet the Performance Criteria detailed in Section 2 of *GSBSM-I-UL*. Proposed members being designated as IRM shall be designed to remain linearly elastic at all load combinations.

312.3.4 SUBMITAL REQUIREMENTS

When designating existing members or designing proposed SRMs or IRMs, submit the following information with the plans as an independent report:

Stage 1 Plan Submittal:

- Nationally recognized method intended for use for determining redundancy.
- Engineer to be overseeing the analysis and their experience with regards to Steel Design, Finite Element Modeling, and any other relevant experience.
- The software or computation method intended for the evaluation redundancy. (System Redundancy Analysis requires FEA modeling)
- The Expected Special inspection interval

Stage 2 Plan Submittal:

- Nationally recognized method used for determining redundancy.
- Engineer that oversaw the analysis and their experience with regards to Steel Design, Finite Element Modeling, and any other relevant experience.
- Engineer who performed the Technical Quality Review and their associated experience.
- The software or computation method used for the redundancy evaluation. (System Redundancy Analysis require FEA modeling)
- A summary of the results of the analysis.
- The recommended special inspection interval.

- Any additional requirements that should be added to the routine inspection of the structure. (i.e. Access considerations or Areas of focus)
- Site specific requirements for the special inspection needed to determine if a fracture has occurred and to potentially find damage that could affect the structural integrity.
- Criteria to define when the member would default back to an NSTM.
- A comprehensive Calculation Package shall be included in the Final Calculation Submittal.

312.3.4.1 SPECIAL REQUIREMENTS FOR THREE-GIRDER STRADDLE BENTS

Stacked Three-Grider Straddle bents are load path redundant.

Stacked Three-Grider Straddle Bents, where the longitudinal girders are “stacked” on top of the Straddle Bent, and a bearing is provided under each I-girder web comprising the straddle, have been nationally recognized as having load path redundancy and no system redundancy analysis is required.

The analysis may consider all bracing, continuous framed girders, deck (and all reinforcement), and continuous concrete barriers as being effective for load transfer.

Integral straddle bents, where the longitudinal girders frame into the side of the straddle bent, will need to be analyzed for system redundancy regardless of the number of bearings that are used under the beam.

Consider fractures at points of maximum load effects at the Strength I load combination in the analysis. If only one bearing is used under the straddle bent, show that the load transfer to the single bearing can be maintained by adding more fracture analysis cases to diaphragms or other critical members as may be necessary.

These high stress values may occur for symmetrically loaded simple supported members at midspan and near the support of one of the exterior girders of the three-girder system.

When the three-girder straddle bent is subjected to the dead load portion of the Redundancy II load combination in the faulted state, the primary steel members with any tension shall be designated as NSTMs if the straddle cap experiences a maximum vertical deflection larger than $L/50$.

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SECTION 400 – REHABILITATION AND MAINTENANCE

401 INTRODUCTION

BDM Sections 100, 200 & 300 apply for the design of Rehabilitation and Maintenance to existing bridges except as noted in BDM Section 100.

Refer to BDM Section 101 for a description of the format, audience, purpose and more related to this Manual.

401.1 GENERAL CONSIDERATIONS

Perform a site visit to the structure and identify/verify defects that may exist, in order to tailor solutions to all problems found. During this site visit, take color photographs and develop sketches showing pertinent details and field verified dimensions. Schedule this visit during the plan development phase of the project.

Verify all pertinent field dimensions. Do not take dimensions directly from old plans without checking them in the field because deviations from plans are common. The Designer is responsible for providing accurate bid information based on field observations and preparing plans that reflect the actual field conditions. Under [C&MS](#) 105.02, the Contractor is responsible for taking measurements of existing structures to accurately join new and existing work. Include the plan note provided in BDM Section 603 entitled “EXISTING STRUCTURE VERIFICATION” to clarify the Department’s contractual approach to uncertainties.

401.2 DESIGN SPECIFICATIONS

Design rehabilitations to existing structures in accordance with the latest edition of the *AASHTO LRFD Bridge Design Specifications* and this Manual.

401.3 DESIGN LOADING

The Design Loading for the rehabilitation of vehicular bridges shall be in accordance with the *AASHTO LRFD Bridge Design Specifications, Section 3*. The Design Loading for bridges carrying high speed (i.e. Design Speed > 45 mph) National Highway System traffic shall include a dead load allowance for future wearing surfaces equal to 0.06-ksf applied to the deck surface for the full length of the bridge from face-to-face of curbs and/or railing. No allowance for future wearing surfaces is required for all other bridges.

C401.1

For individual members, determine whether the best option is to repair or replace. In making this decision, consider cost along with factors such as traffic maintenance, convenience to the public, longevity of the structure, whether the rehab is long term or short term, and the practicality of all options.

C401.3

Include the Vehicular Live Load and Future Wearing Surface allowance in the Proposed Structure Block on the Site Plan as specified in BDM Section 201.1.2.2G.

401.4 DESIGN EXCEPTIONS

Bridge rehabilitation and maintenance projects that do not require a revised load rating according to BDM Section 914 will not require a Design Exception for Design Loading Structural Capacity.

Evaluate bridges that do require a revised load rating according to BDM Section 914 for a Design Exception for Design Load Structural Capacity as follows:

- A. For a bridge that will receive a new overlay according to BDM Section 403.4.1 with a resulting **Operating** level rating factor less than 1.08 for all Ohio Legal Loads defined in BDM Section 908.3, a Design Exception is required.

- B. When the Scope of Services requires rehabilitation of a superstructure resulting in an Inventory level rating factor less than 1.00 for HL-93 loading, a Design Exception is required.

To determine the governing inventory level rating factor, load rate the superstructure in accordance with BDM Section 900.

- C. When the Scope of Services requires substructure and foundation analysis for a rehabilitation resulting in any member of the bridge substructure or foundation with a capacity-to-demand ratio less than 1.00 for the Design Loading defined in BDM Section 401.3, a Design Exception is required.

To determine the governing capacity-to-demand ratio analyze the substructure members specified in the Scope of Services in accordance with the *AASHTO LRFD Bridge Design Specifications*. The capacity-to-demand ratio for each member analyzed shall be determined by rewriting *LRFD Eq. 1.3.2.1-1* as follows:

$$\frac{C}{D} = \frac{\phi R_n}{\sum \eta_i \gamma_i Q_i}$$

C401.4

The purpose for this provision is to identify bridges receiving a new rigid overlay that will require Bridge Posting for Reduced Load Limits in accordance with BDM Section 919. An Owner needs an approved Design Exception in order to fund a bridge rehabilitation project that leaves the structure capacity below Ohio Legal Load limits. Refer to BDM Section 919.3.1H for information on which LFR or LRFR rating method applies.

This requirement is not intended to dictate the level of analysis required for every existing structure. It simply sets the minimum structural capacity for an existing structure or portions of an existing structure that are to be analyzed. The Scope of Services should define the portions of the structure to be analyzed.

With regard to establishing what portions of a structure to analyze, it is not economically feasible to require an analysis for the entirety of every existing bridge in need of rehabilitation. The importance of the route carried by the bridge and the existing condition of the bridge are factors to be considered. Routes of high importance may consist of interstates, major routes, or high traffic roadways connecting urban areas.

In general, the load capacity of existing substructure and foundation elements may be assumed to be adequate for reuse without a detailed structural analysis when:

- A. The substructure elements are in good condition (NBIS Condition Rating of 6 or greater) and show no significant structural distress under existing live load, AND

- B. The proposed service dead load is not greater than 115% of the original designed service dead load at the top of the substructure element (top of bearing seat), AND
- C. There is no significant reconfiguration of load configuration (i.e. changes to bearing locations or substructure fixities).

Additional guidance can be found in BDM Section 912 and BDM Section 309.3.8.2.a.

It may not be feasible to increase the capacity of an existing member to the Design Loading for every structure. Designers should investigate the cost and feasibility to increase capacity to alternative service levels during preliminary engineering and consider requesting a Design Exception.

In addition to the documentation required by [L&D, Vol. 1](#), include a copy of the Load Rating Report with the Design Exception request for Design Loading Structural Capacity. Include rating factors for all rating vehicles defined in BDM Section 908.

Rating vehicles include: Ohio Legal Commercial Vehicles (2F1, 3F1, 4F1 & 5C1); Special Hauling Vehicles (SU4, SU5, SU6 & SU7); and Emergency Vehicles (EV2 & EV3).

Provide a statement in the Design Loading General Note when an approved Design Exception for Design Loading Structural Capacity has been approved.

401.5 PLAN PREPARATION

401.5.1 GENERAL PLAN

In addition to the requirements listed in BDM Section 103, provide a General Plan sheet for:

- A. Deck overlay projects
- B. Deck replacement projects where the bridge deck is variable width or curved
- C. Rehabilitated bridges requiring staged construction.

401.5.2 NARRATIVE OF BRIDGE ALTERNATIVES

In addition to the Narrative of Bridge Alternatives requirements listed in BDM Section 201.1.2.3, include color photographs of the portions of the existing structure to be salvaged. To substantiate the proposed salvage decision, identify all areas of rehabilitation by field investigation.

For steel superstructures, include the findings of the Fatigue Evaluation defined in BDM Section 404.1.2.2.

401.5.3 COST ANALYSIS**C401.5.3**

When a rehabilitation alternate involves salvaging existing concrete members, cost overruns should be anticipated and included in the cost analysis.

401.5.4 FINAL STRUCTURE SITE PLAN**C401.5.4**

In addition to the requirements listed in BDM Section 201.2.1.1, the first item in the “Proposed Structure” data block shall be “Proposed Work” followed by a brief description of the type of work to be done.

Examples description of “Proposed Work”: new composite reinforced concrete deck, new superstructure on existing substructure, or new superstructure on widened substructure

Provide a relatively thorough description (list of work) of the type of work to be done within a plan note entitled “Proposed Work” and include this note on the Final Site Plan.

402 CLEANING BRIDGES**C402**

When specified in the Scope of Services, clean bridges as follows. Clean the three areas of the bridge listed below:

These cleaning provisions apply to bridges which have routine maintenance responsibility provided by ODOT.

- A. Expansion Joints
- B. Scuppers and Drainage Troughs
- C. Bridge Seats under expansion joints

Benefit for Cleaning of bridges:

Perform Bridge Cleaning in accordance to the Bridge Cleaning plan insert sheets located on the Office of Structural Engineering website.

Cleaning Deck Joints – Cleaning deck joints helps the joint perform as intended. Debris that sits on top of an elastomeric seal is packed down by traffic. A sharp object will exert a great deal of pressure as traffic travels over the packed down debris. This damages the elastomeric seals and they will leak and allow debris to fall onto the bridge seat below. Cleaning the deck joints will help keep the joints functioning as designed.

Clearing Debris from Scuppers/Drainage Troughs – Controlling deck drainage is important to maintaining a bridge in good condition. When chloride laden deck drainage does not follow the path that was originally intended, deterioration of bridge components may begin and accelerate. Components of the bridge that remain damp with high chloride content are prime locations for deterioration. Clearing debris from scuppers and drainage troughs will not pond chloride laden drainage on the bridge deck and will increase the life of the bridge deck.

Cleaning Bridge Seats – Debris built up on bridge seats holds moisture against the bearings and seats. The moisture laden debris will cause the steel sections of bearings to rust and deteriorate and cause spalling of the concrete seat. The debris generally deposits on the bridge seat through an open expansion deck joint or a defective deck expansion joint. Keeping the bridge seats free of debris will extend the life of the bridge

403 DECK**C403**

Preparing the existing concrete for any sealing, patching, or overlay is very important for a durable repair. There are many products used in these processes all require a degree of removing deteriorated and damaged concrete, leaving a sound surface for the repair material to bond. BDM Table 403-1 provides a guide for concrete removal types and application to be considered.

Table 403-1

Reference Guide for Concrete Removal Types and Application		
Concrete removal technique	Maximum removal concrete depth	Applications
Grit blast	Surface to 1/16-in per pass	Texturing and cleaning of concrete surface, removing fractured concrete after other method has been applied to remove concrete
Shot blast	Surface to 1/4-in	Texturing and cleaning of concrete surface. Can be used to removing fractured concrete after other method has been applied to remove concrete. Creates more aggressive surface profile than grit blasting
Diamond grind	3/16-in to 1/4-in per pass	Effective in restoring smoothness and skid resistance on existing concrete decks.
Scarification	Depth to top of concrete reinforcement.	Texturing, removal, and cleaning of concrete
Hydro demolition	1/4-in typical (to entire depth)	Texturing, removal, and cleaning of concrete surface, remove concrete around reinforcement bars
Pneumatic hammer (Hand Chipping)	1-in to entire depth	Removal of the entire depth of concrete in localized areas.

403.1 DECK CONDITION SURVEY**C403.1**

For each deck rehabilitation considered on a project, provide a Deck Condition Survey that includes the information listed below. Prepare the survey as near as practicable to the plan preparation stage and before beginning detail design work for deck rehabilitations.

Do not perform any deck evaluation technique unless specified in the Scope of Services.

The Deck Condition Survey provides justification for the deck rehabilitation treatment selected.

There are numerous concrete deck evaluation techniques available to help determine the appropriate treatment option for a deck in need of rehabilitation.

Many of the commonly used techniques are further described in BDM Sections 403.1.1 through 403.1.8. This Manual does not present an exhaustive list of concrete testing methods; however, a combination of the techniques identified herein will provide a sound basis for identifying an appropriate treatment method. Refer to the project Scope of Services to determine which techniques to apply.

- A. All bridge inventory information
- B. Most recent bridge inspection findings
- C. Age and history of existing and previous wearing course(s) including original deck thickness and concrete reinforcement cover (where available)

Provide as much information on the history of the deck that is available including amount of surface removal for previous overlays.
- D. Dates and results of all evaluation techniques performed on the bridge deck

Refer to the Scope of Services for which techniques to employ on a project
- E. Sketches of the top and bottom surface of the deck dimensioning: locations of unsound areas; delamination; cracking/crack density; spalling; patching; discoloration, efflorescence and moisture staining; traffic lane locations; and all testing/sampling locations

Be as accurate as possible locating all signs of deterioration. This information shall be the basis for estimating the amount of unsound deck area.
- F. Photos of the top and bottom surface of the deck
- G. Photos and description of all cores and core holes (if applicable)
- H. Estimate of the percentage of total deck surface area deemed unsound
- I. Distribution of the unsound area per travel lanes
- J. Recommendations for deck rehabilitation treatment (e.g. sealing, patching, overlay, replacement) and estimated service life

The Deck Condition Survey may provide multiple recommendations for treatments. Each recommended treatment method shall include an independent cost estimate, MOT considerations and estimated service life.
- K. Cost estimate for rehabilitation
- L. Maintenance of traffic considerations for rehabilitation

403.1.1 HAMMER SOUND & CHAIN DRAG C403.1.1

When specified in the Scope of Services, perform chain drag sounding on the deck surface in accordance with ASTM D4580. Sound the deck underside where there are indications of delamination, water intrusion, discoloration, spalls, efflorescence or other signs of distress and visually mark areas on and under the deck.

Survey a minimum 40% of the traveled lanes.

The Hammer Sound and Chain Drag technique is the most well-established field collection method and requires little training. This method is labor intensive, requires traffic control, and should be scheduled when traffic noise is minimal.

The survey area should focus upon traffic lanes with visual damage or having the most truck traffic.

403.1.2 CORING

When specified in the Scope of Services perform deck coring as follows.

Core concrete decks according to ASTM C42. Determine the number of cores to be taken using the following criteria:

- A. A minimum of two (2) per bridge for bridges with a deck area less than 2500-ft²
- B. A minimum of three (3) per bridge for bridges with a deck area between 2500-ft² to 5000-ft²
- C. A minimum of four (4) per bridge for bridges with a deck area between 5000-ft² to 10,000-ft² with one additional core for each additional 10,000-ft² or part thereof up to a maximum of 6 total.

Take at least one core from an apparently sound area to compare with core(s) taken from questionable areas. Take cores in questionable areas to verify and further define areas of unsoundness. If it is suspected that full depth repair may be required, take full depth cores or at least core to the bottom mat of concrete reinforcement in those areas.

Inspect the cores and core holes for: crumbling; stratification or delamination zones; soundness of aggregate; and depth and condition of concrete reinforcement. If required by the Scope of Services, test core samples for bond strength according to BDM Section 403.1.9; for compressive strength according to ASTM C39; for permeability according to ASTM C1202 and for chloride concentration according to BDM Section 403.1.6.

Plot the location of all cores on a plan view of the deck and provide a description of the cores and all test results with the Deck Condition Survey report.

Unless otherwise specified in the Scope of Services, the depth of concrete cores shall be taken to the bottom of the lower mat of reinforcement.

Submit the cores to the District Bridge Engineer or owner with proper identification.

C403.1.2

Coring is a destructive technique that provides samples for strength, permeability and chloride concentration testing for determining extent of deterioration. More importantly, this technique also provides visual evidence of the integrity of the deck.

For bridge decks with an asphalt overlay, additional cores may be necessary due to the variability of unknowns hidden under the overlay.

Due to the inability to evaluate the floor condition of bridges with stay-in-place forms, the number of minimum cores listed should be increased by one.

The underside of the deck needs to be considered when determining core locations.

Questionable areas are areas that exhibit signs of greater deterioration compared with concrete to be removed with the uniform concrete removal depth. Questionable areas are illustrated by advanced cracking, leaking through the deck, buildup of efflorescence, widespread rust staining, or discoloration of the underside of concrete deck.

Cores to determine compressive strength containing embedded reinforcing can yield erroneous results. Embedded steel should be avoided if possible when compressive strengths are required, concrete cores can be tested if trimmed to eliminate the reinforcement provided core length-to-diameter (L/D) of 1.00 or greater can be attained.

Core depth should be full depth for decks with stay-in-place forms.

Core depths used to verify existing overlay depth or reinforcing depth need not be deeper than the first mat of concrete reinforcement.

Repair core holes using an ODOT approved non-shrink mortar, [C&MS 705.22](#), installed according to the manufacturer's recommended specifications. Seal repair with HMWM according to [C&MS 512.04](#)

403.1.3 GROUND PENETRATING RADAR (GPR)

When specified in the Scope of Services, perform GPR testing in accordance with ASTM D6087. Operators shall be certified by the manufacturer to operate the equipment and process the data. Additionally, operators shall possess 24 months of documented GPR testing and analysis experience.

C403.1.3

Ground penetrating radar can be used to determine depth to reinforcement, deck thickness, overlay thickness, reinforcement position, and defects in the upper half of the deck. Because the GPR unit can travel as fast as 55 mph while collecting data, there's no need for traffic interruption. The test is nondestructive, so there's no follow-up repair work. Unlike other methods, GPR works on bridge decks covered with asphalt overlays. The results are typically reliable and repeatable. This technique does require advanced expertise and training for data collection, processing and interpretation. This makes this technique more cost effective as the deck area increases. Coring in accordance with BDM Section 403.1.2 may be used to supplement GPR results

403.1.4 INFRARED (IR) THERMOGRAPHY

When specified in the Scope of Services, perform Infrared Thermography in accordance with ASTM D4788.

C403.1.4

Infrared Thermography will detect concrete defects by measuring temperature variations on the surface of an element resulting from differences in thermal conductivity within the element. Defective areas of concrete will heat up faster after sunrise and cool down more quickly after sunset when compared to sound concrete. The best time for an IR survey is between 5 and 9 hours after sunrise or sunset; however, cloudiness and precipitation will reduce effectiveness. Accuracy of results is highly dependent upon collecting data during appropriate environmental conditions. When using vehicle mounted systems, this technique is more cost effective and can be less disruptive to traffic control when compared to coring, and half-cell testing. However, this technique will not provide information about the depth of defect. Coring in accordance with BDM Section 403.1.2 may be used to supplement IR results.

403.1.5 IMPACT ECHO

When specified in the Scope of Services, perform a stress wave-based impact echo survey method conforming to ASTM C1383-98a. Provide color contour map of data analysis indicating thickness of deck, location of delaminations, debonding overlays, or voids in post-tension ducts.

C403.1.5

Impact Echo provides benefits over traditional chain dragging of bare concrete decks. The method can detect and assess delamination at various deterioration stages and can detect deeper delamination, evaluate vertical cracks, detect and characterize conditions around rebars and tendon ducts, and can be used as a material evaluation tool.

Conduct impact echo test in all traveled lanes.

The impact echo data collection for delamination detection purpose is relatively slow and requires lane closure.

403.1.6 CHLORIDE CONCENTRATION TESTING

When specified in the Scope of Services, perform chloride concentration testing as follows.

Analyze the chloride ion content of all cores according to ASTM C1218. For projects with no coring requirements specified, drill 1-in diameter holes in the deck to a maximum depth of 2-in, or as specified in the Scope of Services, with a rotary impact drill. Collect powdered concrete samples at every 0.5-in of hole depth. After collecting each sample and before proceeding further, thoroughly clean hole with compressed air to avoid contaminating future sample collection. Label all sample containers by location and sample depth. Analyze the chloride ion content of all collected samples according to ASTM C1218. Repair all holes according to BDM Section 403.1.2. Determine the number of drill holes using the coring requirements of BDM Section 403.1.2.

Prepare a report of all test results. Include a plan view of the deck that identifies all hole locations and chloride ion concentrations.

Repair core holes using an ODOT approved non-shrink, non-metallic material per [C&MS](#) 705.20, installed according to the manufacturer's recommended specifications.

403.1.7 HALF-CELL POTENTIAL

When specified in the Scope of Services, perform Half-Cell potential testing according to ASTM C876. Assume active corrosion is occurring if an observed rebar electrical potential reading is more negative than -0.35-volts measured by copper-copper sulfate reference half-cell. The maximum spacing between test locations shall be 2-ft measured longitudinally and transversely across the entire deck area.

C403.1.6

The purpose of chloride concentration testing is to determine the extent of chloride ingress into the concrete surface. Corrosion of uncoated mild steel reinforcement due to chloride ingress is a primary cause of damage to reinforced concrete structures.

Unlike the case for uncoated mild steel reinforcement, no published literature presents definitive chloride threshold for epoxy-coated mild steel reinforcement. Generally, decks with epoxy-coated steel reinforcement with the original monolithic overlay do not require chloride concentration testing.

It is recommended all decks being considered for a 2nd generation concrete overlay (or greater) receive chloride concentration testing.

For reference; uncoated mild steel reinforcement, corrosion typically exists when the chloride ion content exceeds 2-lb/yd³.

For epoxy-coated mild steel reinforcement, corrosion is suggested to exist when the chloride ion content exceeds 3.5 to 4.0-lb/yd³. The threshold is based upon uncertainties associated with: the quality of the organic coating of the epoxy; damage that could have occurred during transportation or storage of the epoxy-coated reinforcement; loss of adhesion between the coating and the base metal; or damage caused during the original overlay replacement.

C403.1.7

Half-Cell potential surveys are useful for monitoring the effectiveness of concrete overlay protective systems as the bridge deck ages. This is accomplished by comparing Half-Cell potential readings at the same locations once per year using the same ground. Utilizing a Half-Cell survey to designate specific repair areas is not recommended.

In addition to the reporting requirements of ASTM C876, include in the survey report: the date of survey; general weather conditions; a plan view of the deck area showing all Half-Cell test locations; ground location; voltage measurements; and an equipotential contour map.

Table 403-2

Half-Cell Potential	Corrosion Activity
Less negative than - 0.20-V	90% probability of no corrosion
Between -0.20 and - 0.35-V	Uncertain corrosion activity
More negative than - 0.35-V	90% probability of corrosion

Because coated reinforcement does not provide a good electrical connection for the measuring circuit throughout the deck limits, the ASTM C876 standard indicates that the half-cell potential technique is not suitable for measurements involving epoxy-coated and galvanized steel reinforcement. However, there is evidence measurements can be obtained because of coating defects or damage and unprotected rebar ends. Consideration of half-cell testing should only be considered on 2nd generation or later overlays.

403.1.8 ELECTRICAL RESISTIVITY (ER)

When specified in the Scope of Services, perform Electrical Resistivity testing in accordance with ASTM D3633.

C403.1.8

Electrical Resistivity will detect areas susceptible to corrosion due to moisture and chloride penetration. Delaminated and cracked areas, due to increased porosity, will form preferential paths for fluid and ion flow. This will lead to higher moisture and chloride concentrations and higher concrete electrical conductivity, manifesting as a lower electrical resistivity. Electrical resistivity is typically measured in units of $k\Omega\text{-cm}$ using a device called the Wenner probe. Readings less than $5\text{-}k\Omega\text{-cm}$ indicate a very high corrosion rate and readings greater than $20\text{-}k\Omega\text{-cm}$ indicate a low corrosion rate.

This technique requires traffic control and data collection may be time consuming. This makes this technique less cost effective as the deck area increases. However, it can be very effective for determining the severity of localized areas of corrosion.

403.1.9 BOND STRENGTH TESTING

When specified in the Scope of Services, perform bond testing by one of the following methods

C403.1.9

Bond strength testing of the existing overlays may be required when the complete removal of the existing overlay cannot be performed due to depth of removal located at or below the top mat of steel reinforcing.

On site testing BOND-TEST; conduct a pull-off test in accordance with ASTM C1583, "Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off-Method)."

For cores taken perform a Slant Shear Method, ASTM C 1042.

While typically used as QC/QA method for new construction, bond strength of concrete overlays is critical to the performance and capital importance when deciding if any apportion, there are several procedures to measure the bond strength of a concrete overlay.

A pulloff test measures the tensile bond. It is similar to the ACI method for determining the adhesion of epoxies to concrete substrates (ACI 503R, Ref 88). A field test, in which, a core is drilled but not extracted. The drilling is stopped slightly below the depth of the overlay and the test apparatus is attached. Overlays with test specimen having tensile failures inside the original deck concrete are acceptable to remain in place

The Slant Shear Method is a laboratory test, based on ASTM C 1042, and is the most common method for determining bond strengths of epoxies and latex systems used with concrete. Developed to measure the effectiveness of latex in bonding fresh concrete to hardened concrete. Generally, a bond strength greater than 75% of the original deck concrete is acceptable to remain in place.

403.2 PATCHING

403.2.1 SURFACE

Repair spalls in the monolithic concrete deck wearing surface in accordance with [C&MS](#) 256 using a Type B or C patch material.

Repair localized unsound areas within an existing rigid overlay in accordance either [PN511](#), [C&MS](#) 256, [C&MS](#) 519, or [SS844](#).

C403.2.1

A patching repair is appropriate for bridge decks with spalls in the monolithic concrete deck wearing surface comprising less than 10% of the total deck area.

These patch materials typically require 4-6-hour lane closures.

A patching repair is appropriate for localized unsound areas within an existing rigid overlay comprising less than 15% of the total deck area.

When patching decks containing uncoated reinforcement, the addition of embedded galvanic anodes per [SS844](#) will help protect the existing concrete surrounding patched area.

403.2.2 UNDER DECK

For an under-deck spall with exposed uncoated reinforcement, remove all loose concrete, mechanically clean concrete reinforcement and coat with paints containing zinc dust. Provide galvanizing material conforming to ASTM A780 except aerosol spray applications will not be permitted.

C403.2.2

Spalls located in haunches and spalls exposing coated reinforcement typically do not require repairs

When specified in the Scope of Services to patch an under-deck spall located over vehicular, rail or pedestrian traffic, seal the patch material with and FRP wrap/cover in accordance with [PN519](#) - Composite Fiber Wrap System. The patch material shall be in accordance with [C&MS](#) 519 when the deck reinforcement is coated or with [SS844](#) when the deck reinforcement is uncoated.

403.3 SEALING

Seal all major bridges and mainline priority system bridges with a concrete wearing surface as follows:

- A. For decks that are cracked, seal the deck using HMWM or Gravity-Fed Resin according to [C&MS](#) 512.06 every 10 years.
- B. For decks that are not cracked or only minor hairline cracks, seal the deck using SRS (Soluble Reactive Silicate) according to [C&MS](#) 512.05 every 5 years.

403.3.1 GRAVITY-FED RESIN

Seal deck surface with Gravity-fed Resin in accordance with [C&MS](#) 512.06.

403.3.2 SOLUBLE REACTIVE SILICATE (SRS)

Treat deck surface with Soluble Reactive Silicate in accordance with [C&MS](#) 512.05.

In almost all cases the best option for preventing damaged concrete from falling onto traffic is to confine the patch material with Fiber Reinforced Polymer wraps. The type of wrap will be selected by contractor.

C403.3

Sealing bridge decks will extend their service life. ODOT's major bridges are some of the highest value assets that the Department maintains. A couple of ODOT's major bridges have decks that will be extremely difficult and costly to replace (segmental and cable stayed structures). Maintaining these decks in good condition is critical. Replacing the deck on a mainline priority system bridge can be very disruptive to traffic. Maintaining these decks in good condition is important to minimize disruption to traffic. In general, bridge decks should be sealed when they are in good to fair condition. Bridge decks in poor condition should be programmed for rehabilitation.

C403.3.1

Gravity-fed resins are very low viscosity (250 cps, similar to SAE 30 motor oil or maple syrup) epoxy crack fillers. These products are cost effective when used to flood deck surfaces with widespread deck cracking. Sand is broadcast over the sealed surfaces to increase slip resistance. [C&MS](#) 512.06 specifies a minimum 6-hr cure period after placement before surfaces receive traffic. The six-hour cure period could be expected with temperatures around 70 degrees and sunshine to help with the curing process. The sealers will take about two hours additional cure time with every 10 degrees drop in temperature and nighttime application will also slow the curing process further. Please review the QPL on the OMM website for additional information if you have a short duration closure. These sealers need a 36-hr window of dry weather with temperatures between 40°F and 100°F.

C403.3.2

SRS is a concrete treatment that penetrates the pores in the concrete surface, reacts with the concrete, and creates a by-product that fills the porosity. SRS is not intended as a crack filler. SRS can be used on bridge decks that experience early-age cracks with widths of 0.001-in to 0.08-in. Sealers should be applied before the onset of severe damage to be effective.

Reapply SRS periodically to maintain long-term effectiveness. Studies indicate that SRS may provide 5-7 years of reduced permeability and corrosion protection for the deck surface.

403.3.3 HIGH MOLECULAR WEIGHT METHACRYLATE (HMWM)

Seal deck surface with High Molecular Weight Methacrylate in accordance with [C&MS 512.04](#).

C403.3.3

HMWM sealers are extremely low viscosity (25 cps, similar to antifreeze) multi-component crack sealer/fillers. These products are most cost effective when drizzled into isolated deck cracks and cold joints as specified in [C&MS 511.19](#). These products may also be used to flood deck surfaces but are less cost effective as the deck area increases. [C&MS 512.04](#) specifies a minimum 6-hr cure period after placement before surfaces receive traffic. The six-hour cure period could be expected with temperatures around 70 degrees and sunshine to help with the curing process. The sealers will take about two hours additional cure time with every 10 degrees drop in temperature and nighttime application also slow the curing process further. Please review the QPL on the OMM website for additional information if you have a short duration closure. These sealers need a 36-hr window of dry weather with temperatures between 50°F and 85°F.

HMWM is typically limited to maximum crack widths of 10 mils or less and has an effective life of 5-7 years.

HMWM can be aesthetically displeasing (green tint) and is not intended to be a wearing surface replacement.

HMWM can be used on new decks that have extensive cracks.

403.4 OVERLAY

The amount of uniform depth removed from the existing concrete deck surface shall not expose the top mat of reinforcement.

C403.4

Theoretically, there is no limit to the number of overlays a bridge deck may have during its life cycle provided the uniform depth limit is not exceeded. The depth of removal specified for an overlay shall be based upon existing overlay condition, underlying deck condition and material data collected specified in BDM Section 403.1.

The service life of a first-generation overlay may extend the life of a bridge, when applied to a deck in a condition state of 5 or 6. Subsequent overlays may not have the service life extension as the previous.

Concrete overlays are used to establish a new wearing surface, restore ride quality, and protect the underlying deck. Typically, overlays are the preferred option for bridge preservation. See BDM Table 403-4 when selecting an overlay treatment

Do not specify overlays for the sole purpose of adjusting the profile grade.

Do not specify an overlay on new decks.

Specify RPM's in accordance with the [TEM](#).

Waiver of this requirement should only be considered for bridges that have decks that are not replaceable (e.g. cable stayed bridge decks, Post-tensioned box girder superstructures, and superstructure members with integral wearing surfaces).

For optimal performance of the overlay, RPM's should be placed at the minimum requirements.

403.4.1 RIGID CONCRETE

Provide a rigid concrete overlay removal in accordance with either [SS847](#) – Bridge Deck Repair and Overlay With Concrete Using Scarification And Chipping or [SS848](#) – Bridge Deck Repair And Overlay With Concrete Using Hydro-Demolition. Available overlay material includes:

- A. Micro-Silica Modified
- B. Latex Modified
- C. Superplasticized Dense
- D. Polyester Polymer Concrete (PPC)

C403.4.1

SS847 and SS848 provide different methods of surface preparation for the installation of the rigid concrete overlay. [SS847](#) requires mechanical scarification of the existing concrete surface. [SS848](#) requires mechanical scarification with hydro-demolition of the existing concrete surface.

[SS847](#) is the most appropriate overlay application for the following bridge decks:

- A. Replacing a well-bonded overlay on an existing deck
- B. Placing an overlay on an existing deck with a uniform depth of distress
- C. Replacing an existing debonded overlay with a limited amounts of hand chipping (variable depth removal)

[SS848](#) is the most appropriate overlay for situations with higher amounts of deterioration. [SS848](#) requires less labor to remove unsound concrete beneath the uniform removal thickness because the variable depth material removed by high pressure water in [SS848](#) is removed by chipping hammers in [SS847](#). [SS847](#) becomes less cost effective as the variable depth area increases. However, on bridges with very little to no unsound area, [SS847](#) is more cost effective as the cost to collect the hydro-demolition run-off water is eliminated.

Hydro-demolition ([SS848](#)) is more appropriate on 2nd generation and 3rd generation overlays as these tend to involve a higher quantity of variable depth removal.

One way to reduce costs associated with collection and disposal of hydro-demolition waste water is to identify a location for the disposal that is compliant with the Ohio EPA General Wastewater Disposal System Permit to Install (PTI) for Land Application of Hydro-Demolition Wastewater. More information is available in the Designer Note for [SS848](#).

The following table provides an estimate for the variable thickness quantity for plan development based on the measured area of unsound deck defined in the Deck Condition Survey (BDM Section 403.1). The plan estimate should be increased by a factor of 1.10 when the deck survey data is one winter old.

The Variable Thickness values in the table are for a 1st generation overlay. The factors should be increased by 1.15 for 2nd generation and greater overlays.

Table 403-3

Measured Unsound Area (% Total Deck Area)	Estimating Factor	Variable Thickness (% Total Deck Area)
0-10	3.00	0-30
15	2.33	35
20	2.00	40
25	1.90	47.5
30	1.87	55
35	1.79	62.5
40	1.69	67.5
45	1.56	70
50	1.50	75
60	1.33	80
70	1.21	85
80	1.09	87.5
90	1.00	90

Benefits derived from a deck replacement when compared to an overlay are approximately equal when the amount of unsound deck area is in the range of 60% to 80% of the total deck area. Therefore, unless overriding circumstances exist, replacement of the existing decks should be considered rather than a deck overlay when the unsound area exceeds 70% of the total deck area.

Do not specify a uniform removal depth “D” that will expose the top mat of deck reinforcement. Provide a plan note to specify “D” differing from the default values specified in the applicable overlay specification.

To maintain structural integrity, avoid rehabilitation methods that expose large areas of the top mat reinforcement where tension exists (e.g. cantilevered deck edges and negative moment regions over piers). This is especially critical for slab superstructures.

[SS847](#) and [SS848](#) define a uniform removal depth “D” as the distance from the bottom of the uniform removal to the top of an existing deck without an overlay or to the top of an existing deck after the removal of an existing overlay.

For [SS847](#), the default value for “D” is 0.25-in.

Estimate the quantity for variable thickness bid items using the estimated variable thickness area multiplied by a thickness of 2-in.

Estimate the quantity for the Hand chipping bid items as 10% of the estimated variable thickness area.

Estimate the quantity for full depth repair bid item as the estimated area of full depth repair multiplied by the remaining deck thickness after uniform and variable removal (where applicable).

Do not include a pay item for full depth repair when no full depth repair is anticipated.

403.4.1.1 LATEX MODIFIED (LMC)

The minimum thickness of a Latex Modified concrete overlay is 1.25-in.

For [SS848](#), the default value for “D” is 1.0-in.

For decks with an existing overlay, determine the thickness of the existing overlay from core samples, if available, rather than solely from information in existing plans.

Variable thickness is the depth of removal that exceeds the uniform removal depth. As the name suggest, this will not be a fixed dimension, and determining an accurate average depth is impossible. The value used to estimate the variable depth pay quantity should be reasonable to avoid significant changes in the character of the work. Therefore, to keep the estimate simple, reasonable, and consistent, an average depth of variable thickness of 2-in is recommended.

Overlays using hydrodemolition, may have excessive removal and some “blow-through” may be expected.

Consideration shall be given to adding a plan note and quantity requiring falsework to be placed over traffic prior to performing the hydrodemolition.

C403.4.1.1

LMC materials are batched on-site. The internal structure of the LMC overlay creates a surface with similar permeability as MSC. LMC has a lower elastic modulus than MSC and SDC making it more resistant to cracking than the other two materials. LMC is the most expensive of the three standard rigid overlay materials with a 25% premium over the average combined cost of MSC and SDC materials.

LMC is a good alternative for remote locations due to on site mixing requirements. LMC is not pumpable, and the mobile mixing trucks need to have access to the screed machine. In some instances, the mobile mixers will drive over areas of exposed concrete reinforcement.

The maximum uniform thickness should not exceed 2.5-in. The maximum spot thickness should not exceed 4-in.

Latex-Modified overlays should not be used of bridges with a grade greater than 6%.

LCM may be best suited for second-generation overlays, due to the difficulty of removal from the original concrete deck.

403.4.1.2 MICRO-SILICA MODIFIED (MSC)

The minimum thickness of a Micro-Silica Modified concrete overlay is 1.5-in.

C403.4.1.2

MSC materials are batched at a ready-mix facility. The internal structure of the MSC overlay creates a surface with a lower permeability than SDC.

MSC overlays are the most susceptible to plastic shrinkage cracking under improper placement and curing conditions.

MSC overlay are a good choice for uniform removals with minimal variable depth removal. The maximum uniform thickness should not exceed 3-in and the maximum spot thickness should not exceed 4-in. Thicker MSC overlays will be susceptible to shrinkage cracking.

Micro-silica concretes are very cohesive and behave somewhat differently than conventional concrete in that they are internally very sticky and need a water reducer. Slumps need to be at maximum allowable to achieve the same workability.

403.4.1.3 SUPERPLASTICIZED DENSE (SDC)

The minimum thickness of a Superplasticized Dense concrete overlay is 1.75-in.

C403.4.1.3

SDC materials are batched at a ready-mix facility and are the least complicated for contractors to place and cure.

SDC overlays are best suited for decks with high quantities of deck patching and full depth repairs. There is no maximum limit to the uniform or spot thickness. For a uniform overlay thickness exceeding 5-in, place a 12 x 12 – D12 x D12 epoxy coated or galvanized welded wire reinforcement (WWR) with a 2.5-in minimum cover to the top surface and 1.5-in minimum cover to the prepared surface. The WWR will provide shrinkage and temperature control reinforcement to help control cracking in the overlay.

403.4.2 THIN EPOXY

The minimum thickness of an Epoxy overlay is 0.25-in. Depressions in the deck surface less than 1.5-in can be filled with a mortar made from Polymer Epoxy Overlay material. If depressions in the deck are greater than 1.5-in, add a quantity for deck patching. The Department will pay for repairing these depressions under ITEM 519 – PATCHING CONCRETE BRIDGE DECKS.

Seal cracks with a width greater than 13 mills per [C&MS 512.07](#), Sealing Cracks by Epoxy Injection. The Department will pay for these crack repairs under ITEM 512 – CONCRETE REPAIR BY EPOXY INJECTION.

C403.4.2

Thin Polymer Epoxy Overlays are used as a deck preservation methodology. They serve to seal the deck from chloride intrusion, seal cracks, and to increase surface friction. Thin bonded overlays can aid in minor grade corrections.

This treatment is appropriate for candidates with a deck General Condition Rating of 6 or greater showing signs of cracking but requiring minimal repair.

Minor grade correction required to fill low spots and improve drainage will require additional polymer epoxy overlay courses not covered by the specification. The limits of and number of additional courses shall be specified in the contract plans. Use pay item as per plan, when specifying addition courses

SS858 Thin Polymer (Epoxy) Overlays for Structural Slabs overlays includes the following repair areas:

- A. Concrete bridge decks
- B. Top of backwalls
- C. Approach slabs
- D. Barriers/curbs 3-in above the deck

For bridge decks with an existing thin epoxy overlay to be removed, include pay ITEM 202 – WEARING CORSE REMOVED, AS PER PLAN (units square yard).

403.4.3 WATERPROOFING ASPHALT

Perform work in accordance with [SS856](#).

Do not specify waterproofing fabric or similar under the Waterproofing Asphalt.

The minimum thickness of the Waterproofing Asphalt course is 1.5-in.

If additional areas are to be repaired with thin epoxy, add a note to the plans and change the pay item to include “AS PER PLAN”

A typical General Note is available in BDM Section 603.2- ITEM 202 - WEARING COURSE REMOVED, AS PER PLAN: REMOVE THE EXISTING EPOXY WEARING SURFACE BY MICRO-MILLING.

C403.4.3

This work shall be performed on projects with a completion date prior to November 1st. Waterproofing Asphalt is a consideration for overlay replacements for existing asphalt overlays on bridge decks and non-composite box beam superstructures. When specified at the minimum thickness, Waterproofing Asphalt, not including costs of removing an existing overlay, is about one-third of the average combined cost of a typical waterproofing membrane and asphalt overlay.

Existing asphalt overlay does not require complete removal. Depth of asphalt removed should be based upon existing asphalt overlay condition and whether the structure is capable of additional load from the asphalt.

Table 403-4

ODOT Bridge Deck Overlay Guide									
Type	Specification	Lifespan (years)	Cost (\$/SF)	Typical condition state ranges				thickness (spot max)	Commentary
				CS 2	CS 3	CS 4	Deck Summary		
Sealing (HMWM)	705.15	5 to 7	15 - 17	≤ 5%	≤ 1%	≤ 0%	≤ 7	N/A	<ul style="list-style-type: none"> - Should not be placed on concrete or cementitious patch material less than 28 days old. - Seals cracks - quick operation (1-day) - Very inexpensive aggregate (mason sand) - Not reliant on preparation. - Does not provide wearing surface - Can be aesthetically displeasing (green tint) - Use on New Decks that have extensive cracks. Recommended sealer is applied 6 months after construction of new concrete deck or overlay.
Epoxy Polymer Overlay	SS 858	10 to 15		≤ 10%	≤ 1%	≤ 0%	≤ 6	0.375" (2")	<ul style="list-style-type: none"> - Seals cracks - Typically 1 to 3- day operation - Extremely sensitive to surface preparation - Susceptible to snow plows - Provides aesthetic wearing surface -Increases skid resistance - Dark aggregate retains more heat reducing icing of bridge - Cannot be placed on concrete or cementitious patch material less than 28 days old. - No grade correction with overlay - *Use on New Decks that have extensive cracks whether or not the cracks have been sealed with HMWM.
Polyester Polymer Concrete (PPC)	Plan Note							1.0 to 1.5 (12")	<ul style="list-style-type: none"> - Very Rapid Curing Overlays - Less than 24 hours, night closure - Require mobile mixers - good for remote locations and fast application - Max depth of 12" however not economical
Latex Modified Concrete (LMC)	SS 953 (latex modification) used with SS 847 or SS848	20 to 30	75-85					1.25 to 2.5 (4")	<ul style="list-style-type: none"> - Best suited for a second-generation overlay. LMC overlay are difficult to remove from the original concrete deck, overlay may take multiple passes of the hydrodemolition equipment. Excessive removal and some "blow-through" may be expected - Add Type III cement - Fast Curing Overlays -Weekend closure - Very susceptible to plastic shrinkage cracking - Requires mobile batcher mixer - Typically has higher flexural strengths than plain concrete - Very low chloride permeability - Can be used with a high early strength for weekend overlays
Micro-silica Modified Concrete (MSC)	SS 847 or SS848	20 to 30	55-76	≤ 25%	≤ 15%	≤ 5%	≤ 5	1.5" to 3.0" (should not exceed 4")	<ul style="list-style-type: none"> - Fast Curing Overlays -Weekend closure - Micro-silica is very cohesive and behaves somewhat differently than conventional concrete (very sticky) and needs water reducer, slump needs to be at max allowable to achieve same workability.
Superplasticized Dense Concrete (SDC)	SS 847 or SS848	20 to 30	60-85	≤ 25%	≤ 15%	≤ 5%	≤ 5	1.75" in and greater	<ul style="list-style-type: none"> - Used on typical overlay projects
Asphalt Concrete Overlay (special)	SS 856	10 to 12 Yrs		≤ 20%	≤ 5%	≤ 1%	≤ 6	3" to 4"	<ul style="list-style-type: none"> - Proprietary item - name brand Rosphat - Installed at typical asphalt rates - Asphalt with Mix Modifier (not polymer)
Asphalt Concrete Overlay with waterproofing membrane		10 to 12 Yrs		≤ 25%	≤ 15%	≤ 5%	≤ 5	3" to 4"	<ul style="list-style-type: none"> - Leave membrane in place thru one milling of asphalt, replace membrane on subsequent milling project. - Can be used as second generation deck treatment or when joints need to be replaced. - Can be use on New Decks that have extensive cracks whether or not the cracks have been sealed with HMWM. - Not ideal for horizontally curved bridges. - Not for use on bridges that exceed 4.5% superelevation. - Cost of deck repairs not included.

Existing concrete overlay removed with scarification add\$35 Sq/Ft (2019)

Existing concrete overlay removed with scarification and Hydrodemolition prep add\$55 Sq/Ft

Note hand chipping is an additional cost add\$60 Sq/Ft.

403.5 DECK REPLACEMENT

Perform removal of existing decks using methods that will not damage supporting superstructure members. Refer to BDM Section 603.2 for sample General Notes that address appropriate deck removal methods.

Use a composite deck design to increase the load carrying capacity of existing beams or girders when replacing the concrete deck of a bridge that was originally designed and constructed as non-composite. See BDM Section 404.1.6 for additional information.

For bridges on curved alignments where shoulders are used to maintain traffic, provide temporary modifications to cross-slopes oriented in the wrong direction. Address this issue in the Narrative of Bridge Alternatives and Preliminary Maintenance of Traffic Plan submitted during the Structure Type Study.

During plan development, record elevations at key points along the bottom of all beams/girders (e.g. above beam seats; at quarter points, mid-span points and splice points; and at intermediate points not to exceed 25-ft). Verify this elevation information against the proposed profile grade and deck surface elevations to ensure the minimum deck thickness and determine haunch heights.

Specify that elevations at these same points be recorded during construction before and after the deck removal. The difference in these elevations becomes the superstructure rebound. Multiply this rebound by the per foot weight of the proposed deck and divide by the per foot weight of the existing deck to determine the deflection due to deck placement. Add this deflection to the Final Deck Surface Elevations to get Screed Elevations. Add this deflection to the Final Deck Surface Elevations and subtract the design deck thickness to get Top of Haunch elevations. Provide a table in the plans similar to that shown in BDM Section C403.5 with known information supplied that allows the contractor to complete during construction.

C403.5

For some bridges, the shoulder on the high side of the superelevated section will be sloped downward towards the railing to prevent surface water from crossing the travel lanes. Maintaining temporary lanes on a shoulder sloping towards the railing on the outside of a curve may present a safety concern that should be avoided.

Table 403-5

Location			℄R.A.	1/4 Span 1
Beam 1	A	Final Deck Elev.		
	B	*Bottom of Flange Elev. (Before Removal)		
	C	*Bottom of Flange Elev. (After Removal)		
	D	*Rebound		
	E	Wt. of Exist. Deck		
	F	Wt. of Prop. Deck		
	G	*Deck Deflection		
	H	Deck Thickness		
	J	*Screed Elev.		
	K	*Top of Haunch Elev.		

* - To be completed by Contractor as follows:

$$D = C - B$$

$$G = D \times (F/E)$$

$$J = A + G$$

$$K = A + G - H$$

403.5.1 ELIMINATION OF LONGITUDINAL DECK JOINT

If the total bridge width will be 90-ft or less, eliminate existing longitudinal deck joints. When the longitudinal joint is eliminated, replace structural steel bearings (e.g. rockers and bolsters) with elastomeric bearings.

When the decks of twin structures without elastomeric bearings are being tied together resulting in a total structure width in excess of 60-ft, replace the existing bearings with elastomeric bearings.

C403.5.1

This measurement for bridge width assumes the joint has been removed.

When the joint is removed, transverse movement due to expansion and contraction will be increased. This additional movement cannot be accommodated by rockers and bolsters.

403.5.2 DECK HAUNCH

The minimum haunch above an existing beam/girder is 0.0-in.

When the haunch is used to provide an increase in the profile elevations, reinforce the haunch in areas where the top of the stud does not penetrate at least 2.0-in into the design deck thickness. The maximum stud length shall not exceed 8.5-in. Refer to BDM Figure 308-8 for I-beam haunch reinforcement and BDM Figure 403-1 for steel beam/girder haunch reinforcement.

C403.5.2

Refer to BDM Section 403.5 for information ensuring minimum deck thickness.

The purpose for the haunch reinforcement is to avoid a horizontal shear plane between the top flange and the bottom of the deck and to reduce the size of concrete spall if concrete deteriorates with age.

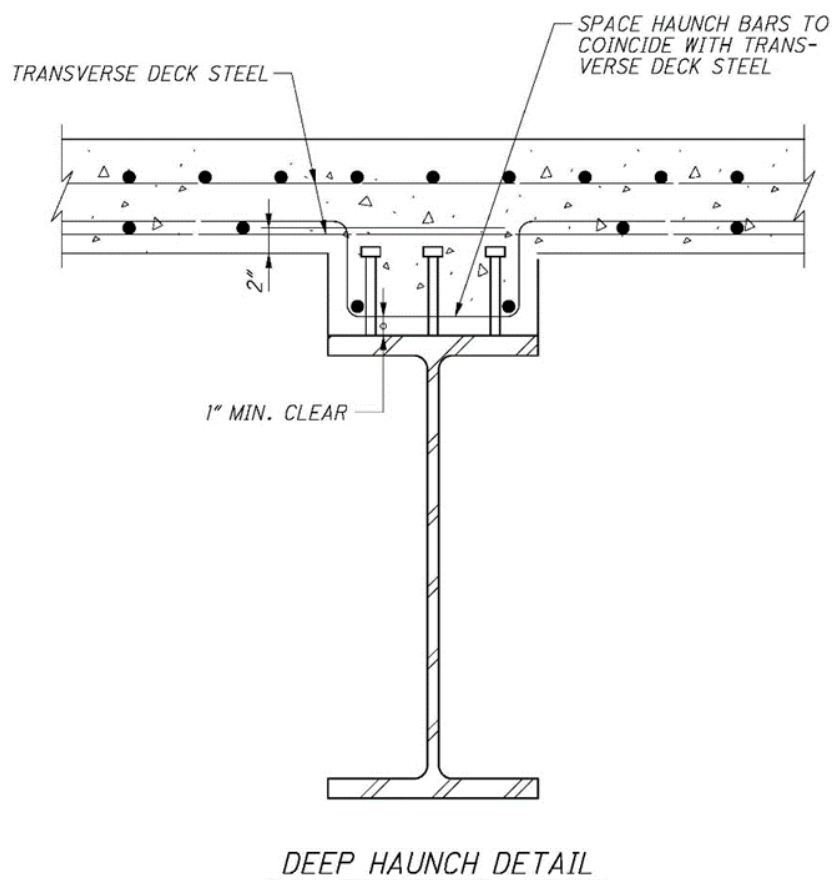


Figure 403-1

403.5.3 CLOSURE POUR

Do not provide a closure pour between adjacent phases of deck placement on an existing superstructure where cross-frames or diaphragms remain fully connected between all phases.

For **phased** deck replacements that include integral or semi-integral diaphragms, **see BDM Section 702.4.2 for concrete end diaphragm placement requirements.**

Seal the phase joint with HMWM in accordance with [C&MS 511](#).

Refer to BDM Section 403.6.1 for closure pour requirements in widening projects.

403.5.4 CONCRETE PLACEMENT SEQUENCE

Provide a concrete placement sequence for superstructures subjected to uplift.

For bridges with intermediate hinges and expansion joints, provide a concrete placement sequence in the plans.

C403.5.3

Fully connected cross-frames in the bay separating phases will resist differential deflections between the phases.

If the diaphragm for the deck phase being placed sets against the preceding phase diaphragm, it will resist the deflection of the superstructure members supporting that phase of deck placement. This can lead to excessive stresses and insufficient cover/deck thickness.

C403.5.4

Without a plan sequence, the Contractor is free to place the deck that best suits their means and methods.

Uplift can become an issue where the end spans of a continuous structure are less than 60% of the adjacent interior span.

Long multiple span steel beam and girder bridges have, in the past, been subdivided into units by means of intermediate expansion joints, located at points of contraflexure, to keep expansion and contraction within the capacities of bearing devices and expansion joints. The hinged structure is more sensitive to placement of deck slab concrete than a fully continuous structure. This sensitivity requires that the sequence of deck concrete placement be carefully planned because (1) the configuration of most intermediate joints makes them susceptible to damage if the deck placement sequence results in large angle changes between the articulated elements of the joints, and (2) the development of composite action in previously placed spans may cause deflections to vary from design deflections and result in a rough profile.

Generally, concrete should be placed on the long cantilever before concrete is placed on the short cantilever, particularly before placement in the span contiguous with the short cantilever.

The most unsatisfactory sequence is to first place concrete in the span contiguous with the short cantilever, especially if concrete is first placed in half of the span immediately adjoining the short cantilever. This sequence produces the maximum angle change between the joint elements.

Where controlled deck placement sequence alone will not provide adequate protection against damage to the joint, provision should be made for attaching part or all of the joint to the main structural elements after the major portion of the concrete is placed. Another alternative is to have a separate deck pour of approximately 36-in, or of a width necessary to accommodate the joint and its proper placement/installation and alignment, to allow for installation of the joint after the rest of the deck has been placed.

403.6 WIDENING

Design the new beams/girders for widened bridges to have the same dead load deflections (± 0.25 -in) as the adjacent existing beam/girder.

403.6.1 CLOSURE POUR

Provide closure pours for widened superstructures constructed in phases as follows:

- A. For widened bridges where the existing deck is removed, and a new or wider deck is to be placed with no superstructure members added, no closure pour is necessary. Refer to Detail A in BDM Figure 403-2.
- B. For widenings where either the existing deck is to remain, or the phase line of a new deck will be between the existing and new superstructure, provide a closure pour in the bay between the new and existing beams/girders. Refer to Detail B in BDM Figure 403-2. Do not fully connect cross-frames (designated B1 in BDM Figure 403-2) in the bay between the new and existing superstructure until after the phase 1 and 2 deck portions have been placed. Complete the phase 3 placement after the cross-frames have been fully connected. Splice rebar within the closure section.
- C. For widenings where the deck's phase line is not between the new and existing superstructure members, provide a closure pour in the bay between the phases. Refer to Detail C in BDM Figure 403-2. Disconnect the existing cross-frames (designated C1 in BDM Figure 403-2) under the closure pour location before the phase 1 deck removal begins.

C403.6

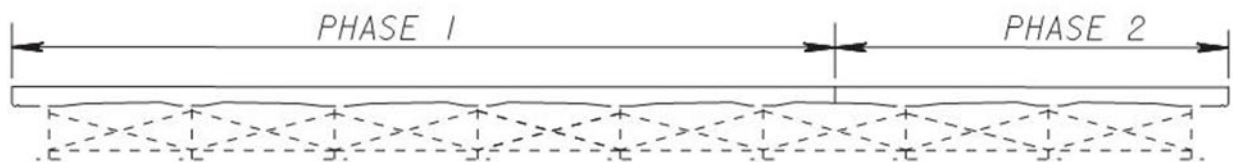
Deflections that differ between adjacent beams/girders will affect deck thickness and concrete cover. These differential deflections will also provide additional loading to bracing members and connections.

C403.6.1

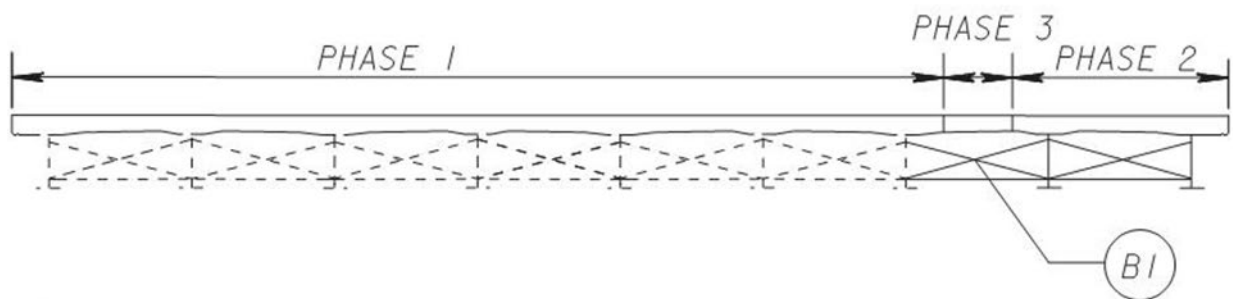
The purpose of the closure pour is to accommodate the differences in deflection that can occur between two phases of construction.

Install the cross-frames between the new and existing beams/girders before the phase 2 pour. Re-install the disconnected cross-frames after the phase 2 pour but before the phase 3 pour. Splice rebar within the closure section.

A



B



C

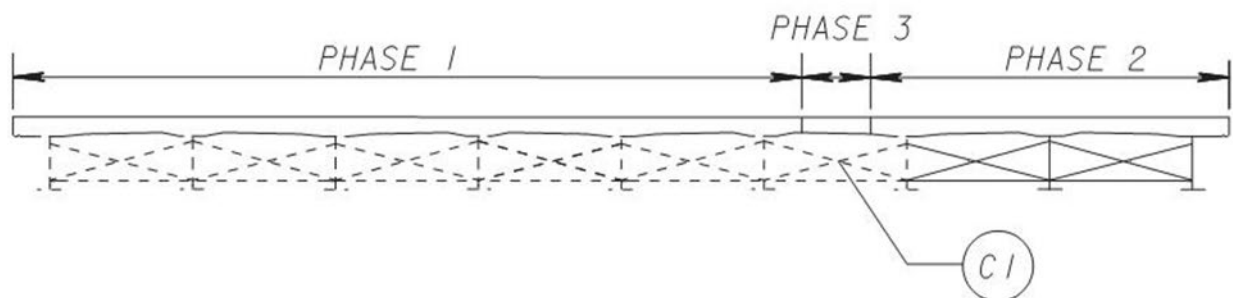


Figure 403-2

The minimum width of the closure section shall be 30-in.

The 30-in dimension accommodates a lap length and provides some transition if the two phases do not align vertically.

Closure pours may be eliminated if the differential deflection is expected to be less than 1/4-in, regardless of superstructure type.

Seal all construction joints with HMWM according to [C&MS 511.19](#).

Forms and falseworks for decks and slabs constructed in phases shall be independent from the adjacent phase of construction.

The forms shall not span across the closure pour until all cross-frames are fully connected following the sequencing specified above.

For superstructure deflections refer to BDM Section 404.2.

403.7 EXPANSION JOINTS

C403.7

Inspect joints for water tightness during the development of rigid overlay plans and repair leaking joints.

Keeping a leaking joint system in place may lead to unnecessary emergency repairs in the future.

Remove and replace joint systems during deck replacements according to the applicable Standard Bridge Drawing.

Trying to keep existing expansion joint systems when replacing the deck may lead to unnecessary cost for removal operations, or more likely, irreparable damage to the joint system.

For widened superstructures, extend the joint armor and furnish new joint seal in one continuous piece or remove and replace the entire existing joints systems according to the applicable Standard Bridge Drawing.

If the existing joint system is leaking or if it is a sliding plate type joint system, replacement of the entire joint is more appropriate than trying to provide an extended sliding plate joint system.

On projects involving stage construction, provide the location of armor joints in the plans.

Many designers consider the detailing of joints to be of secondary importance and merely a nuisance. However improper detailing of joints has frequently caused project delays and caused numerous problems. The joints are important to the longevity of the structure or they would not be included in the work. Designers shall take care to ensure that they are designed and detailed in a professional manner.

403.7.1 STRIP SEAL REPLACEMENT

C403.7.1

For existing strip seal joints which show sign of leakage, identify the joint supplier and specify seal removal and replacement with seal of the same geometry.

It may not always be possible to replace an existing joint seal. In rare cases, the existing seals will still have the manufacturer name visible. More often, it is more practical to contact joint seal suppliers for technical assistance. Sometimes, the suppliers may be able to determine the seal model by a photograph of the seal cross section taken at the joint upturns. If the seal is no longer available or if the joint retainers are damaged, removal of the retainers and installation of adhesive joint seals should be considered.

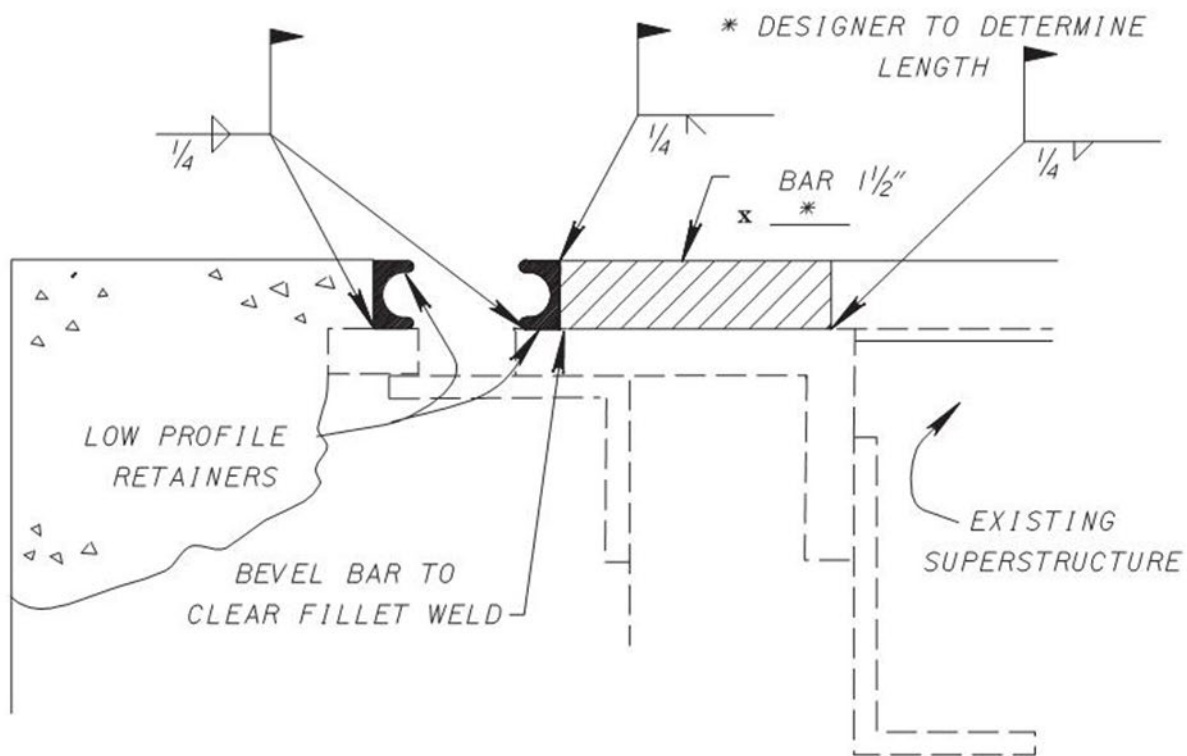
403.7.2 VERTICAL JOINT EXTENSION

For existing sliding plate joints, attach steel retainers on roadway surface of sliding plates according to BDM Figure 403-3.

C403.7.2

Existing sliding plate joints typically end at the curb and toe of concrete bridge railings. The joint armor and steel retainers will need to be extended and bent up the face of the curb or concrete railing. Develop plan details that show existing concrete removals, attachment methods for new steel armor to existing steel, new concrete anchorages, retainer attachments, finished concrete dimensions, concrete reinforcement bar marks and schedule, material requirements and coating requirements. Provide section views from the centerline of the joint looking toward the deck and the backwall, a plan view and cross-section views. Make sure all work is clearly defined and associated with an Item for payment.

This joint repair may not provide sufficient clearance for the larger joint seal installations. Contact seal suppliers to ensure the proposed details will allow installation.



VERTICAL EXTENSION OF STRUCTURAL
EXPANSION JOINTS
INCLUDING ELASTOMERIC STRIP SEALS

Figure 403-3

403.7.3 ADHESIVE JOINT SEALS

For existing sliding plate joints, remove the horizontal leg of the bridging angle and install an adhesive joint seal according to the manufacturer's recommendations and the Scope of Services.

For existing joint systems with leaking seals and damaged retainers, remove the retainers and install an adhesive joint seal according to the manufacture's recommendations and the Scope of Services.

403.8 RAISING & JACKING

Raise the bridge uniformly in order to minimize induced stresses into the superstructure. The maximum differential elevation change between adjacent superstructure members is ¼-in as measured at opposite ends of the same cross-frame. The maximum differential elevation change between adjacent bearings of the same superstructure member is 1-in. Perform work in accordance with [C&MS](#) 501.05.B.5.

Use structural steel to raise the bearing seat elevation 4-in or less.

403.9 BRIDGE DRAINAGE

Retrofit all non-functional drainage systems. Provide replacement drainage according to BDM Section 309.7.

Perform pavement drainage calculations according to the [L&D, Vol. 2](#). If the calculations indicate that the existing scuppers are not necessary, and the deck is not to be replaced, permanently plug the existing scuppers and collect the additional drainage at the end of the bridge. If the deck is to be replaced, remove the scuppers and grind all scupper weld locations smooth.

Extend the lower portion of existing scuppers to remain so that they terminate at least 8-in below the bottom of the bottom flange. Remove and repair or completely replace existing scuppers with extensive section loss due to corrosion.

For overlay projects, extend existing scuppers to remain upward by the thickness of the overlay.

403.10 RAILING

Use this section to determine whether an existing railing system may remain or shall be upgraded to ODOT's current railing design requirements defined in BDM Section 309.4.

C403.7.3

There are many different adhesive joint systems in the marketplace including strip seals and compression seals. Unless otherwise stated in the Scope of Services, contact multiple joint system suppliers to determine the adhesive joint system that best fits the project requirements.

C403.8

If the seat elevation will be raised more than 4-in, consider adding a reinforced concrete extension to the existing bearing seat.

C403.9

Damage often occurs to existing bridges as a result of poorly designed drainage. Proper drainage is extremely important to the longevity of the structure.

This avoids a potential for hazard presented by the drop in surface elevation at the scupper locations.

C403.10

When the portion of the deck supporting bridge railing will be replaced with the proposed construction project, the proposed bridge railing shall meet the design requirements specified in BDM Section 309.4. If only one side of the supporting deck will be replaced on the project, the existing railing on the remaining portion of the deck shall meet TL-3 or better acceptance criteria contained in NCHRP Report 350 or the AASHTO Manual for Assessing Safety Hardware (MASH) or shall meet the design requirements specified in BDM Section 309.4 or shall be replaced with a bridge railing meeting the design requirements specified in BDM Section 309.4.

When the deck supporting bridge railing will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1, the existing railing shall meet TL-3 or better acceptance criteria contained in NCHRP Report 350 or MASH or shall meet the design requirements specified in BDM Section 309.4 or shall be replaced with a bridge railing meeting the design requirements specified in BDM Section 309.4.

Refer to the following sections for rehabilitation requirements of structurally deficient railing systems

For construction projects not involving the complete replacement of existing bridge railing, a mechanical connection between the approach guardrail and the bridge railing is required. Refer to the following sections for more information.

403.10.1 EXISTING STANDARD RAILING TYPES

The railing systems identified in this section were all detailed on ODOT Standard Bridge Drawings.

403.10.1.1 CONCRETE BRIDGE RAILING

403.10.1.1.a BR-1-67

This railing system is commonly referred to as the GM barrier.

The GM barrier does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

This requirement will apply to superstructure replacements, complete deck replacements, deck and slab edge replacements and superstructure widenings.

C403.10.1

These drawings are available under the Archived Drawings link on the OSE website.

C403.10.1.1.a

The GM barrier was developed by General Motors in conjunction with the Texas Transportation Institute using full scale crash testing of large passenger cars in the 1960's. By the early 1970's, there was ample evidence that impacts involving compact passenger cars often resulted in roll over. By the mid-to-late 1970's the use of the GM barrier was discontinued across the U.S.

The GM barrier has not been tested nor evaluated to either NCHRP Report 350 or MASH acceptance criteria.

BR-1-67 was the ODOT standard concrete railing for bridges until it was replaced by BR-1 in 1979.

The GM barrier shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4 when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

GM barrier with a condition rating of 3 or 4 shall be rehabilitated as follows:

- A. Where the bridge railing with a condition rating of 3 or 4 is 20% or less than the total length of bridge railing, this portion of the bridge railing shall be removed and replaced in kind.
- B. Where the bridge railing with a condition rating of 3 or 4 exceeds 20% of the total length of bridge railing, the entire bridge railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4.

For projects where the GM barrier will remain and that have Type 5 guardrail as the approach railing system, use the Bridge Terminal Assembly from archived Roadway Standard Construction Drawing, GR-3 with a final revision date of 1-21-85 to connect the roadway railing to the bridge railing.

For projects where the GM barrier will remain and that have Type MGS guardrail as the approach railing system, construct a Type D Concrete Barrier End Section similar to Standard Roadway Construction Drawing RM-4.6 except the 10-ft barrier transition shall vary from 2.67-ft at the guardrail end to 2.33-ft at the bridge railing end. Use a Type 1 or Type 2 MGS Bridge Terminal Assembly as specified in Roadway Standard Construction Drawings MGS-3.1 or MGS-3.2 respectively.

403.10.1.1.b BR-1 & BR-1-13

This railing system is commonly referred to as the New Jersey barrier.

The owner shall also consider replacing the Type 5 guardrail with the project.

C403.10.1.1.b

The New Jersey barrier was initially introduced in the 1950's and adopted by California and other states throughout the 1960's and 1970's. The barrier was not initially developed with crash testing.

ODOT adopted the New Jersey barrier in 1979 with the BR-1 Standard Bridge Drawing. The initial ODOT BR-1 height was 32-in. In 1994, ODOT introduced two new heights: 36-in and 42-in.

The 32-in and 36-in NJ barrier are accepted under NCHRP Report 350 criteria as TL-4.

The 42-in NJ barrier is accepted under NCHRP Report 350 criteria as TL-5.

The NJ barrier has not been crash tested under the MASH criteria.

The New Jersey barrier does not need to be replaced when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1 that does not increase the deck surface elevation higher than the 3-in toe of the barrier.

Portions of New Jersey barrier with a condition rating of 3 or 4 shall be removed and replaced in kind. For projects that have Type 5 guardrail as the approach railing system, use a Type 1 or Type 2 Bridge Terminal Assembly as specified in current Roadway Plan Insert Sheets GR-3.1 or GR-3.2 respectively to connect the roadway railing to the bridge railing.

For projects that have Type MGS guardrail as the approach railing system, use a Type 1 or Type 2 MGS Bridge Terminal Assembly as specified in Roadway Standard Construction Drawings MGS-3.1 or MGS-3.2 respectively.

403.10.1.1.c SBR-1-13 & SBR-1-20

This railing system is commonly referred to as the Single Slope bridge railing.

The Single Slope bridge railing does not need to be replaced when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

Portions of Single Slope bridge railing with a condition rating of 3 or 4 shall be removed and replaced in kind.

For projects that have Type 5 guardrail as the approach railing system, use a Type 1 or Type 2 Bridge Terminal Assembly as specified in current Roadway Plan Insert Sheets GR-3.1 or GR-3.2 respectively to connect the roadway railing to the bridge railing.

C403.10.1.1.c

The Single Slope bridge railing system was developed as a portable concrete barrier shape by the Texas DOT. It has been adopted by many states as a permanent bridge railing shape.

The Texas 42-in Single Slope shape has been successfully tested and accepted under NCHRP Report 350 criteria as TL-4. The bridge railing has not been crash tested under MASH criteria. However, the Texas DOT has further evaluated the railing system to have equal strength and like geometry to the Manitoba Tall Wall bridge barrier which satisfies MASH TL-5 acceptance criteria.

For projects that have Type MGS guardrail as the approach railing system, use a Type 1 or Type 2 MGS Bridge Terminal Assembly as specified in Roadway Standard Construction Drawings MGS-3.1 or MGS-3.2 respectively.

403.10.1.2 CONCRETE SAFETY CURB AND SIDEWALK RAILING SYSTEMS

403.10.1.2.a AR-1-57

This was ODOT's first railing standard that introduced the design detail commonly referred to as the Safety Curb.

This Safety Curb railing system does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

This Safety Curb railing system shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4 when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

Safety Curb railing system with a condition rating of 3 or 4 shall be rehabilitated as follows:

- A. Where the bridge railing with a condition rating of 3 or 4 is 20% or less than the total length of bridge railing, this portion of the bridge railing shall be removed and replaced in kind.
- B. Where the bridge railing with a condition rating of 3 or 4 exceeds 20% of the total length of bridge railing, the entire bridge railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4.

To connect the roadway approach guardrail to the bridge railing, use a Type F Bridge Terminal Assembly as detailed on archived Roadway Standard Construction Drawing GR-3A with a final revision date of 2-5-82.

403.10.1.2.b BR-1-65

This ODOT Standard Bridge Drawing replaced AR-1-57 to become the standard railing system commonly referred to as the Safety Curb.

This Safety Curb railing system does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

C403.10.1.2.a

The AR-1-57 Safety Curb railing system has not been crash tested.

C403.10.1.2.b

The BR-1-65 Safety Curb railing system has not been crash tested.

This Safety Curb railing system shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4 when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

Safety Curb railing systems with a condition rating of 3 or 4 shall be rehabilitated as follows:

- A. Where the bridge railing with a condition rating of 3 or 4 is 20% or less than the total length of bridge railing, this portion of the bridge railing shall be removed and replaced in kind.
- B. Where the bridge railing with a condition rating of 3 or 4 exceeds 20% of the total length of bridge railing, the entire bridge railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4.

To connect the roadway approach guardrail to the bridge railing, use a Type F Bridge Terminal Assembly as detailed on archived Roadway Standard Construction Drawing GR-3A with a final revision date of 2-5-82.

403.10.1.2.c BR-2-67

This ODOT Standard Bridge Drawing provided details for a sidewalk bridge railing system.

This sidewalk bridge railing does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

This sidewalk bridge railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4 when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

Sidewalk bridge railing with a condition rating of 3 or 4 shall be rehabilitated as follows:

- A. Where the bridge railing with a condition rating of 3 or 4 is 20% or less than the total length of bridge railing, this portion of the bridge railing shall be removed and replaced in kind.
- B. Where the bridge railing with a condition rating of 3 or 4 exceeds 20% of the total length of bridge railing, the entire bridge railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4.

Use the fabricated connection bracket details provided on the drawing to connect the roadway approach guardrail to the bridge railing.

C403.10.1.2.c

The BR-2-67 sidewalk bridge railing has not been crash tested.

403.10.1.2.d BR-2-82

This ODOT Standard Bridge Drawing replaced BR-2-67 to become the standard sidewalk railing system.

This sidewalk railing system does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

This sidewalk railing system shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4 when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

Sidewalk bridge railing with a condition rating of 3 or 4 shall be rehabilitated as follows:

- A. Where the bridge railing with a condition rating of 3 or 4 is 20% or less than the total length of bridge railing, this portion of the bridge railing shall be removed and replaced in kind.
- B. Where the bridge railing with a condition rating of 3 or 4 exceeds 20% of the total length of bridge railing, the entire bridge railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4.

Use the W-Beam Terminal Connector detailed in Roadway Standard Construction Drawing MGS-1.1 to connect the roadway approach guardrail to the bridge railing as shown in BR-2-82.

403.10.1.2.e BR-2-98 & BR-2-15

The BR-2-98 ODOT Standard Bridge Drawing replaced BR-2-82 to become the standard sidewalk railing system. In 2015, [BR-2-15](#) replaced BR-2-98.

This sidewalk railing system does not need to be replaced when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

Portions of either BR-2-98 or [BR-2-15](#) sidewalk bridge railing with a condition rating of 3 or 4 shall be removed and replaced in kind.

Use the Thrie-Beam Terminal Connector detailed in Roadway Standard Construction Drawing MGS-1.1 to connect the roadway approach guardrail to the bridge railing as shown in BR-2-98 and [BR-2-15](#).

C403.10.1.2.d

The BR-2-82 sidewalk railing system has not been crash tested.

C403.10.1.2.e

The BR-2-98 sidewalk railing system is a slightly modified version of the BR27C which was successfully tested with and without a raised sidewalk in accordance with the 1989 AASHTO Guide Specifications for Bridge Railing to a Performance Level two (PL-2). FHWA accepted this testing under the NCHRP Report 350 criteria as TL-4.

403.10.1.3 METAL RAILING SYSTEMS**403.10.1.3.a CSB-1-46, CSB-1-47, CSB-2-46, CSB-2-47, CSB-3-46, CSB-3-47, SSB-1-46, SSB-1-47, SSB-2-46 & SSB-2-47****C403.10.1.3.a**

This railing system was detailed on the 1946 and 1947 simple span steel beam bridge and continuous span steel beam bridge standard drawings. The railing consisted of a tubular steel posts with two steel angle hand rails mounted on a concrete Safety Curb.

This railing system has not been crash tested.

This railing system does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

This railing system shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4 when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

This railing system with a condition rating of 3 or 4 shall be rehabilitated as follows:

- A. Where the railing system with a condition rating of 3 or 4 is 20% or less than the total length of railing, this portion of the railing shall be removed and replaced in kind.
- B. Where the railing system with a condition rating of 3 or 4 exceeds 20% of the total length of railing, the entire railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4.

Use the Bridge Terminal Assembly Type H detailed in archived Roadway Standard Construction Drawing GR-3B with a final revision date of 1-21-85 to connect the roadway approach guardrail to the bridge railing.

403.10.1.3.b CSB-1-55, CSB-1-63, CSB-2-56, & CSB-2-63**C403.10.1.3.b**

This railing system was detailed on the 1955, 1956 and 1963 continuous span steel beam bridge standard drawings. The railing consisted of a wide flange post with a w-beam rail element and a steel angle hand rail mounted to the side of the deck to allow surface drainage over-the-side of the bridge deck.

This railing system was the precursor to the Deep Beam Bridge Guard Rail system which was detailed on Standard Bridge Drawing DBR-1-71 which is discussed in BDM Section 403.10.1.3.d.

This railing system does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

This railing system has not been crash tested.

This railing system shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4 when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

The anchorage to the deck is identical to the current [DBR-2-73](#) system. Therefore, a replacement railing consisting of the posts and railing elements of [DBR-2-73](#) with the [DBR-3-11](#) retrofit that utilizes the existing anchors will allow the final railing design to comply with NCHRP Report 350 TL-3 acceptance criteria.

This railing system with a condition rating of 3 or 4 shall be rehabilitated as follows:

- A. Where the railing system with a condition rating of 3 or 4 is 20% or less than the total length of railing, this portion of the railing shall be removed and replaced in kind.
- B. Where the railing system with a condition rating of 3 or 4 exceeds 20% of the total length of railing, the entire railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4.

Use a Type 4 Bridge Terminal Assembly as specified in current Roadway Plan Insert Sheets GR-3.4 to connect the Type 5 or Type MGS roadway railing to the bridge railing. The height difference between the bridge and the roadway railing shall be accommodated beyond the length of the Bridge Terminal Assembly. Include a plan note for this information in the contract plans.

The height from the deck surface to the top of the w-beam element for this railing system is ± 25 -in. The comparable height of the w-beam element in the Type 5 and Type MGS guardrail are ± 29 -in and ± 31 -in respectively.

This railing system was detailed on the 1955, 1956 and 1963 continuous span steel beam bridge standard drawings. The railing consisted of a wide flange post with a w-beam rail element and a steel angle hand rail mounted to the side of the deck to allow surface drainage over-the-side of the bridge deck.

This railing system was the precursor to the Deep Beam Bridge Guard Rail system which was detailed on Standard Bridge Drawing DBR-1-71 which is discussed in BDM Section 403.10.1.3.d.

403.10.1.3.c CSB-4-50 & CSB-5-50

C403.10.1.3.c

This railing system was a deep beam railing element supported by posts that were fastened to a cantilever arms attached to the fascia beam web.

This railing system has not been crash tested.

This railing system does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

This railing system shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4 when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

ODOT in partnership with Ohio's Local Public Agencies developed an NCHRP Report 350 TL-3 accepted railing that is very similar to the CSB-4-50 and CSB-5-50 railing system. Designers should consider using this tested railing system as an acceptable replacement. Details of this system can be found on the ODOT Office of Statewide Planning & Research - Ohio's Research Initiative for Locals (ORIL) website. The final report was entitled "Evaluation & Design of a TL-3 Bridge Guardrail System Mounted to Steel Fascia Beams."

This railing system with a condition rating of 3 or 4 shall be rehabilitated as follows:

- A. Where the railing system with a condition rating of 3 or 4 is 20% or less than the total length of railing, this portion of the railing shall be removed and replaced in kind.
- B. Where the railing system with a condition rating of 3 or 4 exceeds 20% of the total length of railing, the entire railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4.

Use a Type 4 Bridge Terminal Assembly as specified in current Roadway Plan Insert Sheets GR-3.4 to connect the Type 5 or Type MGS roadway railing to the bridge railing. The height difference between the bridge and the roadway railing shall be accommodated beyond the length of the Bridge Terminal Assembly. Include a plan note for this information in the contract plans.

The height from the deck surface to the top of the w-beam element for this railing system is ± 25 -in. The comparable height of the w-beam element in the Type 5 and Type MGS guardrail are ± 29 -in and ± 31 -in respectively.

403.10.1.3.d DBR-1-71

The DBR-1-71 railing is an open railing system utilizing w-beam guardrail supported on wide flange posts attached to the side of the deck. The system allows surface drainage over-the-side of the bridge deck

This railing system does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

This railing system shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4 when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

This railing system with a condition rating of 3 or 4 shall be rehabilitated as follows:

C403.10.1.3.d

The DBR-1-71 railing system utilizes a standard w-beam guardrail without tubular backup.

This railing system has not been crash tested.

The anchorage to the deck is identical to the current [DBR-2-73](#) system. Therefore, a replacement railing consisting of the posts and railing elements of DBR-2-73 with the [DBR-3-11](#) retrofit that utilizes the existing anchors will allow the final railing design to comply with NCHRP Report 350 TL-3 acceptance criteria.

- A. Where the railing system with a condition rating of 3 or 4 is 20% or less than the total length of railing, this portion of the railing shall be removed and replaced in kind.
- B. Where the railing system with a condition rating of 3 or 4 exceeds 20% of the total length of railing, the entire railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4.

Use a Type 4 Bridge Terminal Assembly as specified in current Roadway Plan Insert Sheets GR-3.4 to connect the Type 5 or Type MGS roadway railing to the bridge railing. The height difference between the bridge and the roadway railing shall be accommodated beyond the length of the Bridge Terminal Assembly. Include a plan note for this information in the contract plans.

403.10.1.3.e DBR-2-73

The [DBR-2-73](#) railing is an open railing system utilizing w-beam guardrail structurally backed with a tubular steel rail element supported on wide flange posts attached to the side of the deck. The system allows surface drainage over-the-side of the bridge deck.

This railing system does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

This railing system shall be upgraded to NCHRP Report 350 TL-3 acceptance using the [DBR-3-11](#) Deep Beam Bridge Retrofit Railing when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

This railing system with a condition rating of 3 or 4 shall be rehabilitated as follows:

- A. Where the railing system with a condition rating of 3 or 4 is 20% or less than the total length of railing, this portion of the railing shall be removed and replaced in kind.
- B. Where the railing system with a condition rating of 3 or 4 exceeds 20% of the total length of railing, the entire railing shall be upgraded using [DBR-3-11](#).

Use a Type 4 Bridge Terminal Assembly as specified in current Roadway Plan Insert Sheets GR-3.4 to connect the Type 5 or Type MGS roadway railing to the bridge railing. The height difference between the bridge and the roadway railing shall be accommodated beyond the length of the Bridge Terminal Assembly. Include a plan note for this information in the contract plans.

The height from the deck surface to the top of the w-beam element for this railing system is ± 27 -in. The comparable height of the w-beam element in the Type 5 and Type MGS guardrail are ± 29 -in and ± 31 -in respectively.

C403.10.1.3.e

The Deep Beam railing system with tubular backup was crash tested under NCHRP Report 230 criteria. After FHWA adopted NCHRP Report 350 acceptance criteria on the NHS, FHWA accepted the Ohio Deep Beam Bridge Railing, [DBR-2-73](#) as NCHRP Report 350 TL-2 compliant.

The height from the deck surface to the top of the w-beam element for this railing system is ± 27.75 -in. The comparable height of the w-beam element in the Type 5 and Type MGS guardrail are ± 29 -in and ± 31 -in respectively.

403.10.1.3.f LONG SPAN GUARDRAIL ACROSS CULVERT

The Long Span Guardrail system utilized a single or nested w-beam element backed by structural steel tubing sections.

This railing system does not meet the minimum TL-3 acceptance criteria in either NCHRP Report 350 or MASH.

Replace the Long Span Guardrail system with a MASH acceptable system when condition rating falls to 3 or 4. Otherwise, maintain the existing Long Span Guardrail system throughout its useful life.

For projects that have Type 5 guardrail as the approach railing system, use a Type 4 Bridge Terminal Assembly as specified in current Roadway Plan Insert Sheets GR-3.4 to connect the roadway railing to the bridge railing.

403.10.1.3.g TBR-91 & TBR-1-11

The Thrie Beam Retrofit railing system was designed as an economical solution to protect the Safety Curb railing systems. This railing system uses a thrie beam railing element blocked out from the concrete railing to line up with the toe of the Safety Curb.

The Thrie Beam Retrofit railing system does not need to be replaced when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

C403.10.1.3.f

The original Plan Insert Sheet provided six designs:

- A. Single w-beam element with single backing tube for a maximum rail span of 10.25-ft.
- B. Nested w-beam element with single backing tube for a maximum rail span of 11.75-ft.
- C. Single w-beam element with two backing tubes for a maximum rail span of 19-ft.
- D. Nested w-beam element with two backing tubes for a maximum rail span of 20.5-ft.
- E. Single w-beam element with three backing tubes for a maximum rail span of 27.75-ft.
- F. Nested w-beam element with three backing tubes for a maximum rail span of 29.25-ft.

This railing system was considered a zero-deflection barrier during vehicle impacts.

This railing system has not been crash tested.

The Midwest Guardrail System, Long-Span Guardrail as detailed on current Roadway Standard Construction Drawing, MGS-2.3 may be an acceptable replacement system when used according to the Lateral Offset requirements specified.

For locations where the existing guardrail span is greater than 25-ft, consider using a Single Slope Type D barrier across the structure span.

This requirement applies only to locations where the existing Long-Span Guardrail is repaired. If the existing Long-Span Guardrail is replaced, use the appropriate transition to guardrail compatible with the replacement system.

C403.10.1.3.g

The Thrie Beam Retrofit system is based on the Michigan 10 gage retrofit of the Type R4 railing. This system was tested under NCHRP Report 230 and evaluated to the AASHTO Guide Specifications for Bridge Railing PL-2 criteria. FHWA later accepted the system as NCHRP Report 350 TL-4.

Portions of Thrie Beam Retrofit railing system with a condition rating of 3 or 4 shall be removed and replaced in kind.

For projects that have Type 5 guardrail as the approach railing system, use a Type 3 Bridge Terminal Assembly as specified in current Roadway Plan Insert Sheets GR-3.3 to connect the roadway railing to the bridge railing.

For projects that have Type MGS guardrail as the approach railing system, use a Type 1 MGS Bridge Terminal Assembly as specified in Roadway Standard Construction Drawings MGS-3.1.

403.10.1.3.h TST-1-99

The Twin Steel Tube railing system is an open railing system that utilizes two structural steel tubes attached to wide flange posts connected to the side of the deck. The system allows surface drainage over-the-side of the bridge deck.

The [TST-1-99](#) railing system does not need to be replaced when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

Portions of [TST-1-99](#) railing system with a condition rating of 3 or 4 shall be removed and replaced in kind.

For projects that have Type 5 guardrail as the approach railing system, use a Type TST Bridge Terminal Assembly as specified in current Roadway Plan Insert Sheets GR-3.6 to connect the roadway railing to the bridge railing.

For projects that have Type MGS guardrail as the approach railing system, use a Type 1 MGS Bridge Terminal Assembly as specified in Roadway Standard Construction Drawings MGS-3.1.

403.10.2 EXISTING NON-STANDARD RAILING TYPES

If the existing railing on the bridge to be rehabilitated is not one of the standard systems detailed in BDM Section 403.10.1, determine if the railing system is based on the design of a successfully tested railing system.

C403.10.1.3.h

The Twin Steel Tube railing system is a slightly modified version of the Illinois DOT's Type SM which was crash tested under the AASHTO Guide Specifications for Bridge Railing PL-2 criteria and later accepted by FHWA as NCHRP Report 350 TL-4.

The original version of TST-1-99, dated 07-06-99, included a modified Type 3 Bridge Terminal Assembly that connected to the bridge railing at two W6x25 posts spaced at 1.5-ft located off of the end of the bridge. This assembly did not utilize a terminal connector at the end of the thrie beam. For projects that include this original detail, connect the thrie beam as shown in the original TST-1-99 drawing using either Roadway Plan Insert Sheet GR-3.6 or Roadway Standard Construction Drawing MGS-3.1, as appropriate, without the terminal connector.

C403.10.2

A listing of railing systems that have been accepted for use on the NHS is provided on the FHWA Office of Safety – Roadway Departure Safety website.

Railing systems that are accepted to NCHRP Report 350 or MASH TL-3 or better, do not need to be replaced when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1. Otherwise, the railing system shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4 when the deck supporting this railing system will be rehabilitated using a rigid overlay specified in BDM Section 403.4.1.

Portions of railing systems that are accepted to NCHRP Report 350 or MASH TL-3 or better with a condition rating of 3 or 4 shall be removed and replaced in kind regardless of the length of deficiency. Otherwise, portions of the railing systems shall be rehabilitated as follows:

- A. Where the railing system with a condition rating of 3 or 4 is 20% or less than the total length of railing, this portion of the railing shall be removed and replaced in kind.
- B. Where the railing system with a condition rating of 3 or 4 exceeds 20% of the total length of railing, the entire railing shall be removed and replaced with railing meeting the design requirements specified in BDM Section 309.4.

For construction projects not involving the complete replacement of existing bridge railing, a mechanical connection between the approach guardrail and the bridge railing is required.

403.11 BEARINGS

Do not use compressible bearings in combination with sliding plate expansion joints.

All bearings at an individual substructure unit shall be the same type. The bearings at any one substructure unit shall be compatible with all the bearings at the other substructure units.

C403.11

If compressible type bearings are used in this situation, the bearing's reaction will be transferred to the bridging angle on the expansion joint causing distress to the end of the deck and top of backwall.

403.11.1 STEEL ROCKER AND BOLSTER BEARINGS, RB-1-55

Only use this bearing type in rehabilitation projects where a match to the existing bearing is required.

This bearing is limited to a 2-in factored movement in one direction from the vertical.

Bevel the upper load plate to match the profile grade for structures where the grade at the bearing is greater than 2 percent. Provide a plan detail of the beveled, upper load plate. The thickness of the upper load plate at the centerline of the bearing (dimension C in the standard drawing) shall be held.

Pier and abutment seats will allow the bearing base plate to achieve full seat area.

The maximum clip shall not remove more than 3-in² of bearing base plate surface area. Investigate that the substructure can accept the increased loading.

403.11.2 SLIDING BRONZE TYPE & FIXED TYPE STEEL BEARINGS

Only use this bearing type in rehabilitation projects where a match to the existing bearing is required.

403.12 VANDAL FENCE

Replace corroding post sleeves.

Do not install VPF on deteriorated/refaced concrete bridge railing.

C403.11.1

This bearing type is presented on Standard Bridge Drawing [RB-1-55](#). The standard drawing also includes maximum load capacity requirements (Service Load I) for this bearing type.

The assumed nominal rolling and sliding resistance of rockers is $0.25 \cdot DL \cdot \frac{r}{R}$, where: DL is the dead load reaction on the rockers, “r” is the radius of the pin, in inches, and “R” is the radius of the rocker, in inches.

The designer may choose, on structures of extreme skew, the alternative of clipping the corner of the bearing plate to save adding additional width to the pier and abutment seats.

C403.11.2

The sliding bronze type expansion bearing is known to freeze up, therefore, not providing the required freedom of movement. This bearing type is normally not recommended even on rehabilitation projects.

This bearing type is found on older steel beam or girder structures and is shown on Standard Bridge Drawing FSB-1-62. This standard is not currently active but copies are available on the OSE Archived Drawings Website. The fixed type bearing shown on Standard Bridge Drawing [FB-1-82](#), originated from the old Standard Bridge Drawing FSB-1-62.

In the design of these bearings for steel bridges, the assumed coefficient of friction of lubricated bronze sliding bearings is 0.10.

404 SUPERSTRUCTURE**404.1 STRUCTURAL STEEL****404.1.1 DAMAGE OR SECTION LOSS****C404.1.1**

When it is necessary to repair a section of a steel member that has been damaged by rust or other means the specifics of the repair details are left to the ingenuity of the designer due to the vast number of possible solutions. When a beam is damaged by a vehicle impact refer to plan insert sheets CRHS for repair techniques. Repairing beams by heat straightening is discussed in BDM Section 404.1.8.

When performing welded repairs in compression zones, ensure the chemistry of the existing steel is such that it can be welded. Perform field non-destructive testing (NDT) on the welds. Specify weld procedures, the type and location of the NDT, and other relevant information in the plan notes.

Review old mill certifications or take samples of the base material for chemical analysis to determine the existing material composition.

Do not perform welded repairs in tension zones. Repair damaged sections in tension zones by bolting new steel to existing steel.

If it is absolutely necessary to perform welded repairs in a tension zone, consult the Office of Structural Engineering for recommendations.

404.1.2 FATIGUE**404.1.2.1 GENERAL****C404.1.2.1**

Fatigue damage, as it pertains to bridges, is categorized as due to either load-induced or distortion-induced (displacement-induced) stresses. Both types of damage are actually load-induced, the former is directly related to the application of live load and the resulting stresses (in plane), while the latter is the result of secondary stresses (out of plane) typically transmitted by a secondary member as it tends to change the shape of, or distort, the primary member because of displacement. Load-induced damage is dependent on stress range, type of detail, and the number of applications of live load and can be accounted for during design. Distortion-induced damage is not directly quantified in the design of a bridge but can be minimized through proper detailing. Two conditions are necessary for distortion-induced damage to occur: a periodic out-of-plane force or displacement; and an abrupt local change in stiffness where the force/displacement is applied.

Only load-induced fatigue is directly addressed by this section. Since the repairs for distortion-induced fatigue damage are specifically dependent upon the existing detail itself, the repair details are largely up to the designer; are beyond the scope of this section; and may require a special analysis. The designer should consult with the Office of Structural Engineering when developing these types of repairs.

For clarity, the following terms are used as defined throughout BDM Section 404.1.2. A “moment plate” is an existing plate welded to the flanges of a member with the purpose of strengthening the section. A moment plate does not connect members together. A “coverplate” is an existing plate that is welded to the flanges of the section to connect two members together.

Repair or retrofit the detail if cracks are detected.

Retrofit all E or and E’ details on **nonredundant steel tension** members.

404.1.2.2 FATIGUE EVALUATION

C404.1.2.2

Perform a fatigue evaluation of existing steel members/details to be re-used or rehabilitated. The evaluation consists of screening the member/detail and performing a remaining fatigue life analysis when necessary.

In addition to the field survey of the structure as required by BDM Section 401.1, during the Structure Type Study, conduct a detailed arm’s length survey of the bridge’s fatigue-prone details to be evaluated. Also review the bridge inspection records to ascertain if cracking was noted during an inspection.

Bridge inspection records will be available from the District or online at <http://biareports.dot.state.oh.us/>.

Submit a fatigue evaluation report to the Department, as part of the Structure Type Study, with recommendations as to whether the members require fatigue related upgrading.

404.1.2.3 DETAILS TO CONSIDER

C404.1.2.3

Evaluate longitudinal rolled steel beams/welded steel girders having welded moment plates in the positive or negative moment regions, transverse floor beams with welded moment plates, and/or Beam Continuity Welds with and without coverplates.

Do not evaluate base metal of rolled beams, welded plate girders without moment plates, cross-frames, cross-frame connections to beams/girders, transverse web stiffeners, longitudinal web stiffeners, lateral bracing, uncracked web welds, riveted members and riveted connections except if member shows signs of distress or is required by the Scope of Services.

If the element of concern has experienced severe corrosion, mechanical damage, or previously repaired fatigue damage it is recommended the detail be replaced during a rehabilitation project.

404.1.2.4 EVALUATION PROCESS

C404.1.2.4

Use the following retrofitting specifications when evaluating the re-use of the existing steel beams/girders for deck replacement projects.

Complete an economic analysis comparing the cost of utilizing the existing steel members with all associated repairs, retrofitting, strengthening, painting, and widening issues considered, versus superstructure replacement to justify the decision to retrofit.

For the economic analysis, use \$10.00 per pound (in 2019 dollars) for the estimated cost of “Structural Steel Repair”.

For an estimation of the weight of steel required for the retrofit, the plate sizes from the retired Bolted Splice Standard Drawing (Standard Bridge Drawing BS-1-93, sheet 2 of 3) may be used with the resulting weight increased by 25 percent if the web is not to be retrofitted and with no increase if the web is to be retrofit. When using the standard drawing, no deduction for bolt holes is necessary. The cost estimate should be based on initial cost.

404.1.2.4.a MOMENT PLATES

C404.1.2.4.a

- A. Retrofit the ends of top and bottom moment plates, without performing remaining fatigue life calculations, when:

1. The bridge is known to contain fatigue cracks in either the weld of the flange plate or the primary member.

Place a plan note in the contract plans describing how the contractor is to inspect the top flange plate welds for cracks after the deck has been removed. Also, define in the plan note how the retrofit of the top flange plate will be paid in the event that cracks are found in the top flange plate welds.

2. The bridge is located on principal arterial roads (interstate, other freeways or express ways) [functional classifications 01 and 02].

- B. Conduct a remaining fatigue life analysis, BDM Section 404.1.2.6, on bridges with a present day single lane ADTT (Average Daily Truck Traffic) exceeding 500 trucks.

Not all possible cracks can be seen during the preliminary field survey. A separate survey will be required after the existing deck has been removed.

Functional classification maps can be obtained from the Office of Technical Services website.

The 500 truck ADTT limit for a single lane was set based on the belief that the bridges that are below this criterion will have sufficient fatigue capacity for the remaining life of the superstructure.

During the field visit described in BDM Section 404.1.2.2 if it is observed that the moment plates do not terminate in the correct location a remaining fatigue life analysis should be conducted on the member based on the in-situ conditions.

Retrofitting is not required when the remaining fatigue life is at least 20 years.

Retrofit both ends of a plate when the remaining fatigue life at either end is less than 20 years. Conduct an economic analysis as detailed in BDM Section 404.1.2.4.

- C. Bridges not meeting the requirements of (A) or (B) do not require a fatigue analysis. No retrofit is required.
- No retrofitting is required on deck repair projects, unless known fatigue cracks exist.

404.1.2.4.b BEAM CONTINUITY WELDS WITH COVERPLATES

C404.1.2.4.b

When Ohio began using the concept of continuous spans, one commonly used standard detail is a fatigue concern. The detail consisted of simple span rolled beams butt welded together at the piers with short plates welded to the top and bottom flanges, roughly centered on the piers. The weld line of the beam webs had coped holes at the top and bottom flange, serving as access holes for the flange welds. The plates are not structural in that they are not serving the part of moment plates and were meant to reinforce the welded flanges, using a “belt and suspenders” logic. These details are illustrated on retired Standard Bridge Drawing SD-1-63, sheet 1, and were generally used with the rolled beams shown in retired Standard Bridge Drawing CSB-1-63. When beams of differing depths were used, the web of the shorter beam was cut horizontally, and its flange was raised to match the depth of the adjacent beam and the gap in the web was then filled with weld. This method of joining typically resulted in the top splice plate being kinked or bent at the center of the pier.

Henceforth, the butt welds detailed in “Beam Splice Detail A” and “Beam Splice Detail B” on sheet 1 of retired standard drawing SD-1-63 will be referred to as the “Beam Continuity Weld” throughout BDM Section 404.

- A. Retrofit the top coverplate of the Beam Continuity Weld, without performing remaining fatigue life calculations, when:

1. The bridge is known to contain fatigue cracks in either the weld of the flange plate or the primary member.

Place a plan note in the contract plans describing how the contractor is to inspect the top flange plate welds for cracks after the deck has been removed. Also define in the plan note how the retrofit of the top flange plate will be paid in the event that cracks are found in the top flange plate welds.

2. The bridge is located on principal arterial roads (interstate, other freeways or express ways) [functional classifications 01 and 02].

Functional classification maps can be obtained from the Office of Technical Services website.

The ends of bottom flange coverplates of the Beam Continuity Weld do not require automatic retrofit. Perform a remaining fatigue life analysis per BDM Section 404.1.2.6 to determine if a retrofit is warranted.

In most cases the stress range at this location will meet the requirements of MBE 7.2.3 or MBE 7.2.4 and a retrofit will not be required.

- B. Conduct a remaining fatigue life analysis, BDM Section 404.1.2.6, on bridges with a present day single lane ADTT (Average Daily Truck Traffic) exceeding 1000 trucks.

The 1000 truck ADTT limit for a single lane was set based on the belief that the bridges that do not meet this criterion will have sufficient fatigue capacity for the remaining life of the superstructure. When the deck is converted from non-composite to composite deck the stress ranges at the areas of concern is reduced enough to provide a remaining fatigue life in excess of 20 years.

Retrofitting is not required when the remaining fatigue life is at least 20 years.

Retrofit both ends of a plate when the remaining fatigue life at either end is less than 20 years. Conduct an economic analysis as detailed herein.

Retrofit the top coverplate of the Beam Continuity Weld at the centerline of the splice (i.e. the location of the flange to flange weld) when retrofitting the ends of coverplates. Refer to BDM Figure 404-2.

Retrofit web welds only when the web is cracked.

- C. Bridges not meeting the requirements of (A) or (B) do not require a fatigue analysis. No retrofit is required.

No retrofitting is required on deck repair projects, unless known fatigue cracks exist.

404.1.2.4.c BEAM CONTINUITY WELD WITHOUT COVERPLATES

C404.1.2.4.c

Retrofit flange and beam welds when cracks are visually detected or where there is physical evidence of cracking.

Examples of physical evidence would include but are not limited to, rust staining and damaged paint.

404.1.2.5 CLASSIFICATION OF DETAILS

C404.1.2.5

Classify the ends of moment plates and ends of Beam Continuity Weld coverplates as either an E or E' detail.

In order to qualify as a Category E detail, all of the following criteria must be met, otherwise it is to be classified as a Category E' detail:

- A. The thickness of the welded plate must be less than 1.0-in
- AND**
- B. The thickness of the top flange of rolled beam must be less than or equal to 0.8-in
- AND**
- C. The welded plate is narrower than the beam flange (with or without welds across the ends of the plate) or the welded plate is wider than the beam flange with welds across the ends of the plate.

For the purpose of a remaining life analysis, evaluate Beam Continuity Welds, riveted built-up sections, riveted members and riveted connections based upon the requirements of a Category C detail.

For the purpose of a remaining life analysis, the complete penetration flange weld of the Beam Continuity Weld is to be evaluated based upon the requirements of a Category C detail.

Classify all other details according to *LRFD Table 6.6.1.2.3-1*.

404.1.2.6 REMAINING FATIGUE LIFE ANALYSIS

Except as noted in BDM Section 404.1.2.4 or the Scope of Services perform remaining fatigue life analyses only on fatigue-prone details.

Fatigue-prone details are all details that fall within Categories D, E, or E' of *LRFD Table 6.6.1.2.3-1*.

If a detail falls within the categories listed above but meets the requirements of MBE 7.2.3, the detail may be considered as not fatigue-prone and a remaining fatigue life analysis is not required for that detail.

Prior to completing the remaining fatigue life analysis detailed below perform an infinite-life check. If the detail meets the infinite-life check no retrofit is required.

Calculate the remaining fatigue life of the fatigue-prone detail per MBE 7.2.5.

A complete penetration flange weld is typically classified as a Category B detail. However, due to the typical poor weld quality and the lack of testing of the Beam Continuity Welds the Department requires this detail to be analyzed as a Category C detail.

C404.1.2.6

Refer to MBE 7.2.4 for guidance on the infinite-life check.

When determining the expected annual growth rate, unless traffic information has been provided by the Department, refer to the ODOT Traffic Monitoring Management System (TMMS) for applicable traffic information. Use all the available traffic data to determine the expected annual growth rate by calculating the average growth rate. For cases when the growth rate is zero or negative, use a very small number for the growth rate (e.g. 0.0001). If historical traffic data is not available use an assumed annual growth rate of 5%.

For the average number of trucks per day in a single lane in the first year the detail was in service $(ADTT_{SL})_0$, if historical truck data is not available use the $(ADTT_{SL})_{PRESENT}$

Unless otherwise stated in the Scope of Services use Evaluation 2 resistance factors.

When the effective stress range is determined using the factored calculated stress range due to the passage of the fatigue truck, use the stress range for an interior beam. The Department does not believe that the stress range on the exterior beams for each fatigue truck passage is accurately represented using the LRFD distribution factors.

404.1.2.7 REMAINING LIFE FATIGUE ANALYSIS SUBMISSION INFORMATION

C404.1.2.7

As a minimum, provide the following in the remaining life fatigue analysis submission:

- A. Identify and locate all fatigue prone details. Provide recommendations regarding whether or not the details require retrofitting and are suitable for retrofitting.
- B. Provide recommendations regarding necessary strengthening of members to meet loading requirements.
- C. Include a summary table showing the following at each fatigue prone detail and location being evaluated: remaining fatigue life; moments; stress ranges; fatigue detail category; live load distribution factor; ADTT, whether ADTT is for one way or total traffic; directional distribution factor; present age of structure in years; impact percentage.

The owner may request the detailed calculations showing the remaining life computations and the economic analysis. Do not submit voluminous pages of computer output. Provide the information in summary table form only.

404.1.3 FATIGUE RETROFIT DESIGN

404.1.3.1 GENERAL

C404.1.3.1

Provide retrofits for details that have cracked or whose remaining fatigue life has been determined to be less than the desired service life. Only place splice plates across the Beam Continuity Weld when the flange and web welds have cracked.

Refer to BDM Section 404.1.2.4.c for additional information on crack detection.

For locations where known cracks exist or where visible active corrosion is present, the faying surfaces on the existing steel members in contact with the new prime coated retrofit plates shall receive surface preparation and zinc prime coat in accordance with [C&MS](#) 514. For all other retrofit locations, surface preparation and prime coating of the faying surfaces on the existing steel members in contact with the new prime coated retrofit plates is not required.

The Department will address the discovery of cracks or active corrosion not previously known to exist prior to deck removal as differing site conditions according to [C&MS](#) 104.02.B.

404.1.3.2 VERTICAL CLEARANCES

Check the vertical clearance for retrofit details that may decrease the vertical clearance.

C404.1.3.2

If the clearance is reduced and is unacceptable to the District, a modified retrofit will be necessary.

A modified retrofit consists of a combination of a partial bolted plate along with a mechanical treatment of the existing weld (i.e. peening). Contact the Office of Structural Engineering for further guidance in developing the details.

404.1.3.3 ENDS OF MOMENT PLATES (BEAM FLANGES)**C404.1.3.3**

The retrofit for moment plates is done by bolting flange plates at the ends of the existing welded plates in accordance with the procedures outlined in the following sections.

This design procedure applies equally to moment plates and coverplate ends on the bottom and top flanges.

- A. Design the retrofit in accordance with the current edition of the *AASHTO LRFD Bridge Design Specifications*) and the Bridge Design Manual except as noted in this section.
- B. Design the bolted flange plates for the entire moment, both from dead load and live load at the section and any contribution from the web is to be discounted.
- C. Design the connection as a slip-critical connection.
- D. Refer to BDM Section 401.3 for the Design Loading.
- E. Design the retrofit so that the stress range in the flange and in the bolted flange plates, does not exceed the constant-amplitude fatigue threshold for a Category B detail.
Standard Bridge Drawing BS-1-93 may be used as a preliminary guide in the initial sizing of the plate and determination of the number of bolts. Do not directly use the bolted plate designs shown on the retired Standard Bridge Drawing, BS-1-93 for the retrofit design.
- F. For plates with tapered ends, place the number of bolts required for design before the beginning of the taper and beyond the end of the taper. Place additional seal bolts in the range of the plate taper.
(See BDM Figure 404-1)

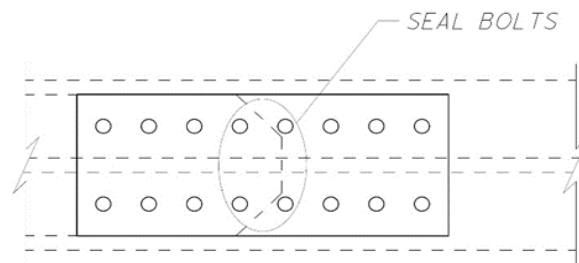


Figure 404-1

- G. Provide and detail filler plates equal to the thickness of the moment plate in the plans, as required.
- H. If the flange is cracked and the crack extends into the beam web, drill a 2-in diameter hole through the web at the tip of the crack to stop the web crack from progressing and specify the exposed steel to be sealed with a non-aerosol paint containing zinc dust. If the web crack is longer than one sixth of the beam depth, splice the web across the cracked section. Detail the drilling of the hole in the plans.

404.1.3.4 ENDS OF COVERPLATES (BEAM FLANGES) C404.1.3.4

The retrofit for Beam Continuity Weld coverplates is done by bolting flange plates at the ends of the existing welded plates and over the flange butt weld in accordance with the procedures outlined in the following sections.

This design procedure applies equally to moment plates and coverplate ends on the bottom and top flanges.

- A. Design the retrofit in accordance with the current edition of the *AASHTO LRFD Bridge Design Specifications* and the Bridge Design Manual except as noted in this section.
- B. Design the bolted flange plates for the entire moment, both from dead load and live load at the section and any contribution from the web is to be discounted, except for when the web is also retrofitted at the Beam Continuity Weld detail.
- C. Design the connection as a slip-critical connection.
- D. Refer to BDM Section 401.3 for the Design Loading.
- E. Design the retrofit so that the stress range in the flange and in the bolted flange plates, does not exceed the constant-amplitude fatigue threshold for a Category B detail.
- F. For plates with tapered ends, place the number of bolts required for design before the beginning of the taper and beyond the end of the taper. Place additional seal bolts in the range of the plate taper. (See BDM Figure 404-1)
- G. Provide and detail filler plates equal to the thickness of the coverplate in the plans, as required.

Standard Bridge Drawing BS-1-93 may be used as a preliminary guide in the initial sizing of the plate and determination of the number of bolts. Do not directly use the bolted plate designs shown on the retired Standard Bridge Drawing, BS-1-93 for the retrofit design.

- H. If the flange is cracked and the crack extends into the beam web, drill a 2-in diameter hole through the web at the tip of the crack to stop the web crack from progressing and specify the exposed steel to be sealed with a non-aerosol paint containing zinc dust. If the web crack is longer than one sixth of the beam depth, splice the web across the cracked section. Detail the drilling of the hole in the plans.
- I. When bolting splice plates at the end of the existing welded plates and over the Beam Continuity Weld, use three individual bolted flange plates. (See BDM Figure 404-2)

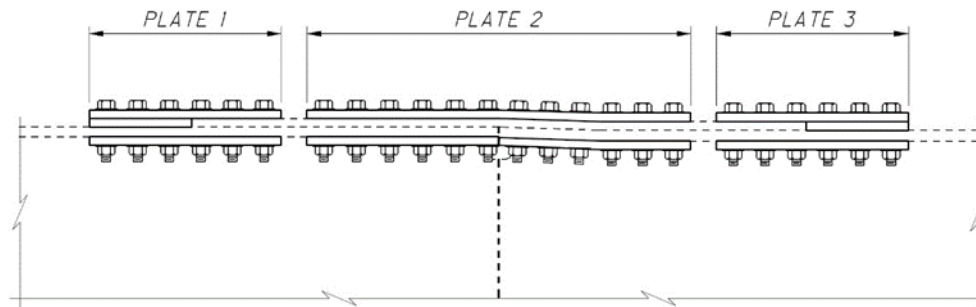


Figure 404-2

404.1.3.5 BEAM WEBS**C404.1.3.5**

Retrofit webs only when web is cracked

- A. The design is to be in accordance with the current edition of the *AASHTO LRFD Bridge Design Specifications* and the Bridge Design Manual except as noted in this section.
- B. When the web retrofit is located under an end of a coverplate retrofit, design the bolted web plates for the applied shear, from both dead load and live load. All the moment is assumed to be taken by the flange. Center the bolted web plates on the ends of the coverplates.

If the web retrofit is required and located at a Beam Continuity Weld, also install the splice plates to the beam flange per BDM Figure 404-2. Design the bolted web plates for the applied shear, from both dead load and live load, the portion of the moment that is carried by the web, and the moment caused by the eccentricity of the bolts.
- C. Design the connection as a slip-critical connection.
- D. Refer to BDM Section 401.3 for the Design Loading.

- E. Design the retrofit so that the stress range in the bolted web plates, does not exceed the constant-amplitude fatigue threshold for a Category B detail. Standard Bridge Drawing BS-1-93 may be used as a preliminary guide in the initial sizing of the plate and determination of the number of bolts. Do not directly use the bolted plate designs shown on the retired Standard Bridge Drawing, BS-1-93 for the retrofit design.
- F. Provide and detail filler plates equal to half the difference in thickness of the beam webs in the plans, as required Refer to BDM Section 404.1.3.6 for thickness requirements.
- G. Arrest the crack by drilling a 2-in diameter hole at the crack tip and specify the exposed steel to be sealed with a non-aerosol paint containing zinc dust.

404.1.3.6 FILLER PLATES**C404.1.3.6**

Consider filler plates 0.25-in or thicker as underdeveloped and include their effects in the design of the fasteners (*LRFD 6.13.6.1.4*). Apply the reduction factor to the fasteners on both sides of the connection to ensure a symmetrical splice. Filler plates need not extend beyond the plates.

Detail fill plates to the nearest 1/16-in in thickness. The minimum fill plate thickness shall be 1/8-in.

When the difference in thickness of the two sections being plated is less than 1/8-in, no filler plate is necessary.

When plating the top flange coverplate for a Beam Continuity Weld detail, determine the thickness of the filler plate by examining the details shown on sheet 1 of retired Standard Bridge Drawing SD-1-63.

Detail filler plates in the plans.

404.1.3.7 FATIGUE CRACKS IN OTHER MEMBERS AND DETAILS**C404.1.3.7**

Since the repair of fatigue cracks in members and details is dependent on the specifics of a situation, it is not practical to provide specific guidelines or repair details, so only a general discussion will be made. To properly devise a repair scheme, it must be determined if the crack resulted from load-induced or distortion-induced stresses. The source of the cracking should also be identified. More than one factor could contribute to cracking. Defects from poor fabrication may need to be considered. Cracking can also occur during shipment and/or erection. The retrofit may need to include modifying the stiffness of an existing connection. The repair should be logically thought out and its influence on the other steel members investigated. A poorly designed repair may worsen the situation as repaired cracks can reinitiate and propagate further into the member, the repair detail itself may experience cracking and new cracking may occur elsewhere if load patterns were modified or member response to loads has been modified.

**404.1.4 BOX GIRDER PIER CAPS
RETROFIT**

Retrofit box girders only when cracking is evident.

C404.1.4

Often box girders were constructed using non-continuous back-up bars that were stitch welded in place. The discontinuity in the back-up bar is of major concern since it acts like a crack in the member and may be a source of crack propagation into the flange or web. The stitch welds may or may not be a problem depending on the stress ranges at their location.

**404.1.5 MISCELLANEOUS FATIGUE
RETROFITS****C404.1.5**

Various retrofits have been used for fatigue prone details such as small web gaps which result in stress concentration and subsequent cracking, intersecting welds, lateral connection plates, longitudinal stiffeners, cracks and many others. The Office of Structural Engineering may be contacted for assistance in determining the best retrofit for specific details.

**404.1.6 STRENGTHENING OF
STRUCTURAL STEEL MEMBERS****C404.1.6**

Install welded stud shear connectors the full length on all steel beam or girder bridges in which the deck is being removed and replaced. Design the stud spacing in accordance with *LRFD 6.10.10*.

Perform field welding and non-destructive testing (NDT) in strict compliance with AWS Specifications, S1011 and the [C&MS](#). Specify the type and location of the NDT in the plans. Do not specify overhead welding in the plans.

Bolted plates in tension zones or compression zones can be used to increase strength.

Consider jacking the stringers to relieve stresses prior to installing cover plates. In this manner the cover plates will carry dead load and live load stresses. If the plates are installed without relieving the stresses, they will carry live load only. This is merely a suggestion as to how extra strength might be obtained if it is needed.

Other methods of increasing the strength are to attach angles or structural shapes to the web or flanges. The possibilities are numerous and is left to the ingenuity of the designer. However, pay strict attention to practicality as well as strength and fatigue requirements. Consult the Office of Structural Engineering for recommendations or to review unusual details before proceeding.

When retrofitting or repairing truss members, provide temporary support where needed.

Many truss members are non-redundant, and their removal could result in the collapse of the structure.

404.1.7 TRIMMING BEAM ENDS

Provide Plan and Elevation views in the plans showing where the beam is to be cut. Also provide pertinent notes and include the work with ITEM 513 - TRIMMING OF BEAM ENDS for payment. Pay attention to the clearance to the end cross-frames and detail their removal and replacement if necessary.

C404.1.7

Trimming of beam ends is sometimes necessary due to tilting of the abutment and closure of the end dam.

In lieu of trimming the beam ends, consider modifying the backwall if backwall removal and replacement is being performed as part of the work. Modifying the backwall would be a viable option if it were necessary to remove and replace the end cross-frames as a result of trimming the beam ends. Another option to consider is converting the existing abutment into a semi-integral abutment as discussed in BDM Sections 404.6 and 405.7.

404.1.8 HEAT STRAIGHTENING

Do not heat straighten **nonredundant steel tension** members.

Provide details showing the location of all repairs and include Item 513 pay items for repair of damaged members.

C404.1.8

Beams or girders that have been struck by trucks or bent by other causes can often be repaired by heat straightening only, or in combination with field welding to install new sections for the damaged steel member portions. The District Bridge Engineer or other ODOT representative with experience in heat straightening should assess the practicality of this type of repair before proceeding. If heat straightening is deemed to be practical, [SS849](#) is available which describes and controls the operation. Refer to the [SS849](#) Designer Notes for more information. Refer to Plan Insert Sheet CRHS for repair and heat straightening details.

404.1.9 HINGE ASSEMBLIES**C404.1.9**

Consideration should be given to removing hinges and making the members continuous.

If the hinge cannot be removed, consideration should be given to the need of providing redundancy in the event of a hinge failure or seismic event.

If a pin and hanger assembly is to be rehabilitated, consult the Office of Structural Engineering for recommendations, including suggested lubrication and NDT requirements.

404.1.10 BOLTS

The Department will permit oversize and slotted holes in rehabilitation work. Design oversize and slotted holes in accordance with *LRFD 6.13.2.4*. Design connections as slip critical.

Since all [C&MS](#) requirements are for standard size holes, specify requirements for the holes and necessary washers in the plans.

The Department will accept the use of blind fasteners (e.g. Huck Bolts) when clearance problems arise. Since the installation specifications in the [C&MS](#) deal strictly with the installation and testing of normal high strength bolts, if blind fasteners are specified, modify the installation specifications by plan note.

404.1.11 EXISTING STEEL COATING SYSTEMS

When estimating the quantity for ITEM 514 - GRINDING FINES, TEARS, SLIVERS ON EXISTING STRUCTURAL STEEL, provide 1 minute for each linear foot of beam/girder to be coated.

404.2 SUPERSTRUCTURE DEFLECTIONS

Design the beams/girders of the widened section so that the live load deflections between the new and existing beams/girders at each end of a lateral bracing location does not exceed $S/100$. Where S is the center to center beam spacing measured in the direction of the lateral bracing.

C404.1.10

Oversize or slotted holes may be desirable in some remedial applications especially where the fit of repair or replacement members or parts becomes tedious.

C404.1.11

If the total existing structure is to be field painted, the coating should be a three-coat paint system consisting of an organic zinc prime coat, an epoxy intermediate coat and a urethane finish coat (formerly referred to as system OZEU) according to [C&MS](#) 514. The OZEU system is better for field application on existing steel since the organic zinc prime coat is more surface-tolerant.

For widened structures, the new steel shall be coated with the inorganic zinc prime coat in the shop and the existing steel shall receive the organic prime coat. The intermediate and finish coats are the same for each system. Refer to BDM Section 308.2.2.1.d for additional coating information.

Field metallizing is another option, but its costs are typically more than twice the cost of OZEU.

C404.2

An effort should be made to use a beam/girder section that closely matches the moment of inertia as the existing beams/girders to keep the relative deflections to a minimum.

404.3 PRESTRESSED CONCRETE I-BEAM

404.3.1 IMPACT DAMAGE

When a prestressed concrete I-beam has been damaged by impact, conduct a detailed survey of the bridge. Complete the survey so that sufficient information is obtained to perform a comprehensive load rating of the bridge's current condition. For these requirements refer to BDM Section 900.

Design all bridge repairs requiring structural strengthening in accordance to [C&MS](#) 501.05.

404.3.1.1 CRACK REPAIR

Repair cracks, 7 to 100 mils in thickness, with epoxy injection per [C&MS](#) 512.07.

404.3.1.2 CONCRETE PATCHING AND BEAM REPAIR

When strengthening is not required, repair damaged beam(s) with the following repair techniques.

Repair spalls with concrete patching when the spall exposes the beam reinforcing. If the reinforcing is not exposed, seal the damaged portion(s) of the beam with an epoxy coating conforming to [C&MS](#) 512.03.

When patching is required, patch the beam according to [SS843](#), [C&MS](#) 519, or [C&MS](#) 520.

Confine the concrete patch with a Fiber Reinforced Polymer (FRP) wrap. Install two layers of unidirectional FRP wrap. Orient the first layer of the FRP wrap parallel to the centerline of the beam. Install this layer on the bottom of the beam flange. Orient the second layer of the FRP wrap perpendicular to the beam centerline. Wrap the second layer of FRP transversely around the bottom flange and outside web of the beam. Install the both layers of FRP a minimum of 12-in beyond the concrete patching in the longitudinal direction. Refer to BDM Figure 404-3 for the limits of the FRP in the longitudinal and transverse direction. Seal all repaired beam area with an epoxy-urethane sealer per [C&MS](#) 512.03.

C404.3.1

When the damage to the bridge/beam causes the bridge load rating to require a Design Exception in accordance with BDM Section 401.4, repair or replacement of the damaged elements is necessary.

Numerous strategies have been developed to structurally repair damaged prestressed concrete beams. However, due to the uniqueness of each damage scenario the repair must be designed to suit each individual case.

C404.3.1.2

The Scope of Services will provide the patching specification based on the type and degree of concrete damage. Patching per [SS843](#) will be used in most cases because it works well in overhead applications without requiring formwork. Patching per [C&MS](#) 520 will likely only be cost effective if a large area of patching is required.

The concrete patch and FRP wrap are not designed to strengthen beam. This application is only intended to provide corrosion protection for the reinforcement. Strengthening the beam with FRP will require a detail design based on the strengthening requirements of the repair.

The FRP repair may be completed with carbon fiber reinforced polymer (CFRP) or E-glass fiber reinforced polymer (EGFRP). However, CFRP is thinner and will be much easier to install in an overhead placement application. The ease of construction will likely offset the increased material cost of the CFRP relative to the EGFRP.

Install FRP wrap in accordance with [PN519](#) – Composite Fiber Wrap System. However, if the repair is being conducted as described in this section the requirement for an Ohio Registered Engineer to prepare, sign, seal and date the cover sheet or submittal letter according to ORC 4733 and OAC 4733-35 is not mandatory.

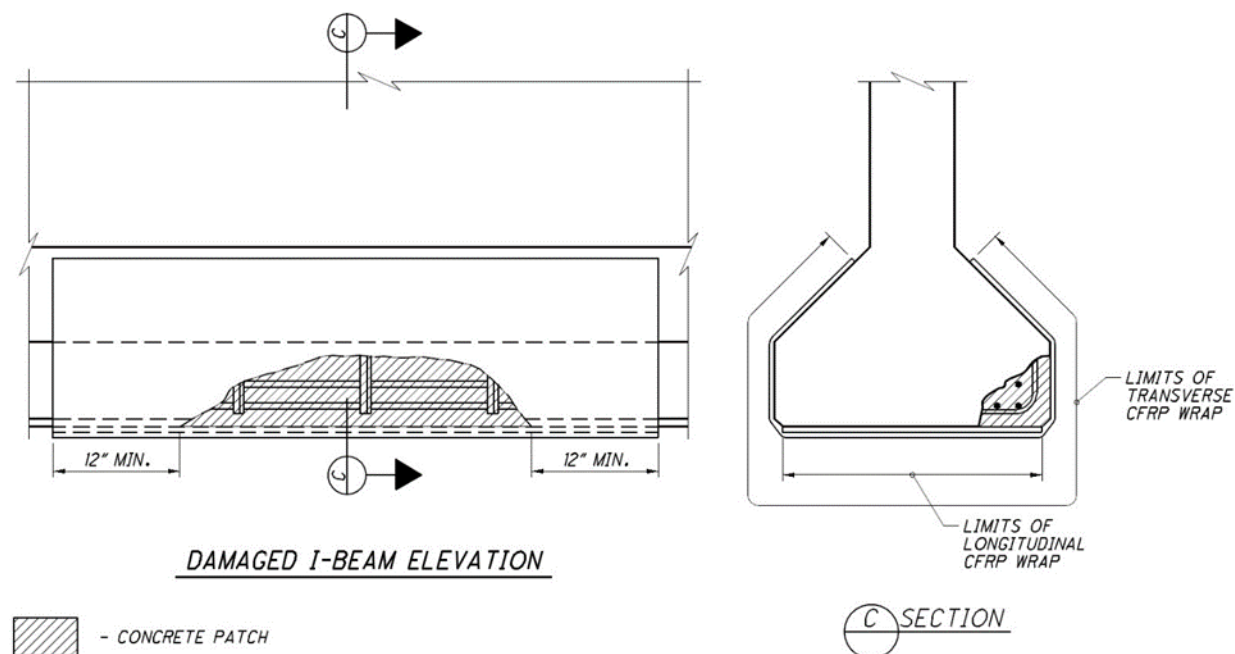


Figure 404-3

404.3.1.3 DAMAGED BEAM REPLACEMENT CRITERIA C404.3.1.3

Independent of the bridge rating, replace the beam when the following damaged is observed during the bridge survey.

- A. Over 25% of the strands have been lost.
- B. The bottom flange is displaced from the horizontal position more than ½-in per 10-ft of girder length.
- C. If the alignment of the girder has been permanently altered by the impact causing a bend point in the beam.
- D. Abrupt lateral offset in the beams cross section indicating that stirrups have yielded.
- E. Concrete damage at harping point resulting in permanent loss of prestressing in the strand.

A strand is considered lost when the effective strand section is reduced by over 25% (e.g. 2 or more of the 7-wire strand bundle is broken).

An example of this type of failure would be when large cracks at the web-flange interface remain open indicating that the vertical reinforcing has yielded.

- F. Severe concrete damage at girder ends resulting in permanent loss of prestressing in the strand.

404.4 PRESTRESSED CONCRETE BOX BEAMS

404.4.1 IMPACT DAMAGE

When a prestressed concrete box beam has been damaged by impact, conduct a detailed survey of the bridge. Complete the survey so that sufficient information is obtained to perform a comprehensive rating of the bridge's current condition. For these requirements refer to BDM Section 900.

Design all bridge repairs requiring structural strengthening in accordance to [C&MS](#) 501.05.

404.4.1.1 CRACK REPAIR

Repair cracks, 7 to 100 mils in thickness, with epoxy injection per [C&MS](#) 512.07.

404.4.1.2 CONCRETE PATCHING AND BEAM REPAIR

When strengthening is not required, repair damaged beam(s) with the following repair techniques.

Repair spalls with concrete patching when the spall exposes the beam reinforcing. If the reinforcing is not exposed, seal the damaged portion(s) of the beam with an epoxy coating conforming to [C&MS](#) 512.03.

When patching is required, patch the beam according to [SS843](#), [C&MS](#) 519, or [C&MS](#) 520.

C404.4.1

When the damage to the bridge/beam causes the bridge load rating to require a Design Exception in accordance with BDM Section 401.4, repair or replacement of the damaged elements is necessary.

Numerous strategies have been developed to structurally repair damaged prestressed concrete beams. However, due to the uniqueness of each damage scenario the repair must be designed to suit each individual case.

C404.4.1.2

The Scope of Services will provide the patching specification based on the type and degree of concrete damage. Patching per [SS843](#) will be used in most cases because it works well in overhead applications without requiring formwork. Patching per [C&MS](#) 520 will likely only be cost effective if a large area of patching is required.

Confine the concrete patch with a Fiber Reinforced Polymer (FRP) wrap. On an exterior beam, install two layers of unidirectional FRP wrap. Orient the first layer of the FRP wrap parallel to the centerline of the beam. Install this layer on the bottom of the beam flange. Orient the second layer of the FRP wrap perpendicular to the beam centerline. Wrap the second layer of FRP transversely around the bottom flange and outside web of the beam. Install the both layers of FRP a minimum of 12-in beyond the concrete patching in the longitudinal direction. Refer to BDM Figure 404-4 for the limits of the FRP in the longitudinal and transverse direction. Seal all repaired beam area with an epoxy-urethane sealer per [C&MS 512.03](#). On an interior beam, only the longitudinal layer is necessary. All other items apply.

Install FRP wrap in accordance with [PN519](#) – Composite Fiber Wrap System. However, if the repair is being conducted as described in this section the requirement for an Ohio Registered Engineer to prepare, sign, seal and date the cover sheet or submittal letter according to ORC 4733 and OAC 4733-35 is not mandatory.

The concrete patch and FRP wrap are not designed to strengthen beam. This application is only intended to provide corrosion protection for the reinforcement. Strengthening the beam with FRP will require a detail design based on the strengthening requirements of the repair.

The FRP repair may be completed with carbon fiber reinforced polymer (CFRP) or E-glass fiber reinforced polymer (EGFRP). However, CFRP is thinner and will be much easier to install in an overhead placement application. The ease of construction will likely offset the increased material cost of the CFRP relative to the EGFRP.

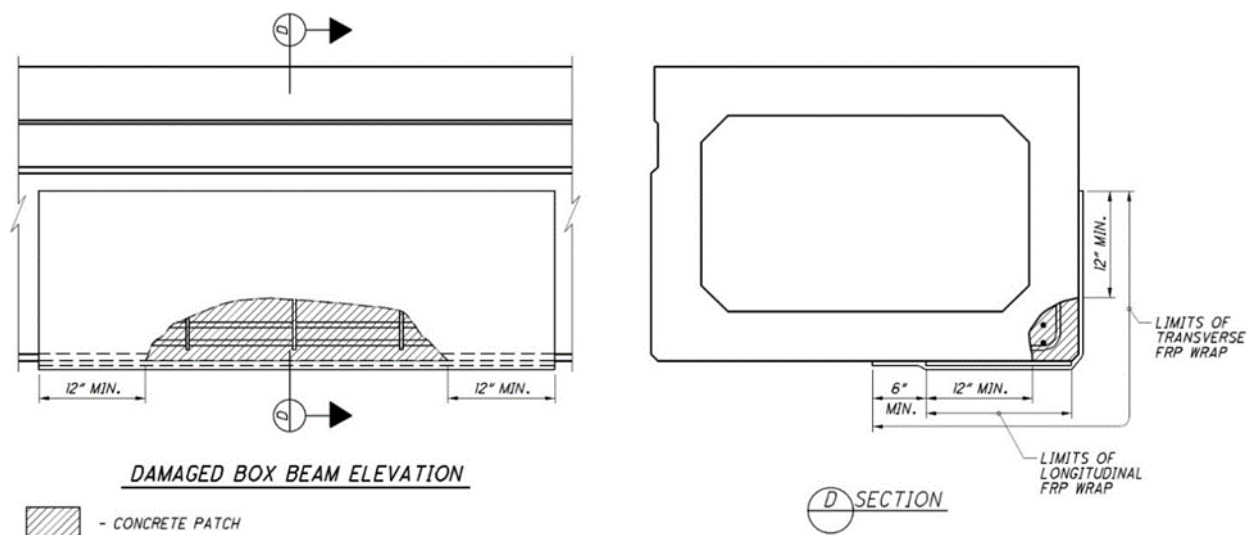


Figure 404-4

404.4.1.3 DAMAGED BEAM REPLACEMENT CRITERIA C404.4.1.3

Independent of the bridge rating, replace the beam when the following damaged is observed during the bridge survey.

- A. Over 25% of the strands have been lost.

Damage severe enough to warrant beam replacement is rare due to the relatively stiff beam section

A strand is considered lost when the effective strand section is reduced by over 25% (e.g. 2 or more of the 7-wire strand bundle is broken).

- B. If the alignment of the girder has been permanently altered by the impact causing a bend point in the beam.
- C. Severe concrete damage at girder ends resulting in permanent loss of prestressing in the strand.

404.4.2 HIGH SKEW STRUCTURE

Do not use box beams with a skew greater than 30-degrees.

C404.4.2

When beam ends that are skewed greater than 30-degrees the asymmetric prestressing force becomes large enough to create a torque in the beams that can lift the beam off its bearings. Then the beam ends will rock when subjected to live load. Ultimately the rocking can cause the bearings to walk from underneath the beams.

When replacing the superstructure of an existing box beam bridge with a skew greater than 30-degrees the beams cannot be replaced in kind.

A possible solution is to use box beams for a 30-degree skew and stagger them along the centerline of bearing. Bearing pads must be installed per [BD-1-11](#) when the beams are being staggered. The anchored bearing secures the bearing from ever moving from below the beam. Refer to BDM Figure 404-5 for additional details. If utilizing this detail, it is important to obtain the accurate beam seat elevations during the survey of the existing structure. Minor adjustments to the beam seat may be required to account for the change in skew and how that if effected by the cross-slope. If grinding the seat is necessary, keep removal limits to 1/2-in or less.

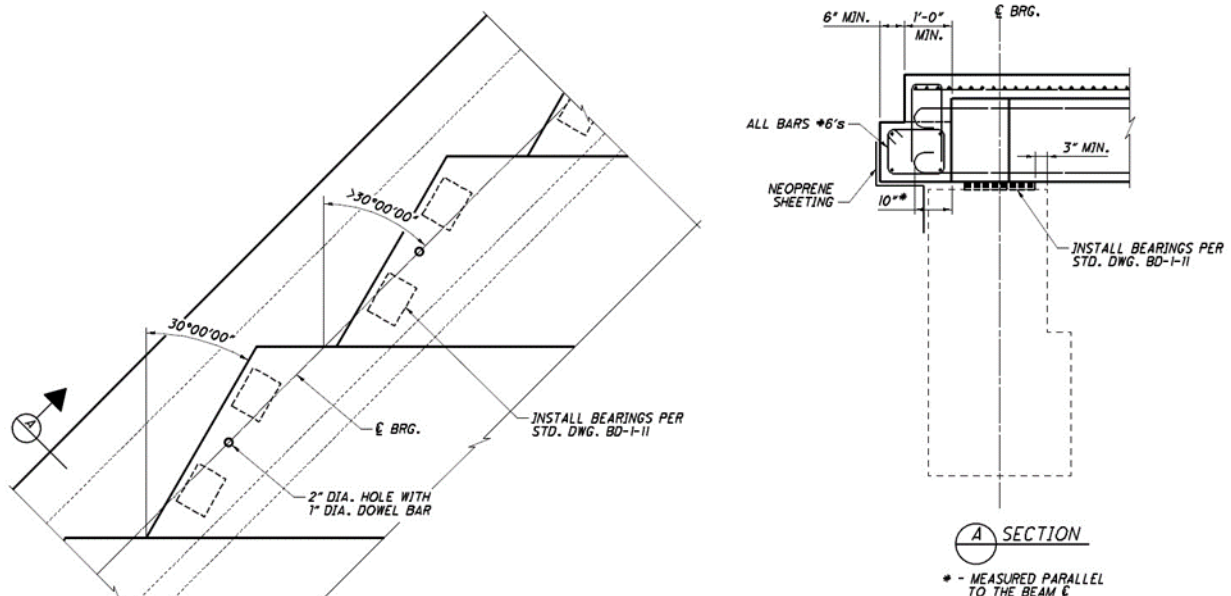


Figure 404-5

404.4.3 COMPOSITE DECK REPAIR

Do not replace composite decks on box beam structures.

C404.4.3

When the composite deck of an adjacent box beam structure needs repaired, the preferred method of repair would be an overlay. Refer to BDM Section 403.4 for additional information.

Complete removal of the composite deck without significantly damaging the top flange of the box beams is unlikely.

When overlaying the existing composite deck is not a viable solution, a complete superstructure replacement should be investigated.

404.5 REINFORCED CONCRETE SLAB BRIDGES

For single span slab bridges where stage construction is used, screen all bridges to determine whether the main reinforcement is parallel to the centerline of the roadway or is perpendicular to the abutment.

C404.5

Early standard drawings called for the main reinforcement to be placed perpendicular to the abutments when the skew angle became larger than a certain value. This angle was revised over the years as new standard drawings were introduced.

Concrete slab bridges should be screened according to the following criteria:

- A. Prior to 1931 the slab bridge standard drawing required the main reinforcement to be placed perpendicular to the abutments when the skew angle was equal to or greater than 20-degrees. This angle was revised to 25-degrees in 1931, 30-degrees in 1933 and finally 35-degrees in 1946. The standard drawing in 1973 required the main reinforcement to be parallel with the centerline of roadway regardless of the skew angle.
- B. If the skew angle of the bridge is equal to or greater than the angles listed above for the year built, a temporary longitudinal bent will have to be designed to support the slab where it is cut. For example, a bridge built in 1938 with a 25-degree skew does not require a bent, however a bridge built in 1928 with a 25-degree skew does require a bent to be designed.
- C. The deck should be inspected in the field to make a visual verification of the direction of the concrete reinforcement.

404.6 SEMI-INTEGRAL ABUTMENT CONVERSIONS

For substructure semi-integral conversion details refer to BDM Section 405.7.

C404.6

When the superstructure and/or deck is being replaced, converting of an existing jointed structure to semi-integral should be considered.

Converting to a semi-integral structure is encouraged where possible due to the decreased maintenance relative to a jointed structure.

The detail presented in BDM Figure 404-6 may be used to alleviate the lateral earth pressure induced into the superstructure. By utilizing these details, semi-integral conversions can be achieved on structures with a fixed pier and curved or dog-legged beams/girders.

If these details are used a fixed bearing is required for bridge stability. Design an expansion device to be installed at the end of the approach slab. Design and install the expansion device in accordance with standard drawing [AS-2-15](#).

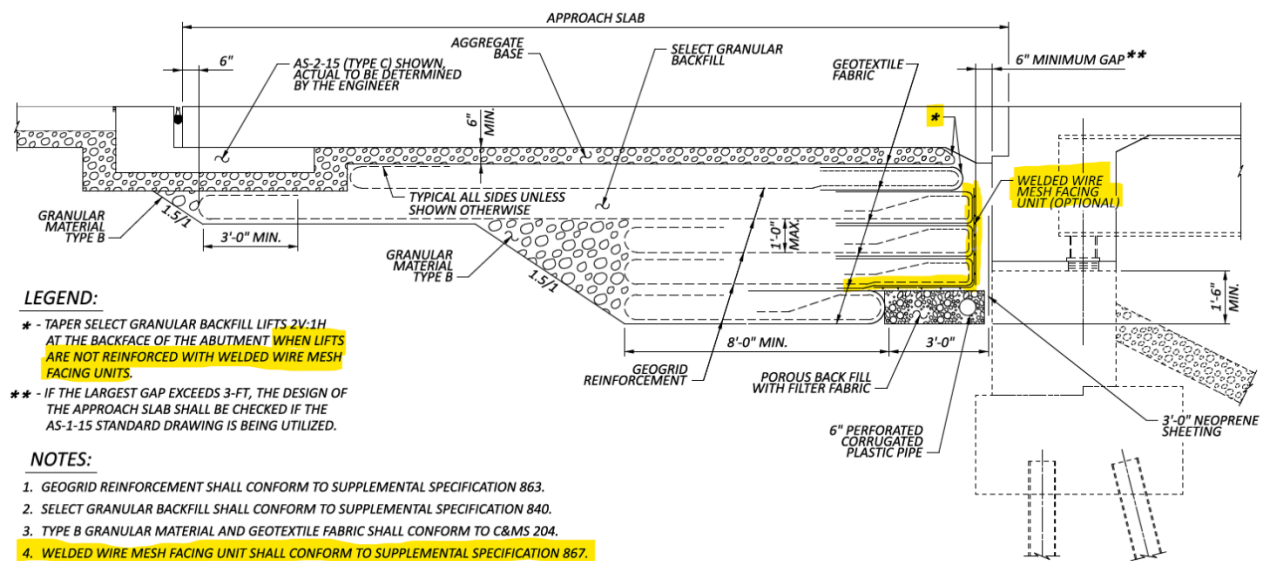


Figure 404-6

404.6.1 STRUCTURAL STEEL

Prior to encasing the beam ends prepare the ends per SSPC SP11 and paint beam ends with non-organic zinc prime coat per [C&MS](#) 514. Extend the limits of the beam preparation and painting 1-ft beyond the limits of the end diaphragm concrete.

After the diaphragm concrete is set, seal the interface between the beam and the concrete with caulk.

**404.6.1.1 INADEQUATE BEARING
CLEARANCE****C404.6.1.1**

The beam end may be coped when the clearance between the beam seat and the bottom of the flange is not sufficient to install the semi-integral bearing device. For additional details on the beam coping refer to BDM Figure 404-7. It is the responsibility of the engineer to ensure the coped beam end has sufficient capacity to carry the required loads.

This detail should only be used as an attempt to avoid performing modifications to the beam seat. Modifications to beam seat can be expensive and labor intensive. There is also a concern that the seat reinforcing will be damaged during the installation of the dowels. If seat modifications are inevitable the beam shall not be coped.

The modification to the existing beam and the components of the coping detail shall be designed by an engineer. Specify the bearing assembly to be either metalized, galvanized per [C&MS 711.02](#), or prime coated per [C&MS 514](#).

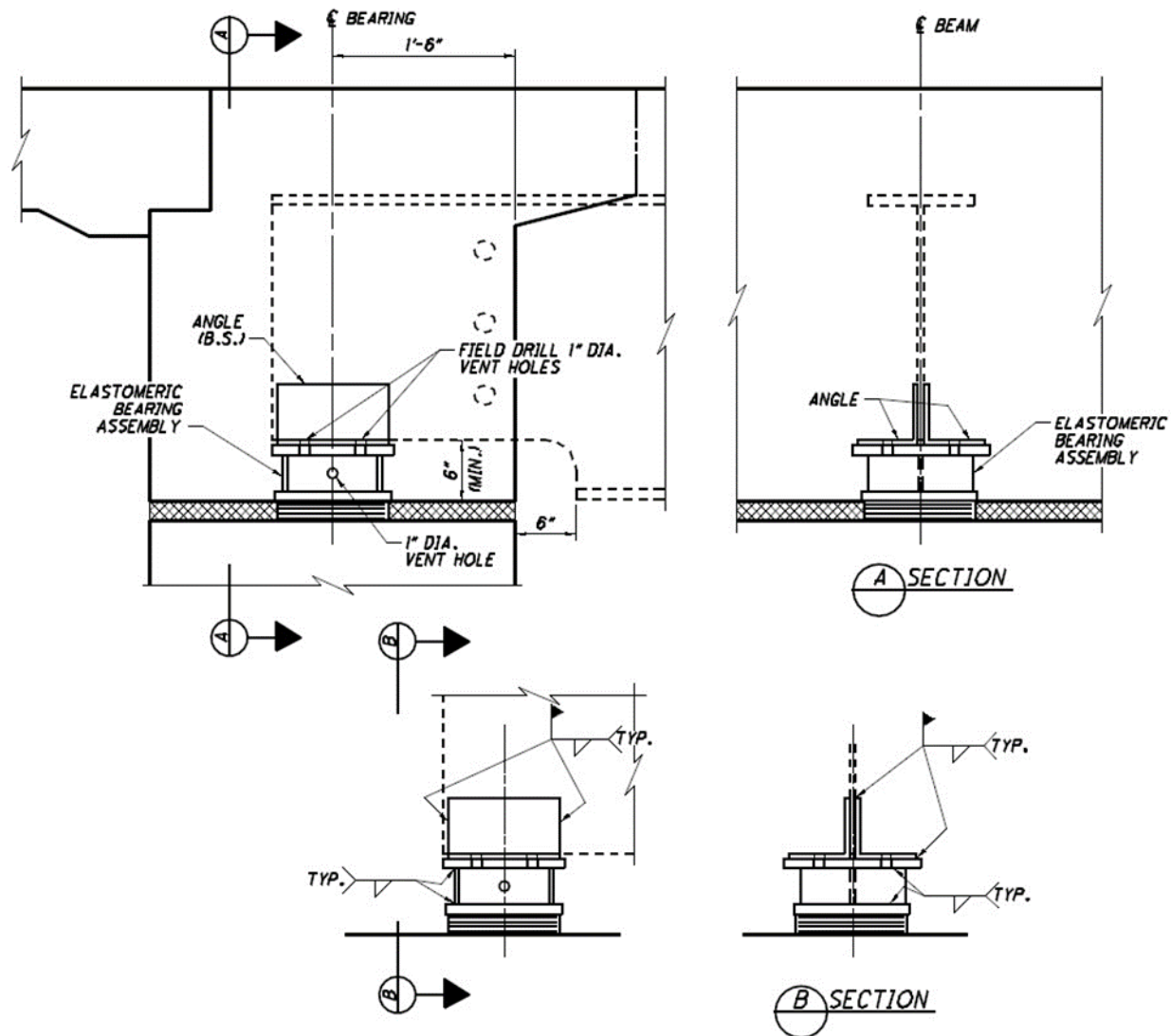


Figure 404-7

405 SUBSTRUCTURE

405.1 SUBSTRUCTURE CONDITION SURVEY

C405.1

Provide a Substructure Condition Survey for every bridge rehabilitation project. Prepare the survey as near as practicable to the plan preparation stage and before beginning detail design work for the rehabilitation. The survey shall include the following information for each substructure unit on the project:

The purpose of the Substructure Condition Survey is to identify and estimate deterioration effects. Focus areas are locations exposed to de-icing chemicals from roadway drainage or concrete disintegration where exposed in splash zones or stream flows.

These surveys are critical in determining the condition of the substructure and whether or not it is economical to reuse. The survey provides documentation of the location and type of distress in the substructure and should be detailed similar to the deck surveys. The surveys will be used later to develop detailed repair plans, if applicable.

- A. All bridge inventory information
- B. Most recent bridge inspection findings
- C. Age and history of existing rehabilitation work on the substructure units
- D. Dates and results of all evaluation techniques performed on the substructure units
- E. Sketches of the substructure units dimensioning; locations of unsound areas; delamination; cracking/crack density; spalling; patching; discoloration, efflorescence and moisture staining; section loss; and all testing/sampling locations

Extra care for should be taken for pile bents, these are the simplest and least expensive piers to construct. This pier type consists of driven piles (H-pile, circular pile, or fluted mono-tubes) with a concrete cap beam cast over the top of the piles to support the superstructure. Steel piling should be checked immediately below the splash zone, water line, and mud lines for deterioration and possible loss of section. High section loss is common these areas due to corrosion from bacterial attack at 3-ft to 6-ft below the water line or mud lines.
- F. Photos and descriptions of all substructure units
- G. Photos and descriptions of all cores and core holes (if applicable)

Abutment and pier concrete reuse may require core tests to determine the quality and strength of the concrete.
- H. Estimate of the total substructure area deemed unsound
- I. Scour assessment (if applicable)

Footings and pilings exposed due to erosion and undermining shall be assessed for loss of bearing capacity and/or section.
- J. Load Rating results for each substructure unit (if required by the Scope of Services)

If elements of the substructure are thought to have sufficient damage that significantly affects the load capacity then a Load Rating should be made.
- K. Recommendations for substructure rehabilitation treatments and estimated service life.

Substructures units with greater than 40% of the element in a condition state of 3 and 4 are candidates for replacement. Upon completion of the rehabilitation work, no portion of a main element should be below a condition state 2.

All exposed piling shall be encased during the rehabilitation project.
- L. Temporary shoring requirements (if necessary)
- M. Cost estimate for rehabilitation
- N. Maintenance of traffic considerations for rehabilitation

405.1.1 SUBSTRUCTURE PROTECTION FOR VEHICLE COLLISION

Refer to BDM Figure 1003-1

405.1.2 SEISMIC REHABILITATION

Evaluate all substructure units scheduled for rehabilitation according to BDM Section 303.1.4.

405.1.3 CORING

If required by Scope of Service, core substructure units concrete according to ASTM C42.

Take a minimum of two (2) cores per substructure unit.

Take at least one core from an apparently sound area to compare with core(s) taken from questionable areas. Take cores in suspicious areas to verify and further define areas of unsoundness.

Repair core holes using an ODOT approved non-shrink mortar, [C&MS 705.22](#), installed according to the manufacturer's recommended specifications. Seal repair with HMWM according to [C&MS 512.04](#).

Inspect the cores for: obvious crumbling; stratification or delamination zones; soundness of aggregate; and depth and condition of concrete reinforcement.

If required by the Scope of Services, test core samples for compressive strength according to ASTM C39 and for permeability according to ASTM C1202.

405.1.4 CHLORIDE CONCENTRATION TESTING

When specified in the Scope of Services, analyze the chloride ion content of all cores according to ASTM C1218.

C405.1.3

Coring is a destructive technique that provides samples for strength, permeability and chloride concentration testing whose results can indicate extent of deterioration. More importantly, this technique also provides visual evidence of the integrity of the substructure unit.

Cores should be taken when:

- A. Original plan or material information is not available.
- B. Determining type of repair material.
- C. Aid in determining integrity of concrete
- D. Multiple materials have been used.

Provide the location of all cores taken. Provide a description of the cores and all test results.

C405.1.4

The purpose of chloride concentration testing is to determine the extent of chloride ingress into the concrete surface. Corrosion of the reinforcement due to chloride ingress is a primary cause of damage to reinforced concrete structures. Corrosion typically exists when the chloride ion content exceeds 2-lb/yd³ [500 ppm].

For projects with no coring requirements specified, drill 1-in diameter holes in the substructure to a maximum depth of 2-in, or as specified in the Scope of Services, with a rotary impact drill. Collect powdered concrete samples at every 0.5-in of hole depth. After collecting each sample, thoroughly clean hole with compressed air before proceeding to avoid contaminating future sample collection. Label all sample containers by location and sample depth. Analyze the chloride ion content of all collected samples according to ASTM C1218. Repair all holes according to BDM Section 403.1.2. Determine the number of drill holes using the coring requirements of BDM Section 403.1.2.

Prepare a report of all test results. Include a sketch that identifies all hole locations and chloride ion concentrations.

405.2 CONCRETE REPAIR

Clearly denote areas of concrete removal on the plans. No corners of the saw cut perimeter shall be less than 90-degrees.

Provide minimum lap splices for replacement reinforcement to existing reinforcement or provide mechanical splices according to [C&MS](#) 509.07.

The material and coating of the replacement concrete reinforcement shall match the existing reinforcement's coating.

When replacement concrete reinforcement is anticipated, provide the following pay item: ITEM 509 – CONCRETE REINFORCEMENT, REPLACEMENT OF EXISTING CONCRETE REINFORCEMENT, AS PER PLAN. Provide a plane note to specify reinforcement to be replaced in-kind.

405.2.1 CONCRETE PATCHING WITH FORMWORK

Repair concrete reinforced with uncoated steel reinforcement according to [SS844](#). Repair concrete reinforced with epoxy coated steel reinforcement according to [C&MS](#) 519 or [SS844](#).

C405.2

Most bridge substructure rehabilitations will require some type of concrete repair. The following sections provides some conventional guidance when to use the several patching options listed in the Departments specifications.

Replacement reinforcement is required for steel damaged during concrete removal or deteriorated steel with 10% or more cross-section reduction. For an estimated quantity for replacement steel, use 5-10% of the existing steel in the repair area.

C405.2.1

The form-and-pour method is the primary method selected to repair existing substructures. The repair should incorporate materials and methods that are compatible with the existing materials, character, and purpose.

Provide a plan note that clearly defines the maximum removal limits for the concrete members under repair.

This work often requires additional information not covered by the [C&MS](#). For example, the Designer should specify limitations for concrete form anchorage placement, and permissible access locations for concrete placement.

The Designer should consider the structural effects of concrete removal extending beyond the limits anticipated and shown in the plans. The purpose should be to prevent an overstress/unstable condition that would require a change order for the addition of temporary support after the contract sale. Example conditions include: excess concrete removals which reduce development and lap splice lengths; and excess compression zone removals causing an unbalanced strain condition.

Designers should consider maximum removal limits at least 25% larger in every direction than the limits shown in the plans for determining structural effects. For projects where removal up to and including the maximum removal limits produce structural deficiencies, provide a pay item for temporary support and include the support locations and support loads in the plans.

Blister concrete repair areas shall meet the minimum concrete cover requirements specified in BDM Section 304.4.9.

For areas where the existing cover to the concrete reinforcement will not permit the minimum spacing for concrete consolidation (*LRFD 5.10.3.1*) between the bars and the form, provide additional cover by extending the surface beyond the original concrete surface with a blister.

405.2.2 TROWELABLE MORTAR

Repair concrete according to [SS843](#), Patching Concrete Structures with Trowelable Mortar.

Show the areas to be repaired with Trowelable Mortar in the plans.

C405.2.2

Trowelable mortar should generally be specified when the repair depth is less than 1.5-in deep, and the repair area is less than 150-ft². Trowelable mortar should also be specified in lieu of pneumatically placed mortar for the case where the depth of patch is equal to or less than 3-in and the quantity is less than 150-ft². The maximum depth of patch that should be attempted with this type of mortar is 3-in.

Trowelable mortar is typically specified for overhead and vertical surfaces of substructures. These hand-applied repair mortars are generally recommended for repairs cosmetic in nature.

Generally, placement thickness should be less than 2-3/4-in on vertical surfaces, and 2-in on overhead surfaces. While, deeper placements can be made, this will require additional time and multiple lifts according to most manufacturer recommendations.

Overhead spalls less than 1-in in depth are not candidates for trowelable mortar unless plan notes are provided to indicate removal depths greater than 1-in.

405.3 PNEUMATICALLY PLACED CONCRETE - SHOTCRETE

Repair concrete according to ITEM 520 – PNEUMATICALLY PLACED CONCRETE - SHOTCRETE.

Seal shotcrete repair areas according to [C&MS](#) 512. Provide a plan note to not apply sealers to shotcrete material for a minimum of 28 days after placement.

C405.3

Shotcrete is used in lieu of conventional concrete patching [C&MS](#) ITEM 519 – PATCHING CONCRETE STRUCTURES, in most instances for reasons of cost or convenience. Shotcrete is advantageous in situations when formwork is cost prohibitive or impractical. Shotcrete operations can often be accomplished in areas of limited access to make repairs to structures and is a good option for large areas (i.e. $\geq 150\text{-ft}^2$) of vertical and overhead patching.

The minimum compressive strength of shotcrete specified in [C&MS](#) 520 is 2000-psi at 3-days and 4000-psi at 28-days. For applications where a higher strength is warranted, the Item will need to be “As Per Plan” with notes defining the minimum compressive strength requirements as well as adjusted compressive strength pay factors defined in [C&MS](#) Table 520.16-1.

405.4 CATHODIC PROTECTION

Use embedded or surface applied galvanic anodes or distributed galvanic anode systems according to [SS844](#) where uncoated steel reinforcement is to be utilized with the following:

- A. Substructure patching
- B. Substructure repairs, where new concrete is placed next to existing concrete.
- C. Substructure widening, where new concrete placed next to existing concrete.
- D. Adding corrosion control in aggressive environments to a repair project.

Consideration should be given to concrete repairs where the epoxy coating on reinforcing steel has been compromised, by corrosion or mechanical means.

C405.4

SS844 uses a passive galvanic cathodic method of protection. The passive galvanic cathodic is based upon the principles of dissimilar metal corrosion and the relative position of specific metals in the galvanic series. Galvanic anodes consist of pure zinc encapsulated within a compressible material or an alkali-activated cementitious shell. The anodes are used to extend the life of standard concrete repairs by protecting reinforcement that remains in chloride-contaminated concrete around the concrete repairs where future corrosion can occur. SS844 specify two types of galvanic cathodic protection:

A discrete galvanic anode with a single point wire connection. (Examples are patching repairs anodes, AKA the Hockey puck. Drill-in plug-type for sound areas of concrete areas).

A distributed galvanic anode system is a long strip anode with a multiple wire connection to the reinforcing grid and distributed over the surface of a concrete structure. The system may be either embedded or surfaced applied.

Galvanic anodes are not effective with the following:

- i. Patch material with electrical resistivity greater than 50,000 ohm-cm. This may include polymer, fly ash, latex modified, and silica fume-based repair materials.

- ii. Additional steps are necessary if the rebars are epoxy coated.
- iii. Low Volume Shotcrete: Repairs performed by low volume shotcrete using Dry-Pak-It methodology..

In the plans, show the number of anodes, spacing requirements, minimum zinc requirement.

Refer to SS844 for additional design information.

Include BDM Detail Note [702.17-1] in the plans.

Provide details showing an electrical connection between both top and bottom mat of bridge deck reinforcing steel.

Discrete and distributed galvanic anodes are normally intended to provide corrosion protection both top and bottom mat of reinforcing steel.

Place anodes along the perimeter of concrete patches or along the interface between new/existing concrete to mitigate the formation of new corrosion sites in the existing concrete.

Corrosion control used for generally sound concrete with high corrosion risk Environment (Steel Potential greater than -350 mV) use either Drill-in plug-type anodes can be placed into drilled holes on a grid pattern in sound concrete to provide distributed protection or surface applied system with electrical leads. Corrosion control of pre-stressed/post-tensioned concrete structures provide an electrical connection between the wire strands and the anodes.

Determine the discrete galvanic anode spacing by the reinforcing steel density ratio. This ratio is equal to the perimeter of the rebar divided by the spacing of that rebar per 12 in² of concrete patch.

The use of galvanic anode or Distributed Anode Systems (DAS) requires the use of hydraulic cement-based material with an electrical resistivity less than 50,000 ohm-cm.

405.5 SUBSTRUCTURE REHABILITATION

C405.5

The important first step of repairing damaged or deteriorated concrete is to correctly determine the cause of damage. Knowing what caused the damage, and reducing or eliminating that cause, will make the repair last longer. If no attempt is made to eliminate the original cause of damage, the repair may fail as the original concrete did, resulting in wasted effort and investment.

405.6 ABUTMENT REHABILITATION

Repair concrete per BDM Section 405.2.

C405.6

There are several strategies for abutment rehabilitation. These strategies are focused around safety, capacity, functionality, operational need, life cycle, and financial constraints at the time of rehabilitation. The amount of analysis needed to determine the appropriate rehabilitation will vary.

Consider the following guidelines for substructure rehabilitation.

- A. After rehabilitation, an abutment may be assumed to have the same life cycle as the rehabilitated superstructure.
- B. Address scour problems or other hydraulics issues.
- C. Perform complete removal and replacement of deteriorated bridge seats beneath and adjacent to the bearings in lieu of remedial patching.
- D. Consider modifying beam seats by placing new concrete in front of the breastwall.

405.7 SEMI-INTEGRAL CONVERSIONS**C405.7**

On projects, where the deck is being replaced, consideration shall be giving to eliminating the expansion joints and converting to a semi-integral superstructure. Bridges without joints provide better protection from water and salt damage to the superstructure and substructure.

The Department will not permit the use of semi-integral abutments founded on a single row of piles.

Because the superstructure is not mechanically attached to the foundation, the abutment foundation shall be stable. Abutments on a single row of piling will deflect which can cause damage to bearings or possibly even a loss of bearing.

Spread footings supported on soil may be considered for semi-integral conversions provided the analysis of all applied loadings do not cause a settlement, bearing capacity or sliding problem for the foundation.

Abutments founded on two or more rows of piling, drilled shafts and spread footings on rock do not require additional analysis in order to convert to semi-integral.

The semi-integral design is appropriate for a total unfactored expansion/contraction movement of 3-in or less.

This movement equates to approximately a 250-ft expansion length for steel assuming a temperature range of 150°F and coefficient of thermal expansion of $6.5 \times 10^{-6}/^{\circ}\text{F}$ or a 500-ft expansion length for concrete assuming a temperature range of 80°F and coefficient of thermal expansion of $6.0 \times 10^{-6}/^{\circ}\text{F}$.

Detail the geometry and layout of the approach slab, wingwalls, curbs, sidewalks, utilities and transition concrete bridge railing to permit the anticipated longitudinal movement of the superstructure.

Wingwalls details shall accommodate the relative movement of between the walls and the approach slab since the superstructure and approach slab move together. If the approach slab were connected to turned back wings in any manner, then movement of the entire superstructure would be restricted. Also refer to BDM Section 306.2.2.6 for further discussion.

For bridges with skews, design the rehabilitated substructures for the forces caused by superstructure expansion/contraction. Refer to BDM Section 306.2.2.6 for more information. These forces could be transferred with Diaphragm Guides, wingwalls or guided bearings. Installing new Diaphragm Guides and guided bearings may be difficult because of the primary steel in the bearing seats and should therefore be considered when bearing seats are also being replaced. These forces may also be too large for a typical turned back wingwall to resist.

Do not place the proposed approach slab on top of turnback wingwalls. There shall be a minimum on 6-in, measured vertically, between the removal limits of the turnback wingwall and the bottom of the proposed approach slab.

This will likely lead to the turnback wingwalls needing to be widened from their original position. Doweling into the existing wingwall and footing is often an effective method for constructing the proposed wingwalls.

Do not install the bridge and/or approach railing on top of turnback wingwalls. Locate the railing on top of the approach slab.

405.8 PIER REHABILITATION

C405.8

Repair concrete according to BDM Section 405.2.

Seal all surfaces of piers adjacent to roadways as described in BDM Section 306.1.2 with epoxy-urethane or non-epoxy sealer according to [C&MS](#) 512.

Protect pier supports added to widened substructures according to BDM Section 1003.10 S3.6.5.1.

The Department will consider single column and two column substructure units that support more than one primary superstructure member as non-redundant unless otherwise supported by a structural analysis. The Department will consider single column substructure units that support non-redundant superstructure members as non-redundant. Connecting a non-redundant type of substructure unit into an adjacent unit may eliminate a redundancy issue and prevent the need to provide additional pier protection.

Designers should investigate the cost and feasibility of connecting all single column substructure units into adjacent units. Even though single column substructure units that support one primary redundant superstructure member may be assumed as redundant, connecting the single column into an adjacent substructure unit will provide additional resistance to loss in the event of a

vehicle collision regardless of the pier protection required.

Provide sufficient details and plan notes to clearly identify the limits and methods of the proposed repairs on the plans

405.9 PIER CAP STRENGTHENING

C405.9

The shear capacity of existing reinforced concrete pier caps may be a common item contributing to an inventory level rating factor less than 1.00 for the Design Loading defined in BDM Section 401.3.

The most common method of analysis for concrete pier caps is the Sectional Method (*LRFD 5.7.3*) which may produce overly-conservative results for deep members. Alternatively, the Strut-and-Tie Method (*LRFD 5.8.2*) yields more accurate results; however, the STM analysis requires more effort and experience than the Sectional Method. Fortunately, a tool, STM-CAP.xlsm, is now available to simplify STM modeling of pier caps. More information regarding the use of this tool, as well as design examples, are available in the final report, Evaluation of Reserve Shear Capacity of Bridge Pier Caps Using the Deep Beam Theory.

Refer to BDM Section 401.4 for more information related to Design Exception requirements and considerations.

Regardless of the method used to increase the structural capacity of a pier cap, perform an analysis and detail the rehabilitation to represent the correct sequence of load application.

Provide a Composite Fiber Wrap System in accordance with [PN519](#). Clearly dimension the FRP locations in the plans and specify the factored capacity increase (in Kips) required for each location identified. Include pay items for the work necessary to restore damaged and/or unsound concrete substrate deeper than 0.5-in and larger than 4-in². Provide a pay item for epoxy injection when cracks in the existing substrate are greater than 10 mils (0.010-in).

Designate each appurtenance attached to existing members to be wrapped to be removed and reinstalled. The Department will pay for this work separately from the Composite Fiber Wrap System.

For example, if the existing pier cap is not jacked and supported to remove dead load, then the strengthening methodology will not contribute to the dead load carrying capacity of the member.

The Department considers the use of externally applied composite FRP as the most feasible and economical method to address strengthening of pier caps in good condition. [PN519](#) does not include work to repair concrete surfaces. This work needs to be paid for under separate pay items.

Other methods to consider for strengthening pier caps experiencing varying degrees of deterioration include externally applied post-tensioning and normally reinforced concrete encasement. In both cases, all steel shall be encased in concrete with adequate cover and sufficient room to permit the flow of concrete and aggregate.

405.10 PIER COLUMNS**405.10.1 PIER COLUMN WRAPPING**

Provide a Composite Fiber Wrap System in accordance with [PN519](#). Clearly dimension the FRP locations in the plans and specify the factored capacity increase (in Kips) or confining stress required for each location identified.

Include pay items for the work necessary to restore damaged and/or unsound concrete substrate deeper than 0.5-in and larger than 4-in². Provide a pay item for epoxy injection when cracks in the existing substrate are greater than 10 mils (0.010-in).

Designate appurtenances attached to existing members receiving FRP to be removed and reinstalled. The Department will pay for this work separately from the Composite Fiber Wrap System.

405.10.2 PIER COLUMN SHIELDING**405.11 FOUNDATIONS**

When the Scope of Services for a rehabilitation project requires an assessment of an existing foundation, perform analysis of both the structural and geotechnical resistance of the existing foundation to be reused in accordance with BDM Section 305.

The Department will allow the reuse of bridge foundations for rehabilitation projects provided each of the following items are met:

- A. There are no scour issues.
- B. The foundations are structurally sound.
- C. The foundations are not exhibiting settlement or rotational issues.
- D. Foundations are not timber.

If conditions A through D are met, and if the proposed service dead load has not increased by greater than 15% compared to the original designed service dead load, the foundations may be reused without any evaluation of their geotechnical resistance.

C405.10.1

When 15% or more of the surface area contains corroded steel reinforcement, provide a layer of concrete encasement to be placed around it. To add a concrete encasement:

- A. Place anchor bars in existing solid concrete using expansion anchors or epoxy anchors.
- B. Place horizontal and vertical reinforcement or welded wire reinforcement in the encasement.
- C. Place forms and pour concrete with no less than a 6-in thickness.
- D. Provide FRP

C405.10.2

For pier columns located adjacent to roadways or in range of de-icing solutions, snow, and ice spray from snow and ice removal operations, shielding products such as Poly Salt Armor® or similar may be used.

C405.11

Reusing existing bridge foundations can save considerable costs and shorten delivery times significantly.

The determination of structural soundness requires consideration of section loss due to corrosion. Refer to BDM Section 405.11.1 for more information on corrosion rates.

405.11.1 EXISTING STEEL PILES

To determine the factored structural resistance for existing steel piling, utilize a corrosion rate of 0.47-mils/yr (0.00047-in/yr) for piles buried in non-aggressive natural soils, and a total service life equal to the current age plus an additional 75-yr service life. If the evaluation of the site conditions indicates a potential pile deterioration or corrosion problem, determine the appropriate corrosion loss rate for carbon steel per Eurocode 3, Part 5, Section 4.4 for the environmental conditions at the site.

Provide pile encasement for exposed steel piling for bridges over waterways. The pile encasement shall extend a minimum of 3-ft below the ground line/stream bottom.

405.11.2 DRIVEN PILES

Determine the geotechnical resistance of existing piling to be reused in a rehabilitation project using the following criteria:

- A. For driven pile foundations where the dynamic load testing records can be found and reviewed, determine the geotechnical resistance using a resistance factor equal to 0.70 applied to the measured nominal end of initial driving (EOID) resistance or the Restrike resistance, whichever was measured last. If more than one pile in the foundation unit was tested, use the value for the pile with the lowest measured resistance.
- B. For driven pile foundations where no dynamic test results are available for (A) above, but pile driving logs are available from the installation of the existing piling, determine the geotechnical resistance using the WSDOT Pile Driving Formula and a dynamic resistance factor of 0.55.

C405.11.1

Steel piling should be inspected immediately below the splash zone or water line for deterioration and possible loss of section. High section loss has occurred in some areas due to corrosion from bacterial attack at 3-ft to 6-ft below the water line. Calculated structural resistance of existing piling and pile caps shall account for measured and estimated section loss.

A form of pile encasement for new bridges is detailed on Standard Bridge Drawing [CPP-1-08](#). For existing bridges, the polyethylene pipe may be cut and half and held in place after installation with steel straps. The top of the encasement shall be located no more than 1-ft from the bottom of the pile cap and the concrete fill shall be sloped to drain.

Alternatively, the exposed piling can be coated in accordance with ITEM 514 – PAINTING OF STRUCTURAL STEEL or [SS845](#) – SHOP AND FIELD METALLIZING OF STRUCTURAL STEEL.

C405.11.2

This requirement assumes dynamic test results (EOID or Restrike) are available and provide the minimum driven pile nominal resistance.

This requirement assumes pile driving logs are available from the installation of the existing piling. The WSDOT Pile Driving Formula is discussed in publication WA-RD 610.1, "Development of the WSDOT Pile Driving Formula and Its Calibration for Load and Resistance Factor Design (LRFD)" (Allen, 2005).

- C. For foundations where: (1) the minimum driven pile resistance is unknown for (A) above and (2) pile driving logs are not available for (B) above, determine the geotechnical resistance using the pile resistance (minimum bearing capacity) from the original plans. For plans produced prior to LRFD, this value is allowable; multiply by a factor of 2 to determine the nominal geotechnical resistance, and then apply a resistance factor of 0.55 for the factored geotechnical resistance.
- D. For foundations where (A), (B) and (C) above do not apply, but the installed pile dimensions are definitively known, determine the geotechnical resistance using static analysis in accordance with BDM Section 305.3 and appropriate resistance factors from BDM Table 305-1.

This requirement assumes the original plans are available and provide the pile resistance (minimum bearing capacity). The length of the existing pile need not be known, nor do boring logs or the soil profile need to be available.

This requirement assumes the length of existing pile is known and boring logs are available. If the length of the existing pile is unknown, non-destructive test methods can be performed on the existing piling to determine their length. If boring logs are not available, a new subsurface investigation will be necessary to generate new boring logs.

If none of the above are applicable, the piles may not be considered for re-use.

405.11.3 MICROPILES

Do not reuse existing micropile foundations. New micropiles may be used to supplement other existing foundations for increased resistance.

C405.11.3

Micropiles can withstand axial and/or lateral loads and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in access- restrictive environments and in all soil types and ground conditions. Micropiles can be installed at angles using the same type of equipment used for ground anchor and grouting projects.

405.11.4 SPREAD FOOTINGS

Calculate structural and geotechnical resistance of spread footing foundations in accordance with BDM Section 305.2. Do not modify the footprint for an existing spread footing founded on soil. If it is necessary to widen or increase loading for a structure supported on a spread footing, supplement the spread footing with deep foundation elements.

C405.11.4

The foundation soil beneath a new extension of a spread footing will consolidate under the increased loading. This consolidation will lead to settlement, rotation, or structural damage.

Any increase in dead loading to a spread footing founded on soil will result in additional settlement and the potential to exceed bearing resistance. Include these effects in the analysis and design of the existing or replacement superstructure.

Refer to BDM Section C405.7 when considering spread footings for semi-integral abutments.

405.11.5 DRILLED SHAFTS

Calculate structural and geotechnical resistance of drilled shaft foundations in accordance with BDM Section 305.4.

405.11.6 CONTINUOUS FLIGHT AUGER (CFA) PILES

Calculate structural and geotechnical resistance of CFA pile foundations in accordance with BDM Section 305.6.

405.12 WIDENING

Refer to BDM Section 406.1 for information regarding the use of anchors and dowels.

405.12.1 ABUTMENT WIDENING

Make abutments and footings continuous by splicing new reinforcement to either:

- A. Dowels anchored into the existing substructure.
- B. The existing substructure's reinforcement exposed by concrete removal operations.

Seal abutment surfaces according to BDM Section 303.1.

405.12.2 PIER WIDENING

Provide separate caps for widenings requiring three or more columns.

405.12.3 FOUNDATION WIDENING

Support widened foundations on piles, drilled shafts or spread footings on rock to limit differential settlement.

C405.11.5

Drilled shafts socketed into bedrock will typically provide sufficient capacity for reuse; however, friction drilled shafts may present a capacity issue if the loading on the existing shafts increases.

C405.11.6**C405.12****C405.12.1**

It is typically much more cost effective to detail splices using dowels than by removing concrete and splicing onto the existing reinforcement. Refer to BDM Section 405.12 for restrictions on anchor dowels.

C405.12.2

Consider connecting new caps for multi-columned piers, in which two or less columns are required, to the existing caps and avoid non-redundant members.

For bridge widening projects, the pier cap may be widened and cantilevered off the existing stem. Refer to BDM Section 405.12 for restrictions on anchor dowels.

Refer to BDM Section C405.8 for more information on pier protection for non-redundant members.

C405.12.3

Consider differential foundation settlements. For example, if it is required to widen a bridge adjacent to an existing spread footing, it is possible that the existing foundation has settled as much as it is going to. However, if the widened portion is placed on a new spread footing, then that portion will settle with respect to the original and distress to the structure will result.

**405.13 MECHANICALLY STABILIZED
EARTH (MSE) WALLS**

Refer to the Standard Drawing, MSE Wall Repair, for repairs to MSE walls. MSE Wall Repairs are located on the [OGE website](#).

405.14 SUBSTRUCTURE DRAINAGE

Retrofit all dysfunctional drainage systems. Provide replacement drainage according to BDM Sections 200 and 300.

Perform pavement drainage calculations according to the [L&D, Vol. 2](#). If the calculations indicate that the existing scuppers are not necessary, and the deck is not to be replaced, permanently plug the existing scuppers and collect the additional drainage at the end of the bridge. If the deck is to be replaced, remove the scuppers and grind all scupper weld locations smooth.

For existing scuppers to remain, extend the length so that the termination is at least 8-in below the bottom of the bottom flange. Remove and repair or completely replace existing scuppers with extensive section loss due to corrosion.

405.15 BRIDGE SCOUR

For all bridges over waterways, perform a Scour Appraisal in accordance with the [ODOT Manual of Bridge Inspection](#), Section 3.4.3.7

For bridges carrying ODOT's Priority System (SNBI Item B.H.01.01 – Code P) and located within a Federal Aid project limits, include all work necessary to eliminate scour criticality as part of the project.

405.15.1 SCOUR DEFINITIONS

Check Flood – Scour event as defined by the scour check flood frequency specified in L&D Volume 2 Section 1008.10.1. Scour associated with this event indicates the maximum amount of foundation material removed to allow the structure to remain in service.

Design Flood – Scour event as defined by the scour design flood frequency specified in L&D Volume 2 Section 1008.10.1. Scour associated with this event is the maximum amount of foundation material removed that does not affect the load carrying ability of the structure.

Observed Scour – A void in foundation material beneath or adjacent to a structure, resulting from the scouring action of a waterway, and documented during a Scour Assessment.

Scour – The removal of foundation material beneath or adjacent to a structure resulting from the scouring action of a waterway.

Scour Analysis – The process required to determine an estimated depth of scour for the design and check flood events. This process includes geotechnical exploration, topographic and hydrographic survey, hydraulic modelling, and scour computations.

C405.14

Damage often occurs to existing bridges as a result of poorly designed drainage. Proper drainage is extremely important to the longevity of the structure.

C405.15

Scour Appraisal – The risk-based and data-driven determination of a bridge’s vulnerability to scour, resulting from the least stable result of scour that is either observed, or estimated through a Scour Evaluation or a Scour Assessment.

Scour Assessment – An activity performed by competent individuals to identify, locate, measure and document Observed Scour by inspecting existing conditions and comparing to past conditions. AssetWise form entitled "SNBI Scour Appraisal" represents an acceptable Scour Assessment.

Scour Countermeasures – Material, devices, or structures designed to prevent, slow, or stop the occurrence of scour.

Scour Critical – A bridge condition that indicates structural instability is possible due to any future scouring action of a waterway. This condition can be illustrated by analysis when: (1) The calculated scour depth is lower than the foundation elevation; (2) Observed Scour produces a foundation capacity-to-demand ratio less than 1.0; (3) When the foundation for a bridge over or adjacent to a waterway is unknown.

Scour Evaluation – An interdisciplinary team approach involving hydraulic, structural, and geotechnical expertise to determine the scour vulnerability of an existing or proposed structure over a waterway. A Scour Evaluation includes a Scour Assessment, Scour Analysis and resulting structural evaluation.

Scour Plan of Action – Documented monitoring plan for known and potential Scour Critical deficiencies. A Scour POA template can be found in the ODOT Manual of Bridge Inspection, Appendix H.

405.15.2 OBSERVED SCOUR

C405.15.2

At existing structures where Observed Scour exists, perform maintenance at the site to reduce the affected structure’s vulnerability to scour. Document all maintenance activities and monitor for scour during routine bridge inspections.

For maintenance activities that may reduce a structure’s vulnerability to scour, refer to the ODOT Bridge Maintenance Manual for recommended causes and repairs to Stream Migration, Stream Degradation and Abutment Scour.

If either of the following conditions exist, perform a Scour Evaluation in accordance with BDM Section 405.15.3.

- A. Scour recurs after performing prior scour maintenance activities.
- B. The Observed Scour depth is lower than the bottom of footing.

405.15.3 SCOUR EVALUATION

C405.15.3

Perform a Scour Evaluation for the following:

- A₁. Structures with Observed Scour as noted in BDM Section 405.15.2.A

B₁. All bridges carrying ODOT's Priority System (SNBI Item B.H.01.01 - Code P) and located within a Federal Aid project limits, with a SNBI Item B.AP.03 - Code C, D, E or U.

The Scour Evaluation process is defined as follows:

A ₂ . Perform a Scour Assessment	Continue to Step B.
B ₂ . Review all available documentation from:	Continue to Step C.
1. Historical site aerial imagery	
2. Historical plans	
3. Scour Assessment	
C ₂ . Evaluate documentation for evidence of:	For locations with any of these streambed degradations listed, continue to Step D, otherwise skip to Step E.
1. Observed Scour	
2. Horizontal and vertical channel movement	
3. Upstream and downstream meandering	
D ₂ . Evaluate Scour Countermeasure options	This part of the process shall involve expertise from hydraulic, structural and geotechnical engineering for preliminary consideration of feasible Scour Countermeasures. Continue to Step E.
E ₂ . Determine if an existing hydraulic model of the project site is available.	If an existing model is available, continue to Step F, otherwise skip to Step G.
F ₂ . Review the hydraulic model and perform scour computations.	The computations shall provide scour elevations for the Design Flood and Check Flood events defined in <i>LRFD</i> 2.6.4.4.2. Skip to Step J.
G ₂ . Perform topography and hydrography survey and geotechnical exploration	Steps G, H and I represent a Scour Analysis. The computations shall provide scour elevations for the Design Flood and Check Flood events defined in <i>LRFD</i> 2.6.4.4.2. Continue to Step J.
H ₂ . Develop hydraulic model	
I ₂ . Perform scour computations	
J ₂ . Compare Observed Scour elevations to Design Flood and Check Flood scour elevations.	If the Observed Scour elevation is lower than the Design Flood scour elevation, continue to Step K.
	If the Observed Scour elevation is not lower than the Design Flood scour elevation, skip to Step M.
	If there is no Observed Scour, but the foundation is unknown, skip to Step M; otherwise skip to Step R.
K ₂ . Determine the nominal geotechnical and structural resistances of the foundation elements due to the following two conditions:	Determine foundation element resistance accord to BDM Section 305. Continue to Step L.
1. Observed Scour	If the existing foundation is unknown, then a spread footing founded on soil can be assumed to determine a resistance.
2. Check Flood scour	
L ₂ . Determine if the results of Step K are acceptable in accordance with BDM Section 401.4.	If the foundation capacity-to-demand ratio is 1.0 or greater for the Observed Scour, a Design Exception is not warranted. Continue to Step M.

If the foundation capacity-to-demand ratio is less than 1.0 for the Observed Scour, submit a request for a Design Exception. Skip to Step N.

If the foundation capacity-to-demand ratio is less than 1.0 for the Observed Scour and the Design Exception request is denied, Skip to Step O.

If foundation deformation (e.g. settlement, rotation, etc.) is identified during the Scour Assessment, consider the resulting load effects to the superstructure when determining the need for a Design Exception in accordance with BDM Section 401.4.

M₂. Develop plans for Scour Countermeasures. Include the Design Flood and Check Flood scour elevations in the General Notes.

This part of the process shall involve expertise from hydraulic, structural and geotechnical engineering for final consideration of Scour Countermeasures.

For bridges with unknown foundations, the Design and Check Flood scour elevations shall both be equal to the bottom of footing elevation. Skip to Step P. Otherwise, the Observed Scour elevation shall be provided as the Design Flood elevation in the Plans. Skip to Step Q.

N₂. Upon approval of the Design Exception request, develop plans for Scour Countermeasures. Include the Design Flood and Check Flood scour elevations in the General Notes.

This part of the process shall involve expertise from hydraulic, structural and geotechnical engineering for final consideration of Scour Countermeasures.

The Observed Scour elevation shall be provided as both the Design Flood and Check Flood elevations in the Plans. Skip to Step P.

O₂. Develop plans for Scour Countermeasures and rehabilitation measures to increase the nominal geotechnical and structural resistances of the foundation elements for Design Flood and Check Flood scour elevations to be included in the General Notes.

This part of the process shall involve expertise from hydraulic, structural and geotechnical engineering for final consideration of Scour Countermeasures and foundation rehabilitation.

Continue to Step P.

P₂. Implement Scour Countermeasures and structural upgrades as necessary.

Continue to **Step Q.**

Q₂. Perform routine inspection of the bridge.

End of Scour Evaluation.

406 MISCELLANEOUS REHABILITATION TOPICS

406.1 ADHESIVE ANCHOR/DOWEL DESIGN

Do not specify an adhesive anchor/dowel system for the following applications:

A. Deck edge replacement for non-slab bridges

The preferred repair would be to reuse the existing transverse deck steel in the deck edge replacement. The deck edge for some slab bridge may be able to be designed using dowels.

- | | |
|---|--|
| B. Replacing a concrete railing on an existing deck | The preferred repair would be to rebuild the deck edge and build the railing per the applicable standard. |
| C. Retrofitting an abutment with a diaphragm guide. | The preferred repair would be to remove the existing abutment concrete with a jack hammer while leaving the existing reinforcing and placing the diaphragm reinforcement around the existing reinforcement. Since SICD-2-14 utilizes a closed loop, solutions that use 2 U-shape bars or headed rebar may help with placing the new reinforcing around the existing reinforcing steel. |

Design adhesive anchor/dowel systems in accordance to *AASHTO LRFD 5.13*.

Specify an adhesive anchor/dowel system when the anchor/dowel is subjected to a constant tension force or the factored tension load on the anchor/dowel is 15% or greater of the anchor/dowel's factored tensile capacity.

For example, if the factored tensile strength of 5/8 -in anchor is 10-kips and the anticipated ultimate tensile load on the anchor is equal to or greater than 1.5 kips, design the anchor using an adhesive anchor system.

Use an adhesive anchor/dowel system when the anchor/dowel is subjected to a factored shear load on the anchor/dowel is 30% or greater of the anchor/dowel's factored shear capacity.

When performing a backwall replacement on a jointed structure, design the dowels as an adhesive dowel system when the height of the wall exceeds 5'-0". The height of the wall is measured from the top of the beam seat to the top of the wall. This dimension is variable across the width of the abutment, utilize the average height of the wall when determining if the dowels for the wall need to be designed using an adhesive system.

Provide an adhesive anchor/dowel systems that meets the acceptance criteria of *ICEES AC308 - Post-installed Adhesive Anchors in Concrete Elements*.

Use either galvanized steel reinforcement or uncoated steel reinforcement as specified in BDM Section 304.4. Refer to BDM Section 304.4.1.1 for when it is applicable to use uncoated steel reinforcement.

When specifying an adhesive anchor/dowel system, list a minimum of two adequate systems in the contract plans.

Provide the factored tensile and shear loads for the anchors/dowels in the contract plans. Refer to BDM Section 602.8 for an example note.

406.2 REDESIGNATION OF NON-REDUNDANT MEMBERS

For guidance on the redesignation of existing non-redundant members, refer to BDM Section 312.

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SECTION 500 – TEMPORARY STRUCTURES

501 GENERAL

This section is a supplement to [C&MS 502, Structures For Maintaining Traffic](#). All design guidelines of [C&MS 502](#) apply.

502 PRELIMINARY DESIGN C502

For the Structure Type Study, show the grade, alignment, approximate location and width of the temporary structure on the Preliminary Structure Site Plan.

For the Preliminary Design Report, show the grade and the alignment of the temporary structure on the Final Site Plan. When the temporary structure can adequately be shown on the Site Plan for the permanent bridge, a Site Plan for the temporary structure is not required. The required Site Plan information shall be in accordance with BDM Section 200. Submit the preliminary design of the temporary structure concurrently with the Preliminary Design Report Submission for the permanent structure.

Determine the roadway width, hydraulic design, clearance requirements, and all other design parameters in conjunction with the development of the preliminary design.

502.1 HYDRAULICS

The hydraulics design shall be in accordance with the L&D, Vol. 2, Section 1011.

503 DETAIL DESIGN C503

The temporary structure detail plans shall be complete and independent of the permanent structure plans. The temporary structure detail plans shall include:

- A₁. General Plan and Elevation
- B₁. General notes
- C₁. Estimated quantities

- D₁. Concrete Reinforcement bar list
- E₁. All necessary detail plan sheets.

The table provided for Estimated quantities is “For Estimating Purposes Only”.

The Department will pay for temporary structure under one Lump Sum bid item:

ITEM 502 - STRUCTURE FOR MAINTAINING TRAFFIC.

Design temporary bridge structures as permanent structures in accordance with the *AASHTO LRFD Bridge Design Specifications* and this Manual. The allowance for Future Wearing Surface shall be 0.0 ksf. The temporary bridge plans shall include Design Data in the General Notes as defined in BDM Section 602.

For ice pressure loads, assume the thickness of ice to be 6-in, with a 200-psi effective ice strength. Assume the force to act at the level of the design year highwater elevation.

The bridge railing for the temporary structures shall meet the requirements of BDM Section 309.4.

The following requirements apply:

- A₂. Specify the allowable design unit stresses of the lumber in the plans. State whether timber sizes are full sawn or standard dressed sizes.
- B₂. The nominal thickness of wood plank or strip floor shall be 3-in minimum.
- C₂. Timber floors shall be securely fastened to the stringers and stringers shall be securely fastened to the substructure units.
- D₂. Field welded connections shall meet require the Nondestructive Testing requirements specified in [C&MS](#) 513.
- E₂. The road surface on the temporary structure shall have antiskid characteristics, crown, drainage and superelevation in accordance with all ODOT and AASHTO publications.

If the Designer elects to use a flexible railing system or standard portable concrete barrier, the design and detailing of the bridge shall allow for the deflection characteristics of the railing system.

Generally, a temporary structure should be designed to be easily constructed and removed with minimal cost. The following items should be considered when designing a temporary bridge:

- A. Timber decks, H-pile bents, and simple spans are commonly used.
- B. Locally available lumber should be specified.
- C. When circumstances permit, all or part of the existing bridge may be used for the run-around.
- D. Bolted connections are preferred and generally are more economical.
- E. Designs that minimize debris accumulation should be considered.
- F. Shop Drawings are not required. Adequate plan details need to be provided.

504 GENERAL NOTES

C504

The designer should provide plan note(s) with the Temporary Structure plans similar to the following:

- A. The Contractor may substitute used or alternate members for the members shown on the Temporary Structure Plans, provided that the strength of the substitute or alternate member is equal to or greater than the original member. Maintain waterway opening size and required clearances. Submit calculations for the substitute or alternate member according to [C&MS](#) 502. Use only new bolts.
- B. Structural steel need not be coated.

When ITEM 513 - STRUCTURAL STEEL is specified in the temporary structure plans, only the following [C&MS](#) descriptions shall apply:

- A₁. Straightening [C&MS](#) 513.11
- B₁. Holes for High Strength and Bearing Bolts

[C&MS](#) 513.19

C₁. High Strength Steel Bolts, Nuts and Washers [C&MS](#) 513.20

D₁. Welding [C&MS](#) 513.21

E₁. Nondestructive Testing [C&MS](#) 513.25

F₁. Shipping, Storage and Erection [C&MS](#) 513.26

When Item 511 is specified in the temporary structure plans, [C&MS](#) 511 surface finish requirements are not required.

Include the following notes in the Structure General Notes. In the roadway plans the pay item description “614 Maintenance of Traffic” shall include an “as per Plan.”

Coordination with the roadway plans for this item is required.

A₂. MAINTENANCE: Maintain all portions of the temporary structure in good condition with regard to strength, safety and rideability. The Department will consider this maintenance to be incidental to ITEM 614 - MAINTAINING TRAFFIC. Maintain the waterway opening shown on the plans at all times. If debris accumulates within the waterway opening or on any part of the structure promptly remove the debris. The Department will compensate for debris removal according to [C&MS](#) 109.05.

B₂. CLOSING OF THE TEMPORARY STRUCTURE:

If for any reason or at any time the temporary structure's ability to safely carry traffic is in question, immediately take the actions necessary to protect traffic, repair and reopen the temporary structure. When closing a temporary structure for this purpose, immediately notify the Engineer and the appropriate law enforcement agency. Water elevations exceeding the design (5) year highwater elevation or an excessive accumulation of debris within the waterway opening shall be sufficient reasons to close the temporary structure. Mark the design (5) year highwater elevation with fluorescent paint on the temporary structure, at a visible location. The Department will consider the costs associated with closing the temporary structure to be incidental to ITEM 614 - MAINTAINING TRAFFIC.

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SECTION 600 – TYPICAL GENERAL NOTES

601 DESIGN REFERENCES

601.1 GENERAL

This section contains various typical general notes. The designer needs to assure that the typical notes are complete and apply to the specific project. These notes may need to be revised or specific notes must be written to conform to the actual conditions that exist on each individual project.

601.2 STANDARD DRAWINGS AND SUPPLEMENTAL SPECIFICATIONS

The designer shall list all Standard Bridge Drawings and Supplemental Specifications that apply, giving date of approval or latest revision date, if revised.

The Designer shall also ensure the listed Standard Bridge Drawings and Supplemental Specifications are transferred to the project plans title sheet and match the information on the title sheet.

[601.2-1] REFER to the following Standard Bridge Drawing(s):

_____ Dated (revised) _____

_____ Dated (revised) _____

_____ Dated (revised) _____

and to the following Supplemental Specification(s):

_____ Dated _____

_____ Dated _____

601.3 DESIGN SPECIFICATIONS

The designer shall include the following note specifying the design specifications used on the structure. If the note is not correct, then the note should be revised with the correct criterion that describes the design specifications for the structure.

[601.3-1] DESIGN SPECIFICATIONS: This structure conforms to the Xth Edition* of the “LRFD Bridge Design Specifications” adopted by the American Association of State Highway and Transportation Officials, XXXX* and the ODOT Bridge Design Manual, 2020.

NOTE TO DESIGNER:

* Designer should fill-in current edition.

The use of note [601.3-1] stipulates the use of live load distribution and designs based on AASHTO specifications, assumptions and standard beam theory design. For the vast majority of ODOT bridges this criterion is not only adequate but also advantageously conservative. There are structure types which, due to either AASHTO’s own limitations or the type of structure, require specific live load distribution factors or other analysis methods other than classical beam theory to analyze the structure. Some examples may include a highly skewed slab bridge, a curved steel girder bridge, cable stayed bridges, etc. If the structure’s analysis required the use of 2D or 3D models including grillage, finite element, finite strip, classical plate solutions the following note should be added.

[601.3-2] SPECIAL DESIGN SPECIFICATIONS: This bridge required the use of a # (# two or three) dimensional model using the ## (## grillage, finite element, finite strip, classical plate theory, etc.) design method to analyze the structure. The computer program used for structural analysis was . The bridge components designed by this method and the live load distribution factors used were:

Dead Load Distribution: (The designer is to explain the assumptions used in how the dead load was applied and distributed)

Live Load Distribution Factors:

Exterior Members- for wheel (or axle) load & for lane load moments.

- for wheel (or axle) load & for lane load shears

Interior Members- for wheel (or axle) load & for lane load moments.

- for wheel (or axle) load & for lane load shears

NOTE TO DESIGNER: Modify the wording of the note as necessary. Also amend the Design Specifications note **[601.3-1]** with the wording “excepted as noted elsewhere in the plans”

602 DESIGN DATA

The designer shall include the following pertinent design information with the Structure General Notes for all bridge plans:

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 1001.2

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.

NOTE TO DESIGNER: Refer to BDM Section 1001.3 for guidance.

602.2 DESIGN LOADING

For bridges designed for highway loads, provide the following Design Loading note:

[602.2-1] DESIGN LOADING includes:

Vehicular Live Load: HL-93 *

Future Wearing Surface (FWS) of 0.0XX ** kips/ft²

This bridge received an approved Design Exception for Design Loading Structural Capacity. ***

NOTE TO DESIGNER:

* HL-93 vehicular live loading is shown. Update designation for vehicular live loading if different.

** Designer should fill-in FWS allowance based on BDM Section 303.1.2 or 401.3.

*** Include this portion of the note ONLY when an approved Design Exception for Design Loading Structural Capacity has been granted.

For bikeway/pedestrian bridges that will not accommodate vehicular traffic, provide the following Design Loading note:

[602.2-2] DESIGN LOADING includes:

Pedestrian Live Load: 0.090 kips/ft²

For bikeway/pedestrian bridges subject to vehicular traffic, provide the following Design Loading note:

[602.2-3] DESIGN LOADING includes:

Pedestrian Live Load: 0.090 kips/ft²

Vehicular Live Load: H15-44 (Truck only)

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 QC (3) , with (4) -in max. aggregate size - compressive strength 4.5 ksi (drilled shaft)

Concrete Reinforcement:

Uncoated Steel Reinforcement – minimum yield strength 60-ksi (5)

Epoxy Coated Steel Reinforcement - minimum yield strength 60-ksi (5)

GFRP Reinforcement (5)

Galvanized Steel Reinforcement – minimum yield strength 60-ksi (5)

Chromium Steel Reinforcement, Type (6) – minimum yield strength 100-ksi (5)

Stainless Steel Reinforcement – minimum yield strength 60-ksi (5)

Structural Steel - ASTM A709 Grade (7) - yield strength (7) ksi

Steel H-piles - ASTM A572 - yield strength 50 ksi

Steel Sheet Piling – ASTM A572 Grade (8) - yield strength (8) ksi

Steel CIP piles – ASTM A252 Grade (9) - yield strength (9) 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) Class QC5 Concrete for drilled shafts < 7'-0", Class QC4 Concrete for drilled shafts $\geq$ 7'-0"
- (4) For all drilled shafts, specify the maximum coarse aggregate size based on the spacing of the concrete reinforcement. See BDM Section 305.4.4.3 for guidance.
- (5) Provide a list describing the general location of the reinforcement in the structure. (e.g. deck, bridge railing, pier, abutment, etc.)
- (6) Type CS, CM or CL
- (7) 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
- (8) Grade 50 – yield strength 50 ksi, or Grade 55 – yield strength 55 ksi, or Grade 60 – yield strength 60 ksi, or Grade 65 – yield strength 65 ksi, or
- (9) Grade 2 – yield strength 35-ksi, or
Grade 3 – yield strength 45-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].

[602.3-2] DESIGN DATA:
Concrete for prestressed beams:
Compressive Strength (final) - (1) ksi
Compressive Strength (release) - (2) ksi
Welded Wire Fabric:
Yield Strength – 70 ksi (4)
Prestressing strand:
Area = (3) in²
Ultimate Strength = 270 ksi
Initial stress = 202.5 ksi (Low relaxation strands)

NOTE TO DESIGNER:

- (1) Specify 28-day compressive strength from the following range: 5.5 – 7.0 ksi
- (2) Specify compressive strength at release from the following range: 4.0 – 5.0 ksi
- (3) Specify prestressing strand area from the following: 0.167 in², or 0.217 in²
- (4) Reference to Welded Wire Fabric applies to I-beams only.

602.4 FOR RAILWAY PROJECTS

For structures carrying railroad traffic, provide notes [602.3-1]; [602.3-2] (if necessary); and the following notes on the project plans:

[602.4-1] DESIGN SPECIFICATIONS: This structure conforms to the requirements of the "Manual for Railway Engineering" by the American Railway Engineering and Maintenance-of-way Association, XXXX * Edition.

CONSTRUCTION AND MATERIAL SPECIFICATIONS: State of Ohio, Department of Transportation, dated January 1, XXXX. *

NOTE TO DESIGNER: Note [601.3-2] may be required if special criteria or distributions have been used for the design of this rail structure. See [601.3-2] and determine if a modified note is required for inclusion. Fill-in items above marked "*" with current edition.

Provide the following note, modified as necessary to meet AREMA and/or a specific railroad criterion, with all railroad structures.

[602.4-2] DESIGN LOADING: Cooper E-80 with diesel impact

602.5 MONOLITHIC WEARING SURFACE

Furnish the following note for concrete bridge decks.

[602.5-1] MONOLITHIC WEARING SURFACE is assumed, for design purposes, to be 1 inch thick.

602.6 SEALING OF CONCRETE SURFACES

Use the following notes when permanent anti-graffiti coatings are required:

[602.6-1] ITEM 512 SEALING OF CONCRETE SURFACES, AS PER PLAN, (PERMANENT GRAFFITI PROTECTION):

Apply a permanent graffiti coating qualified according to S1083 that is compatible with the concrete sealer over which it is applied. Apply the graffiti coating in accordance with the manufacturer's printed instructions.

602.7 BRIDGE SCOUR

Furnish the following note in all bridge plans where Scour Evaluations are performed:

[602.7-1] SCOUR ELEVATIONS: The Design Flood and Check Flood scour elevations are provided below:

	Rear Abutment	Pier 1	Pier 2	Forward Abutment
Design Flood				
Check Flood				

NOTE TO DESIGNER: Adjust the table to fit the project as necessary. Refer to BDM Section 305.1.6 or BDM Section 405.15 as appropriate for more information.

602.8 ADHESIVE ANCHOR/DOWEL SYSTEMS

When specifying an adhesive anchor/dowel system, provide the following Adhesive Anchor/Dowel note:

[602.8-1] Adhesive Anchor/Dowel System

Install adhesive anchors/dowels according to the manufacture's installation instructions published in the ICC-ES Reports listed below.

The holes for the adhesive anchors shall be drilled with a hammer drill and carbide bit prior to the installation of the anchors, clean and dry the holes in a manner consistent with the manufacture's requirements for dry concrete.

<https://icc-es.org/evaluation-report-program/reports-directory/>

Select from one of the following approved products:

Adhesive Anchor/Dowel System Option 1 – ICC-ES ESR Report Number

Adhesive Anchor/Dowel System Option 2 – ICC-ES ESR Report Number

Factored Anchor/Dowel Loads: Tension – ____ kips

Shear – ____ kips

The Contractor may use a substitute adhesive anchor/dowel system that meets the acceptance criteria of ICEES AC308 - Post-installed Adhesive Anchors in Concrete Elements. The substitute anchors shall meet or exceed the capacity of the adhesive anchor/dowel systems listed above.

Adhesive anchors/dowels shall not be substituted with mechanical anchors.

The contractor shall supply documentation sealed by an Ohio registered Professional Engineer ensuring that the substitute adhesive anchor/dowel system provides sufficient capacity for this application in accordance with *AASHTO LRFD Section 5.13*. Install the anchors according to the manufacturer's installation instructions published in the ICC-ES Report.

NOTE TO DESIGNER:

Provide the factored loading for each independent anchor/dowel design in the plans. Refer to BDM Section 406 for guidance.

603 EXISTING STRUCTURE REMOVAL NOTES**603.1 GENERAL REMOVAL NOTES**

The following sample notes will serve as a guide in composing the note(s) for the removal of the existing structure. Modify the notes as required to fit the conditions. Use the following note if it is the desire of the owner to salvage any portion of the bridge.

[603.1-1] REMOVAL OF EXISTING STRUCTURE: Carefully dismantle the _____ and store along the right-of-way for disposal by the State's forces.

Describe the degree of care to be exercised in the removal in sufficient detail to allow accurate bidding. If this option is used, the pay item shall be "as per plan".

Use the following note when removal of structure to 1 foot below ground line as specified in [C&MS 202](#) will not fill the specific requirements of the project.

[603.1-2] ITEM 202, PORTIONS OF STRUCTURE REMOVED, AS PER PLAN:

Remove abutments to Elev. _____ Remove piers to Elev. _____

Use the following note when special protection of an existing structure to be incorporated into a new structure is required.

[603.1-3] ITEM 202, PORTIONS OF STRUCTURE REMOVED, AS PER PLAN: This item shall include the elements indicated in the plans and general notes and that are not separately listed for payment, except for wearing course removal. Items to be removed include all existing materials being replaced by new construction and miscellaneous items that are not shown to be incorporated into the final construction and are directed to be removed by the Engineer. The Department will not permit the use of explosives, headache balls and/or hoe-rams. Do not begin work until the Engineer accepts the method of removal and the weight of hammer shall be approved by the Engineer. Perform all work in a manner that will not cut, elongate or damage the existing concrete reinforcement to be preserved. Chipping hammers shall not be heavier than the nominal 90-pound class. Pneumatic hammers shall not be placed in direct contact with concrete reinforcement that is to be retained in the rebuilt structure. Submit construction plans according to [C&MS 501.05](#).

When temporary support is not provided for concrete members to be repaired and the Plans show defined removal areas, provide the following note describing the maximum removal limits.

[603.1-4] MAXIMUM REMOVAL LIMITS: Sound the concrete to determine the limits of the concrete to be removed and compare these limits to the areas shown in the plans. If new areas are discovered or if the dimensions of the plan areas increase by more than 25% in any direction, document the areas and notify the Engineer for evaluation two weeks prior to removal. The Engineer will determine if patching in discrete sections/stages is needed or if the installation of temporary falsework is required.

NOTE TO DESIGNER: The 25% amount shown in this note is a minimum as mentioned in BDM Section 405.2.1. If a larger percentage of removal is considered acceptable, adjust this note accordingly.

603.2 CONCRETE DECK REMOVAL PROJECTS

Use the following removal note for concrete deck removal projects, where the existing superstructure is to remain. Delete the portions in the note that are not appropriate for the specific project.

[603.2-1] ITEM 202, PORTIONS OF STRUCTURE REMOVED, AS PER PLAN DESCRIPTION: This work consists of the removal of concrete decks including sidewalks, concrete bridge railings, metal railings, deck joints and other appurtenances from steel supporting systems (beams, girders, cross-frames, etc.). The provisions of Item 202 apply except as specified by the following notes. Perform work carefully during deck removals to protect portions of such systems that are to be salvaged and incorporated into the proposed structure. The Department will not permit the use of explosives, headache balls and/or hoe ram type of equipment. Submit construction plans according to [C&MS 501.05](#).

PROTECTION OF STEEL SUPPORT SYSTEMS: Before deck slab cutting begins, draw the outline of primary steel members in contact with the bottom of the deck on the surface of deck. Drill small diameter pilot holes 2 inches outside these lines to confirm the location of flange edges. Deck cuts over or within 2 inches of flange edges shall not extend lower than the bottom layer of concrete reinforcement in the deck slab. Cuts made outside 2 inches of flange edges may extend the full depth of the deck. Perform work carefully during cutting of the deck slab to avoid damaging steel members that are to be incorporated into the proposed structure. Replace or repair steel members damaged by the deck slab cutting operations at no cost to the project. At least 7 days before performing repair work, submit a proposed repair plan, developed by an Ohio registered professional engineer to the Engineer. Obtain the Engineer's approval before performing repair.

PROTECTION OF PRESTRESSED CONCRETE SUPPORT SYSTEMS: Before deck slab cutting begins, draw the outline of primary prestressed concrete members in contact with the bottom of the deck on the surface of deck. Drill small diameter pilot holes 2 inches outside these lines to confirm the location of the edges of those members. Deck cuts over or within 2 inches of flange edges shall not extend lower than the bottom layer of concrete reinforcement in the deck slab. Cuts made outside 2 inches of flange edges may extend the full depth of the deck. Perform work carefully during cutting of the deck slab to avoid damaging prestressed concrete members that are to be incorporated into the proposed structure. Replace or repair prestressed concrete members damaged by the deck slab cutting operations at no cost to the project. At least 7 days before performing repair work, submit a proposed repair plan, developed by an Ohio registered professional engineer to the Engineer. Obtain the Engineer's approval before performing repair.

REMOVAL METHODS: The Contractor may remove concrete by cutting and by means of hand operated pneumatic hammers employing pointed or blunted chisel type tools. For removals over structural members (prestressed box beam, I-beam, steel beam steel girder, etc.), the Contractor may use a hammer heavier than 35 pounds but not to exceed 90 pounds unless approved by the Engineer. Removal methods over structural members shall ensure adequate depth control and prevent nicking or gouging the primary structural members. Due to the possible presence of attachments (e.g., finishing machine, scupper and form supports, etc.) to existing structural members, perform work carefully during deck removal to avoid damaging structural members that are to remain. Replace or repair structural members damaged by the removal operations at no cost to the project. At least 7 days before performing repair work, submit a proposed repair plan, developed by an Ohio registered professional engineer to the Engineer. Obtain the Engineer's approval before performing repair.

DECK REMOVALS - COMPOSITE DECK DESIGNS – STEEL SUPERSTRUCTURES: Due to the presence of welded studs to the existing structural steel, submit a detailed procedure of the deck removal with the Engineered Drawings according to [C&MS 501.05](#). The procedure shall include all details, equipment and methods to be used for removal of the concrete over the flanges and around the studs. Replace or repair main steel and studs damaged by the removal operations at no cost to the project. At least 7 days before performing repair work, submit a proposed repair plan according to [C&MS 501.05.C](#) to the Engineer to replace or repair structural steel and studs damaged by the removal operations. The Department will not pay for damage repairs.

DECK REMOVALS -COMPOSITE DECK DESIGNS – PRESTRESSED SUPERSTRUCTURES: Due to the presence of composite concrete reinforcement between the deck and the prestressed beam flanges, submit a detailed procedure of the deck removal with the Engineered Drawings according to [C&MS 501.05](#). The procedure shall include all details, equipment and methods of removal over the prestressed beams and around the composite reinforcing steel. Replace or repair prestressed members and composite concrete reinforcement. Submit a proposed repair plan according to [C&MS 501.05.C](#) to the Engineer to replace or repair prestressed members and composite reinforcement damaged by the removal operations. The Department will not pay for damage repairs.

EXISTING WELDED ATTACHMENTS: Remove existing welded attachments (e.g., finishing machine and form supports; and supports for scuppers and bulb angles which are to be removed) located in the designated tension portions of the top flanges of existing steel members and grind the flange surfaces smooth. Carefully grind parallel to the flanges.

MEASUREMENT & PAYMENT: The Department will measure the quantity of removals on a lump sum basis. The Department will pay for the accepted quantities of removals at the contract price for ITEM 202 - PORTIONS OF STRUCTURE REMOVED, AS PER PLAN. For modifications to or extensions of existing concrete substructure members, include the following notes in an Item 202, as per plan note.

[603.2-2] DECK REMOVALS – DECK EDGE REHABILITATION: If the contractor's means and methods require drilling holes through the bridge deck, all holes shall be drilled from the bottom of the deck upward. Drilling holes from the top of the deck is not permitted. Falsework intended to form the new deck edge may be used as a work platform to aide in the deck edge removal. If forms become damaged during concrete removal, the damaged portions shall be repaired or replaced prior to pouring the concrete. Prior to placing concrete, thoroughly clean all surfaces of the forms that contact the bottom of the existing bridge deck to ensure a snug fit between the forms and existing bridge deck. The portion of the form to be in contact with the fresh concrete shall be free of laitance, silt, dirt, shavings, sawdust, loose and built-up rust, and other debris. All holes drilled through the bridge deck shall be repaired with trowelable mortar in accordance with Supplemental Specification 843.

If a splitter is used to demolish the deck edge, holes shall be spaced no further than 12-in apart, measured longitudinally along the bridge deck. Each hole shall be used to split the concrete; skipping holes will not be permitted. Any damage to the existing structure that is to remain will be repaired at no additional cost to The Department. If the damage to the remaining structure is significant, at the direction of the Engineer, provide a repair plan in accordance with C&MS 501.05.C.

NOTE TO DESIGNER:

When using BDM General Note [603.2-2], BDM General Note [603.2-3] shall also be included in the plans.

[603.2-3] CUT LINE CONSTRUCTION JOINT PREPARATION: Prior to removing the deck edge, place a 1-in (+0-in,-1/4-in) deep saw cut at the boundaries of proposed concrete removals. If there are integral concrete pier caps within the proposed removal limits, also sawcut the deck concrete along the interface of the deck and pier cap. Remove concrete to a rough surface. Leave the existing concrete reinforcement, if required in the plans, in place. Install dowel bars if specified. Prior to concrete placement abrasively clean joint surfaces and existing exposed reinforcement to remove loose and disintegrated concrete and loose rust. Thoroughly clean the joint surface and exposed reinforcement of all dirt, dust, rust or other foreign material by the use of water, air under pressure, or other methods that produce satisfactory results. Existing steel reinforcement does not have to have a bright steel finish but remove all pack and loose rust. Thoroughly drench existing concrete surfaces with clean water and allow to dry to a damp condition before placing concrete.

[603.2-4] SUBSTRUCTURE CONCRETE REMOVAL: Remove concrete by means of approved pneumatic hammers employing pointed and blunt chisel tools. The Department will not permit hydraulic hoe-ram type hammers. The weight of the hammer shall not be more than 35 pounds for removal within 18 inches of portions to be preserved. Outside the 18-in limit, the contractor may use hammers not exceeding 90 pounds upon the approval of the Engineer. Do not place pneumatic hammers in direct contact with concrete reinforcement that is to be retained in the rebuilt structure.

604 TEMPORARY STRUCTURE CONSTRUCTION

Include the applicable portions of the following temporary structure note on the plans if the bridge roadway width is other than 23-ft, or if the use of the existing structure is part of the temporary road. See BDM Section 500 for additional information.

[604-1] TEMPORARY STRUCTURE roadway width shall be _____ feet. The existing structure may be moved and used for the temporary structure without strengthening.

605 EMBANKMENT CONSTRUCTION

For all substructure units where embankment construction is involved, provide appropriate embankment construction notes in the Structure General Notes. Consult the Office of Geotechnical Engineering for the recommended notes to use at a specific project site.

605.1 FOUNDATIONS ON PILES IN NEW EMBANKMENTS

The following construction method should minimize the effect of lateral forces acting on substructure units and their piles.

For structures with abutments on piles placed in new embankments use the following note:

[605.1-1] PILE DRIVING CONSTRAINTS: Prior to driving piles, construct the spill through slopes and the bridge approach embankment behind the abutments up to the level of the subgrade elevation for a minimum distance of (1) behind each abutment. Do not begin the excavation for the abutment footings and the installation of the abutment piles until after the above required embankment has been constructed and a (2) calendar day waiting period has elapsed. The Engineer may adjust the length of the waiting period based on settlement platform readings. After the specified waiting period has elapsed, drive abutment piles to the UBV* or to refusal on bedrock.

NOTE TO DESIGNER:

- (1) Generally, 200-ft. Optionally, this distance may be defined by station-to-station dimensions.
- (2) Estimate the length of the waiting period by determining the time required for 90% of primary settlement to occur. If the designer determines that a waiting period is not necessary, this portion of the plan note may be removed.
- * Typically, choose one or the other (UBV or to refusal on bedrock), based on whether friction piles or piles to refusal on bedrock are being driven. If setup is to be utilized in the design of friction piles, replace UBV with "the driving criteria determined by setup restrike testing".

For structures with abutments and piers on piles placed in new embankments use the following note:

[605.1-2] PILE DRIVING CONSTRAINTS: Prior to driving piles, construct the spill through slopes and the bridge approach embankment behind the abutments up to the level of the subgrade elevation for a minimum distance of (1) behind each abutment. Do not begin the excavation for the abutment footings and the installation of the abutment and pier piles, for pier(s) (2), until after the above required embankment has been constructed and a (3) calendar day waiting period has elapsed. The Engineer may adjust the length of the waiting period based on settlement platform readings. After the specified waiting period has elapsed, drive the abutment and pier piles, for pier(s) (2) to the UBV* or to refusal on bedrock.

NOTE TO DESIGNER:

- (1) Generally, 200-ft. Optionally, this distance may be defined by station-to-station dimensions.
- (2) Identify specific piers.
- (3) Estimate the length of the waiting period by determining the time required for 90% of primary settlement to occur. If the designer determines that a waiting period is not necessary, this portion of the plan note may be removed.
- * Typically, choose one or the other (UBV or to refusal on bedrock), based on whether friction piles or piles to refusal on bedrock are being driven. If setup is to be utilized in the design of friction piles, replace UBV with "the driving criteria determined by setup restrike testing".

For structures with wall type abutments on piles placed in new embankment use the following note:

[605.1-3] PILE DRIVING CONSTRAINTS: Prior to driving piles at the abutments, construct the bridge approach embankment behind the abutments up at a 1:1 slope from the top of the heel of the footing* to the subgrade elevation and for a minimum distance of 250-ft behind the abutments. Do not begin the installation of the abutment piles until after the above required embankment has been constructed and a (1) calendar day waiting period has elapsed. The Engineer may adjust the length of the waiting period based on settlement platform readings. After the specified waiting period has elapsed, drive abutment piles to the UBV** or to refusal on bedrock. After the footing and the breastwall have been constructed, construct the embankment immediately behind the abutments up to the beam seat elevation and on a 1:1 slope up to the subgrade elevation prior to setting the beams on the abutments.

NOTE TO DESIGNER:

- * In some cases, the bottom of the heel may be used.
- (1) Estimate the length of the waiting period by determining the time required for 90% of primary settlement to occur. If the designer determines that a waiting period is not necessary, this portion of the plan note may be removed.
- ** Typically, choose one or the other (UBV or to refusal on bedrock), based on whether friction piles or piles to refusal on bedrock are being driven. If setup is to be utilized in the design of friction piles, replace UBV with "the driving criteria determined by setup restrike testing".

For MSE wall supported abutments with driven piles use the following note **when it is not necessary to use a settlement waiting period:**

[605.1-4] PILE DRIVING CONSTRAINTS: Prior to driving abutment piles to the Ultimate Bearing Value (UBV) or to refusal on bedrock, construct the MSE wall and the bridge approach embankment behind the abutment up to the bottom of the footing for a minimum distance of (1) behind each abutment. The Contractor may pre-drive abutment piles before constructing MSE walls. Pre-driving consists of installing the abutment piles into the soil only as far as necessary so that the pile will remain vertical during MSE wall construction. If pre-driving piles, install pile sleeves around piles before constructing the MSE wall. Provide at least 3-ft of pile above the top of the pile sleeve to meet the requirements of [C&MS 507.09](#) regarding splices. Do not drive abutment piles to the UBV* or to refusal on bedrock until after the above required MSE wall and embankment have been constructed. In order to remove any negative skin friction that developed during the **MSE wall and embankment construction**, drive each abutment pile a distance of at least 0.5-in.

If not pre-driving abutment piles, install the abutment piles through pile sleeves after the above required MSE wall and embankment have been constructed and the specified waiting period has elapsed.

NOTE TO DESIGNER:

Generate an MSE abutment section detail drawing based on BDM Figure 201-3, but specific to the details of the proposed bridge abutment and place it in the abutment details for the bridge or in the MSE wall typical sections.

- (1) Generally, 200-ft. Optionally, this distance may be defined by station-to-station dimensions.
- * Typically, choose one or the other (UBV or to refusal on bedrock), based on whether friction piles or piles to refusal on bedrock are being driven. If setup is to be utilized in the design of friction piles, replace UBV with "the driving criteria determined by setup restrike testing".

For MSE wall supported abutments with driven piles use the following note when it is necessary to use a settlement waiting period to avoid downdrag on the abutment piles:

[605.1-5] PILE DRIVING CONSTRAINTS: Prior to driving abutment piles to the Ultimate Bearing Value (UBV) or to refusal on bedrock, construct the MSE wall up to the bottom of the footing and construct a temporary wire-faced MSE wall and the bridge approach embankment behind the abutment up to the level of the subgrade elevation for a minimum distance of (1) behind each abutment per the abutment section detail drawing. The Contractor may pre-drive abutment piles before constructing MSE walls. Pre-driving consists of installing the abutment piles into the soil only as far as necessary so that the pile will remain vertical during MSE wall construction. If pre-driving piles, install pile sleeves around piles before constructing the MSE wall. Provide at least 3-ft of pile above the top of the pile sleeve to meet the requirements of C&MS 507.09 regarding splices. Do not drive abutment piles to the UBV* or to refusal on bedrock until after the above required MSE wall and embankment have been constructed and a (2) calendar day waiting period has elapsed. The Engineer may adjust the length of the waiting period based on settlement platform readings. After the specified waiting period has elapsed, drive abutment piles to the UBV* or to refusal on bedrock. In order to remove any negative skin friction that has developed during the waiting period, drive each abutment pile a distance of at least 0.5-in.

If not pre-driving abutment piles, install the abutment piles through pile sleeves after the above required MSE wall and embankment have been constructed and the specified waiting period has elapsed.

NOTE TO DESIGNER:

Generate an MSE abutment section detail drawing based on BDM Figure 201-9, but specific to the details of the proposed bridge abutment, and place it in the abutment details for the bridge or in the MSE wall typical sections.

(1) Generally, 200-ft. Optionally, this distance may be defined by station-to-station dimensions.

(2) Estimate the length of the waiting period by determining the time required for 90% of primary settlement to occur. If the designer determines that a waiting period is not necessary, this portion of the plan note may be removed.

* Typically, choose one or the other (UBV or to refusal on bedrock), based on whether friction piles or piles to refusal on bedrock are being driven. If setup is to be utilized in the design of friction piles, replace UBV with "the driving criteria determined by setup restrrike testing".

605.2 FOUNDATIONS ON SPREAD FOOTINGS IN NEW EMBANKMENTS

The following construction method helps to eliminate any lateral forces on the foundation due to the construction of the embankment and/or settlement of the subgrade under the embankment. For stub abutments on spread footings being constructed in new embankments provide note [605.3-1] or [605.3-2] and the following note:

[605.2-1] CONSTRUCTION CONSTRAINTS:

Prior to constructing the spread footing foundations, construct the bridge approach embankments behind the abutment up to a 1:1 slope from the bottom of the heel of the footing to the subgrade elevation and for a minimum distance of (1) behind the abutments. Do not begin the construction of the spread footing foundations until after the above required embankment has been constructed and a (2) calendar day waiting period has elapsed. The Engineer may adjust the length of the waiting period based on settlement platform readings. After the abutment footing and breastwall are completed and prior to setting superstructure members, construct the embankment immediately behind the abutment up to the beam seat elevation and on a 1:1 slope up to the subgrade elevation, with Type B granular material conforming to 703.16.C.

NOTE TO DESIGNER: Modify the note, as appropriate, for piers constructed on a spread footing foundation.

(1) Generally, 250-ft. Optionally, this distance may be defined by station-to-station dimensions.

(2) Estimate the length of the waiting period by determining the time required for 90% of primary settlement to occur. If the designer determines that a waiting period is not necessary, this portion of the plan note may be removed.

For wall type abutments on spread footings with no new embankment provide note [605.3-1] or [605.3-2] and the following note:

[605.2-2] CONSTRUCTION CONSTRAINTS: Fill the void created by excavating for the abutment footings with Type B granular material, [C&MS](#) 703.16.C. After the footing and the breastwall have been constructed, fill the void behind each abutment up to the beam seat elevation and from the beam seat up on a 1:1 slope to the subgrade elevation prior to constructing the backwall and setting the beams on the abutment.

605.3 EMBANKMENT CONSTRUCTION NOTE

In an attempt to reduce settlements of the roadway approaches, specify the placement of embankment materials in 6-in lifts. Include one of the following plan notes in the Project General Notes and make reference to the work defined below at the appropriate locations within the plans.

Note that Item 203 is a roadway quantity and coordination with the roadway plans is necessary.

To define the limits of measured pay quantities for bridges with wall-type abutments, provide excavation, backfill, and embankment diagrams (or a composite diagram, where suitable), using schematic abutment cross-sections, showing the boundaries between structure and roadway excavation, and between structure backfill and roadway embankment.

[605.3-1] ITEM 203 EMBANKMENT, AS PER PLAN: Place and compact embankment material in 6 inch lifts for the construction of the approach embankment between stations ** to ** .

NOTE TO DESIGNER:

** The approximate limits should be 100-ft behind each abutment

[605.3-2] ITEM 203 EMBANKMENT, AS PER PLAN: Place and compact embankment material in 6-in lifts for the construction of the approach embankment.

605.4 UNCLASSIFIED EXCAVATION

Compute and use pay items for Item 503 as follows:

When an excavation includes 10 yd³ or more of rock (or shale), itemize the quantity of rock excavation separately under:

ITEM 503 - ROCK (OR SHALE) EXCAVATION

When the rock (or shale) excavation is under 10 yd³, do not itemize the rock (or shale) excavation separately. Provide the following pay item:

ITEM 503 - UNCLASSIFIED EXCAVATION, INCLUDING ROCK (AND/OR SHALE)

When excavation includes no rock (or shale), provide the following pay item:

ITEM 503 - UNCLASSIFIED EXCAVATION

In computing the quantity of Item 503 excavation, the designer should confirm that all removals under items 201, 202 or 203 have been excluded, according to [C&MS](#) 503.01. Generally, the basis of payment for Item 503 should be yd³. Lump sum quantities may be used if authorized by the District and with the understanding that cost may be higher than when specific quantities are used.

605.5 PROPRIETARY RETAINING WALLS

For projects with proprietary retaining wall systems supporting bridge abutments on pile foundations, provide the following note:

[605.5-1] PROPRIETARY RETAINING WALL DATA: The proprietary wall supplier shall design the internal stability of a mechanically stabilized earth (MSE) wall in accordance with SS840 to support the abutment. The design for internal stability shall include a nominal (i.e. unfactored) horizontal strip load due to friction (FR) from the superstructure of _____ k/ft applied perpendicular to the face of wall at the base of the concrete footing. This strip load does not include earth pressure loads from the abutment backfill. However, the proprietary wall supplier shall include earth pressure loads from the abutment backfill in the design calculations.

NOTE TO DESIGNER:

The note above applies to the design of abutments supporting expansion bearings only. Longitudinally applied superstructure loads are assumed to be transferred to the substructure as a friction loads (FR) equal to the nominal frictional resistances supplied by the bearings (see BDM Section 303.1.3). This assumption does not apply to fixed bearings. For fixed bearings, provide revised versions of these notes that list all applicable longitudinally applied superstructure loads transferred to the substructure through the bearing connections.

Provide the following note, with the blanks filled in as appropriate for each individual project, if there are mechanically stabilized earth (MSE) retaining walls or abutments.

[605.5-2] FOUNDATION BEARING RESISTANCE: The (1) reinforced soil mass, as designed, produces a maximum Service Limit State bearing pressure of (2) kips per square foot and a maximum Strength Limit State bearing pressure of (2) kips per square foot. The factored bearing resistance is (3) kips per square foot.

NOTE TO DESIGNER:

- (1) Specify the location of the MSE wall.
- (2) Specify the maximum factored bearing pressures.
- (3) Specify the factored bearing resistance according to *LRFD 10.6.3* and *11.10.5.4* and BDM Section 305.2.1.

605.6 FOUNDATIONS ON DRILLED SHAFTS IN NEW EMBANKMENTS

The following construction methods should minimize the effect of lateral and downdrag loads acting on substructure units and their drilled shafts.

For structures with abutments on drilled shafts placed in new embankments use the following note:

[605.6-1] SHAFT DRILLING CONSTRAINTS: Prior to drilling shafts, construct the spill through slopes and the bridge approach embankment behind the abutments up to the level of the subgrade elevation for a minimum distance of (1) behind each abutment. Do not begin the excavation for the abutment footings and the drilling of the abutment shafts until after the above required embankment has been constructed and a (2) calendar day waiting period has elapsed. The Engineer may adjust the length of the waiting period based on settlement platform readings.

NOTE TO DESIGNER:

- (1) Generally, 200-ft. Optionally, this distance may be defined by station-to-station dimensions.
- (2) Estimate the length of the waiting period by determining the time required for 90% of primary settlement to occur. If the designer determines that a waiting period is not necessary, this portion of the plan note may be removed.

For structures with abutments and piers on drilled shafts placed in new embankments use the following note:

[605.6-2] SHAFT DRILLING CONSTRAINTS: Prior to drilling shafts, construct the spill through slopes and the bridge approach embankment behind the abutments up to the level of the subgrade elevation for a minimum distance of (1) behind each abutment. Do not begin the excavation for the abutment footings and the drilling of the abutment and pier shafts, for pier(s) (2), until after the above required embankment has been constructed and a (3) calendar day waiting period has elapsed. The Engineer may adjust the length of the waiting period based on settlement platform readings.

NOTE TO DESIGNER:

- (1) Generally, 200-ft. Optionally, this distance may be defined by station-to-station dimensions.
- (2) Identify specific piers.
- (3) Estimate the length of the waiting period by determining the time required for 90% of primary settlement to occur. If the designer determines that a waiting period is not necessary, this portion of the plan note may be removed.

For structures with wall type abutments on drilled shafts placed in new embankment use the following note:

[605.6-3] SHAFT DRILLING CONSTRAINTS: Prior to drilling shafts at the abutments, construct the bridge approach embankment behind the abutments up to a 1:1 slope from the top of the heel of the footing* to the subgrade elevation and for a minimum distance of 250-ft behind the abutments. Do not begin the drilling of the abutment shafts until after the above required embankment has been constructed and a (1) calendar day waiting period has elapsed. The Engineer may adjust the length of the waiting period based on settlement platform readings. After the footing and the breastwall have been constructed, construct the embankment immediately behind the abutments up to the beam seat elevation and on a 1:1 slope up to the subgrade elevation prior to setting the beams on the abutments.

NOTE TO DESIGNER:

- * In some cases, the bottom of the heel may be used.
- (1) Estimate the length of the waiting period by determining the time required for 90% of primary settlement to occur. If the designer determines that a waiting period is not necessary, this portion of the plan note may be removed.

606 FOUNDATIONS

606.1 PILES DRIVEN TO BEDROCK

The following note generally will apply where steel-H piles are to be driven to bedrock:

[606.1-1] PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained when the pile penetration is an inch or less after receiving at least 20 blows from the pile hammer. Select the hammer size to achieve the required depth to bedrock and refusal.

The total factored load is (1) kips per pile for the (2) abutment piles. The total factored load is (1) kips per pile for the (2) pier piles.

Abutment piles:

(3) piles (4) feet long, order length

Pier piles:

(3) piles (4) feet long, order length

Provide plain cylindrical casings with a minimum pile wall thickness of (5) inch for the cast-in-place reinforced concrete piles.

NOTE TO DESIGNER:

- (1) Specify the total factored load according to BDM Section 305.3.5.2.

- (2) Specify the location of piles for each total factored load.
- (3) Specify the size of pile (e.g. HP 10 x 42 or 12-in diameter).
- (4) Specify the order length according to BDM Section 305.3.5.2.
- (5) Specify the minimum pile wall thickness for cast-in-place reinforced concrete piles. Determine the minimum pile wall thickness from a pile drivability analysis. Remove this sentence if all piles are H-piles.

The following note, modified to fit the conditions, will apply where piles are located within a waterway and the scour depth is significant.

[606.1-2] PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained when the pile penetration is an inch or less after receiving at least 20 blows from the pile hammer. Select the hammer size to achieve the required depth to bedrock and refusal.

The total factored load is (1) kips per pile for the (2) abutment piles. The abutment piles were designed to accommodate (3) ft. of scour. The total factored load is (1) kips per pile for the (2) pier piles. The pier piles were designed to accommodate (3) ft. of scour.

Abutment piles:
(4) piles (5) feet long, order length

Pier piles:
(4) piles (5) feet long, order length

Provide plain cylindrical casings with a minimum pile wall thickness of (6) inch for the cast-in-place reinforced concrete piles.

NOTE TO DESIGNER:

- (1) Specify the total factored load according to BDM Section 305.3.5.2.
- (2) Specify the location of piles for each total factored load.
- (3) Specify the depth of anticipated scour.
- (4) Specify the size of pile (e.g. HP 10 x 42 or 12-in diameter).
- (5) Specify the order length according to BDM Section 305.3.5.2.
- (6) Specify the minimum pile wall thickness for cast-in-place reinforced concrete piles. Determine the minimum pile wall thickness from a pile drivability analysis. Remove this sentence if all piles are H-piles.

The following note, modified to fit the conditions, will apply where downdrag loads on the piles are anticipated.

[606.1-3] PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained when the pile penetration is an inch or less after receiving at least 20 blows from the pile hammer. Select the hammer size to achieve the required depth to bedrock and refusal.

The total factored load is (1) kips per pile for the (2) abutment piles. The abutment piles include an additional (3) kips of factored load per pile to account for possible downdrag loading. The total factored load is (1) kips per pile for the (2) pier piles.

Abutment piles:
(4) piles (5) feet long, order length

Pier piles:
(4) piles (5) feet long, order length

Provide plain cylindrical casings with a minimum pile wall thickness of (6) inch for the cast-in-place reinforced concrete piles.

NOTE TO DESIGNER:

- (1) Specify the total factored load according to BDM Section 305.3.5.2.
- (2) Specify the location of piles for each total factored load.
- (3) Specify the anticipated factored downdrag loading.
- (4) Specify the size of pile (e.g. HP 10 x 42 or 12-in diameter).
- (5) Specify the order length according to BDM Section 305.3.5.2.
- (6) Specify the minimum pile wall thickness for cast-in-place reinforced concrete piles. Determine the minimum pile wall thickness from a pile drivability analysis. Remove this sentence if all piles are H-piles.

606.2 FRICTION TYPE PILES

The following notes, modified to fit the specific conditions for the foundation required, will apply in all cases except where the piles are to be driven to bedrock. Provide the actual calculated Ultimate Bearing Value as shown below:

[606.2-1] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is (1) kips per pile for the (2) abutment piles. The Ultimate Bearing Value is (1) kips per pile for the (2) pier piles.

Abutment piles:

(3) piles (4) feet long, order length

(5) Dynamic load testing items

Pier piles:

(3) piles (4) feet long, order length

(5) Dynamic load testing items

Provide plain cylindrical casings with a minimum pile wall thickness of (6) inch for the cast-in-place reinforced concrete piles.

NOTE TO DESIGNER:

- (1) Specify the Ultimate Bearing Value according to BDM Section 305.3.5.2.
- (2) Specify the location of piles for each Ultimate Bearing Value.
- (3) Specify the size of pile (e.g. HP 10x42 or 12-in diameter).
- (4) Specify the order length according to BDM Section 305.3.5.2
- (5) Specify the number of dynamic load testing items according to BDM Section 305.7.1.
- (6) Specify the minimum pile wall thickness for cast-in-place reinforced concrete piles. Determine the minimum pile wall thickness from a pile drivability analysis. Remove this sentence if all piles are H-piles.

The following note, modified to fit the conditions, will apply where piles are located within a waterway and the scour is anticipated.

[606.2-2] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value (UBV) is (1) kips per pile for the (2) abutment piles. The UBV is (1) kips per pile for the pier piles. The UBV for the pier piles includes an additional (3) kips per pile due to the possibility of losing (7) ft. of frictional resistance due to scour. Drive the pier piles to the UBV or a tip elevation of (8), whichever is deeper.

Abutment piles:

(4) piles (5) feet long, order length

(6) Dynamic load testing items

Pier piles:

(4) piles (5) feet long, order length

(6) Dynamic load testing items

Provide plain cylindrical casings with a minimum pile wall thickness of (9) inch for the cast-in-place reinforced concrete piles.

NOTE TO DESIGNER:

- (1) Specify the Ultimate Bearing Value according to BDM Section 305.3.5.2.
- (2) Specify the location of piles for each Ultimate Bearing Value.
- (3) Specify the additional amount of Ultimate Bearing Value according to BDM Section 305.3.5.2.
- (4) Specify the size of pile (e.g. HP 10 x 42 or 12-in diameter).
- (5) Specify the order length according to BDM Section 305.3.5.2.
- (6) Specify the number of dynamic load testing items according to BDM Section 305.7.1.
- (7) Specify the scour depth.
- (8) Specify the tip elevation according to BDM Section 305.3.2.1.
- (9) Specify the minimum pile wall thickness for cast-in-place reinforced concrete piles. Determine the minimum pile wall thickness from a pile drivability analysis. Remove this sentence if all piles are H-piles.

Provide the following note when Static Load Testing is required according to BDM Section 305.7.2. Modify the note as necessary to fit the specific condition.

[606.2-3] **ITEM 506** STATIC LOAD TEST Perform dynamic testing on the first two production piles to determine the required blow count for the specified Ultimate Bearing Value. Perform the static load test on either pile. Do not over-drive the selected pile. Drive the third and fourth production piles to 75% and 85% of the determined blow count, respectively and perform dynamic testing on each. The test piles and the reduced capacity piles shall not be battered. After installation of the first four production piles, cease all driving operations at the substructure for a minimum of * days. After the waiting period, perform the static load test, and then perform pile restrikes on the four piles (two restrike test items). Perform a CAPWAP analysis on each pile tested for every dynamic load test and every restrike test. The Engineer will review the results of the pile restrikes and establish the driving criteria for the remaining piling represented by the testing. Submit all test results to the Office of Geotechnical Engineering.

If the restrike test results indicate that any of the four piles did not achieve the required UBV, drive the pile to the established driving criteria.

For subsequent static load tests, upon completion of a 10,000-ft increment of driven length, repeat the above procedure for the initial static load test. If necessary, the Engineer will revise the driving criteria for the remaining piling accordingly.

NOTE TO DESIGNER:

* The typical wait period is 7 days. If substantial pile setup is expected, consult BDM Table 606-1 for a recommended wait period. Potential impact on the construction schedule may also prove to be the limiting factor.

The following note, modified to fit the specific conditions for the foundation required, will apply when uplift loads control the design of the pile. In this case, the piles are typically driven to a pile tip elevation and dynamic load testing of the pile is not performed.

[606.2-4] PILES DRIVEN TO TIP ELEVATION FOR UPLIFT: Drive the piles to the pile tip elevation shown on the plans. Do not perform dynamic load testing on piles driven to a tip elevation. Select the hammer size to achieve the required depth. Provide plain cylindrical casings with a minimum pile wall thickness of (1) inch for piles driven to a tip elevation.

Abutment piles:

(2) piles (3) feet long, order length

NOTE TO DESIGNER:

- (1) Specify the minimum pile wall thickness for cast-in-place reinforced concrete piles. Determine the minimum pile wall thickness from a pile drivability analysis. Remove this sentence if the piles are H-piles.
- (2) Specify the size of pile (e.g. HP 10 x 42 or 12-in diameter).
- (3) Specify the order length according to BDM Section 305.3.5.2.

606.3 STEEL PILE POINTS OR SHOES

Where steel points or shoes are required, add one of the following sentences to the end of the provided plan note from BDM Section 606.1 or 606.2, as appropriate. See BDM Section 305.3.5.2 for guidance.

Where steel H-piles are specified add the following note:

[606.3-1] Use steel pile points to protect the tips of the proposed steel H-piles at (1). *

Where CIP reinforced concrete pipe piles are specified add the following note:

[606.3-2] Use conical steel pile points to protect the tips of the proposed steel CIP reinforced concrete pipe piles at (1). *

Where steel open-ended pipe piles are specified add the following note:

[606.3-3] Use open-end cutting shoes to protect the tips of the proposed steel open-ended pipe piles at (1). *

When driving open-ended pipe piles to refusal on bedrock, and driving stresses at the bedrock contact are expected to exceed the compressive driving resistance of the pipe pile, provide the following note:

[606.3-4] ITEM 507, PILING, MISC.: (2) DIAMETER PIPE PILES, FURNISHED, AS PER PLAN: The minimum pile wall thickness for the bottom (3) feet of the piles is be (4) inches at (1). Perform welding of this pile section in shop.

NOTE TO DESIGNER:

* Include ITEM 507, STEEL POINTS OR SHOES in the estimated quantities.

- (1) Specify the substructure unit(s) or location(s) of piles to be protected by steel points or shoes.
- (2) Specify the diameter of the open-ended pipe piles in inches.
- (3) Specify the length of the thickened section of the open-ended pipe piles. Provide one pile diameter when the angle between the axis of the pile and the bedrock surface is greater than 60°; otherwise, provide two pile diameters.
- (4) Specify the thickness of the thickened section of the open-ended pipe piles.

606.4 PILE SPLICES

Provide the following note when H-piles are specified.

[606.4-1] PILE SPLICES: In lieu of using the full penetration butt welds specified in [C&MS 507.09](#) to splice steel H-piles, the Contractor may use a manufactured H-pile splicer. Furnish splicers from the following manufacturer:

Associated Pile and Fitting Corporation

8 Wood Hollow Rd. Plaza 1

Parsippany, New Jersey 07054

Install and weld the splicer to the pile sections in accordance with the manufacturer's written assembly procedure supplied to the Engineer before the welding is performed.

606.5 PILE ENCASEMENT

The following note shall be used where capped pile piers and steel "H" piles are being used for a bridge structure crossing a waterway. The exposed steel piling corrodes at the waterline, or near there. The note should not be used if the capped pile pier standard drawing is being used as standard drawing already specifies pile encasement methods.

[606.5-1] ITEM SPECIAL - PILE ENCASEMENT

Encase all steel H-piles for the capped pile piers in concrete conforming to [C&MS 511](#) ($f'_c = 4.0$ -ksi). Provide a concrete slump between 6 to 8 inches with the use of a superplasticizer. Place the concrete within a form that consists of polyethylene pipe ([C&MS 707.33](#)), or PVC pipe ([C&MS 707.42](#)). The encasement shall extend from 3 feet below the finished ground surface up to the concrete pier cap. Position the pipe so that at least 3 inches of concrete cover is provided around the exterior of the pile.

The Department will measure pile encasement by the number of feet. The Department will determine the sum as the length measured along the axis of each pile from the bottom of the encasement to the bottom of the pier cap. The Department will pay for accepted quantities at the contract price for ITEM - SPECIAL, PILE ENCASEMENT.

606.6 SPREAD FOOTING FOUNDATIONS

Provide the following note, with the blanks filled in as appropriate for each individual project, if there are abutments, piers, or retaining walls which are supported by spread footings.

[606.6-1] FOUNDATION BEARING RESISTANCE: (1) footings, as designed, produce a maximum Service Limit State bearing pressure of (2) kips per square foot and a maximum Strength Limit State bearing pressure of (2) kips per square foot. The factored bearing resistance is (3) kips per square foot.

NOTE TO DESIGNER:

- (1) Specify the location of the spread footing.
- (2) Specify the maximum factored bearing pressures.
- (3) Specify the factored bearing resistance according to *LRFD 10.6.3* and BDM Section 305.2.1.

When abutments or piers are supported by spread footings and significant settlement is predicted in accordance with BDM Section 305.2.1.1, include the following note to require that reference monuments be constructed in each footing. The purpose of the reference monuments is to document the performance of the spread footings, both short and long term.

[606.6-2] ITEM 623, MONUMENT ASSEMBLY, AS PER PLAN: Install a reference monument at each end of each spread footing. The reference monument shall consist of a #8, or larger, epoxy coated rebar embedded at least 6" into the footing and extended vertically 4 to 6 inches above the top of the footing. Install a six-inch diameter, schedule 40, plastic pipe around the reference monument. Center the pipe on the reference monument and place the pipe vertical with its top at the finished grade. The pipe shall have a removable, schedule 40, plastic cap. Permanently attach the bottom of the pipe to the top of the footing.

Establish a benchmark to determine the elevations of the reference monuments at various monitoring periods throughout the length of the construction project. The benchmark shall be the same throughout the project and shall be independent of all structures.

Record the elevation of each reference monument at each monitoring period shown in the table below.

Provide the Engineer an electronic document in portable document format (pdf) with the completed tables after the final measurements are recorded. The document will become part of the District's project plan records.

PID Number: *	Maximum Factored Bearing Pressure: *		
Bridge Number: *	Structure File Number: *		
Benchmark Location:			
Footing Location: *			
Monitoring Period	Date of Measurement	Left Monument	Right Monument
Within 24-hr after footing concrete is placed			
Within 24-hr before placement of superstructure members			
Within 24-hr before deck placement			
Within 24-hr after deck placement			
Within 24-hr before application of traffic loading on footing			

NOTE TO DESIGNER:

- * The Designer shall modify items marked with an asterisk to describe the pier and/or abutment footing location, PID number, bridge number, SFN and maximum factored bearing.

When retaining walls are supported by spread footings and significant settlement is predicted in accordance with BDM Section 305.2.1.1, include the following note to require that reference monuments be constructed in each footing. The purpose of the reference monuments is to document the performance of the spread footings, both short and long term.

[606.6-3] ITEM 623, MONUMENT ASSEMBLY, AS PER PLAN: Install reference monuments in the retaining wall footing at the locations shown in the Table below. The reference monument shall consist of a #8, or larger, epoxy coated rebar embedded at least 6" into the footing and extended vertically 4 to 6 inches above the top of the footing. Install a six-inch diameter, schedule 40, plastic pipe around the reference monument. Center the pipe on the reference monument and place the pipe vertical with its top at the finished grade. The pipe shall have a removable, schedule 40, plastic cap. Permanently attach the bottom of the pipe to the top of the footing.

Establish a benchmark to determine the elevations of the reference monuments at various monitoring periods throughout the length of the construction project. The benchmark shall be the same throughout the project and shall be independent of all structures.

Record the elevation of each reference monument at each monitoring period shown in the table below.

Provide the Engineer an electronic document in portable document format (pdf) with the completed tables after the final measurements are recorded. The document will become part of the District's project plan records.

PID Number: *		C-R-S, PID *		
Wall: *				
Benchmark Location:				
Monument Locations:	Date of measurement	Monument #1 *	Monument #2 *	Monument #3 *
Maximum Factored Bearing Pressure:	*	*	*	*
Monitoring Period				
Within 24-hr after footing concrete is placed:				
Within 24-hr after stem concrete is placed:				
Within 24-hr after concrete bridge railing is placed:				
Project completion:				

NOTE TO DESIGNER:

- * The Designer shall modify items marked with an asterisk to describe the project county-route-section (C-R-S) designation and PID, wall number or name, monument locations (baseline, stations, and offsets), and maximum factored bearing pressures. Specify a number of reference monuments appropriate for the length and geometry of each wall. At least one reference monument should be included per each of the foundation sub-units for which bearing pressures and resistances are calculated in the SFE Report, or wherever there is a change in geometry, such as a step in the footing; with one monument per every approximately 50-ft to 100-ft of wall length; including one reference monument at each end of the wall, located approximately 1.0-ft in from the ends of the wall footing. Modify the table to include a column for each reference monument if there are more than four.

Provide the following note if the footing excavation is mainly bedrock and the footings are to be at an elevation no higher than plan elevation:

[606.6-4] FOOTINGS: Place footings in bedrock at the elevation shown.

Provide the following note where footings are to be founded in bedrock at an elevation no higher than plan elevation.

[606.6-5] FOOTINGS shall extend a minimum of 3 inches* into bedrock or to the elevation shown, whichever is lower.

NOTE TO DESIGNER:

- * Shall be greater than 3-in if required by design considerations.

Provide the following note where footings are to be founded in bedrock, and where the encountering of bedrock at an elevation considerably above plan elevation may make it desirable to raise the footing to an elevation not above the specified maximum in order to affect an appreciable saving:

[606.6-6] FOOTINGS shall extend a minimum of 3 inches* into bedrock. If necessary due to poor bedrock material, the footings should be lowered. If the low point of the bedrock surface occurs 2 feet or more above plan elevation, the final footing elevations may be raised, upon approval by the Office of Geotechnical Engineering, but to an elevation not higher than ** feet. Stepping of individual footings will not be permitted unless shown on the plans.

NOTE TO DESIGNER:

- * Shall be greater than 3-in if required by design considerations.
- ** The maximum elevation allowed should assure that minimum soil cover over the footing is obtained; clearance from the superstructure to the finished ground elevation meets standards; quality of bedrock material at that elevation is adequate; and minimum embedment into the bedrock material will not be adversely affected.

606.7 DRIVEN PILES

Provide the following note whenever it is necessary to specify a pile driving hammer minimum or maximum rated energy for driven piling.

[606.7-1] PILE DRIVING: Use a pile driving hammer with a rated energy of (1) (2) foot-pounds to install the piles. Ensure that stresses in the piles during driving do not exceed (3) pounds per square inch.

NOTE TO DESIGNER:

- (1) Specify if the pile driving hammer rated energy a minimum (use “not less than” in the plan note) or a maximum (use “not greater than” in the plan note).
- (2) Specify the hammer rated energy in accordance with BDM Section 305.3.1.2.
- (3) Specify the limiting pile driving stress in accordance with BDM Section 305.3.1.2. If piles of multiple types or steel grades are to be driven, specify the pile groups and locations separately in the note.
(For example: “Use a pile driving hammer with a rated energy of not less than 47,000 foot-pounds to install the piles. Ensure that stresses in the piles during driving do not exceed 40.5 pounds per square inch.”)

Provide the following note whenever vibration monitoring is specified. See BDM Section 305.3.6 for guidance.

[606.7-2] ITEM 530 SPECIAL – STRUCTURAL SURVEY AND MONITORING OF VIBRATION

Monitor ground vibrations caused by pile driving to minimize the potential for (1).

Retain an experienced vibration specialist to establish the acceptable vibration limits and to perform the vibration monitoring. Use a vibration specialist that is an expert in the interpretation of vibration data, and who meets one of the following criteria: 1) is a registered engineer with at least two years of proven experience in monitoring vibrations on similar construction projects, or 2) has at least five years of proven experience in monitoring vibrations on similar construction projects. Do not use a vibration specialist that is an employee of the Contractor.

Submit a resume of the credentials of the proposed vibration specialist at or before the Preconstruction Meeting. Include in the resume a list of construction projects on which the vibration specialist was responsibly in charge of monitoring the vibrations. List a description of the projects, with details of the vibration interpretations made on the project. List the names and telephone numbers of project owners with sufficient knowledge of the projects to verify the submitted information. Obtain the Engineer’s acceptance of the vibration specialist before beginning any pile driving work. Allow 30 days for the review of this documentation.

Use seismographs capable of continuously recording the peak particle velocity for three mutually perpendicular components of vibration, and of providing a permanent record of the entire vibration event. Use a sufficient number of seismographs to provide redundancy in case one device should fail. Submit a plan of the proposed seismograph locations to the Engineer for review.

The vibration specialist shall perform the following:

1. Measure the ambient ground vibrations near existing structures before pile driving begins.
2. Establish vibration limits to minimize potential damage to existing structures and explain why they are being used to the Engineer before driving piles near existing structures.
3. Monitor ground vibrations during pile driving.
4. Immediately inform the Contractor and Engineer if the vibration limits are reached or exceeded.
5. Furnish the data recorded and include the following:
 - A. Identification of seismograph.
 - B. Distance and direction of seismograph from pile driving.
 - C. Start time and duration of pile driving.
 - D. List of piles driven during each monitoring interval.

Immediately suspend all pile driving if the vibration limits are reached or exceeded. Evaluate alternative construction procedures, such as prebored holes, to reduce the vibrations.

Submit three copies of the final report which contains all measurements, interpretations, and recommendations to the Engineer.

The Department will pay for this item at the contract lump sum price for ITEM 530 SPECIAL – STRUCTURAL SURVEY AND MONITORING OF VIBRATION. The Department will pay the final twenty percent after the Engineer receives the final report.

The Department will pay according to [C&MS](#) 109.05 for alternative construction procedures that the Engineer determines are necessary to reduce vibrations.

NOTE TO DESIGNER:

(1) Specify the reason for vibration monitoring, and the structures that are the subject of monitoring:

- Damage to: _____ (e.g., “Damage to the 60-in brick sewer running from northwest to southeast and located approximately 30-ft from the proposed rear abutment”)
- Unacceptable levels of vibration annoyance with respect to human response at: _____ (e.g., “Unacceptable levels of vibration annoyance with respect to human response at Marion General Hospital”)
- Interference with building function of: _____ (e.g., “Interference with building function of Holman Testing Laboratories”)

Whenever a preconstruction condition survey is required, add the following note to the bottom of Plan Note 606.7-2. See BDM Section 305.3.6 for guidance.

Before pile driving begins, conduct a condition survey of all existing buildings, structures, and utilities within 200-ft of the pile driving work. The purpose of the survey is to document the condition of the buildings, structures, or utilities prior to pile driving, so that claims of damage caused by the pile driving can be verified.

Retain an experienced vibration specialist to perform or supervise the condition survey. Use a vibration specialist that meets the qualification requirements for vibration monitoring.

Record the condition of existing structures and building materials, using written text, photographs, and video recordings. Inspect interior walls, ceilings, and floors that are accessible. Inspect the exterior of the building that is visible from ground level. Also record the location, size, and type of all cracks and other structural deficiencies.

If owners or occupants fail to allow access to the property for the preconstruction condition survey, send a certified letter to the owner or occupant. Document the notification effort and the certified letter in the report.

Submit three copies of a report to the Engineer that summarizes the preconstruction condition of the buildings, structures, and utilities, and that identifies areas of concern.

Payment for the preconstruction condition survey is inclusive of the contract lump sum price for ITEM 530 SPECIAL – STRUCTURAL SURVEY AND MONITORING OF VIBRATION.

Provide the following note whenever piles are to be driven to the full estimated length and use setup and a waiting period with restrikes to prove the final ultimate bearing value (UBV).

[606.7-3] PILES DRIVEN TO FULL ESTIMATED LENGTH WITH PILE/SOIL SETUP

The Ultimate Bearing Value (UBV) is (1) kips per pile for the (2) (3) piles. Part of the UBV will be achieved through pile/soil setup, which is a time dependent increase in resistance that occurs in some soils.

Notify the Engineer at least 5 days before driving piles so that the Engineer can notify the District Geotechnical Engineer, the Office of Construction Administration, and the Office of Geotechnical Engineering.

Drive the first two piles at the (3) to the full estimated length of (4) feet. Perform dynamic load testing on both piles while driving. If the dynamic load testing during the initial driving (before the

waiting period) indicates a pile has achieved the required UBV prior to driving the full estimated length, stop driving and notify the Engineer. After driving and testing the first two piles, drive the remaining piles in the substructure to the same depth as the first two piles. After driving all piles to the estimated length, cease all driving operations at the substructure for a period of * days. Include the waiting period as a separate activity in the progress schedule. After the waiting period, perform pile restrikes on both of the first two piles (one restrike item). Submit all test results to the Engineer. If the restrike test results indicate that both piles achieved the required UBV, all piles in the substructure may be accepted by the Engineer.

If the restrike test results indicate that either of the two piles did not achieve the required UBV, immediately notify the Engineer so that the Engineer can notify the District Geotechnical Engineer, the Office of Construction Administration, and the Office of Geotechnical Engineering. The Engineer will review the test results and establish additional restrike testing or driving criteria for the piling in the substructure with the assistance of the District Geotechnical Engineer, the Office of Construction Administration, and the Office of Geotechnical Engineering.

If directed by the Engineer, perform additional restrike testing or drive all piles in the substructure to the established driving criteria. The Department will pay for splicing of the piles beyond the estimated length provided in the plans under [C&MS](#) 109.05 with a negotiated price per splice.

This plan note includes a quantity of one each ITEM 523 DYNAMIC LOAD TESTING, AS PER PLAN and a quantity of one each ITEM 523 RESTRIKE, AS PER PLAN per each substructure unit.

NOTE TO DESIGNER:

- (1) Specify the Ultimate Bearing Value according to BDM Section 305.3.2.
 - (2) Specify the size of pile (e.g. 12-inch diameter cast-in-place reinforced concrete or HP10x42).
 - (3) Specify the location of piles (e.g. rear abutment, rear and forward abutment, or pier 2).
 - (4) Specify the estimated length according to BDM Section 305.3.5.2.
- * Specify the waiting period. This is generally related to the amount of setup expected, the greater the expected percent setup or setup factor, the longer the wait time. Calculate the expected setup factor as $f_{su} = UBV / EOID$ and use the following table as a guide for wait times.

Table 606-1

Setup Factor (f_{su})	Percent Setup (%)	Wait Time (Days)
$1.0 \leq f_{su} \leq 1.1$	0 – 10	1
$1.1 < f_{su} \leq 1.2$	10 – 20	3
$1.2 < f_{su} \leq 1.4$	20 – 40	7
$1.4 < f_{su} \leq 1.6$	40 – 60	14
$1.6 < f_{su} \leq 2.0$	60 – 100	30
$f_{su} > 2.0$	> 100	45

Provide the following note whenever piles are to be driven to variable specified lengths and then use setup and a waiting period with restrikes to determine the length to achieve the final ultimate bearing value (UBV).

[606.7-4] PILES DRIVEN TO TIP ELEVATION FOR PILE/SOIL SETUP

The Ultimate Bearing Value (UBV) is (1) kips per pile for the (2) (3) piles. Part of the UBV will be achieved through pile/soil setup, which is a time dependent increase in resistance that occurs in some soils.

Notify the Engineer at least 5 days before driving piles so that the Engineer can notify the District Geotechnical Engineer, the Office of Construction Administration, and the Office of Geotechnical Engineering.

Drive the first two piles in each substructure to the tip elevation specified for each substructure. Drive the third and fourth piles to 75% and 85% of the length of the first two piles. Perform dynamic load testing on all four piles while driving. After driving the four piles, cease all driving operations at the substructure for a minimum of 10 days. Include the waiting period as a separate activity in the progress schedule. After the waiting period, perform pile restrikes on all four piles (two restrike items). Perform a CAPWAP analysis on each pile tested for every dynamic load test and every restrike test.

Submit all test results to the Engineer. The Engineer will review the test results and establish driving criteria for the piling in the substructure with the assistance of the District Geotechnical Engineer, the Office of Construction Administration, and the Office of Geotechnical Engineering. Perform the driving criteria with pile setup for the first stage of bridge construction. Do not order piles for subsequent phases until after the driving criteria has been established with setup. The Department will adjust the furnished pile quantities based on the restrike test results.

If the dynamic load testing indicates a pile has achieved the required UBV above the tip elevation during the initial driving (before the waiting period), stop driving and notify the Engineer. If the restrike test results indicate that any of the four piles did not achieve the required UBV, drive the pile to the established driving criteria. The Department will pay for splicing of the piles beyond the estimated length provided in the plans under [C&MS](#) 109.05 with a negotiated price per splice.

This plan note includes a quantity of 2 each ITEM 523 DYNAMIC LOAD TESTING, AS PER PLAN and a quantity of 2 each ITEM 523 RESTRIKE, AS PER PLAN per each substructure unit.

NOTE TO DESIGNER:

- (1) Specify the Ultimate Bearing Value according to BDM Section 305.3.2.
- (2) Specify the size of pile (e.g. 12-inch diameter cast-in-place reinforced concrete or HP10x42).
- (3) Specify the location of piles.

Provide the following note whenever piles are to be driven pre-construction with a site-specific evaluation of driving criteria through a test pile program.

[606.7-5] TEST PILE PROGRAM

The Ultimate Bearing Value (UBV) is (1) kips per pile for the (2) (3) piles. Part of the UBV will be achieved through pile/soil setup, which is a time dependent increase in resistance that occurs in some soils.

Notify the Engineer at least 5 days before driving piles so that the Engineer can notify the District Geotechnical Engineer, the Office of Construction Administration, and the Office of Geotechnical Engineering.

At each substructure, drive a test pile, identical to the production piles, and using the same production pile hammer, to a length equal to 125% of the estimated length. Perform continuous dynamic monitoring and testing of the test pile during driving from 50% to 125% of the estimated length. Perform two CAPWAP analyses on the test pile, one at 75% of the estimated length, and one at 125% of the estimated length. After driving the test pile, cease all driving operations at the substructure for 7 days. At the end of 7 days, perform a pile restrike on the test pile, with a CAPWAP analysis. Continue the waiting period to 45 days after the end of initial drive. At the end of the 45 days, perform a second pile restrike on the test pile, with a CAPWAP analysis.

Submit all test results to the Engineer. The Engineer will review the test results with the assistance of the District Geotechnical Engineer, the Office of Construction Administration, and the Office of Geotechnical Engineering. The Engineer will develop a profile of cumulative setup with depth. At each elevation along the pile, setup is defined as the difference between the side resistance calculated in the CAPWAP analyses for the end of initial drive and for each of the restrikes. Tip resistance shall be the most conservative result from the four CAPWAP analyses (for the initial drive at 75% and 125% of the estimated length, for the 7 day restrike, and for the 45 day restrike). From these analyses, the engineer will determine a depth at which the UBV was achieved at the 45 day restrike, and for that depth, will subtract the cumulative setup to establish a target end of initial drive resistance (EOID).

Drive the first two production piles in the substructure to the established target EOID. Perform dynamic load testing on both piles, with a CAPWAP analysis on one of the two piles. (If the test pile was driven within the foundation footprint and can be incorporated into the foundation as a production pile, then only one additional production pile need be dynamically tested with a CAPWAP analysis.) After driving the first two production piles, again cease all driving operations at the substructure for 7 days. At the end of 7 days, perform a pile restrike on the first production pile(s), with a CAPWAP analysis on the same pile that received a CAPWAP analysis at the end of initial drive. Include each of the above waiting periods as separate activities in the progress schedule.

The Engineer will compare the 7 day restrike and CAPWAP analysis results for the production pile and for the test pile and will compare the plots of cumulative setup with depth developed from each 7 day restrike and CAPWAP analysis.

If the production pile 7 day restrike demonstrates the same or greater resistance than the test pile 7 day restrike at the same tip elevation, then use the more conservative of the end of initial drive dynamic testing results – between the test pile and production pile – to establish driving criteria according to [C&MS 507.05](#) and [523.04](#) for the remaining production piles for the established target EOID. Drive all remaining production piles to these driving criteria.

If the production pile 7 day restrike demonstrates lesser resistance than the test pile 7 day restrike at the same tip elevation, then reduce the cumulative setup with depth plot from the test pile 45 day restrike by the same percentage that the production pile fell short of the test pile 7 day restrike cumulative setup. From this reduced plot, the engineer will determine a greater depth at which the UBV was achieved at the 45 day restrike. All production piles will be driven to this depth, including the production pile that demonstrated lesser resistance at the 7 day restrike. The Department will pay for splicing of the piles beyond the estimated length provided in the plans under [C&MS 109.05](#) with a negotiated price per splice.

This plan note includes a quantity of 1 each ITEM 507 STEEL PILES, MISC.: TEST PILE FURNISHED; a quantity of 1 each ITEM 507 STEEL PILES, MISC.: TEST PILE DRIVEN; a quantity of 3 each ITEM 523 DYNAMIC LOAD TESTING, AS PER PLAN; and a quantity of 3 each ITEM 523 RESTRIKE, AS PER PLAN per each substructure unit.

NOTE TO DESIGNER:

- (1) Specify the Ultimate Bearing Value according to BDM Section 305.3.2.
- (2) Specify the size of pile (e.g. 12-inch diameter cast-in-place reinforced concrete or HP10x42).
- (3) Specify the location of piles.

When using piles placed in prebored holes in bedrock, include the following plan notes in the structure General Notes. Modify the plan notes as necessary to fit the type and size of pile used, and the type of abutment.

[606.7-6] ITEM 507 – PREBORED HOLES, AS PER PLAN:

Prebored holes shall extend at least (1)-ft into bedrock at each pile. The diameter of the prebored hole shall be a minimum 2-in larger than the diagonal dimension of the pile. The Contractor is responsible for maintaining an open hole.

The prebored holes shall be clean and free of all deleterious materials prior to backfilling operations. Backfill the void between the pile and the prebored hole with Class QC Misc. concrete up to *the Top of Rock elevation. Above the Top of Rock, backfill the void to the bottom of footing elevation *with granular material conforming to 703.11, Structural Backfill Type 2, except 100 percent of the material shall pass through a 3/4-in sieve. Payment for the prebored holes includes the backfill material.

NOTE TO DESIGNER:

- (1) Specify the minimum extension into bedrock for the prebored holes. Typically, this is 5-ft for bedrock with unconfined compressive strength of 1500-psi or greater or 10-ft for bedrock with unconfined compressive strength less than 1500-psi. However, increase the value as necessary to meet the requirements of pile lateral stability or uplift resistance.
- * Remove the underlined content if the prebored holes are to be backfilled with Class QC Misc. concrete up to the bottom of footing elevation. For MSE wall abutments, revise the plan note to say: “to the bottom of the pile sleeve”.

[606.7-7] ITEM 507 – STEEL PILES (1), FURNISHED, AS PER PLAN

This work consists of furnishing and placing steel piles into prebored holes. Place each pile vertically within the hole so it is not inclined more than one inch between the top and bottom. Support the pile so that it does not move during placement of backfill material. *Do not remove supports from the pier piles until they can fully support the pier caps.

The total factored load is (2)-kips per pile for the (3) abutment piles. The abutment piles include an additional (4)-kips of factored load per pile to account for possible downdrag loading.

The total factored load is (2)-kips per pile for the pier (3) piles. The pier piles were designed to accommodate (5)-ft of scour.

(3) Abutment Piles:

(1) Piles (6)-ft long, Order Length

Pier (3) Piles:

(1) Piles (6)-ft long, Order Length

NOTE TO DESIGNER:

- (1) Specify the size of pile (e.g. HP12x53).
- (2) Specify the total factored load for each substructure according to BDM Section 305.3.2.
- (3) Specify the location of piles for each total factored load.
- (4) Specify the anticipated factored downdrag loading. If there is no anticipated downdrag, remove this sentence. If downdrag is anticipated at other substructures, repeat this sentence with the appropriate modifications.
- (5) Specify the anticipated depth of scour at the substructure. If there is no anticipated scour, remove this sentence. If scour is anticipated at other substructures, repeat this sentence with the appropriate modifications.
- (6) Specify the order length according to BDM Section 305.3.5.2.
- * Include the underlined content if the piles are to support capped-pile piers; otherwise remove this sentence.

606.8 DRILLED SHAFTS

Use the following drilled shaft notes when applicable for the specific project. Revise the note for the project conditions and the different drilled shaft designs, if any, on the project.

[606.8-1] **ROCK-SOCKETED DRILLED SHAFTS:** The maximum factored load to be supported by each drilled shaft is * kips at the abutments and * kips at the piers. This load is resisted by side resistance within a portion of the bedrock socket and by tip resistance. At the abutments, the factored side resistance is * kips, assumed to act along the bottom * feet of the bedrock socket, and the factored tip resistance is * kips. At the piers, the factored side resistance is * kips, assumed to act along the bottom * feet of the bedrock socket, and the factored tip resistance is * kips.

NOTE TO DESIGNER:

- * Complete the loads and dimensions in this note. Abutment and Pier sections of the note should be removed or revised as required.

[606.8-2] **FRICTION DRILLED SHAFTS:** The maximum factored load to be supported by each drilled shaft is * kips at the abutments and * kips at the piers. This load is resisted by frictional side resistance along the length of the drilled shaft and by tip resistance. At the abutments, the factored side resistance is * kips, assumed to act along the bottom * feet of the drilled shaft, and the factored tip resistance is * kips. At the piers, the factored side resistance is * kips, assumed to act along the bottom * feet of the drilled shaft, and the factored tip resistance is * kips.

NOTE TO DESIGNER:

- * Complete the loads and dimensions in this note. Abutment and Pier sections of the note should be removed or revised as required.

[606.8-3] **LATERALLY LOADED DRILLED SHAFTS:** The maximum factored lateral load and bending moment to be supported by each drilled shaft are * kips, and * kip-feet, respectively. These loads produce a maximum factored bending moment of * kip-feet, and a maximum factored shear of * kips, within the drilled shaft.

NOTE TO DESIGNER:

- * Complete the loads and dimensions in this note. If the maximum factored lateral loading of drilled shafts varies between substructure units, specify the drilled shaft groups and locations separately in the note.

The following note, modified to fit the specific conditions for the foundation required, will apply when uplift loads control the design of the drilled shaft. In this case, the drilled shafts are typically installed to a tip elevation.

[606.8-4] **DRILLED SHAFTS INSTALLED TO TIP ELEVATION FOR UPLIFT:** Uplift loading controls the length of the drilled shafts at the *. Install the drilled shafts to the tip elevation shown on the plans.

NOTE TO DESIGNER:

- * Specify the substructure locations where uplift controls.

Provide the following note whenever a demonstration drilled shaft is specified. See BDM Section 305.4.4.6 for guidance.

[606.8-5] **ITEM 524 - DRILLED SHAFTS, MISC.: DEMONSTRATION DRILLED SHAFT**
PART 1: DESCRIPTION

This work consists of all labor, materials, equipment and incidentals to construct a demonstration drilled shaft for testing and evaluation to verify the proposed construction methods for the production drilled shafts.

Complete the installation of the demonstration drilled shaft within (1) days of contract award date. The Department will consider the demonstration drilled shaft installation complete after receiving written acceptance from the Engineer.

If the vertical deflection of the demonstration drilled shaft during load testing is less than 5% of the drilled shaft nominal diameter, it may be re-used as a production drilled shaft.

PART 2: MATERIALS

The demonstration drilled shaft shall use the same concrete mix design and steel reinforcement as the production drilled shafts.

PART 3: EXECUTION

Submit a drilled shaft installation plan to the Engineer for acceptance in accordance with the requirements of [C&MS 524.03](#). Construct at least one demonstration drilled shaft in the area shown in the plans and in accordance with the accepted written installation. If there is no area shown in the plans for installation, propose an installation location to the Engineer. Upon construction of the demonstration drilled shaft, and receipt of testing and evaluation results confirming the demonstration drilled shaft has been installed in accordance with contract documents, the Engineer will issue a letter accepting the installation plan for the construction of the subsequent production drilled shafts.

If modification(s) to the installation plan are made, whether due to the testing and evaluation results or for other reason, the Department will require construction of an additional demonstration shaft constructed in accordance with the modified installation plan, at no additional cost.

The diameter, length, reinforcing, installations methods, and other miscellaneous details of the demonstration shaft shall be the same as the production drilled shafts.

Submit the location of the demonstration shaft to the Engineer for acceptance. Locate the demonstration drilled shaft such that no interference occurs with the foundations of existing or proposed structures, the proposed maintenance of traffic, or existing or proposed utilities.

Test the demonstration drilled shaft by Thermal Integrity Profiling (TIP) according to ASTM D7949, Method B; by Crosshole Sonic Logging (CSL) according to ASTM D6760; and by high-strain dynamic testing according to ASTM D4945.

PART 4: MEASUREMENT AND PAYMENT

The Department will measure demonstration drilled shaft by the number of feet, measured along the axis of the drilled shaft from the required bottom elevation of the shaft to the proposed top plan elevation.

In addition to the provisions of [C&MS 524.17](#), the Department will pay for accepted quantities of demonstration drilled shaft after installation of the demonstration shaft and after being provided with written testing and evaluation results acceptable to the Engineer.

The contract price is full compensation for furnishing and installing drilled shafts in accordance with the above requirements, including mobilization, site access, and final removal of the shaft to 36 inches below final grade.

The Department will pay for testing and evaluation of the accepted demonstration shaft separately. The Department will not pay for testing and evaluation of additional demonstration drilled shafts.

The Department will pay for accepted quantities at the contract price as follows:

ITEM 524 – DRILLER SHAFTS, MISC.: DEMONSTRATION DRILLED SHAFT

NOTE TO DESIGNER:

- (1) This is generally a period of from 50 to 200 days.

Provide the following note whenever drilled shaft integrity testing by Thermal Integrity Profiling (TIP) is specified. See BDM Section 305.4.5 for guidance.

[606.8-6] ITEM 894 - THERMAL INTEGRITY PROFILER (T.I.P.) TEST

Perform integrity testing on (1) of the drilled shafts at the (2) by Thermal Integrity Profiling (TIP). Perform TIP testing per ASTM D7949, “Standard Test Methods for Thermal Integrity Profiling of Concrete Deep Foundations,” Method B, and per Supplemental Specification 894.

NOTE TO DESIGNER:

- (1) Specify the number of drilled shafts to be tested (e.g. all, 10%, one, 50%, etc.).
- (2) Specify the substructure locations where testing is to be performed.

Provide the following note whenever drilled shaft integrity testing by Crosshole Sonic Logging (CSL) is specified. See BDM Section 305.4.5 for guidance.

[606.8-7] ITEM 524 - DRILLED SHAFTS, MISC.: CSL TESTING, (1)" DIA. SHAFT

Perform integrity testing on (2) of the drilled shafts at the (3) by Crosshole Sonic Logging (CSL). Perform CSL testing per ASTM D6760, “Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing,” and per the project special provisions.

NOTE TO DESIGNER:

- (1) Specify the diameter of drilled shafts to be tested. Because of the difference in labor for different diameter drilled shafts, a separate pay item is typically specified for each diameter of drilled shaft to be tested.
- (2) Specify the number of drilled shafts to be tested (e.g. all, 10%, one, 50%, etc.).
- (3) Specify the substructure locations where testing is to be performed.

Provide the following note whenever field verification of nominal axial resistance of drilled shafts by high-strain dynamic testing is specified in lieu of static load testing. See BDM Sections 305.7.2 and 305.7.3 for guidance.

[606.8-8] ITEM 524 - DRILLED SHAFTS, MISC.: HIGH-STRAIN DYNAMIC TESTING OF DRILLED SHAFTS

Perform field verification of nominal axial resistance on (1) of the drilled shafts at the (2) by high-strain dynamic testing. Perform high strain dynamic testing per ASTM D4945, “Standard Test Method for High-Strain Dynamic Testing of Deep Foundations,” and per the project special provisions.

NOTE TO DESIGNER:

- (1) Specify the number of drilled shafts to be tested (e.g. all, 10%, one, 50%, etc.).
- (2) Specify the substructure locations where testing is to be performed.

607 MAINTENANCE OF TRAFFIC

Notes concerning maintenance of traffic often are required for bridge work, especially in phased construction projects. The designer is responsible for any bridge maintenance of traffic notes being coordinated with the project's overall maintenance of traffic plans. Any phased construction lane widths, temporary or construction vertical and horizontal clearances, or construction access requirements must match requirements in project's maintenance of traffic plans.

608 RAILROAD GRADE SEPARATION PROJECTS**608.1 CONSTRUCTION CLEARANCE**

Obtain the actual dimensions used in the text of this note from the "Agreement" (a legal document signed by the Director and Railroad). To help limit project construction problems, validate those dimensions with the district railroad coordinator before the note is considered complete. Revise the note to define the agreed upon restraints, including items such as short term clearances, if different than the construction clearances; maximum period of time for restricted clearances; or other project specific controls.

[608.1-1] CONSTRUCTION CLEARANCE: Maintain a construction clearance of * feet horizontally from the center of tracks and * feet vertically from a point level with the top of the higher rail, and * feet from the center of tracks, at all times.

NOTE TO DESIGNER:

* The Designer shall fill in the dimensions.

608.2 RAILROAD AERIAL LINES

Modify the note below to match the specific requirements of the "Agreement" (a legal document signed by the Director and Railroad). Contact the District railroad coordinator to confirm whether the railroad will move, maintain, or re-construction their lines or other cable systems attached to the bridge or whether the note must specify this scope of work as part of the project.

[608.2-1] RAILROAD AERIAL LINES will be relocated by the Railroad. Use all precautions necessary to see that the lines are not disturbed during the construction stage and cooperate with the Railroad in the relocation of these lines. The cost of the relocation will be included in the railroad force account work.

609 UTILITY LINES

The District Utility Coordinator shall coordinate utilities. The District utilities coordinator shall be contacted for required notes. If existing utilities contain asbestos or other hazardous materials, plan notes will be required to identify the location of the utility and identify the material. The Designer shall assure any plan notes added are coordinated with the bid item descriptions to assure the Contractor properly bids the item.

610 REHABILITATION OF EXISTING STRUCTURES**610.1 EXISTING STRUCTURE VERIFICATION**

Provide the following note on plans for altering, widening or repairing existing structures in order to qualify the plans and to ensure that the Contractor's obligation is clearly stated. The inclusion of this note in the plans does not relieve the designer of the requirements stated in BDM Sections 401 and 403:

[610.1-1] EXISTING STRUCTURE VERIFICATION: Details and dimensions shown on these plans pertaining to the existing structure have been obtained from plans of the existing structure and from field observations and measurements. Consequently, they are indicative of the existing structure and the proposed work but they shall be considered tentative and approximate. The Contractor is referred to [C&MS](#) Sections 102.05, 105.02 and 513.04*. Base contract bid prices upon a recognition of the uncertainties described above and upon a prebid examination of the existing structure. However, the Department will pay for all project work based upon actual details and dimensions that have been verified in the field.

* Delete the reference to 513.04 if structural steel is not involved.

610.2 CONCRETE REINFORCEMENT REPLACEMENT

Place the following note in the plans where the preserved existing concrete reinforcement which projects from the existing structure after partial removal is to be lapped with new reinforcement.

[610.2-1] ITEM 509 – CONCRETE REINFORCEMENT, REPLACEMENT OF EXISTING CONCRETE REINFORCEMENT, AS PER PLAN: Replace all existing reinforcing bars deemed by the Engineer to be unusable because of corrosion. The Department will measure the replacement concrete reinforcement by the number of pounds accepted in place. Replace all existing steel reinforcement bars which are to be incorporated into the new work and are deemed by the Engineer to be made unusable by concrete removal operations with new concrete reinforcement of the same size, coating, and material at no cost to the Department.

NOTE TO DESIGNER: Include a bid item as defined above with a specific weight of steel reinforcement.

On rehabilitation plans where new steel reinforcement may require field bending and cutting, use the following note. Clearly designate in the plans the bar marks to which this note applies.

[610.2-2] ITEM 509 - EPOXY COATED STEEL REINFORCEMENT, AS PER PLAN: In addition to the provisions of Item 509, field bend and/or field cut the steel reinforcement designated in the plans, as necessary, in order to maintain the required clearances and bar spacings. Repair all damage to the epoxy coating, as a result of this work, according to [C&MS](#) 709.00.

NOTE TO DESIGNER: For uncoated steel reinforcement applications, use ITEM 509 – UNCOATED STEEL REINFORCEMENT, AS PER PLAN and remove the coating repair requirement.

610.3 REHABILITATION – STRUCTURAL STEEL

Use the following note on bridge rehabilitation projects where repair or replacement of members not designed to carry tension live loads (i.e. cross-frames, bearing plates, etc.) consist of materials readily available from a structural warehouse (i.e. angles, channels, bars, etc.) and must be field fabricated to dimensions obtained in the field after contract letting. The recommended bid item quantity for rehabilitation work is in pounds rather than Lump Sum. The Designer should adequately define all steel members to be included in this pay item.

[610.3-1] ITEM 513 - STRUCTURAL STEEL MEMBERS, MISC., AS PER PLAN: Perform work for field fabricated members according to Item 513, except as modified herein. The Department will not require the contractor performing field fabrication to be pre-qualified as specified in S1078. Submit a written letter of material acceptance in accordance with [C&MS](#) 501.06, to the Engineer. Provide the Engineer “as-built” drawings according to [C&MS](#) 513.06, except [C&MS](#) 501.04 does not apply. The following members are included in this item: _____, _____ and _____.

610.4 REFURBISHED BEARINGS

When the following note is used, a separate plan note and pay quantity for jacking or temporary support of the superstructure is required. Revise this note, as appropriate, to describe the work for the type of bearing being refurbished.

[610.4-1] ITEM 516 - REFURBISHING BEARING DEVICES, AS PER PLAN: This item shall include all work necessary to properly align bridge bearings as well as their cleaning and painting. Included shall be the disassembly of the bearings, hand tool cleaning (grinding if necessary), painting according to Item 514, replacement of any damaged sheet lead with preformed bearing pads ([C&MS 711.21](#)), installation of any necessary steel shims of the same size as the bearings to provide a snug fit, realignment of the upper bearing plate by removing existing welds and rewelding so that the bearings are vertically aligned at 60° F, lubricating sliding surfaces, and reassembly of the bearings. Assure all bearings are shimmed adequately and that no beams and/or bearing devices are “floating”. At no additional cost to the State, the Contractor may install new bearings of the same type as the existing in place of refurbishing the bearings. All work shall be to the satisfaction of the Engineer. Payment for all of the above described labor and materials will be made at the contract price bid for ITEM 516 - REFURBISH BEARING DEVICES, AS PER PLAN.

610.5 JACKING BRIDGE SUPERSTRUCTURES

Use the following note, modified as necessary, where jacking and/or temporary support of the existing superstructure is required. Modifications to this note are often not being performed by the designers. Use of this note without a review of the project may add un-necessary requirements to the jacking process or, in reverse, not be restrictive enough. Designers are again cautioned to appropriately review this note before incorporation into a set of plans.

[610.5-1] ITEM 516, JACKING AND TEMPORARY SUPPORT OF SUPERSTRUCTURE, AS PER PLAN: This work consists of raising or re-positioning existing structures to the dimensions and requirements defined in the project plans. Submit construction plans in accordance with [C&MS 501.05](#). If, during the jacking operations, cracking of the concrete superstructure, separation of the concrete deck from the steel stringers, or other damage to the structure is visually observed, immediately cease the jacking operation and install supports to the satisfaction of the Engineer. Analyze the damage and submit a method of correction to the Engineer for approval. Epoxy inject all beams that separate from the deck for the distance of the separation in accordance with [C&MS 512.07](#). The Department will not pay for the cost of this epoxy injection or other required repairs. The bridge bearings shall be fully seated at all contact areas. If full seating is not attained, submit a repair plan to the Engineer. The Department will not pay for the repair costs to ensure full seating on bearings. The Department will measure this work on a lump sum basis. The Department will pay for the accepted quantities at the contract price for ITEM 516, JACKING AND TEMPORARY SUPPORT OF SUPERSTRUCTURE, AS PER PLAN.

610.6 FATIGUE MEMBER INSPECTION

When re-decking a continuous beam bridge containing top flange fillet-welded cover plates and/or field butt-welded beams, provide the following note to facilitate the Engineer's inspection of the welded connections.

- [610.6-1]** INSPECTION OF EXISTING STRUCTURAL STEEL: The Engineer will visually inspect all existing butt-welded splices and/or top flange cover plate fillet welds to ensure the welds, plates and beams or girders are free of defects and cracks. If necessary, remove all deck slab haunch forms immediately adjacent to such welds that may interfere with the Engineer's inspection. The inspection will not take place until the top flanges are cleaned according to [C&MS 511.07](#), but it will be done before the deck slab reinforcement is installed. The Department will pay for the cost associated with this inspection with ITEM 511 - SUPERSTRUCTURE CONCRETE. The Engineer will report all cracks found to the Office of Construction Administration, Bridge Construction Specialist, along with specific information on location of the cracks, length, and depth so an evaluation and repair or replacement recommendation can be made.

610.7 SEMI-INTEGRAL CONVERSION

Use the following note for surface preparation and coating of existing beam ends inside the semi-integral diaphragm.

- [610.7-1]** ITEM 514 – FIELD PAINTING, MISC.: COATING OF BEAM ENDS: Prior to encasing the beam ends, prepare the ends per SSPC SP10 or SSPC SP11 to bare metal achieving a 1.5 to 3.5 mil profile. Paint the beam ends with organic zinc prime coat per [C&MS 514](#). Provide the prime coat thickness as per [C&MS 514.20](#). Extend the limits of the beam preparation and painting 1-ft beyond the limits of the end diaphragm concrete.

After the diaphragm concrete is set, seal the interface between the beam and concrete with caulk.

The Department will pay for all above labor and at the contract bid price for ITEM 514 – FIELD PAINTING, MISC: COATING OF BEAM ENDS.

611 MISCELLANEOUS GENERAL NOTES

611.1 BEARING PAD SHIMS, PRESTRESSED

Add the following note to ensure proper seating of prestressed concrete box beams for skewed bridges.

- [611.1-1]** BEARING PAD SHIMS: Place 1/8" thick preformed bearing pad shims, plan area ____ inches by ____ inches, under the elastomeric bearing pads where required for proper bearing. Furnish two shims per beam. The Department will measure this item by the total number supplied. The Department will pay for accepted quantities at the contract price for ITEM 516 - 1/8" PREFORMED BEARING PADS. Any unused shims will become the property of the State.

NOTE TO DESIGNER: The plan area of the shim pad shall be the same as the elastomeric bearing.

611.2 CLEANING STEEL IN PATCHES

Use this note with all concrete patching bid items that refer to the cleaning requirements specified in 519.04

- [611.2-1]** ITEM 519 - PATCHING CONCRETE STRUCTURES, AS PER PLAN: Prior to the surface cleaning specified in [C&MS 519.04](#) and within 24 hours of placing patching material, blast clean all surfaces to be patched including the exposed steel reinforcement. Acceptable methods include high-pressure water blasting with or without abrasives in the water, abrasive blasting with containment, or vacuum abrasive blasting.

611.3 COFFERDAMS AND EXCAVATION BRACING

Use this note when the plans include detail designs for temporary shoring.

[611.3-1] ITEM 503 - COFFERDAMS AND EXCAVATION BRACING, AS PER PLAN:

The design shown on the plans for temporary support of excavation is one representative design that may be used to construct the project. The Contractor may construct the design shown on the plans or prepare an alternate design to support the sides of excavations. If constructing an alternate design for temporary support of excavation, prepare and provide plans in accordance with [C&MS 501.05](#). The Department will pay for the temporary support of excavation at the contract lump sum price for Cofferdams and Excavation Bracing. The Department will not make additional payment for providing an alternate design.

611.4 DECK PLACEMENT NOTES**611.4.1 FALSEWORK AND FORMS**

Use the following note when web depths greater than 84 in. are specified.

[611.4.1-1] ITEM 511 - CLASS QC2 CONCRETE, SUPERSTRUCTURE, AS PER PLAN *

Locate the lower contact point of the overhang falsework at least ** inches $\pm$ 2 in. above the top of the girder's bottom flange. The bracket contact point location requirements of [C&MS 508](#) do not apply.

NOTE TO DESIGNER:

- * Modify the pay item description to fit the specific project requirements.
- ** The minimum dimension for the location for the lower point of contact should be 76 in. below the bottom of the top flange. Designers should verify the acceptability of the design within the range of tolerance specified.

611.4.2 DECK PLACEMENT DESIGN ASSUMPTIONS

Use the following note on projects requiring mechanized finishing machines to place deck concrete except as noted. This note is not required for composite deck placement on adjacent box beam superstructures.

[611.4.2-1] DECK PLACEMENT DESIGN ASSUMPTIONS:

The following assumptions of construction means and methods were made for the analysis and design of the superstructure. The Contractor is responsible for the design of the falsework support system within these parameters and will assume responsibility for superstructure analysis for deviation from these design assumptions.

An eight wheel finishing machine with a maximum wheel load of kips.

A minimum out-to-out wheel spacing at each end of the machine of 103".

A maximum spacing of overhang falsework brackets of 48 in.

A maximum distance from the centerline of the fascia girder to the face of the safety handrail of 65".

NOTE TO DESIGNER: Refer to BDM Section 309.3.8.1 for design information regarding finishing machine loads.

611.5 VANDAL PROTECTION FENCING

For bridges where non-standard vandal protection fencing is provided in accordance with BDM Section 309.5 and the bridge is constructed in phases, provide the following plan note:

[0-1] VANDAL PROTECTION FENCING

Install fencing for each construction phase prior to opening that phase to vehicular and/or pedestrian traffic.

611.6 PRECAST WALLS

Until precast alternative wall designs are included on the Office of Geotechnical Engineering's Approved List, this section has been retired.

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SECTION 700 – TYPICAL DETAIL NOTES

701 SUBSTRUCTURE DETAIL

701.1 STEEL SHEET PILING

Place the following note on the substructure or retaining wall sheet with the details of steel sheet piling that is to be left in place.

[701.1-1] STEEL SHEET PILING left in place shall have a minimum section modulus of _____ in³ per foot of wall.

701.2 BRIDGE SEAT REINFORCEMENT

For structures that contain bearing anchors, place one of the two following notes on an appropriate abutment or pier detail sheet near the "Bearing Anchor Plan". Where the Contractor is allowed the option of presetting bearing anchors (or formed holes), or of drilling bearing anchor holes, provide the first note. Where drilling of anchors into the bridge seat is required, provide the second note. (Formed holes are not practical for prestressed concrete box beam bridges.)

[701.2-1] BRIDGE SEAT REINFORCEMENT, SETTING ANCHORS: Accurately place concrete reinforcement in the vicinity of the bridge seat to avoid interference with the drilling of bearing anchor holes or the pre-setting of bearing anchors.

[701.2-2] BRIDGE SEAT REINFORCEMENT, SETTING ANCHORS: Accurately place concrete reinforcement in the vicinity of the bridge seat to avoid interference with the drilling of anchor bar holes.

701.3 BRIDGE SEAT ELEVATIONS FOR ELASTOMERIC BEARINGS

Where bridge seats have been adjusted to compensate for the vertical deformation of elastomeric bearings, place the following note with the necessary modifications on the appropriate substructure detail sheet.

[701.3-1] BRIDGE SEAT ELEVATIONS have been adjusted upward _____ inches at abutments and _____ inches at piers to compensate for the vertical deformation of the bearings.

701.4 PROPER SEATING OF STEEL BEAMS AT ABUTMENTS

For a steel beam bridge with concrete backwalls and sealed deck joints employing superstructure support or armor steel of considerable stiffness where there is a possibility of individual beams being lifted off of their bearings in a clamping operation, a note similar to the following shall be provided:

[701.4-1] INSTALLATION OF SEAL: During installation of the support/armor for the superstructure side of the expansion joint seal, observe the seating of beams on bearings to assure that positive bearing is maintained.

701.5 BACKWALL CONCRETE PLACEMENT FOR PRESTRESSED BOX BEAMS

For prestressed concrete box beam bridges where the placement of the wingwall concrete above the bridge seat needs to occur after the beams have been erected to allow for the tolerances of the beam fit-up and for beam erection clearances, provide the following note:

[701.5-1] ABUTMENT CONCRETE: Do not place the abutment concrete above the bridge seat construction joint until the prestressed concrete box beams have been erected.

702 SUPERSTRUCTURE DETAILS**702.1 STEEL BEAM DEFLECTION AND CAMBER**

For steel beam or built-up girder bridges provide a table similar to BDM Table 702-1 on a structural steel detail sheet. Tabulation is required regardless of the amount of deflection and is required for all beams or girders, if the deflection is different.

Show the deflection and camber data as described in BDM Section 308.2.2.1.f. The table shall include bearing points, midspan points, splice points and additional points if necessary, at maximum 25-ft increments. Unique geometry may require even closer spacing.

Table 702-1

STRUCTURAL STEEL DEFLECTION & CAMBER TABLE								
DEFLECTION AND CAMBER								
	SPAN NUMBER (1)							
Point (2)	Bearing Pt	1/4 Pt	Pt (3)	Mid Span	Pt (3)	1/4 Pt	Splice Pt	Bearing Pt
Deflection due to weight of steel								
Deflection due to remaining deadload (4)								
Adjustment required for vertical curve								
Adjustment required for horizontal curve								
Required Shop Camber								

- (1) Table is only an example. For multiple span structures include each span and the bearing points in a single table.
- (2) Define distance and position
- (3) Additional points required in a span if the distance between bearing points, 1/4 points, mid span and/or splice points exceeds 25-ft.
- (4) Do not include a separate wearing surface that is not installed during the project

702.2 STEEL NOTCH TOUGHNESS REQUIREMENT (CHARPY V-NOTCH)

CVN material is a requirement to help assure fracture toughness of primary material. Designers using this note should understand not only why CVN is specified but what is a primary member. BDM Section 308.2.2.1.h helps with the definition of primary members and specially highlights that cross-frames of curved steel structures are also primary members, because they transmit forces necessary to provide equilibrium. Designers are reminded they must indicate specific pieces, members, shapes, etc. that are primary members.

Place the following note on a structural steel detail sheet for bridges having main load-carrying members that must meet minimum notch toughness requirements:

[702.2-1] CVN: Where a shape or plate is designated (CVN), furnish material that meets the minimum notch toughness requirements as specified in [C&MS](#) 711.01.

702.3 HIGH STRENGTH BOLTS

For all structural steel superstructures, place the following note on the structural detail sheet:

[702.3-1] HIGH STRENGTH BOLTS shall be _____ diameter A325 unless otherwise noted.

702.4 CONCRETE PLACEMENT SEQUENCE NOTES

Also see BDM Section 701.4 notes.

702.4.1 CONCRETE INTERMEDIATE DIAPHRAGM FOR PRESTRESSED CONCRETE I-BEAMS

If the design plans do not reference Standard Bridge Drawing [PSID-1-13](#), provide the following note.

[702.4.1-1] INTERMEDIATE DIAPHRAGMS: Do not place the deck concrete until all intermediate diaphragms have been properly installed. If concrete diaphragms are used, complete the installation of the intermediate diaphragms at least 48 hours before deck placement begins. Concrete shall conform to [C&MS](#) 511 with a design strength of 4.5 ksi.

702.4.2 SEMI-INTEGRAL OR INTEGRAL ABUTMENT CONCRETE PLACEMENT FOR DIAPHRAGMS

Hardened concrete end diaphragms restrain the movement and rotation of beam/girder ends that occur during deck placement. This restraint will increase stress in both the beam/girder and diaphragm. Factors that can contribute to detrimental stress increases include large structure skew and phased construction. When these factors exist, hardened diaphragms shall be avoided during the deck placement. BDM Table 702-2 provides requirements for concrete diaphragm placement.

Table 702-2

Description of Superstructure	Note
No phased construction, and Steel superstructures with skew < 30°, or Prestressed I-beam superstructure with skew < 10°	702.4.2-1
No phased construction, and Steel superstructures with skew ≥ 30°, or Prestressed I-beam superstructure with skew ≥ 10°	702.4.2-2
Phased construction with closure pour, and Steel superstructures with skew < 30°, or Prestressed I-beam superstructure with skew < 10°	702.4.2-3
Phased construction with closure pour, and Steel superstructures with skew ≥ 30°, or Prestressed I-beam superstructure with skew ≥ 10°	702.4.2-4
Phased construction without closure pour, and Steel superstructures with skew < 30°, or Prestressed I-beam superstructure with skew < 10°	702.4.2-5
Phased construction without closure pour, and Steel superstructures with skew ≥ 30°, or Prestressed I-beam superstructure with skew ≥ 10°	702.4.2-6

Designers shall consider the absence of restraint at the diaphragm location when calculating the unbraced length of beam/girder flanges. If temporary bracing is required, provide appropriate details in the plans. Temporary end bracing should be oriented perpendicular to beam/girder webs.

Use the following notes as prescribed in BDM Table 702-2:

- [702.4.2-1]** ABUTMENT DIAPHRAGM CONCRETE: Place the diaphragm concrete encasing the structural member ends with the deck concrete or at least 48 hours before placement of the deck concrete. If placed separately, locate a horizontal construction joint in the diaphragm as shown on PSID-1-13, sheet 7 of 10 for prestressed I-beam superstructures or as shown on SICD-1-21 for steel superstructures and place remaining diaphragm concrete with the deck.
- [702.4.2-2]** ABUTMENT DIAPHRAGM CONCRETE: Place the diaphragm concrete encasing the structural member ends after the deck placement in the adjacent span is complete. Procedures that place the abutment diaphragm with the deck concrete may be approved by the Engineer if the placement submittal can assure that the deck concrete in the adjacent span will be placed before concrete in the diaphragm has reached its initial set.

NOTE TO DESIGNER: (Applies only to [702.4.2-2])

Locate the deck construction joint parallel to the centerline of the abutment offset 1-ft from the face of the diaphragm toward the span.

- [702.4.2-3]** ABUTMENT DIAPHRAGM CONCRETE, PHASED CONSTRUCTION: Place the diaphragm concrete encasing the structural member ends of an individual phase with the deck concrete or at least 48 hours before placement of the deck concrete. If placed separately, locate a horizontal construction joint in the diaphragm as shown on PSID-1-13, sheet 7 of 10 for prestressed I-beam superstructures or as shown on SICD-1-21 for steel superstructures and place remaining diaphragm concrete with the deck. Place closure pour concrete in the diaphragm and deck concurrently.

NOTE TO DESIGNER: (Applies only to [702.4.2-3])

If a closure pour is required in the deck per BDM Section 309.3.8.5, provide a closure pour in the diaphragm as well. Locate the closure pour in the diaphragm, as near as practical, to mid-bay and orient the vertical construction joints parallel to the centerline of the adjacent beam/girders. Provide 3-in chamfers at the acute corners.

[702.4.2-4] ABUTMENT DIAPHRAGM CONCRETE, PHASED CONSTRUCTION: Place the diaphragm concrete encasing the structural member ends of an individual phase after the deck placement in the adjacent span is complete. Procedures that place the abutment diaphragm with the deck concrete may be approved by the Engineer if the placement submittal can assure that the deck concrete in the adjacent span will be placed before concrete in the diaphragm has reached its initial set. Place closure pour concrete in the diaphragm and deck concurrently.

NOTE TO DESIGNER: (Applies only to [702.4.2-4])

If a closure pour is required in the deck per BDM Section 309.3.8.5, provide a closure pour in the diaphragm as well. Locate the closure pour in the diaphragm, as near as practical, to mid-bay and orient the vertical construction joints parallel to the centerline of the adjacent beam/girders. Locate the deck construction joint parallel to the centerline of the abutment offset 1-ft from the face of the diaphragm toward the span. Provide 3-in chamfers at the acute corners.

[702.4.2-5] ABUTMENT DIAPHRAGM CONCRETE, PHASED CONSTRUCTION: Place the diaphragm concrete encasing the structural member ends of an individual phase with the deck concrete.

NOTE TO DESIGNER: (Applies only to [702.4.2-5])

Do not use this note where a deck closure pour is required per BDM Section 309.3.8.5.

[702.4.2-6] ABUTMENT DIAPHRAGM CONCRETE, PHASED CONSTRUCTION: Place the diaphragm concrete encasing the structural member ends of an individual phase after the deck placement in the adjacent span is complete. Procedures that place the abutment diaphragm with the deck concrete may be approved by the Engineer if the placement submittal can assure that the deck concrete in the adjacent span will be placed before concrete in the diaphragm has reached its initial set.

NOTE TO DESIGNER: (Applies only to [702.4.2-6])

Do not use this note where a deck closure pour is required per BDM Section 309.3.8.5. Locate the deck construction joint parallel to the centerline of the abutment offset 1-ft from the face of the diaphragm toward the span.

702.5 CONCRETE DECK SLAB DEPTH AND PAY QUANTITIES

For all steel beam and girder bridges with a concrete deck, provide the following note that describes how the quantity of deck concrete was calculated.

[702.5-1] DECK SLAB CONCRETE QUANTITY: The estimated quantity of deck slab concrete is based on the constant deck slab thickness, as shown, plus the quantity of concrete that forms each beam/girder haunch. The estimate assumes a constant haunch thickness of inches and a haunch width equal to the top flange width. Deviate from this haunch thickness as necessary to place the deck surface at the finished grade.

The haunch thickness was measured at the centerline of the beam/girder, from the surface of the deck to the bottom of the top flange minus the deck slab thickness. The area of all embedded steel plates has been deducted from the haunch quantity in accordance with 511.23.

NOTE TO DESIGNER: The note above applies to both rolled beams and plate girders, and measures the haunch to the top of the web (bottom of top flange). The note above applies to new structures with beams/girders placed parallel to the profile grade line. A constant depth haunch may not be practical for new structures whose beams/girders are not placed parallel to the profile grade line. In these special cases, the note shall be modified to fit the exact conditions.

702.6 PRESTRESSED CONCRETE I-BEAM BRIDGES

For prestressed concrete I-beam bridges with concrete deck, compute the concrete topping depth over the top of the beams according to BDM Section 308.2.3.4.b. Provide a longitudinal cross section showing the topping thickness at each centerline of bearing and at each midspan.

In addition to the deck elevation tables required by BDM Section 702.12, provide the following notes in the plans for every beam:

[702.6-1] CAMBER:

Estimated camber at Day 0 (D_0) is _____ inches.

Estimated camber at Day 30 (D_{30}) is _____ inches.

Deflection due to remaining dead load (e.g. concrete deck, cross-frames, diaphragms, bridge railing, utilities, etc.) is _____ inches.

Provide the following note in the plans for each substructure unit:

[702.6-2] SEAT ELEVATIONS:

The beam seat elevations were calculated using the assumed camber D_{30} with a minimum sacrificial haunch thickness of 2 inches. Refer to sheet ____/____ for camber information. The contractor shall adjust seat elevations based upon the actual beam camber at the time of fabrication if available.

NOTE TO DESIGNER: Refer to BDM Section 308.2.3.4.b for description of camber values. If the minimum sacrificial haunch depth differs from 2-inches, revise the note accordingly. Due to the camber of prestressed beams being difficult to determine exactly, the Contractor may need to adjust the beam seat elevations based on the actual beam camber values. To accommodate this adjustment, Designers shall detail vertical reinforcement in the bearing seats with adjustable lap splices such that the minimum lap length coincides with D_{30} . Do not include deflection due to the weight of FWS.

[702.6-3] DECK SLAB THICKNESS FOR CONCRETE QUANTITY: The estimated quantity of deck concrete is measured according to [C&MS 511](#). In addition to the design slab thickness, the quantity includes a variable haunch thickness that provides an allowance for: vertical grade adjustment, beam camber and additional sacrificial haunch thickness.

NOTE TO DESIGNER: Delete “vertical grade adjustment” from the above note when the project does not include such an adjustment.

Use the following note when the length of the I-beam, measured along the grade, differs from the length, measured horizontally, by more than 3/8-in:

[702.6-4] NOTE TO FABRICATOR: The dimensions measured along the length of the beam, marked with a * , do not contain an allowance for the effect of the longitudinal grade. Include the proper allowance for these dimensions in the Shop Drawings.

NOTE TO DESIGNER: Indicate the dimensions that require a grade adjustment with an asterisk or some other easily recognizable symbol and include that symbol in the note above.

702.7 PRESTRESSED CONCRETE BOX BEAM BRIDGE

For prestressed composite concrete box beam bridges with concrete deck, compute the concrete topping depth over the top of the beams according to BDM Section Provide a longitudinal cross section showing the topping thickness at each centerline of bearing and at each midspan.

In addition to the deck elevation tables required by BDM Section 702.12.1, provide the following notes in the plans for every beam:

[702.7-1] CAMBER:

Estimated camber at Day 0 (D_0) is _____ inches.

Estimated camber at Day 30 (D_{30}) is _____ inches.

Deflection due to remaining dead load (e.g. concrete deck, diaphragms, bridge railings, utilities, etc.) is _____ inches.

Provide the following note in the plans for each substructure unit:

[702.7-2] SEAT ELEVATIONS:

The beam seat elevations were calculated using the assumed camber D_{30} with a minimum sacrificial haunch thickness of _____ inches. Refer to sheet ____/____ for camber information. The contractor shall adjust seat elevations based upon the actual beam camber at the time of fabrication if available.

NOTE TO DESIGNER: Refer to BDM Section 308.2.3.3.e for description of camber values. Due to the camber of prestressed beams being difficult to determine exactly, the Contractor may need to adjust the beam seat elevations based on the actual beam camber values. To accommodate this adjustment, Designers shall detail vertical reinforcement in the bearing seats with adjustable lap splices such that the minimum lap length coincides with D_{30} . Do not include deflection due to the weight of FWS.

[702.7-3] DECK SLAB THICKNESS FOR CONCRETE QUANTITY: The estimated quantity of deck concrete is measured according to [C&MS](#) 511. In addition to the design slab thickness, the quantity includes a variable haunch thickness that provides an allowance for: vertical grade adjustment and beam camber.

NOTE TO DESIGNER: Delete “vertical grade adjustment” from the above note when the project does not include such an adjustment.

Use the following note when the length of the box beam, measured along the grade, differs from the length, measured horizontally, by more than 3/8-in:

[702.7-4] NOTE TO FABRICATOR: The dimensions measured along the length of the beam, marked with a *, do not contain an allowance for the effect of the longitudinal grade. Include the proper allowance for these dimensions in the Shop Drawings.

NOTE TO DESIGNER: Indicate the dimensions that require a grade adjustment with an asterisk or some other easily recognizable symbol and include that symbol in the note above.

For prestressed non-composite box beam bridges with asphalt wearing surface, compute the topping depth over the top of the beams according to BDM Section 308.2.3.3.e. Provide a longitudinal cross section showing the thickness of each Item 441 surface course at each centerline of bearing and at midspan. Refer to BDM Figure 702-1. Additionally, provide the following note in the plans for every beam:

[702.7-5] CAMBER:

Estimated camber at Day 0 (D0) is _____ inches.

Estimated camber at Day 30 (D30) is _____ inches.

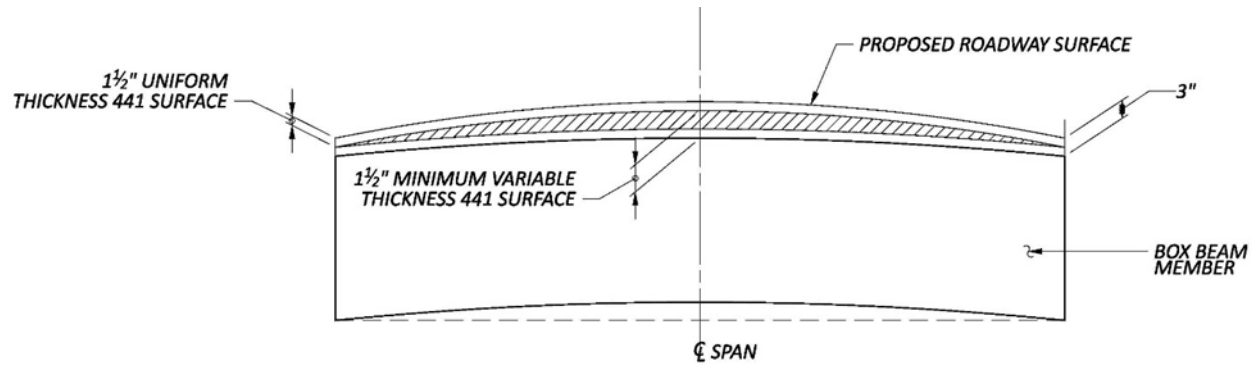
Deflection due to remaining dead load (e.g. asphalt wearing surface, diaphragms, bridge railings, utilities, etc.) is _____ inches.

Provide the following note in the plans for each substructure unit:

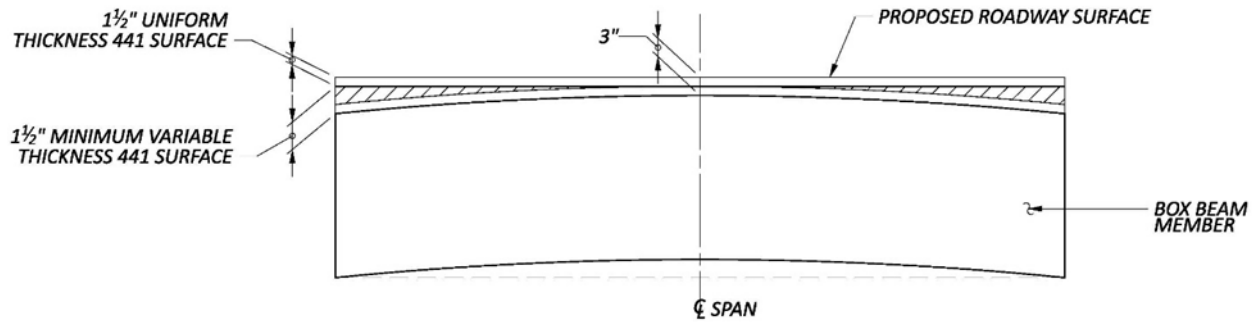
[702.7-6] SEAT ELEVATIONS:

The beam seat elevations were calculated using the assumed camber D30. Refer to sheet ____/____ for camber information. The contractor shall adjust seat elevations based upon the actual beam camber at the time of fabrication if available. No adjustment shall be made to the overlay thickness at midspan.

NOTE TO DESIGNER: Refer to BDM Section 308.2.3.3.e for description of camber values. Due to the camber of prestressed beams being difficult to determine exactly, the Contractor may need to adjust the bearing seat elevations based on the actual beam camber values. To accommodate this adjustment, Designers shall detail vertical reinforcement in the bearing seats with adjustable lap splices such that the minimum lap length coincides with D30. Do not include deflection due to the weight of FWS.



ASPHALT THICKNESS DIAGRAM
(CREST VERTICAL CURVE)



ASPHALT THICKNESS DIAGRAM
(STRAIGHT GRADE OR SAG VERTICAL CURVE)

Figure 702-1

702.8 ASPHALT CONCRETE WEARING COURSE

Place note [702.8-1] on the plans for prestressed concrete box beam bridges having an asphalt concrete wearing course. If the nominal thickness of 441 varies from the 1.5-in shown, revise the note accordingly.

While this note specifies how to place only the two 441 bid items, the designer should recognize that two ITEM 407 - TACK COAT items are also required. One tack coat is applied before the first surface course. The other tack coat is applied between the first and second surface courses.

[702.8-1] ASPHALT CONCRETE WEARING COURSE shall consist of a variable thickness of 441 asphalt concrete surface course, Type 1, PG70-22M and a second 1½" thickness of 441 asphalt concrete surface course, Type 1, PG70-22M. Place the first 441 surface course in two operations. The first portion of the course shall be of 1½" uniform thickness. Feather the second portion of the course to place the surface parallel to and 1½" below final pavement surface elevation.

702.9 PAINTING OF A588/A709 GRADE 50 STEEL

Provide the following note for bridge superstructures using unpainted A588/A709 Grade 50W steel and having deck expansion joints at the abutments. Modify the note accordingly for structures with intermediate expansion joints. Bridges with an integral or semi-integral type abutment will not require painting of the beam ends.

[702.9-1] PARTIAL PAINTING OF A709 GRADE 50W STEEL: Paint the last 10 ft of each beam/girder end adjacent to the abutments including all cross-frames and other steel within these limits. The prime coat shall be 708.01. The top coat color shall closely approach Federal Standard No. 595B - 20045 or 20059 (the color of weathering steel).

702.10 ERECTION BOLTS

Where erection bolts are specified for attaching cross-frames on steel girder or rolled beam bridges, and the expected dead load differential deflection at each end of the cross-frames is less than or equal to 0.5-in provide the following note. (Do not use the note if standard drawing GSD-1-19 is being referenced.)

[702.10-1] ERECTION BOLTS: The hole diameter in the cross-frames and girder stiffeners shall be 3/16" larger than the diameter of the erection bolts. Erection bolts shall be high strength bolts and shall remain in place. Supply two hardened washers with each high strength bolt. Fully torque the bolts or use a lock washer in addition to the two hardened washers. Furnish erection bolts as part of Item 513.

702.11 WELDED ATTACHMENTS

Provide the following note on plans for steel beam or girder bridges:

[702.11-1] WELD ATTACHMENT of supports for concrete deck finishing machine to areas of the fascia stringer flanges designated "Compression". Do not weld attachments to areas designated "Tension". Fillet welds to compression flanges shall be at least 1" from edge of flange, be no more than 2" long, and be at least 1/4" for thicknesses up to 3/4" or 5/16" for greater than 3/4" thick.

702.12 DECK ELEVATION TABLES**702.12.1 SCREED ELEVATION TABLES**

Screed elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam, composite box beam and other superstructure types with cast-in-place concrete decks. Screed elevations are not required for slab bridges. The general criteria for screed elevation tables are defined in BDM Section 309.3.3.3. Refer to BDM Table 702-3 for an example screed table for structural steel members and BDM Table 702-4 for an example screed table for composite box beams. In lieu of a table format, the designer may supply screed elevations through the use of a deck plan view showing elevations and stations of the points required in BDM Section 309.3.3.3.

In addition to the screed elevation table or diagram, provide a screed elevation note similar to the one below to define the elevations that are given. The screed elevation locations should be identified on the transverse section.

[702.12.1-1] SCREED ELEVATIONS shown represent the theoretical deck surface location prior to deflections caused by deck placement and other anticipated dead loads.

Table 702-3

STRUCTURAL STEEL SCREED TABLE								
Point (2)	SPAN NUMBER (1)							
	Station	Station	Station	Station	Station	Station	Station	Station
	Bearing Pt	1/4 Pt	Pt (3)	Mid Span	Pt (3)	Splice Pt	1/4 Pt	Bearing Pt
Left Curb Line								
Phased Const Line								
Centerline								
Right Curb Line								

- (1) Detail all spans.
- (2) Station all Points for screeds.
- (3) Additional points required in a span if the distance between bearing points, 1/4 points, mid span and/or splice points exceeds 30-ft. If the distance does exceed 30-ft, locate the additional point midway between standard points.

Table 702-4

COMPOSITE BOX BEAM SCREED TABLE								
Point (2)	SPAN NUMBER (1)							
	Station	Station	Station	Station	Station	Station	Station	Station
	Bearing Pt	1/4 Pt	Pt (3)	Mid Span	Pt (3)	1/4 Pt	Bearing Pt	
Left Curb line								
Phased Const. Line								
Centerline								
Right Curb Line								

- (1) Detail all spans
- (2) Station all Points for screeds.
- (3) Additional points required in a span if the distance between bearing points, 1/4 points and/or mid span points exceeds 30-ft. If the distance does exceed 30 -ft, locate the additional point midway between standard points.

702.12.2 TOP OF HAUNCH ELEVATION TABLES

Top of haunch elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam and other superstructure types requiring deck falsework. Top of haunch elevations are not required for slab bridges. The general criteria for top of haunch elevation tables are defined in BDM Section 309.3.3.3.

In addition to the top of haunch elevation table, provide a top of haunch elevation note similar to the one below to define the elevations that are given. The top of haunch elevation locations should be identified on the transverse section.

- [702.12.2-1]** TOP OF HAUNCH ELEVATIONS shown represent the theoretical location of the bottom of the deck above the beam/girder haunch prior to deflections caused by deck placement and other anticipated dead loads.

702.12.3 FINAL DECK SURFACE ELEVATION TABLES

Final deck surface elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam, composite box beam and other superstructure types with cast-in-place concrete decks including slab bridges. The general criteria for final deck surface elevation tables are defined in BDM Section 309.3.3.3.

In addition to the final deck surface elevation table, provide a final deck surface elevation note similar to the one below to define the elevations that are given.

- [702.12.3-1]** FINAL DECK SURFACE ELEVATIONS shown represent the deck surface location after all anticipated dead load deflections have occurred.

702.13 ELASTOMERIC BEARING MATERIAL REQUIREMENTS

Use the following note for elastomeric bearings designed in accordance with *LRFD 14.7.6 (i.e. Method A)*

- [702.13-1]** ELASTOMERIC BEARINGS: The elastomer shall have a hardness of (50 or 60) durometer. The bearings were designed in accordance with Section 14.7.6 (Method A) of the AASHTO LRFD Bridge Design Specifications. The Long-term Compression Proof Load Test (AASHTO Standard Specifications for Highway Bridges, Division II, Section 18.7.2.6) is not required.

Use the following note for elastomeric bearings designed in accordance with *LRFD 14.7.5 (i.e. Method B)*

- [702.13-2]** ELASTOMERIC BEARINGS: The elastomer shall have a hardness of (50 or 60) durometer. The bearings were designed in accordance with Section 14.7.5 (Method B) of the AASHTO LRFD Bridge Design Specifications. Perform the Long-term Compression Proof Load Test in accordance with the AASHTO Standard Specifications for Highway Bridges, Division II, Section 18.7.2.6 and 18.7.4.5.

702.14 BEARING SEAT ADJUSTMENTS FOR SPECIAL BEARINGS

Provide the following plan note in project plans that specify specialized bearings such as pot, spherical or disc. This note is intended to ensure that the contractor builds the bearing seats to the proper elevation in the event that the bearing manufacturer adjusts the height of the bearing from the height assumed in the design plans.

- [702.14-1]** The pier and abutment beam seat elevations are based on bearing heights provided in the table below. If the Contractor's selected bearing manufacturer has a design that does not conform to the heights provided in the table, adjust the bearing seat elevations at no additional cost to the state. Adjust the location of concrete reinforcement horizontally as necessary to avoid interference with the bearing anchor bolts. Maintain the minimum concrete cover and minimum spacing required by the project plans. If the concrete reinforcement cannot be moved to provide the required position for the anchor bolts, the Contractor's bearing manufacturer shall re-design the bearings to accommodate an acceptable anchor bolt configuration.

Table 702-5

	Rear Abutment	Pier No #	Forward Abutment
Member Line 1			
Member Line 2			
Member Line 3			
Member Line 4			

702.15 HAUNCHED GIRDER FABRICATION NOTE

For steel haunched girders, add the following note on the design plan sheet that shows an elevation view of the typical haunched girder section defining web size, flange size, depth of member, CVN, etc.

- [702.15-1]** HAUNCHED GIRDERS: Near the bearing, at the intersection of the horizontal bottom flange with the curved (haunched) portion of the bottom flange, the Contractor's fabricator shall hot bend the flange in accordance with AASHTO LRFD Bridge Construction Specifications, Section 11.4.3.3.3 or provide a full penetration weld, with 100% radiographic inspection.

702.16 NSTM FABRICATION NOTE

For structures that contain fracture critical components and members, place the following note in the design plans.

- [702.16-1]** NSTM: All items designated NSTM (, including ____)* are Nonredundant Steel Tension Members and Components and shall be furnished and fabricated according to the requirements of Section 12 of the AASHTO/AWS Bridge Welding Code D1.5.

* - Include this additional wording if there exists nonredundant or fracture critical components such as welds, attachments, etc. that are not easily or clearly identified in the plan details. Write descriptions of such components as specific as necessary to prevent any possible confusion during fabrication.

702.17 CATHODIC PROTECTION

The following note is to be used when galvanic anodes or Distributed Anode Systems (DAS) are to be incorporated into a concrete placement. The pay item associated with note shall correspond to the material or specification placed either C&MS 511, C&MS 519, C&MS 520, C&MS 843. Each pay item shall be "as per plan".

- [702.17-1]** Repair concrete shall be hydraulic cement-based material with a electrical resistivity less than 50,000 ohm-cm according to ASTM C 1760. Do not use non- conductive repair materials such as magnesium ammonium phosphate concrete and epoxy mortars or bonding agents. Concrete mixes containing high levels of supplementary cementitious materials such as silica fume, ground-granulated blast furnace slag, latex, fly ash or metakaolin may not meet the resistivity requirement.

The galvanic anode size and spacing is based on achieving a current density for the Extremely High corrosion risk category with a __ (10, 20,30) year installation. Supply anodes with a minimum core of __ grams of zinc. See sheets __/__ for distribution.

NOTE TO DESIGNER: The design year for the installation shall be noted in the note. Supply the minimum number of grams for each anode or minimum number of grams per ft for each DAS. Show spacing on appropriate plan sheets.

703 SITE PLAN REQUIREMENTS FOR SECTION 401 AND 404 OF THE CLEAN WATER ACT

For waterway crossing projects, include the following information on the Preliminary Structure Site Plan. Refer to BDM Section 201.1.2.2 for additional information.

[703-1] For this project, permits for Sections 401 and 404 of the Clean Water Act, are based on the limits of temporary construction fill placed in “Waters of the United States” as shown below. If either of the limits provided are exceeded, then a 404/401 permit modification will be required. If a permit modification is required, refer to SS832.09 for the application requirements.

Plan Area of Temporary Fill Material = ____ acres

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SECTION 800 – NOISE BARRIERS

801 INTRODUCTION

C801

When noise barriers are necessary, the Office of Environmental Services will furnish the required noise barrier height, length and location(s).

Include the detail design for noise barriers in the Stage 2 Detailed Design Review Submission.

Design specifications for ground mounted precast concrete noise barrier walls are provided in Standard Bridge Drawing [NBS-1-09](#). Associated designer notes are provided in Design Data Sheet [NBSDD-1-09](#). The Department permits the use of noise barrier walls consisting of material types other than precast concrete. These wall types are pre-approved according to the requirements of the Department's Standard Procedure 27-005(SP) for new products and this Manual. Alternate noise barrier material types currently approved include: metal; fiberglass; brick or masonry; and acrylic. A complete listing of approved noise barrier suppliers for material types other than precast concrete are provided in the [ODOT Noise Analysis Manual](#).

802 DESIGN CONSIDERATIONS

802.1 NOISE BARRIER FOUNDATIONS

Design foundations for noise barriers walls in accordance with [GDM](#) Section 1600.

802.2 NOISE BARRIER AESTHETICS

Provide Baseline aesthetic treatments unless otherwise directed in the Scope of Services.

Baseline aesthetic treatments for noise barriers include:

- A. Concrete wall panels with an Ashlar stone pattern form liner, or other approved formliner. Provide a surface treatment detail in the project plans. The Office of Environmental Services will approve formliners other than Ashlar stone pattern. Other wall panel materials (steel, fiberglass, etc.) will not require a form liner.
- B. Except when mounted on bridges and other structures, specify concrete posts.
- C. Provide a cap on noise barrier panels and posts to create a shadow line. Caps for posts should be of the same material as the posts and shall be integral with or mechanically fastened to the top of the post. Caps for panels shall be integral with the panel.

- D. General dimensions for the cap are:
1. 6-in high
 2. 4-in wider than the post and panel and centered so the cap horizontally extends 2-in beyond the panel or posts vertical surfaces. (Other options may be acceptable depending on limits of manufacturing process and final visual effect.)
- E. Baseline colors are beige, light gray, tan and plain uncoated concrete. The coating material used to produce the color is to be approved by the Office of Environmental Services. Determine the noise barrier color(s) in conjunction with the District Aesthetic Coordinator. The posts, caps and joints may have a color(s) different from the panels, but the color combination shall be visually appealing.

Color choices may also be limited due to noise panel material types. Agencies and/or designers specifying both color and wall panel material type should assure that the color is available.

Refer to ODOT's [Aesthetic Design Guidelines](#) for requirements to specify Enhanced aesthetic treatments.

Clearly define the aesthetic requirements for each project in the noise barrier plan general notes.

The notes shall be detailed enough to assure detail finishes, form liners, caps, colors, coatings, application requirements, special design dimensions, etc. are not only adequately specified to assure the visual effect but to also assure quality of construction and materials.

803 DETAIL DESIGN SUBMISSION REQUIREMENTS

803.1 NOISE BARRIER PLAN SHEET ORDER

The noise barrier plan sheets shall be an individually numbered plan subset within the project plans. The plan sheet order shall conform to the following:

- A. Noise Barrier Schematic Plan
- B. Noise Barrier Typical Sections
- C. General Notes
- D. Noise Barrier Subsummary
- E. Plan & Profile Sheets
- F. Cross Sections
- G. Noise Barrier Data Tables
- H. Miscellaneous Noise Barrier Details

Ensure that the soil profile/foundation investigation sheets are included in the project plans.

803.2 NOISE BARRIER PLAN REQUIREMENTS

The noise barrier plans for Stage 2 Detailed Design Review shall include the following:

A. Noise Barrier Schematic Plan

The scale (1-in = 200-ft) shall be shown in bar format. A north arrow shall be provided. Show all reference lines (e.g. centerlines of construction, baselines of ramps, etc.). Provide roadway stationing tick marks at 100-ft. intervals.

Each noise barrier on the project shall be given a unique designation (e.g. Noise Barrier #1, Noise Barrier #2, etc.) and unique stationing. The noise barrier stationing shall be independent of the roadway stationing. Provide the noise barrier station; the roadway station; and the offset from the roadway reference line for the beginning and ending points for each wall on the project.

B. Noise Barrier Typical Sections

1. Identify the roadway centerline and dimension the outside traffic lanes, paved shoulder width and graded shoulder width.
2. Dimension the location and identify the type of roadside barrier protection (if required).
3. Dimension the location of the noise barrier with respect to the roadway centerline or the face of the roadside barrier protection (if available).
4. Provide the paved shoulder and graded shoulder cross slopes.
5. Provide the limiting stations where the typical section is applicable.
6. Provide typical grading details around the noise barrier in cut sections and fill sections. Include earth berm dimensions and cross slopes, distance from the noise barrier to the ditch centerlines, ditch dimensions, foreslope and backslope rates, and erosion control or seeding details.
7. Provide typical grading details around the noise barrier in a backslope location, including typical drainage and backfill details, and erosion control or seeding details.

C803.2

Sample Noise Barrier Plans are available on the Office of Structural Engineering website.

The purpose of the schematic plan is to show the location of each noise barrier wall on the project and provide baseline geometry for the wall.

The baseline geometry for the wall may be provided in tabular format. At a minimum, the geometry shall provide the noise barrier station; the roadway station and offset; and the bearing from for all points of tangency along the barrier alignment.

Noise barrier located within the clear zone requires protection in the form of guardrail or concrete barrier. Measure the clear zone to the nearest face of the post.

C. General Notes

Provide general notes, not included in the standard bridge drawings or elsewhere in the project plans, that are necessary to complete the project plans and properly construct the noise barrier. For wall types other than precast concrete, include the approved supplier contact information provided in the [ODOT Noise Wall Analysis Manual](#).

D. Noise Barrier Subsummary

Provide a noise barrier subsummary as described in Section 1307 of the [L&D, Vol. 3](#), which includes all quantities required to construct the noise barriers that are not summarized elsewhere in the project plans.

E. Plan and Profile Sheets

Plan and profile sheets shall conform to Section 1309 of the [L&D, Vol. 3](#), and shall detail the horizontal and vertical alignments of the noise barrier as follows:

1. In addition to the requirements of the [L&D, Vol. 3](#), include the following information in the plan view (using a 1-in=20-ft horizontal scale):
 - a. All horizontal alignment data for the noise barrier.

Provide the noise barrier stationing at the begin point, end point and all intermediate deflection points of the barrier alignment; the bearing direction for all noise barrier segments; the deflection angle at all deflection points.

- b. Graphically accurate representation of the panel segments and post locations including alignment centerline. In order to avoid congestion, post and panel labels should not be shown.
 - c. All existing and proposed utilities, boundary lines, drainage facilities and other roadway incidentals (e.g. signs, fences, etc.) located within the construction limits of the noise barrier.
 - d. All soil boring locations and designations.
 - e. All proposed roadway, erosion control and drainage items and features to be constructed as part of the noise barrier installation.
 - f. The locations of all existing and proposed reference monuments. Locate all proposed monuments on the roadway side of the noise barrier and relocate any existing monument as necessary to place it on the roadway side of the barrier. (Provide a pay item to relocate every existing Reference Monument located on the outside of proposed noise walls.)
 - g. Proposed construction limits encompassing the area that will be disturbed by the noise barrier construction.
2. In addition to the requirements of the [L&D, Vol. 3](#), include the following information in the profile view (using a 1-in=5-ft vertical scale and a 1-in=20-ft horizontal scale with vertical gridlines at 10-ft increments, horizontal gridlines at 25-ft increments and no grid subdivisions):
- a. A profile of the existing ground line taken along the centerline of the proposed noise barrier, with elevations provided every 25-ft and at abrupt elevation changes.
 - b. All existing and proposed underground utilities and drainage facilities located within the construction limits of the noise barrier. Label the disposition of existing items.
 - c. The acoustic profile line and elevations determined from the preliminary data in the project noise analysis report.
 - d. The length of all panel spans.
 - e. Each panel number or designation.
 - f. A profile of the final ground line taken along the centerline of the proposed noise barrier with elevations provided every 25-ft and at abrupt elevation changes.
 - g. The top and bottom elevations of each noise barrier panel between adjacent posts.

- h. The locations of noise barrier panels with special architectural surface treatments, form liners or icons that differ from the normal surface treatment for the project.
- i. The locations of drilled shafts with a diameter greater than 30-in or other special foundation types.
- j. The approximate top of bedrock elevation at each soil boring location (where applicable).

F. Cross Sections

Provide cross sections along the centerline alignment of each noise barrier. The cross section shall conform to Section 1310 of the [L&D, Vol. 3](#) and include the following:

- 1. Earth berms or other special grading that is required to construct the noise barrier at the proper elevation.
- 2. Proposed ditches required to properly convey stormwater runoff around the noise barrier.
- 3. Profiles of proposed drainage facilities within the construction limits of the noise barrier (not including the noise barrier underdrains) that are not shown elsewhere in the project plans.

G. Noise Barrier Data Tables

H. Miscellaneous Noise Barrier Details

The noise barrier cross sections may be shown on the roadway cross sections. If provided with the roadway sections, in addition to the roadway station, provide the noise barrier station at each cross-section. Also include cross-references in the noise barrier plan subset listing the location of the noise barrier cross sections in the overall plan set.

Provide completed data tables for each noise barrier similar to those shown in the sample noise barrier plans.

Provide details not provided in the standard drawings or elsewhere in the project plans. Examples include: special architectural surface treatments or other aesthetic details; special post or panel cap details; noise barrier transitions at bridges; special panel details to accommodate fire hose connections; etc.

803.3 ADDITIONAL DETAIL DESIGN SUBMISSION REQUIREMENTS

Prior to beginning a noise wall design, schedule a conference call with the noise wall construction plan preparer/designer, the Office of Environmental Services, and District, to discuss the details and expectations of the design.

C803.3

The reason for this requirement is to help ensure the design will meet ODOT's expectations and to avoid/minimize potential revisions/comments to the design during ODOT review.

804 NOISE BARRIERS – APPROVAL OF WALL DESIGNS C804

Individual manufacturers of noise barrier for material types other than precast concrete shall submit panel designs. A modification to an approved wall design requires resubmission to the Department for approval. The Department will not allow a modified design to be used on a construction project prior to its approval.

If approved, the Department will add those designs to the ODOT Approved Reflective and Absorptive Barrier Suppliers List.

Environmental, structural and acoustic design for the walls shall meet the requirements of BDM Sections 805.1, 805.2 and 0.

There are two types of noise barriers: reflective and absorptive.

805 NOISE BARRIER SUBMISSION REQUIREMENTS

Manufacturers interested in having their noise barrier design approved shall submit an approval package to the Office of Environmental Services.

As a minimum, the submission package shall show compliance with the following design requirements:

- A. Environmental BDM Section 805.1
- B. Structural BDM Section 805.2
- C. Material BDM Section 805.3

Submit three copies of the complete submission package to the Office of Environmental Services. Include the specific product trade name; company address; and name, phone number, and email address of a technical representative available to answer questions during the product review period. The Department will evaluate the submission and provide a written decision to the manufacturer no later than 60 days after the submission package is received.

805.1 ENVIRONMENTAL DESIGN REQUIREMENTS

The Manufacturer's wall system shall show compliance with the Department's Aesthetic limitations provided in BDM Section 802.2 and the following Acoustic requirements:

- A₁. Reflective noise barriers - Minimum STC (Sound Transmission Class) = 30
- B₁. Absorptive noise barriers:
 - 1. Minimum STC (Sound Transmission Class) = 30
 - 2. Minimum NRC (Noise Reduction Coefficient) = 0.70

All barrier material submitted shall be acoustically tested at an independent laboratory capable of performing the following tests:

- A₂. ASTM Standard Test Method for Sound Absorption and Sound Absorption Coefficients by Reverberation Room Method
- B₂. ASTM C423 and E795 (Latest editions)
- C₂. ASTM E90 and E413 (Latest editions)

805.2 STRUCTURAL DESIGN REQUIREMENTS

As a minimum, the structural design of the Manufacturer's wall system shall conform to the current edition of the "*AASHTO LRFD Bridge Design Specifications*" and this Manual.

The structural design submission shall also include:

- A. All design assumptions including:
 - 1. Physical and mechanical strengths of the component raw materials
 - 2. Physical and mechanical strengths of the final composite material
 - 3. Design method(s) and governing specifications used
 - 4. Design safety factors used including information on why the factors were chosen and how the material's environmental and loading durability affect those factors.
 - 5. Certified test data for material properties specified in the design.
- B. Complete design calculations annotated with references to governing specifications:
 - 1. Signed, sealed and dated by an Ohio registered professional engineer
 - 2. Include all fabrication, shipping, handling and erection loads
- C. Fabrication and construction drawings for every component of the wall system showing wall details, dimensions, connections and any other information required to define the wall system.

**805.3 MATERIAL DESIGN
REQUIREMENTS**

The material design submission shall include:

- A. Test data documenting the physical and mechanical properties used for structural design.
- B. Test data documenting any long term decrease in physical and/or mechanical properties due to fatigue, creep, bond deterioration, etc.
- C. Test data documenting material durability to environmental variables including: UV, temperature, moisture, freeze-thaw, fire, salt, petroleum, pH, etc.
- D. Test data documenting material's performance to temperature changes expected under service conditions.
- E. Test data documenting durability of any applied coatings used to protect material from environmental deterioration.

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SECTION 900 – BRIDGE LOAD RATING

901 PURPOSE

The purpose of this Section is to provide consistency and uniformity in the procedures, guidelines and policies for determining safe live load carrying capacity or load rating of the highway bridges in the State of Ohio.

902 SCOPE

The guidelines, policies and recommendations provided in this Section are meant to assist bridge owners and bridge raters by establishing evaluation practices that meet the Ohio Revised Code (ORC), the National Bridge Inspection Standards, Specifications for National Bridge Inventory, ODOT Bridge Design Manual and American Association of State Highway Transportation Officials.

The intent of this Section is to establish standardized load rating procedures conforming to FHWA reporting requirements and posting bridges in the State of Ohio. The provisions of BDM Section 900 apply to ODOT bridges. All provisions of BDM Section 900 may also be applied to non-ODOT bridges at the discretion of the bridge owner.

903 APPLICABILITY

903.1 APPLICABILITY OF AASHTO DESIGN SPECIFICATIONS

BDM Section 900 is consistent with the current edition of AASHTO LRFD Bridge Design Specifications and Standard Specifications for Highway Bridges (14th Edition). Where this Section is silent, the current AASHTO LRFD Bridge Design Specifications or Standard Specifications for Highway Bridges shall govern.

903.2 APPLICABILITY TO HIGHWAY BRIDGES

The provisions of this Section apply to all highway structures in Ohio that qualify as bridges in accordance with the definition for a bridge set herein. These provisions may be applied to smaller structures which do not qualify as bridges.

904 QUALITY MEASURES

To maintain the accuracy and consistency of load rating, bridge owners should implement appropriate quality assurance and quality control (QA/QC) measures. Typical quality control procedures include the use of checklists to ensure uniformity and completeness, the review of reports and computations by a person other than originating individual and periodic field review of the inspection teams and their work.

Each load rating analysis shall be performed under the supervision of an Ohio registered professional engineer (i.e. the load rater) who will sign and stamp (seal) the final load rating report before submission to the bridge owner.

905 DEFINITIONS AND TERMINOLOGY

ASR: Allowable Stress Rating (also known as Working Stress Rating)

ADT: Average Daily Traffic volume

ADTT: Average Daily Truck Traffic volume

AssetWise: AssetWise Asset Reliability is ODOT's bridge inventory and appraisal data collection system. AssetWise went live on May 11, 2020.

BR100: A load rating summary form (spreadsheet) internally developed by ODOT.

Bridge: A structure, including supports, erected over a depression or an obstruction such as water, highway, bikeway or railway; and having a roadway to carry vehicular traffic and having an opening measured along the centerline of the roadway of 10-ft or more between under-copings of abutments or spring lines of arches or extreme ends of openings for multiple boxes. It may also include multiple pipes where the clear distance between openings is less than half that of the smaller contiguous opening.

Bridge Asset Name (BAN): A combination of 3-letter County Abbreviation – Route Number – County Log Point, in miles, followed by special designation, if any (e.g., HAM-71-10.680 R); 3-letter County Abbreviations are given in ODOT L&D Manual, Volume I

Bridge Appraisal Rating (BAR): Lowest of the appraisal summary ratings of the deck, superstructure and substructure or culvert.

Bridge Owner: A public or private entity that has jurisdiction over the bridge or an agency having major maintenance responsibility for a bridge. Generally, an entity responsible for the major maintenance of a bridge is considered the owner of the bridge.

Bridge Rating Engineer (BRE): The State Bridge Rating Engineer managing the state bridge rating program.

Buried Structure: A structure, including a flat slab, an arch, a frame, a box section, etc. that has a fill or pavement material of 2-ft or more on top of it.

Collapse: A major change in the geometry of the bridge rendering it unfit for its intended use

Condition Rating: The result of the assessment of the functional capability and the physical condition of a bridge's components by considering the extent of deterioration and other defects. Generally, Condition Rating is evaluated on a scale "0" through "9" (where "9" is the best).

Control Authority Program Manager (CAPM): The designated person at a public transportation entity (Control Authority) responsible for overseeing FHWA's National Bridge Inspection Program for that entity.

Controlling Legal Load Rating Factor: Lowest (controlling) rating factor for the State's legal and AASHTO loads.

County Log Point: Distance in miles from the point where a route enters the county or the starting point of a route within the county traveling in the up-station direction from south-to-north or west-to-east.

Design Load: Live load for which the bridge was designed.

District Bridge Engineer (DBE): The Engineer in the respective ODOT district office managing the bridges within the district.

Emergency Vehicle (EV): An emergency vehicle, as defined in the Fixing America's Surface Transportation Act (FAST Act), is designed to use under emergency conditions to transport personnel and equipment to support the suppression of fires and mitigation of other hazardous situations (23 U.S.C. 127(r)(2)). The GVW limit for an EV is 86,000 pounds. The statute imposes the following additional limits:

- A. 24,000 pounds on a single steering axle.
- B. 33,500 pounds on a single drive axle.
- C. 62,000 pounds on a tandem axle; or
- D. 52,000 pounds on a tandem rear drive steering axle.

Failure: A condition where a limit state is reached or exceeded. This may or may not involve collapse or other catastrophic occurrences.

Federal Bridge: A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the centerline of the roadway of more than 20 feet between under copings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it includes multiple pipes, where the clear distance between openings is less than half that of the smaller contiguous opening. (23 CFR 650.305)

FHWA: Federal Highway Administration – U.S. Department of Transportation

GVW: Gross Vehicle Weight

Interstate System: The interstate routes and roadways within reasonable access to the interstate system.

Inventory Rating: Load ratings based on the inventory level allow comparisons with the capacity for new structures and, therefore, result in a live load that can safely utilize a structure for an indefinite period of time.

Legal Load: The maximum load for each vehicle configuration, including the weight of the vehicle and its payload, permitted by law for the state in which the bridge is located.

LFR: Load Factor Rating

Limit State: A condition beyond which a bridge or a component ceases to satisfy the criteria for which it was designed.

Load Effect: The response (axial force, shear force, bending moment, torque, etc.) in a member or an element due to the loading.

Load Factor: A load multiplier accounting for the variability of the loads, the lack of accuracy in analysis and the probability of simultaneous occurrence of different loads.

Load Rater: An individual person responsible for the load rating of a bridge. The Load Rater shall be a professional engineer registered in the State of Ohio.

Load Rating: The analysis to determine the safe vehicular live load carrying capacity of a bridge.

Long Span Bridge: Any single or multi-span bridge that has at least one span greater than 200-ft

LRFD: Load and Resistance Factor Design

LRFR: Load and Resistance Factor Rating

MBE: AASHTO Manual for Bridge Evaluation

MUTCD: Manual on Uniform Traffic Control Devices, A document published by the FHWA that sets standards for traffic control devices used on public roads.

NBI: National Bridge Inventory; the aggregation of structure inventory and appraisal data collection to fulfill the requirements of NBIS and SNBI.

NBI Performance Measures (Good/Fair/Poor): Starting January 1, 2018, NBI performance measures are determined as follows:

When the lowest rating of any of the 3 NBI items for a bridge (#58-Deck, #59-Superstructure, #60-Substructure) is 7, 8, or 9, the bridge is classified as Good. When the rating of NBI item for a culvert (#62-Culverts) is 7, 8 or 9, the culvert will be classified as Good.

When the lowest rating of any of the 3 NBI items for a bridge (#58-Deck, #59-Superstructure, #60-Substructure) is 5 or 6, the bridge is classified as Fair. When the rating of NBI item for a culvert (#62-Culverts) is 5 or 6, the culvert will be classified as Fair.

When the lowest rating of any of the 3 NBI items for a bridge (#58-Deck, #59-Superstructure, #60-Substructure) is 4, 3, 2, 1 or 0, the bridge is classified as Poor. When the rating of NBI item for a culvert (#62-Culverts) is 4, 3, 2, 1 or 0, the culvert will be classified as Poor.

When the lowest rating of the 3 NBI Items 58-Deck, 59-Superstructure, 60-Substructure for a bridge or the Item 62-Culvert for a culvert is 4, 3, 2, 1, or 0, the bridge or culvert shall be classified as Poor bridge.

NBIS: National Bridge Inspection Standards; Federal regulations establishing requirements for the bridge inspection organization, its inspection procedures, the frequency of inspection, the qualification of personnel, inspection reports, and preparation and maintenance of bridge inventory records. The NBIS applies to all structures defined as NBIS bridges located on or over all public roads.

Nominal Resistance: Resistance of a component or connection to load effect, based on its geometry, permissible stresses, and specified strength of materials.

Non-Buried Structure: A structure, including a flat slab, an arch, a frame, a box section, etc., that has a fill or pavement material of less than 2-ft on top of it.

Non-ODOT Bridge: A bridge on which ODOT does not have jurisdiction or major maintenance responsibility.

ODOT: Ohio Department of Transportation

ODOT Bridge: A bridge on which ODOT has jurisdiction or major maintenance responsibility.

Ohio Legal Vehicles: A set of vehicles which are legally allowed to operate on the public roads in Ohio without any permit. It consists of commercial legal loads (S-2F1, S-3F1, and S-5C1; AASHTO Type 3, Type 3S2, and Type 3-3, SHVs, SU4, SU5, SU6, and SU7); and emergency vehicle EV2; EV3 is legal to operate on Interstate system.

Operating Rating: Load ratings based on the operating rating level generally describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at operating level may shorten the life of the bridge.

ORC: Ohio Revised Code (as amended and adopted)

OSE: ODOT Office of Structural Engineering

OMUTCD: Ohio Manual of Uniform Traffic Control Devices (current edition)

Pavement of a Roadway: The pavement of a roadway includes all the paved or unpaved portions of a roadway including graded shoulders that may support vehicular traffic.

PDF: Portable Document Format, a type of industry standard electronic file format developed by the Adobe Corporation

Permit Load: A live load, which has a gross weight, axle weight, or distance between axles not conforming with State statutes for legally configured vehicles.

Posting: Signing a bridge for load restriction

Preliminary Design Date: The date when Federal-aid funds are obligated for the studies or design activities related to identification of the type, size, and/or location of bridges. For ODOT projects following the Project Development Process (PDP), this date corresponds to the initiation of Step 1 for a Minimal Project, Step 3 for a Minor Project or Step 6 for a Major Project.

Quality Assurance: The use of sampling and other measures to assure the adequacy of quality control procedures in order to verify and measure the quality level of the entire bridge inspection and load rating program.

Reasonable Access to Interstate: At least one-road-mile from interchange to an interstate highway.

Reliability Index: A computed quantity defining the relative safety of a structural element or structure expressed as the number of standard deviations that the mean of the margin of safety falls on the safe side.

Resistance Factor: A resistance multiplier accounting for the variability of material properties, structural dimensions, workmanship, and the uncertainty in the prediction of resistance

RF: Rating Factor, an indicator of live load carrying capacity of a member or a bridge for a specific truck or load.

Routine Permit Load: A live load, which has a gross weight, axle weight, or distance between axles not conforming with State statutes for legally configured vehicles, authorized for unlimited trips over an extended period of time to move alongside other heavy vehicles on a regular basis.

Safe Posting Load/Weight: Weight of a certain vehicle or truck which can safely cross a bridge repeatedly; It is usually determined by multiplying RF with the GVW or determined by the engineer and posted on the bridge sign when a bridge posting is required; posting load cannot be more than the GVW of the legal truck

Service Limit State: Limit state related to stress, deformation and cracking.

Serviceability: A term that denotes restrictions on stress, deformation, and crack opening under regular service conditions

Serviceability Limit State: Collective term for service and fatigue limit states

SFN/NBI/SNBI Bridge Number: Structure File Number (SFN) of a structure is a unique identification number assigned to a structure. It is a seven-digit number and does not change during the service life of the bridge.

SNBI: Specifications for the National Bridge Inventory

SHV: AASHTO Special Hauling Vehicles (SU4, SU5, SU6, SU7)

Strength Limit State: Safety limit state relating to strength and stability.

Superload: In Ohio, a Superload is any permitted highway vehicular load with the total gross load equal to or more than 120,000 pounds (60 tons)

Target Reliability: A desired level of reliability in a proposed evaluation.

TEM: Traffic Engineering Manual, published and maintained by the ODOT Office of Roadway Engineering

Total Bridge Length: Total length of the bridge measured along the roadway centerline; it is usually measured from back-to-back of backwalls; it can be “Bridge Limits,” if given on bridge plans.

906 REFERENCES

906.1 REFERENCES FROM OHIO REVISED CODE

Selected references from the ORC related to bridge load rating and posting are as follows:

4513.34 Written permits for oversized vehicles

Effective: October 09, 2021

5501.49 Lift bridge inspection

Effective: June 30, 2007

5577.01 Load limits on highways definition

Effective: June 30, 1993

5577.02 Operation of vehicle on highways in excess of prescribed weights forbidden

Effective: June 30, 2021

5577.03 Weight of load - width of tire

Effective: October 01, 1953

5577.04 Maximum axle load, wheel load, gross weights, for pneumatic tired vehicles

Effective: June 29, 2001

5577.041 Maximum axle load, wheel load, and gross weights for solid-tired vehicles

Effective: November 21, 1967

5577.042 Weight provisions for farm, log and coal trucks and farm machinery

Effective: July 27, 2011

5577.043 Permissible weight variations for certain vehicles

Effective: July 27, 2011

5577.044 Weight restrictions for vehicles fueled by natural gas.

Effective: June 30, 2023

5577.045 Fire engine weight exemptions; permits for overweight vehicles.

Effective: June 30, 2021

5577.05 Prohibition against violation

Effective: July 01, 2013

5577.07 Reduction of weight and speed during times of thaw and moisture

Effective: September 28, 1973

5577.071 Reduction of weight of vehicle or load or speed on deteriorated or vulnerable bridge

Effective: May 31, 1988

5577.08 Classification of roads and bridges by board of county commissioners

Effective: May 31, 1988

5577.081 Surface mining vehicles overload limit to use designated roads.

Effective: September 29, 2017

5577.15 Application of size and weight provisions of chapter

Effective: July 03, 2019

5591.42 Carrying capacity of bridges - warning notice.

Effective: November 02, 1989

906.2 REFERENCE PUBLICATIONS

- A. AASHTO, LRFD Bridge Design Specifications, most current Edition, and all subsequent Interims
- B. AASHTO, the Manual for Bridge Evaluation, most current Edition, and all subsequent Interims
- C. AASHTO, Standard Specifications for Highway Bridges, 17th Edition
- D. AASHTO, Guide Specifications for Fatigue Evaluation of Existing Steel Bridges, most current Edition, and all subsequent Interims
- E. AASHTO, Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges
- F. AASHTO, Manual for Maintenance Inspection of Bridges
- G. AASHTO, Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members, most current edition, and all subsequent Interims

- H. AASHTO Bridge Rating and AASHTO Bridge Design Software, developed by Baker Corp, Moon Township, Pittsburgh
- I. Duncan, J.M., 1979, “Design Studies for Aluminum Structural Plate Box Culverts,” Kaiser Aluminum and Chemical Sales, Inc.
- J. FHWA, 1995, “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges, Report No. FHWA-PD-96-001, December 1995
- K. FHWA, 2022, “Specifications for the National Bridge Inventory, Publication No. FHWA-HIF-22-017, March 2022
- L. NCSA, “Load Rating & Structural Evaluation of In-Service Corrugated Steel Structures,” & Design Data Sheet No. 19, National Corrugated Steel Pipe Association (NCSA, 202-452-1700)

907 BRIDGE FILES (RECORDS)

Bridge owners shall maintain a complete, accurate and current record of each bridge under their jurisdiction. Complete information, in good usable form, is vital to the effective management of bridges. Such information provides a record that may be important for repair, rehabilitation, replacement, and future planning.

Items that shall be assembled as part of the bridge record are discussed below. Some or all of the information pertaining to a bridge may be stored in electronic format as part of a bridge management system.

907.1 CONSTRUCTION PLANS

Each bridge record should include one clear and readable set of all drawings used to construct, repair and/or rehabilitate the bridge. In lieu of hard copies, the construction plans may be stored in an electronic format in such a way that clear and readable paper copies can easily be reproduced from the electronic records.

907.2 CONSTRUCTION & MATERIAL SPECIFICATIONS

Each bridge record should include the reference to the construction and material specification used during the construction of the bridge. Where general technical specifications were not used, only the special technical provisions need to be incorporated in the bridge record.

907.3 SHOP AND WORKING DRAWINGS

One set of all shop and working drawings approved for the construction or repair of a bridge should be saved or preserved as a part of the bridge record.

907.4 AS-BUILT DRAWINGS

If available, each bridge record should include one set of final drawings showing the “as-built” condition of the bridge, complete with signature of the individual responsible for recording the as-built conditions.

907.5 CORRESPONDENCE

Include all pertinent letters, memoranda and notices of project completion, telephone memos and other related information directly concerning the bridge in chronological order in the bridge record.

907.6 INVENTORY DATA

A complete inventory of a bridge in the ODOT AssetWise shall be done as soon as the bridge is opened to traffic. FHWA mandates, per 23CFR650.313 (b) that an initial inspection shall be performed for each new, replaced, rehabilitated, and temporary bridge as soon as practical, but within 3 months of the bridge opening to traffic. Initial inventory can be completed using the bridge plans. However, a history of dates of physical closing or opening of the traffic on the bridge should be maintained in the bridge record.

907.7 INSPECTION HISTORY

Each bridge record shall include a chronological record of the date and the type of all inspections performed on the bridge. When available, scour, seismic, wind and fatigue evaluation studies; fracture critical information; deck evaluations; field load testing, and corrosion studies should be part of the bridge record.

907.8 PHOTOGRAPHS

Each bridge record shall at least contain photographs of the bridges showing the top view, approach views and the elevation. For bridges crossing over waterways and relief, include photos of the hydraulic openings on the upstream and the downstream sides. Other photos necessary to show major defects, damages, or other important features such as utilities on or under the bridge should also be included.

907.9 RATING RECORDS

The bridge record shall include a complete record of the determination of bridge's load-carrying capacity.

907.10 ACCIDENT DATA

Details of accidents or damage occurrences including date, description of accident, member damage and repairs, as supported by photographs and investigation reports, shall be included in the bridge record.

907.11 MAINTENANCE AND REPAIR HISTORY

Each bridge record shall include a chronological record documenting the maintenance and repairs that have occurred since the initial construction of the bridge. Include details such as date, description of project, contractor cost and related data for in-house projects as well as contracted work.

907.12 POSTING HISTORY

Each bridge record shall include a summary of all loads posting and rescinding actions taken for the bridge, including load capacity calculations, date of posting and description of signing used.

908 GENERAL**908.1 APPLICATION**

For load rating of new bridges, BDM Sections 911 through 925 shall apply.

For load rating of existing bridges, BDM Sections 911 through 924 and 926 shall apply.

908.2 INVENTORY AND OPERATING RATING LOADS

Inventory and Operating Rating Loads are shown in BDM Figure 908.2-1 and BDM Figure 908.2-2

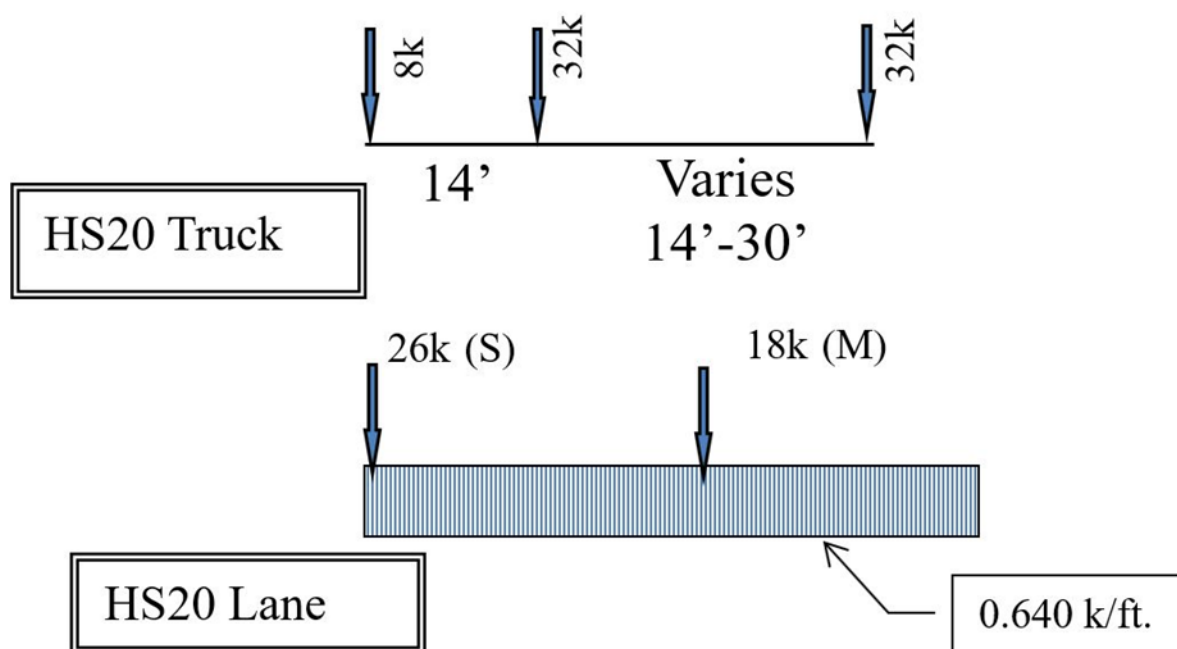


Figure 908.2-1: AASHTO HS20 LOADING

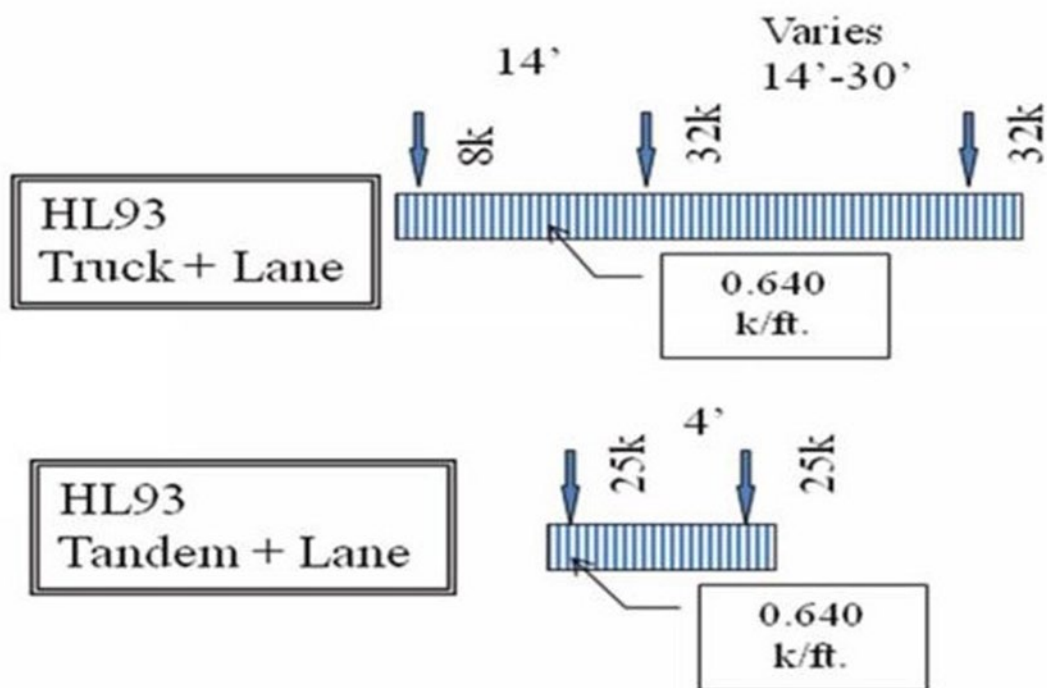


Figure 908.2-2: AASHTO HL93 LOADING

908.3 RATING LOADS

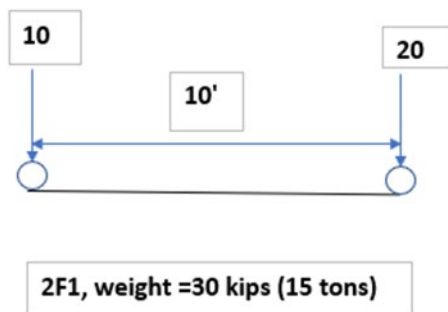
All bridges shall be load rated for all commercial legal vehicles (S-2F1, S-3F1, S-5C1, SU4, SU5, SU6, SU7, TYPE 3, TYPE 3S2, TYPE 3-3) emergency vehicles (EV2, EV3). ODOT bridges shall also be rated for state permit loads shown in this section by policy. Rating for permit loads is used for internal planning and screening. Owners of non-ODOT bridges may decide to rate bridges for permit loads of their choice. Agencies which issue routine permits shall load rate their bridges for routine permit loads, required by FHWA.

For all span lengths the critical load effects shall be taken as the larger of the following (Refer AASHTO MBE 6A.4.4.2.1.):

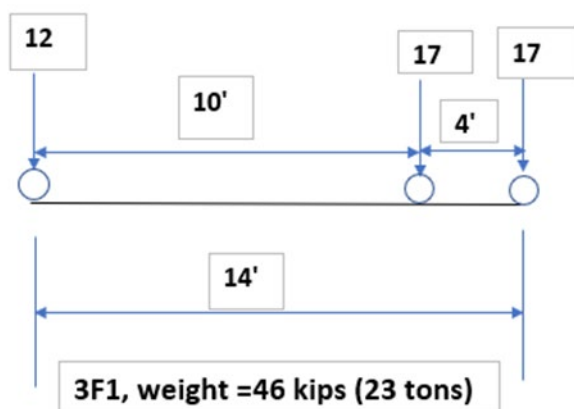
- For all load effects, all legal loads applied separately.
- For negative moments and reactions at interior supports, a lane load of 0.2 kip per linear ft. combined with two AASHTO Type 3-3 vehicles multiplied by 0.75 heading in the same direction separated by 30 ft.

908.3.1 COMMERCIAL LEGAL VEHICLES

All Axle Loads in kips



All Axle Loads in kips

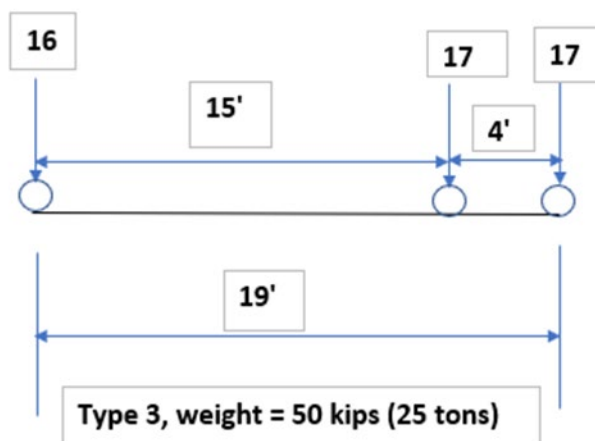


All Axle Loads in kips

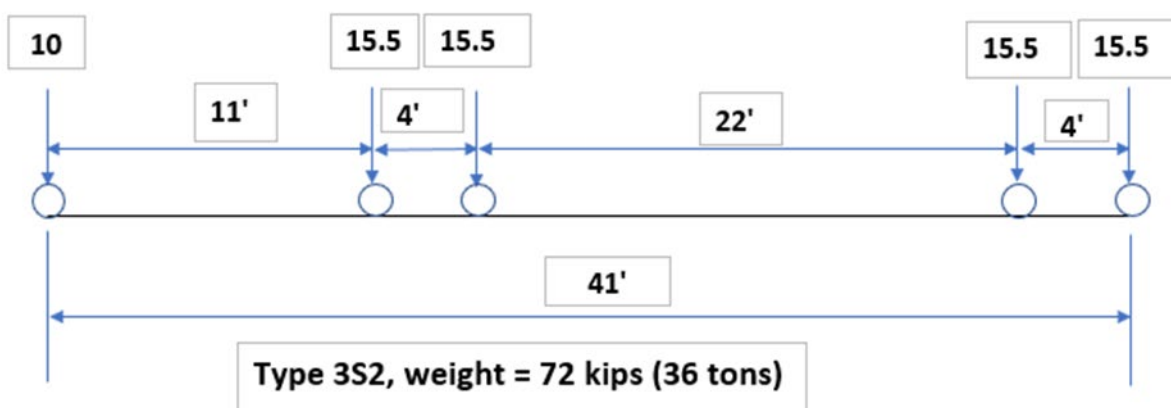


Figure 908.3-1: OHIO LOADS

All Axle Loads in kips



All Axle Loads in kips



All Axle Loads in kips

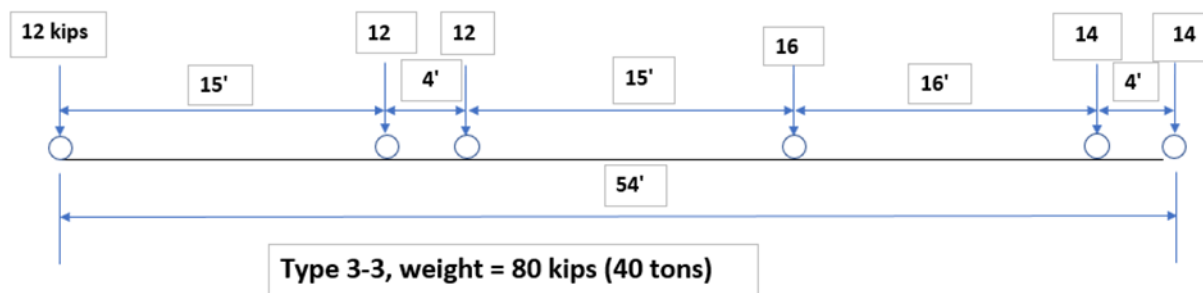
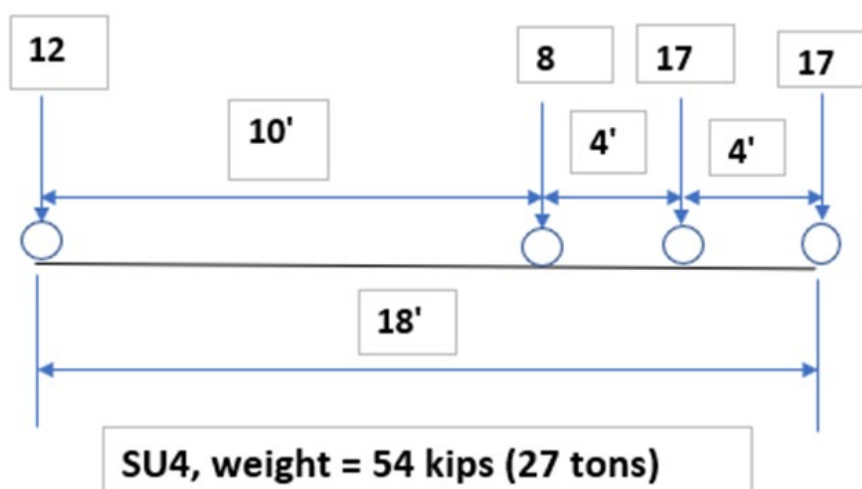
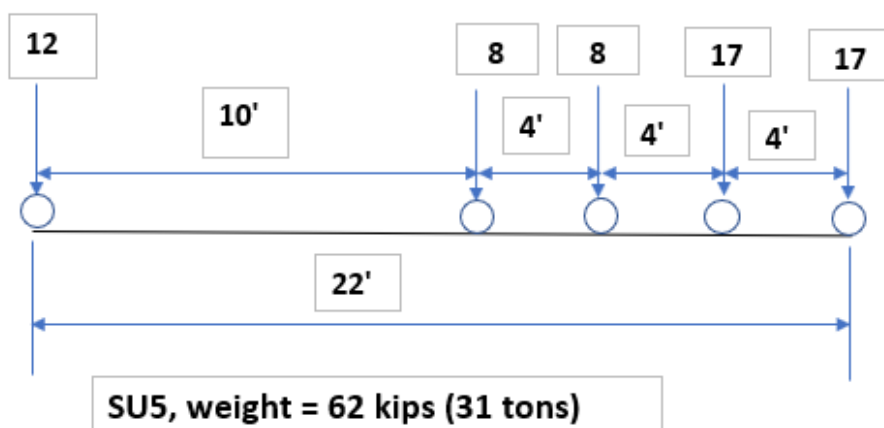
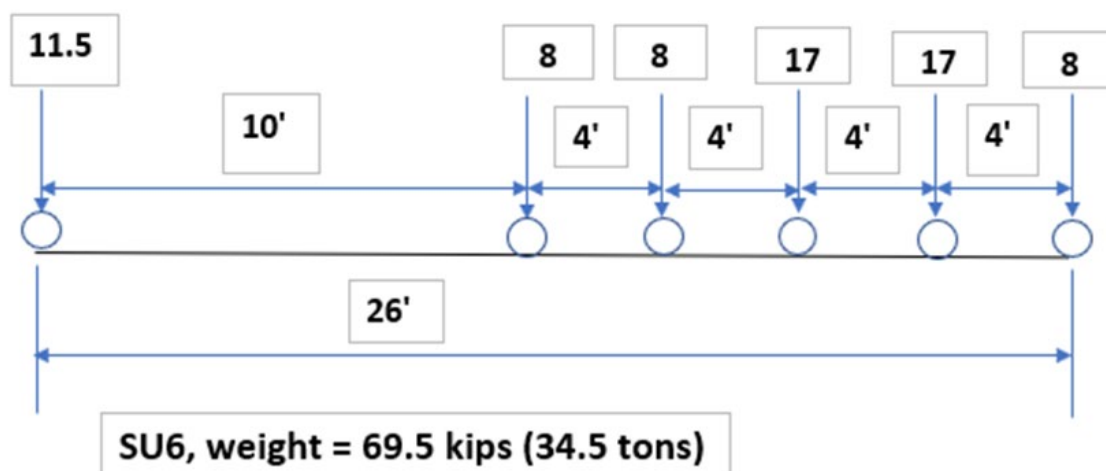


Figure 908.3-2: AASHTO LOADS

All Axle Loads in kips



All Axle Loads in kips**All Axle Loads in kips**

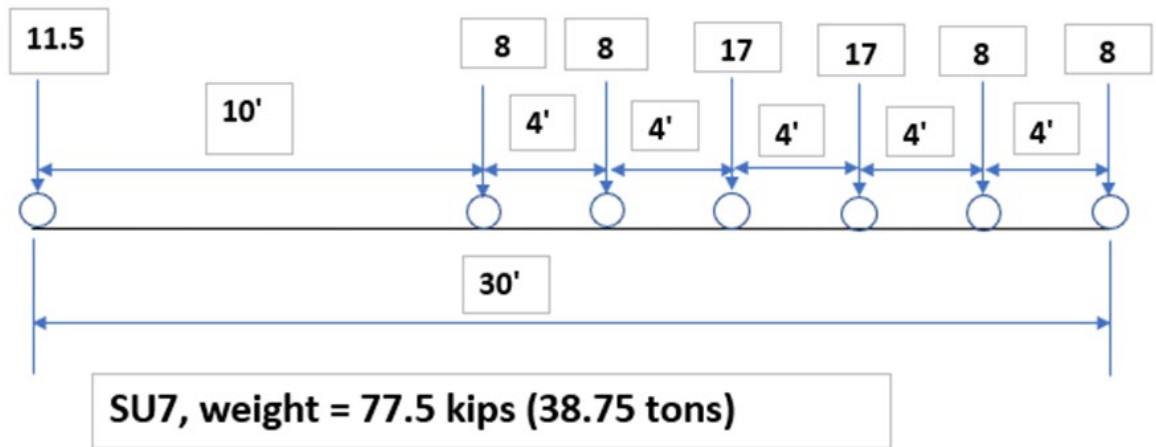
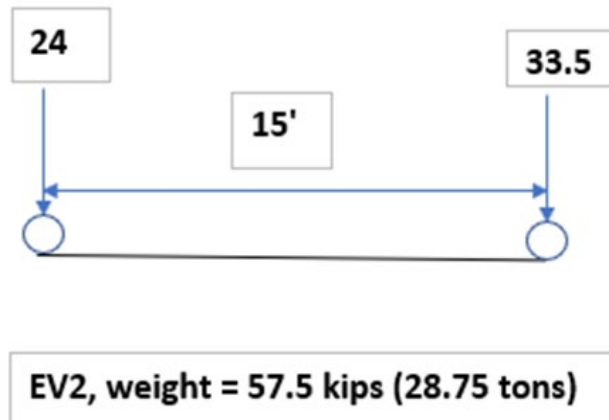
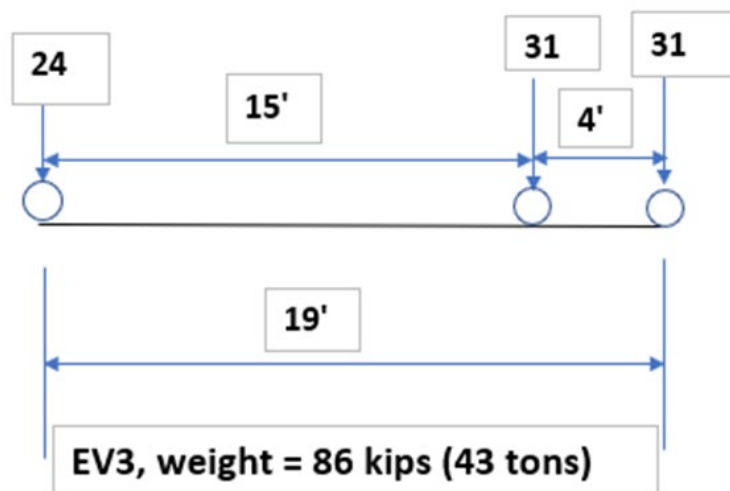
All Axle Loads in kips

Figure 908.3-3: SPECIAL HAULING VEHICLES (SHV)

908.3.2 EMERGENCY VEHICLES**All Axle Loads in kips**

All Axle Loads in kips**Figure 908.3-4: EMERGENCY VEHICLES (EVs)**

908.3.3 STATE PERMIT VEHICLES

All Axle Loads in kips



All Axle Loads in kips

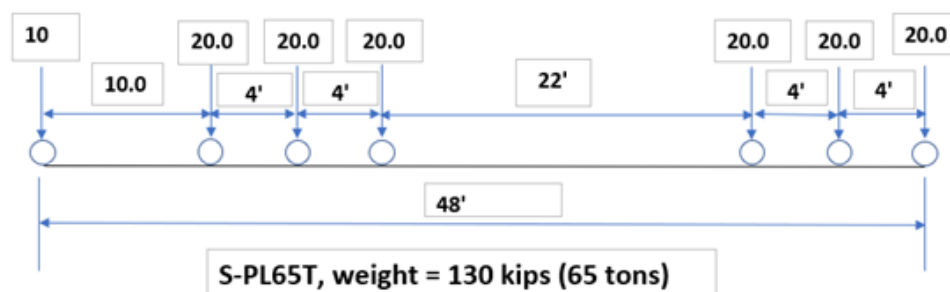


Figure 908.3-5: STATE PERMIT LOADS

909 UNIT WEIGHTS & DENSITIES

The following assumptions should be made while performing the load rating analysis unless more accurate site information is available:

A. Unit weight of asphalt	145 lb. /ft ³
B. Unit weight of concrete	150 lb. /ft ³
C. Unit weight of latex modified concrete	150 lb. /ft ³
D. Unit weight of soil	120 lb. /ft ³
E. Unit weight of steel	490 lb. /ft ³
F. Water density	62.4 lb. /ft ³

910 STRUCTURES EXEMPT FROM LOAD RATING CALCULATIONS

The following types of structures are exempt from load rating calculations under the provisions of this Section.

- A. Circular plastic pipes
- B. Concrete pipes (circular, or elliptical)
- C. Buried metal frames
- D. Junction chambers
- E. Manhole flat slab tops, catch basins, inlet tops
- F. Structures that do not carry vehicular traffic
- G. Structures not carrying a public road
- H. Structures meeting requirements of Section 911

910.1 EXEMPT STRUCTURES THAT NEED A BR100

All structures exempt under this section, carrying highway traffic and maintained in the ODOT bridge inventory (have SFN/NBI Number) shall have a load rating summary form (BR100) filed based on the original design load of the structure and current field inspection. If the design load is unknown, the load rating summary form shall be filed based on rational evaluation.

When there are no signs of distress, structural damage or deterioration, load rating factors of 1.00 for inventory, 1.30 for operating and 1.500 for legal loads shall be used.

911 STRUCTURES UNDER DEEP FILL

The intent of the guidelines of this section is to provide conservative recommendations on rating of structures under deep fill. It is not intended to discourage anyone from conducting detailed analysis of structures under deep fill. When reasonable concerns arise, or when field conditions or observations warrant detailed modeling or testing, such analysis and testing should be carried out.

Experience has shown that when a structure has 6.5-ft or more fill on the top, measured from the roadway centerline, the live load effect on the structure due to vehicular traffic is insignificant. ODOT recommends following rating factors for structures under deep fill when detailed calculations are not performed.

The rating factors of 1.00 for inventory, 1.30 for operating, 1.5 for legal loads and 1.3 for EV3 may be coded for LRFR method of rating for structures in Good and Fair condition, when full analyses have not been conducted. For structures in poor conditions appropriate bridge appraisal factors shall be applied to the above recommended rating factors, per Section 929.3.

912 WHICH PORTION OF BRIDGES SHALL BE LOAD RATED

Any structural member of a bridge that could carry vehicular traffic shall be load rated. Typically, the structural members of the bridge superstructure are load rated. Substructure elements, such as pier caps and columns, should be analyzed for their load carrying capacities in situations when they are either scoped to be analyzed or when the bridge owner or the rating engineer has reason to believe that the capacity of a substructure element may control the capacity of the bridge.

913 PROCEDURE FOR LOAD RATING**A. New Bridges**

1. Load rate new (proposed) bridges at the design stage per BDM Section 925.
2. The Project Manager shall forward the load rating report, bridge structure plans and data input files to the OSE Bridge Rating Engineer for review.

B. Existing Bridges

1. Perform a field inspection of the existing bridge according to the ODOT Manual of Bridge Inspection to determine its condition and the effectiveness of the various members for carrying load, if included in the scope. All information shown in the Bridge Inventory and the Inspection Reports shall also be carefully reviewed.
2. If a field inspection of the bridge is not a part of the Scope of Services, as a minimum, review the most current inspection report (and inspection notes, if available).
3. Perform the load rating per BDM Section 926.
4. Use the date of construction to determine the yield stresses for the construction materials in older bridges for which plan information is not available.
5. For a load rating being requested to the OSE, the District Bridge Engineer (DBE) shall make available to the OSE, Bridge Rating Engineer (BRE), a complete inspection report (including comments), bridge photographs, field measurements and a copy of the previous rating calculation sheets. BRE will review the submitted material, analyze the bridge, and return a copy of the final rating summary (BR100) to the DBE, along with any recommendations.
6. For load rating performed by the consultant and tasked by the District, the DBE, or Project Manager (PM) shall forward the load rating report, bridge structure plans, and data input files to the BRE for review. The BRE shall review the submittal and send the comments to the DBE/PM or the recommended BR100 to the DBE or PM.
7. For load rating performed by the consultant and tasked by the OSE, the consultant shall submit the load rating report, bridge structure plans, and data input files to the BRE for review. The BRE shall review the submittal and send the recommended BR100 to the DBE, after all comments are addressed.
8. Using pertinent current information and load rating analysis, the District Bridge Engineer/Bridge Owner shall determine and record the Inventory, Operating and Legal Load Ratings in AssetWise.
9. The District Bridge Engineer/Bridge Owner shall keep the final rating summary, rating calculations or computer output (if any) and the rating report along with any recommendations in the bridge file.

C. Influence Line Connected Rating Spreadsheets

1. If directed and included in the Scope of Services, prepare Microsoft Excel spreadsheets connected to influence lines/surfaces to analyze a permit vehicle of up to 25 axles.
2. The Project Manager shall forward the load rating report, bridge structure plans, data input files and spreadsheets to the BRE for review.

914 WHEN LOAD RATING SHALL BE REVISED

The load rating of a bridge should be revised when:

- A₁. The superstructure is replaced
- B₁. The existing deck is replaced with a new deck
- C₁. The existing deck width has changed or there is an addition of new beam(s) or girder(s) in the cross-section
- D₁. There is a change in the dead load on the superstructure, like addition or removal of wearing Surfaces, addition or removal of sidewalks, parapets, railings, etc.
- E₁. There is a physical change in the condition of a structural member of the bridge, which may change the capacity of the structural member.
- F₁. Rusting or damage to a slab, beam, girder, or other structural element has resulted in section loss and change in capacity.
- G₁. There is structural damage to steel, due to a hit by a vehicle or excessive deflection or elongation under temperature or highway loads.
- H₁. The bridge appraisal rating drops below 5 and every time it drops any step further below.
- I₁. When exposed or broken prestressing strands are discovered. For each broken or exposed strand, consider the adjacent strand(s) in the same row as ineffective.
- J₁. When a bridge is rated based on “Field evaluation and documented engineering judgement (code:0)” and the bridge appraisal rating drops a step.

The load rating of a bridge does not need to be revised when:

- A₂. The change in the thickness of external wearing surface is less than 1/2-in.
- B₂. The change in the dead load on a beam member is not more than 10-lbs/ft.

915 ANALYSIS OF BRIDGES WITH SIDEWALKS

Pedestrian loads on sidewalks are not typically considered in a load rating analysis of a highway bridge, regardless of if a sidewalk on the bridge is present or not. If a bridge owner has reasons to believe that the sidewalk loads shall be included in the load rating analysis of a bridge, a pedestrian load of 75-lbs/ft² shall be applied to all sidewalks wider than 2-ft and considered simultaneously with the live load in the vehicle lane.

When pedestrian load is present, the pedestrian load effect multiplied with applicable load factor should be subtracted from the capacity of the member at the location being investigated when calculating the RF.

For bridges load rated according to the AASHTO Standard Specifications for Highway Bridges, AASHTO Table 3.22.1A applies. For bridges load rated according to the AASHTO LRFD Bridge Design Specifications, refer to BDM Section 924.2.

Pedestrian loads shall not be considered when performing Special or Permit Load Analysis as per BDM Section 917.

916 ANALYSIS FOR MULTILANE LOADING

- A. Traffic lanes to be used for rating purposes shall be in accordance with AASHTO Specifications.
- B. For rating analysis of floor beams, trusses, non-redundant girders or other non-redundant main structural members, position identical rating vehicles in one or more of the through traffic lanes on the bridge, spaced and shifted laterally on the deck within the traffic lanes to produce the maximum stress in the member under consideration.
- C. Apply the multiple presence factors of AASHTO Standard Specification for Highway Bridges, Section 3.12 or AASHTO LRFD Bridge Design Specification, Section 3.6.1, accordingly.
- D. For analysis of longitudinal members, position a single EV3 vehicle in one lane of the bridge. If multiple lanes are present, apply the heaviest commercial legal loads in the remaining lanes. For analysis of transverse members, place the EV3 in one lane and adjust its lateral position to induce the maximum stress in the target member. If multiple lanes are present, apply the heaviest commercial legal loads in other lanes, also laterally shifted and spaced to maximize stress effects on the transverse members.

917 ANALYSIS FOR PERMIT OR SPECIAL VEHICLE**917.1 ANALYSIS OF BRIDGE FOR PERMIT OR SPECIAL VEHICLE WHEN SCOPED**

When the Scope of Services requires a structure to be analyzed for a special load, define the special load to be included in the analyses in the Scope.

Two analyses shall be performed as per BDM Sections 917.1.1 through 917.1.4, at the operating level.

917.1.1 FIRST ANALYSIS OF BRIDGE WITH THREE OR MORE LANES

- A. In the right-most lane, place the permit or special vehicle positioned to produce the maximum live load effect on the component to be rated.
- B. In the adjacent lanes, simultaneously place single 5C1 vehicles. These vehicles shall be positioned to produce the maximum live load effect on the component to be rated. No partial 5C1 vehicles shall be used. If applicable, BDM Section 916 shall apply.

917.1.2 FIRST ANALYSIS OF BRIDGE WITH TWO LANES

- A. In the right-most lane, place the permit or special vehicle positioned to produce the maximum live load effect on the component to be rated.
- B. In the adjacent lane, place a series of 5C1 vehicles positioned to produce maximum live load effect on the component to be rated. The 5C1 vehicles should be spaced in the longitudinal direction, such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36-ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used. If applicable, BDM Section 916 shall apply.

917.1.3 FIRST ANALYSIS OF BRIDGE WITH A SINGLE LANE

In the traffic lane, place the permit or special vehicle positioned to produce the maximum live load effect on the component to be rated.

917.1.4 SECOND ANALYSIS OF BRIDGE FOR PERMIT OR SPECIAL VEHICLE

Place the permit or special vehicle positioned on the bridge to produce the maximum live load effect on the component to be rated. No other vehicle shall be placed on the bridge.

917.2 ANALYSIS FOR PERMIT LOADS S-PL60T AND S-PL65T

Bridges shall be analyzed for the permit loads S-PL60T and S-PL65T given in BDM Section 908.3 at the permit level for unlimited trips mixed with the regular traffic. The rating factors shall be recorded in the BR100.

918 LOAD RATING OF LONG SPAN BRIDGES**918.1 WHEN THE LOAD RATING SHALL BE DONE**

Perform the load rating of long span bridges according to BDM Sections 925.3.3, 926.2.2, or 926.3.2.

918.2 HOW THE LOAD RATING SHALL BE DONE**918.2.1 INVENTORY & OPERATING LEVEL RATING USING HL93 LOADING**

- A. The HL93 live load, shown in BDM Figure 908.2-2, shall be used and applied as per AASHTO LRFD Design Specification.
- B. Multiple presence factors shall be applied as per BDM Section 916.

918.2.2 INVENTORY & OPERATING LEVEL RATING USING HS20 TRUCK

- A. The AASHTO HS20 truck or lane loading, whichever controls, shall be applied as per AASHTO.
- B. Multiple presence factors shall be applied as per BDM Section 916.

918.2.3 LOAD RATING FOR OHIO LEGAL LOADS

- A. **The Legal Loads shown in BDM Section 908.3 shall be used.** The provisions of BDM Sections 912 through 916 and 917 shall apply.
- B. Multiple presence factors shall be applied as per BDM Section 916.

918.2.3.1 LONG SPAN BRIDGES WITH THREE OR MORE LANES

- A. In the right-most lane, place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 30-ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.
- B. In all other lanes in the same direction, simultaneously place single 5C1 vehicles. These vehicles shall be positioned to produce the maximum live load effect on the component to be rated.
- C. For bridges with two-way traffic, apply the live load for the opposing traffic in the same manner as the one-way traffic.

918.2.3.2 LONG SPAN BRIDGES WITH TWO LANES

- A. In the right-most lane, place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 30-ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.
- B. For bridges with one-way traffic, in the other lane simultaneously place a single 5C1 vehicle positioned to produce the maximum live load effect on the component to be rated.
- C. For bridges with two-way traffic, in the other lane place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 30-ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

918.2.3.3 LONG SPAN BRIDGES WITH A SINGLE LANE

The live load shall be a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 30-ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

918.2.3.4 BRIDGES WITH SPAN LENGTHS GREATER THAN 200 FEET

For span lengths greater than 200 feet, critical load effects shall be created by placing AASHTO Type 3-3 load multiplied by 0.75 and combined with a lane load of 0.2 kip per linear ft. Refer to AASHTO MBE 6A.4.4.2.1.

919 BRIDGE POSTING FOR REDUCED LOAD LIMITS

919.1 BRIDGE POSTING POLICY

When the maximum load effects under Ohio law and AASHTO MBE exceed the load carrying capacity of a bridge, load posting must be considered following ODOT's posting policy, to ensure the safety of the traveling public. ODOT's load posting policy is given below. It is applicable to all bridges carrying public highway traffic.

1. A bridge that is not already posted shall be posted for reduced commercial legal loads if the rating factor for any commercial legal load falls below the posting threshold of 1.08. For emergency vehicles (EVs), the posting threshold that requires posting is a rating factor below 1.00.
2. For bridges on the interstate system and all ODOT bridges not on interstate system, if a bridge is not posted for commercial legal loads, and the rating factor of any of the emergency vehicles (EV2 or EV3) is below 1.00, it will be posted for reduced loads of Emergency Vehicles only.
3. The posting load for any vehicle shall not be more than the legal gross vehicle weight (GVW).
4. The minimum load posting value is 3 tons. Bridges not capable of carrying a minimum gross legal weight of 3 tons shall be closed.

919.2 REFERENCE

Ohio Revised Code, Section 5591.42:

919.3 PROCEDURE FOR BRIDGE POSTING

The Procedure outlined in this section is to be followed for posting or rescinding warnings of bridge strength deficiencies on ODOT bridges. Owners of non-ODOT bridges may modify and adapt the guidelines given in this section to post or rescind warnings of bridge strength deficiencies on their bridges.

919.3.1 BRIDGE POSTING FOLLOWING LOAD RATING

- A. A load rater performs the bridge load rating per the ODOT Bridge Design Manual (BDM).
- B. For an existing or in-service bridge, the bridge shall be load rated based on current dead loads and the last field inspection report. The current operating status, inspection comments, photographs, and condition rating of structural elements shall be considered in the load rating.
- C. Any structural deficiencies discovered during the most current field inspection, as recorded in the bridge field inspection report, shall be considered during the load rating process. The Control Authority Program Manager (CAPM) shall contact the load rater to request to reanalyze a bridge in service.
- D. If load rating is performed by load testing, the test load configuration shall be noted.
- E. The Load Rating Report shall be signed, sealed, and dated by an Ohio registered Professional Engineer. The load rating results shall be communicated to the CAPM who will enter/update the load rating results in the ODOT AssetWise.
- F. If the load rating determines a need for the bridge posting, then the bridge will be analyzed by both LFR and LRFR methods. The method which produces generally higher rating factors shall be used to determine the need for bridge posting and posting loads. The BR100 shall be prepared based on the method selected.
- G. Subsequent to load rating analysis, if it is determined that the bridge needs to be posted for reduced loads (i.e., below Ohio Legal Loads), the CAPM shall note in the AssetWise, "Bridge Posting Required." The CAPM shall initiate the process to have the posting signs installed on the bridge no later than 30 days after a load rating determines a need for such posting (FHWA - October 1, 2019.) The date of posting of sign shall be recorded in the AssetWise.

For ODOT bridges, the date on Director's letter of posting will be used as the date of posting determination.

For Ohio Turnpike bridges, the date when the Chief Engineer approves the bridge posting will be used as the date of posting determination.

For county bridges that require a resolution from the county commissioners, the date of resolution will be used as the date of posting determination. For other county bridges, the date of rating on the BR100 will be used as the date of posting determination.

For city bridges that require a resolution from the city council, the date of resolution will be used as the date of posting determination. For other city bridges, the date of rating on the BR100 will be used as the date of posting determination.

H. It will be the responsibility of the CAPM to periodically verify that the posting signs are in place.

919.3.2 PROCEDURE FOR PLACING POSTING ON ODOT BRIDGES

- A. The ODOT District Bridge Engineer shall submit a written bridge posting request according to BDM Section 919.6 to the OSE Bridge Rating Engineer.
- B. After the Director signs the posting request:
 - 1. The OSE Bridge Rating Engineer shall send a copy to each of the following:
 - a. District Bridge Engineer
 - b. Manager, ODOT Special Hauling Permits
 - c. Superintendent of State Highway Patrol
 - d. Board of County Commissioners
 - e. Respective County Engineer's Office
 - 2. The District Roadway Services Manager shall prepare, erect, and maintain all necessary signs until the bridge is either strengthened or replaced or the posting is rescinded per Section 919.6. The sign placement may require the maintaining agency to obtain a permit to install the bridge signs from the local county/city if a route is carried inside local jurisdiction.
 - 3. The District Bridge Engineer shall update all bridge inventory and inspection records to show the latest official posted capacity.
- C. When posting of a bridge is determined necessary and no unusual or special circumstance at the bridge dictates otherwise:
 - 1. Weight restriction regulatory signs shall be used. Follow the MUTCD, and Ohio Supplement to MUTCD. An example of the standard wording to be used on the signs is given in BDM Figure 919.3.2-1. Sign placement at the bridge and advance warning signs shall be placed in accordance with TEM 201-9.1.
 - 2. Weight limits on the load posting sign shown in BDM Figure 919.3.2-1 (R12-H6) are the total GVW of the vehicle which can safely cross the bridge without causing stresses above safe limits. The first silhouette on top represents a single unit vehicle. 2 Axle to 5 Axle weight limits represent the maximum GVW of single unit vehicles having the specified number of axles. 6+ Axles represents the maximum GVW of single unit vehicles having six or more axles. The second silhouette represents the maximum GVW of semi-trailer combination and the third silhouette represents the maximum GVW of truck and full trailer combination. Regardless of how much of the total weight of the vehicle is actually on the bridge, the GVW must be equal or less than the limit next to the applicable vehicle. No posting sign indicates that vehicles of legal configuration are allowed to cross up to the legal GVW.

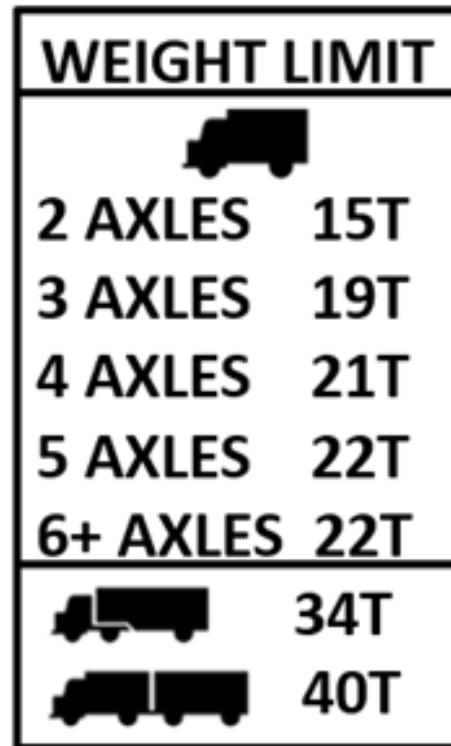


Figure 919.3.2-1: LOAD POSTING SIGN (R12-H6)

919.4 PROCEDURE FOR RESCINDING POSTING OF ODOT BRIDGES

- A. When a posted bridge has been strengthened or replaced and no longer needs posting, the District Bridge Engineer shall forward to the Bridge Rating Engineer a written request to rescind the existing signed posting. The request shall include a complete statement of the reason for the action as specified in BDM Section 919.6.
- B. If a bridge is permanently taken out of service (a bridge number is retired), the DBE shall notify the BRE of this occurrence so that the BRE removes the bridge from the list of posted bridges.
- C. The Bridge Rating Engineer shall review the request submitted by the District Bridge Engineer and upon concurrence, shall forward to the Director a request to rescind the posting.
- D. The Bridge Rating Engineer shall distribute copies of the rescind notice as described in BDM Section 919.3.2.

919.5 PROCEDURE FOR CHANGING POSTING OF ODOT BRIDGES

When the rated capacity of a posted bridge changes, to require a revised posting level, the procedures in BDM Section 919.3 apply. Additionally, the existing posting must be rescinded as set forth in BDM Section 919.4.

919.6 REQUIRED INFORMATION FOR POST, RESCIND AND CHANGE REQUESTS FROM DISTRICTS

The following minimum information is required on all post, rescind and change requests:

- A. Posting Request (Reduction in Load Limits)
 - 1. Current Bridge Asset Name
 - 2. SNBI Bridge Number/SFN
 - 3. Feature intersected (over or under bridge)
 - 4. Posting Load for each Ohio Legal Load or EV load
 - 5. Controlling legal load rating factor
 - 6. Explanation as to why the posting is required
 - 7. If the bridge is scheduled for rehabilitation or replacement. If so, when.
- B. Rescinding Request (Removal of posted load limits)
 - 1. Current Bridge Asset Name
 - 2. SNBI Bridge Number/SFN
 - 3. Feature intersected (over or under bridge)
 - 4. Existing posting (weight limits currently in effect for each of the Ohio Legal Loads or EV loads)
 - 5. Date existing posting was effective
 - 6. Explanation as to why the posting restrictions can now be removed
 - 7. New load rating summary for the rehabilitated or new structure, if applicable
- C. Change Request (Revision of Existing Posted Limits)
 - 1. Current Bridge Asset Name
 - 2. SNBI Bridge Number/SFN
 - 3. Feature intersected (over or under bridge)
 - 4. Existing posting (weight limit currently in effect for each of the Ohio Legal Loads)
 - 5. Revised posting request (revised weight limit for each of the Ohio Legal Loads or EV loads)
 - 6. Date of existing posting was effective
 - 7. Explanation as to why posting change is necessary

919.7 POSTING FOR EMERGENCY VEHICLES

When a bridge is analyzed and determined to be incapable of carrying full loads of the EVs, and the bridge is not posted for the reduced commercial legal loads, it shall then be posted for the reduced weight limits of EVs that can cross the bridge without causing stresses above safe limits. MUTCD R12-7 sign (shown in BDM 919.7-1) shall be used. The single axle weight limit shall be for the maximum load of a single axle. The tandem weight limit shall be the maximum load on a tandem or dual axle. The Gross shall be the maximum GVW.

If a bridge is posted for reduced commercial legal loads, it shall not be posted for EVs simultaneously. Sign placement shall be in accordance with the TEM 201-9.

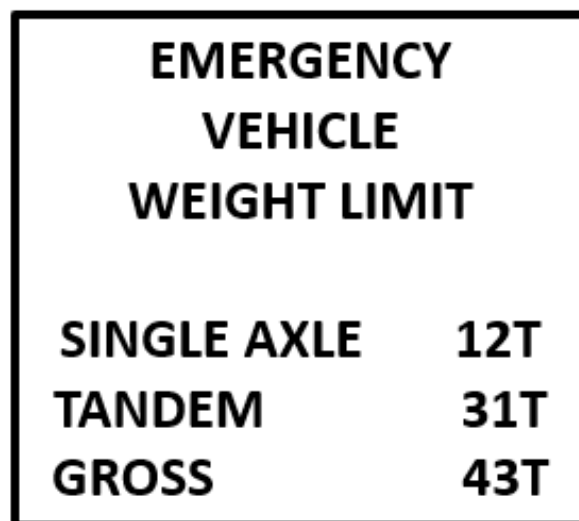


Figure 919.7-1: EMERGENCY VEHICLE SIGN (R12-7)

920 SOFTWARE TO BE USED FOR LOAD RATING OF ODOT BRIDGES

920.1 LIST OF ODOT PREFERRED LOAD RATING PROGRAMS

The bridge models developed during the load rating analyses are being used by ODOT Superload system to analyze and process hundreds of overload-permits every day. Superload system is configured to analyze bridges on a route in a fast and efficient batch process and minimize the permit analysis turnaround time. ODOT uses preferred programs to maintain and update the bridge models in the Superload system. ODOT has licenses of a few other load rating and analysis programs but not every analysis program is interfaced with the Superload system. The bridges which are not modeled using preferred programs require to be analyzed manually for permit loads, which substantially increases the process time.

- A. AASHTOWare BrR: BrR is a load rating and analysis program developed and licensed by AASHTO. BrR can rate the bridges by LRFR and LFR methods. BrR can load rate a variety of bridge types including reinforced concrete box culverts and curved beam bridges. ODOT has a site license for BrR. Through a special pricing option, Counties, Cities and Consultants working on bridges in Ohio can obtain a stand-alone single user license of the BrR program from AASHTO. Please contact the ODOT Bridge Rating Engineer in the Central Office, should you need help.
- B. AASHTOWare BrD: BrD is a bridge design check program developed and licensed by AASHTO. BrD can perform design check of a bridge by LRFR and LFR methods for compliance with current AASHTO Specifications. BrD program is fully compatible with BrR program, as data files created in BrD program can be used in BrR program to load rate a bridge. ODOT has a site license for BrD from AASHTO. Through a special pricing option, Counties, Cities and Consultants working on bridges in Ohio can obtain a stand-alone single user license of the BrD program from AASHTO. Please contact the ODOT Bridge Rating Engineer in the Central Office, should you need help.

920.2 OTHER LOAD RATING PROGRAMS

For the analysis of the structures that cannot be accurately modeled using ODOT's Preferred Load Rating Programs stated in 920.1, contact the OSE Bridge Rating Engineer for software pre-approval prior to performing any load rating. The Department will not accept load ratings performed using any software not on ODOT's preferred load rating program list or pre-approved for a bridge. Currently, ODOT licenses following additional bridge analysis programs:

- A. Midas Civil: Midas Civil is a finite element analysis program by Midas IT Co., Ltd., Use this program to load rate only complex bridges that cannot be accurately modeled using the AASHTO BrR. (<http://www.midasuser.com>).

- B. Eriksson Culvert: This program can load rate reinforced concrete flat-topped 3-sided frames and 4-sided boxes buried under the fill by LRFR and LFR methods. (www.erikssonsoftware.com)
- C. MDX Software: MDX software can only be used to load rate ODOT slab-girder/beam bridges that have horizontal curvature of more than 4 degrees. This program supports Load Factor or Load and Resistance Factor methods. Do not use this program to load rate straight or low curvature bridges that can be accurately modeled using the AASHTO BrR. (<http://www.mdxsoftware.com/>)
- D. DESCUS I: Design and Analysis of Curved I-girder Bridge Systems, marketed by OPTI-MATE, Inc. Use the most current version of the software; (www.opti-mate.com)

921 LOAD RATING REPORT SUBMISSION

The load rating reports shall be submitted to the ODOT project manager (PM), ODOT DBE or the respective owner (in case of a non-ODOT bridge). For an ODOT or ODOT-let bridge, the DBE or the PM shall send electronic copies of all files, including the bridge plans, rating model input files, BR100 and load rating report files to the OSE for review.

For the design check of new (proposed) bridges in accordance with BDM 925, submit the following as a minimum:

1. Error-free electronic input files of the load-rating models
2. Proposed bridge plans and shop drawings (if applicable)
3. Supporting calculations (if applicable)
4. A screenshot showing the design load rating factors at both the inventory and operating levels

The final load rating submission shall include:

- a) Electronic copies of all input data files of final bridge rating models, complete, error free and ready to run.
- b) Electronic copy of the BR100 form. BR100 form shall be submitted both in PDF and Excel formats. The PDF copy of the BR100 shall be signed, sealed, and dated by an Ohio Registered Professional Engineer.
- c) Electronic copy of the Load Rating Report including BR100 form. The PDF copy of the Load Rating Report shall be signed, sealed, and dated by an Ohio Registered Professional Engineer.

For existing bridges, the report shall state how the material properties were determined. Any specific details about the current conditions and bridge geometry shall be listed.

The report summary must list final inventory and operating loads and ratings of each structure unit, and the final ratings of the entire bridge summarized in a tabular form on BR100. The controlling stress (flexure, shear, tension, compression, etc.), member, location and limit state shall be stated in the Comment section of BR100. For complex structures, rating factors of critical members shall also be included in the rating report.

The load rating summary (BR100) shall include the names of the rater and an independent reviewer. One of them must have an active PE license with the Ohio Board of Professional Engineers and Surveyors. PE Number(s) shall be added.

The engineer who performed the load rating shall be responsible for incorporating any changes in the input files recommended by ODOT subsequent to its review.

An example of a Bridge Load Rating Report Summary (BR100) is shown in BDM Figure 921-1.

An example of a Bridge Load Rating Report Summary (BR100) is shown in BDM Figure 921 1.

 BRIDGE LOAD RATING SUMMARY REPORT OFFICE OF STRUCTURAL ENGINEERING OHIO DEPARTMENT OF TRANSPORTATION										
SNBI Bridge Number (SFN)	Bridge Asset Name		District	GPS Coordinates						
3601951	HIG-124-14.06		9	LATITUDE:	LONGITUDE:					
				39.178551	-83.544244					
Original Year Built/Rebuilt	Total Br Length	Material-Continuity	Span Type	Feature Intersected						
1954/2001	199.5	S01-2	G02	Rocky Fork Creek						
SPECIAL ASSUMPTIONS & COMMENTS	Modeled from rehab plans (1999), ODOT standard drawings, and inspection records. 3-Span Continuous (15 000mm, 29 900, 15 000 c/c brgs) steel beams with composite reinforced concrete deck on a tangent alignment. No additional wearing surface included (Monolithic). Member =B1 & B3 (Fascia) @ Span 2 - (30.0%) (98.26ft along the member) : Limit State = STRENGTH-I Steel Flexure Stress Min Rating = 1.334, Live Load = SU7,									
	Please type or select on right using drop down arrow									
Load Rating Purpose:	Update Analysis Model and Software									
Bridge Appraisal Rating (0-9):	7 - Good									
Load rating Software:	AASHTO BrR									
Software version:	7.6.1									
Rating Source:	Plan information available for load rating analysis									
Load Rating Method:	LRFR - Load & Resistance Factor Rating									
Design Loading:	HS20									
STRUCTURE RATING SUMMARY										
OHIO & AASHTO LEGAL VEHICLES					DESIGN AND OPERATING RATINGS					
Legal Load	GVW (Tons)	No of Axles	Rating Factor RF	Safe Weight (Tons)	Loading Type	Rating by RF				
S-2F1	15	2	3.038	15.00	HL93 Loading	Inventory 0.846, Operating 1.097				
S-3F1	23	3	2.039	23.00	Recommendation	No Load Posting is Recommended				
S-5C1	40	5	1.663	40.00	Sign Posting Recommendation:					
Type 3	25	3	2.010	25.00						
Type 3-3	40	6	1.632	40.00						
Type 3S2	36	5	1.677	36.00						
SPECIALIZED HAULING VEHICLES (SHV)										
SU4	27	4	1.788	27.00						
SU5	31	5	1.619	31.00						
SU6	34.75	6	1.454	34.75						
SU7	38.75	7	1.334	38.75						
EMERGENCY VEHICLES (EV) Check box if rating for EV3 ☒					Permit Load (PL) Analysis **					
					Loading Type	GVW (Tons)	No of Axles	Rating Factor	Calculated Load (Tons)	
EV2	28.75	2	1.948	28.75	S-PL 60T	60	6	1.839	111.54	
EV3	43	3	1.344	43.00	S-PL 65T	65	7	1.390	90.35	
Controlling Legal Load RF					133%	1.33	PL Analysis Method			
					Load & Resistance Factor Rating (LRFR)					
AGENCY/FIRM/OFFICE		ODOT-OSE								
	Name	PE Number	Phone Number	Email	Report Date:	2025-07-15				
Rated By	Amjad Waheed	55865	6147529972	amjad.waheed@dot.ohio.gov						
Reviewed By	Hemapriya Krishnaraj	89950	6143874117	hemapriya.krishnaraj@dot.ohio.gov						

ODOT bridges to be analyzed for permit trucks by policy.

BR-100 (7/2025)

Figure 921-1: BR100 - LOAD RATING SUMMARY FORM

922 LOAD RATING USING AASHTO BrR PROGRAM**922.1 GENERAL**

BrR is a load rating program licensed from AASHTO. BrR runs on Microsoft Windows and can load rate a variety of bridges by ASR, LFR as well as LRFR methods.

Include all legal loads, including AASHTO loads, SHVs, EVs, and Ohio loads in the rating and posting analyses. Also, analyze all ODOT bridges for permit loads. For non-ODOT bridges, check with the responsible owner before analyzing for permit loads. The BrR vehicle library and rating templates can be customized to include all loads. Alternatively, ODOT's BrR library and rating templates can be requested from OSE.

Always create girder system models in BrR. For complex framing plans, consult with the Rating Engineer in OSE.

The current and future wearing surface loads shall be applied on typical section of the system model. Do not apply wearing surface loads as member loads on individual members of the system model.

Specify concrete deck reinforcement, for multi-span steel beam/girders with composite concrete deck. Make sure extra reinforcement in composite decks over piers is also specified in BrR models.

Compute and fill out all live load distribution factors for LFD and LRFR methods in the BrR model. Make sure you verify the BrR models run without Secs for both LFD and LRFR methods and produce reasonable rating factors.

922.2 BrR LOAD RATING REPORT SUBMISSION

The load rating report shall be submitted to the Project Manager, the District Bridge Engineer or the non-ODOT bridge owner.

922.3 BrR COMPUTER INPUT AND OUTPUT FILES

Submit the error-free and working electronic copies of the input file exported as an "XML" file. Do not submit output files of BrR program unless directed to do so.

923 LOAD ANALYSIS USING ERIKSSON-CULVERT PROGRAM**923.1 GENERAL**

Eriksson Culvert program can be used to analyze reinforced concrete three-sided flat-topped frames and four-sided box sections. Include all legal loads, including AASHTO loads, SHVs, EVs, and Ohio loads in the rating and posting analyses. Also, analyze all ODOT bridges for permit loads. For non-ODOT bridges, check with the responsible owner before analyzing for permit loads.

If haunch dimensions vary, use the smallest dimension in the analysis.

923.2 ERIKSSON CULVERT LOAD RATING REPORT SUBMISSION

The load rating report and input files shall be submitted to the Project Manager, the District Bridge Engineer or the non-ODOT bridge owner.

923.3 ERIKSSON COMPUTER INPUT AND OUTPUT FILES

Use the SNBI Bridge Number (SFN) of the bridge with prefix "R" to name and appropriate extension to name the input and output files. For example, if the SNBI Bridge Number of a bridge is 4729854, the input and output file names should be "R4729854.dat" and R4729854.cus and R4729854.xml, etc.

924 LOAD RATING OF BRIDGES USING LRFR SPECIFICATIONS**924.1 APPLICABILITY OF AASHTO DESIGN SPECIFICATIONS**

This Section is consistent with the current *AASHTO LRFD Bridge Design Specifications*. Where this Section is silent, the current *AASHTO LRFD Bridge Design Specification* shall govern.

924.2 GENERAL LOAD RATING EQUATION

The following general equation shall be used in determining the load rating of each component and connection subject to a single force effect (axial force, flexure, or shear) [MBE 6A.4.2]:

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P) - (\gamma_{PL})(PL)}{(\gamma_{LL})(LL)(1 + IM/100)}$$

For Strength Limit States:

$$C = \phi_c \cdot \phi_s \cdot \phi \cdot R_n$$

Where the following lower limit shall apply:

$$\phi_c \cdot \phi_s \geq 0.85$$

For Service Limit States:

$$C = f_R$$

Where:

C = Capacity

DC = Dead load effect due to structural components and attachments

DW = Dead load effect due to wearing surface and utilities

f_R = Allowable stress specified in LRFD Code

IM = Dynamic load allowance expressed as percentage (%)

LL = Live Load effect

P = Permanent loads other than dead loads, such earth pressure, shrinkage etc.

PL = Pedestrian Load effect only to be applied when a sidewalk is present

RF = Rating Factor

R_n = Nominal member resistance

γ_{DC} = Load factor for DC load

γ_{DW} = Load factor for DW load

γ_P = Load factor for P load = 1.0

γ_{LL} = Evaluation live load factor

γ_{PL} = Load factor for PL load = 1.0

ϕ_c = Condition factor

ϕ_s = System factor

ϕ = LRFD Resistance factor

For Limit States and factors see BDM Section 924.3.

924.3 LIMIT STATES AND LOAD FACTORS FOR LOAD RATING BY LRFR METHOD

Strength is the primary limit state for load rating; Service and Fatigue limit states are selectively applied in accordance with the provisions of *AASHTO Manual of Bridge Evaluation* [MBE 6A.4.2]:

For Inventory and Operating Rating for AASHTO HL93 Loading, use the following limit states and load factors:

Table 924.3-1: LRFR Design Load Limit States and Load Factors [MBE Table 6A.4.2.2-1]

Bridge Type	Limit State	Dead Load γ_{DC}	Dead Load γ_{DW}	HL93 Loading	
				Inventory γ_{LL}	Operating γ_{LL}
Steel	Strength I	1.25	1.50	1.75	1.35
	Service II	1.00	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.80	
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35
	Service III	1.00	1.00	0.80*	
Wood	Strength I	1.25	1.50	1.75	1.35

*Use $\gamma_{LL} = 1.0$ when time-dependent losses are calculated per *LRFD Design Article 5.9.5.4*

For rating for Legal Loads, use the following limit states and load factors:

Table 924.3-2: Legal Loads Limit States and Load Factors [MBE Table 6A.4.4.2.3a-1]

Bridge Type	Limit State	Dead Load γ_{DC}	Dead Load γ_{DW}	Ohio Legal Loads γ_{LL}
Steel	Strength I	1.25	1.50	1.45
	Service II	1.00	1.00	1.30
Reinforced Concrete	Strength I	1.25	1.50	1.45
Prestressed Concrete	Strength I	1.25	1.50	1.45
	Service III	1.00	1.00	1.00
Wood	Strength I	1.25	1.50	1.45

For Rating for Special and Permit Loads, use the following limit states and load factors:

Table 924.3-3: Permit Load Limit States and Load Factors [MBE Table 6A.4.5.4.2a-1]

Bridge Type	Limit State	Dead Load γ_{DC}	Dead Load γ_{DW}	Routine Permit Loads* γ_{LL}	Permit or Special Loads** γ_{LL}
Steel	Strength II	1.25	1.50	1.40	1.20
	Service II	1.00	1.00	1.00	1.00
Reinforced Concrete	Strength II	1.25	1.50	1.40	1.20
	Service I	1.00	1.00	1.00	1.00
Prestressed Concrete	Strength II	1.25	1.50	1.40	1.20
	Service I	1.00	1.00	1.00	1.00
Wood	Strength II	1.25	1.50	1.40	1.20

* For unlimited crossings/multiple trips

**For all permit or special loads, single trip & limited crossing, other than routine permits

***For EV3, use $\gamma_{LL} = 1.20$ for bridges on Interstate system; use $\gamma_{LL} = 1.10$ for all others

924.4 DYNAMIC LOAD ALLOWANCE (IM)

- A. A dynamic load allowance of 33% shall be used for all non-buried bridges, except for fatigue evaluation.
- B. For fatigue evaluation, a dynamic load allowance of 15% shall be used.
- C. Dynamic load allowance shall only be applied to the truck or tandem portion of HL93 loading (dynamic load allowance shall not be provided to the lane portion).
- D. Dynamic load allowance shall not be applied to wood components of a bridge.
- E. Dynamic allowance may be ignored for slow moving (speed less than 10 mph), special or permit loads under controlled conditions.
- F. For buried bridges, dynamic allowance (IM) shall be taken as:

$$IM = 33 (1.0 - 0.125 DE) \geq 0\% \quad [AASHTO 3.6.2.2-1]$$

Where:

DE = the minimum depth of cover above the structure (ft.)

924.5 CONDITION FACTOR (ϕ_c)

A Condition Factor shall be applied to the calculated capacity of the structure, as follows:

Table 924.5-1: Condition Factors [MBE 6A.4.2.3]

Structural Condition of a Member	Bridge Appraisal Rating	Condition Factor ϕ_c
Good or Satisfactory	6 or higher	1.00
Fair	5	0.95
Poor	4 or lower	0.85

924.6 SYSTEM FACTOR (ϕ_s)

System factors are multiplied to the nominal resistance to reflect the level of redundancy of the complete superstructure [MBE 6A.4.2.4]. Bridges that are less redundant will have their factored member capacities reduced.

The following system factors may be used for Flexural and Axial Effects:

Table 924.6-1: System Factors [MBE 6A.4.2.4]

Superstructure Type	ϕ_s
Welded members in two-girder/truss/arch bridges	0.85
Riveted members in two-girder/truss/arch bridges	0.90
Multiple eye bar members in truss bridges	0.90
Three-girder bridges with girder spacing 6-ft.	0.85
Four-girder bridges with girder spacing $\leq$ 4-ft.	0.95
Floor beams with spacing $>$ 12.0-ft. and non-continuous stringers	0.85
Redundant stringer subsystems between floor-beams	1.00
All other girder and slab bridges	1.00

924.7 RESISTANCE FACTOR (ϕ)

Resistance factor (ϕ) for the load rating has the same value as for a new design as given in *AASHTO LRFD Specification*. Also, $\phi = 1.0$ for all non-strength limit states [MBE C6A.4.2.1]. See appropriate section in the LRFD Specification for recommended values for resistance factors [LRFD 5.5.4.2, 6.5.4.2, 8.5.2, 12.5.5].

Some of the commonly used Resistance Factors are given here:

Table 924.7-1: Resistance Factors

Type	ϕ
Tension controlled reinforced concrete section	0.90
Tension controlled prestressed concrete section	1.00
Shear and torsion in normal weight concrete	0.90
Flexure in steel	1.00
Shear in steel	1.00
Axial Compression in steel only	0.95
Axial Compression in composite	0.90
Shear connectors, steel	0.85
Web crippling, steel	0.80
For block shear	0.80
For shear rupture in connection element	0.80
For weld metal in partial penetration and fillet weld	0.80
Flexure in wood	0.85
Shear in wood	0.75
Wood connections	0.65
RC cast-in-place buried box structures in flexure	0.90
RC cast-in-place buried box structures in shear	0.85
RC precast buried box structures in flexure	1.00
RC precast buried box structures in shear	0.90
RC precast buried 3-sided structures in flexure	0.95
RC precast buried 3-sided structures in shear	0.90
Structural steel pipe, minimum wall area and buckling	1.00
Structural steel pipe, minimum longitudinal seam strength	0.67

924.8 EFFECT OF SKEW

Effect of skew on the distribution of live loads shall be considered according to *AASHTO LRFD Specifications (LRFD 4.6.2.2.2 and 4.6.2.2.3)*.

925 LOAD RATING OF NEW BRIDGES**925.1 LOADS TO BE USED FOR LOAD RATING**

- A. All new and replacement bridges, whose preliminary design was started **after October 1, 2010**, and requiring load rating, shall be load rated by the AASHTO LRFR method to comply with FHWA requirements. The load to be used for inventory and operating rating based on the LRFR method shall be AASHTO's HL93 loading (truck and lane or tandem and lane).
- B. Newly designed timber bridges shall be load rated using the LRFR method.
- C. All legal loads used for analysis shall have transverse spacing, between the centerline of wheels and wheel-groups of 6-ft.
- D. Long span bridges shall use the special load configurations given in BDM Section 918.
- E. The inventory and operating ratings for AASHTO design loads inventory and operating ratings for the AASHTO HL93 loading shall be expressed in terms of rating factors expressed in terms of rating factors, rounded to the nearest third decimal point.
- F. The ratings for each of the legal loads shall be expressed in terms of rating factors of respective legal load, rounded to the nearest third decimal point. The controlling legal load rating factor shall be the smallest rating factor of all legal loads.
- G. For a bridge being designed, analysis shall be performed, and rating factors shall be checked at two stages:
 - 1) **Design Check:** Perform the analysis of the bridge for the Design Loading specified in plans or design documents. Apply the future wearing surface load (FWS), as specified in the design plans or design documents. The minimum acceptable inventory RF for specified design loading at inventory level is 1.00. If the RF at inventory level (for design loading) is less than 1.00, the design shall be revised to bring up the inventory RF to 1.00 or higher. Submit the new or proposed bridges for design check review of ODOT.
 - 2) **Final Load Rating:** After the design check is completed and approved, load rate the bridge for the current loading conditions (without adding the FWS load). Submit the final load rating report (BR100 form) for the bridge based on the current loading conditions.
- H. The Bridge Owner may also require load rating to be done for special loads in addition to those specified above. The owner shall include full configurations of the special load used in the analysis, including, but not limited to, axle weights and spacing, number of tires on each axle, tire gauges and overall dimensions of the load. All special loads are to be analyzed by the method specified LRFR method of analysis per BDM Section 917 unless specified otherwise by the Owner.

925.2 LOAD RATING OF NEW BURIED BRIDGES**925.2.1 GENERAL**

- A. All new pre-cast and cast-in-place bridge structures under the fill of two feet or more and supporting vehicular loads shall be load rated according to the provisions of this Section.
- B. The ODOT preferred load rating programs shall be used to load rate the structure.
- C. For structures under 6.5-feet or more fill on top, see BDM Section 911.

925.2.2 HOW THE LOAD RATING SHALL BE DONE

- A. All spans that are designed to carry vehicular traffic shall be load rated.
- B. The load rating analysis shall be based on the final design plans. At the inventory or design level, the load rating shall be equal to or greater than the Design Loading.
- C. All structural members shall have actual net section and current conditions incorporated into the member analysis.
- D. All dead loads are to be calculated based on the final field conditions. Future dead loads shall be applied unless directed otherwise.
- E. Unless more accurate soil data exists, calculate the rating based on a lateral pressure as specified in AASHTO.
- F. Apply a live load surcharge as per AASHTO.
- G. The effect of soil-structure interaction shall be considered as per AASHTO.
- H. Assume hinged connections between the walls and the top and bottom slabs unless there is adequate concrete reinforcement continuous between the slab and the walls at the joint. In that case, continuity between the slab and the walls can be assumed.

925.2.3 CAST-IN-PLACE CONCRETE BOX & FRAME STRUCTURES

- A. Cast-in-place bridges shall be design-checked, and load rated by the designer of the bridge.
- B. The AASHTO BrR program shall be used to load rate the structure.

925.2.4 PRECAST CONCRETE STRUCTURES**925.2.4.1 PRECAST CONCRETE BOXES OF SPAN GREATER THAN 12-FT**

- A. The load rating analysis for precast concrete box culverts of spans up to 20 feet is provided in the Supplemental Specifications [SS940](#). The manufacturer shall submit the actual information about the dimensions and reinforcing bars/cage to the OSE along with the shop drawings before the placement of structure.
- B. The load rating analysis will be performed by the OSE using the AASHTO BrR program.

925.2.4.2 PRECAST CONCRETE BOXES OF SPAN EQUAL TO OR LESS THAN 12-FT

- A. The manufacturer shall submit the actual information about the dimensions and reinforcing bars/cage to the OSE along with the shop drawings before the placement of structure.
- B. The design check and load rating analysis will be performed by the OSE using the AASHTO BrR program.

925.2.4.3 PRECAST FRAMES, ARCHES, AND CONSPANS & BEBO TYPE STRUCTURES

- A. The design check for the design loading (including FWS loading if specified) and the final load rating analysis shall be performed by the manufacturer. Refer to BDM Section 925.1.
- B. The load rating report along with the completed BR100 shall be submitted along with the shop drawings before the placement of the precast units.
- C. Use the design software to load rate the bridge.
- D. Provide all input files to the bridge owner.

925.2.4.4 PRECAST CONCRETE BOXES NOT DESIGNED BY ASTM OR SS940 SPECIFICATIONS

- A. Precast concrete boxes, not designed and manufactured according to ASTM or ODOT Supplemental Specifications, SS940 shall be designed and load rated by the manufacturer.
- B. AASHTO BrR program shall be used to load rate the precast structure. BrR input files shall be submitted to the bridge owner.
- C. The load rating report along with the completed BR100 in Excel and PDF formats shall be submitted along with the shop drawings before the placement of the precast units.

925.3 LOAD RATING OF NON-BURIED STRUCTURES**925.3.1 GENERAL**

- A. All structures, including arch structures, frames, box sections, etc., having a fill or pavement thickness of less than 2-ft on top shall be load rated according to the provisions of this Section.
- B. All main structural members of the superstructure affected by the live load shall be analyzed.

925.3.2 HOW THE LOAD RATING SHALL BE DONE

- A. The designer shall analyze and load rate all spans that are designed to carry vehicular traffic.
- B. The design check and load rating analysis shall be based on the final design plans.
- C. All members shall have actual net section and current conditions incorporated into the member analysis.
- D. Bridge members designed as non-composite with the deck slab should be analyzed as non-composite.
- E. All dead loads are to be calculated based on the actual field conditions.
- F. The total thickness of the composite concrete slab shall be used in load rating for the calculations of section properties. Do not subtract for the monolithic wearing surface.
- G. Live load distribution factors, as defined in the current AASHTO LRFD, shall be used.

925.3.3 WHEN THE BRIDGE LOAD RATING SHALL BE DONE

The load rating of new bridges shall be done as per the following sub-sections:

925.3.3.1 BRIDGES DESIGNED UNDER MAJOR OR MINOR PLAN DEVELOPMENT PROCESS

For bridges designed under the Major or Minor Plan Development Process (PDP), perform the design check and the load rating, and include with the load rating report in the Stage 2 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 2 and prior to contract sale, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the load rating report to the District Bridge Engineer.

925.3.3.2 BRIDGES DESIGNED UNDER MINIMAL PLAN DEVELOPMENT PROCESS

For bridges designed under the Minimal Plan Development Process (PDP), perform the design check and the load rating, and include with the load rating report in the Stage 3 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 3 and prior to contract sale, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.

925.3.3.3 BRIDGES DESIGNED UNDER DESIGN-BUILD PROCESS

Unless otherwise indicated in the project scope, perform the design check and the load rating report for bridges designed as part of a Design Build project with the Stage 2 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 2, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.

925.3.4 BRIDGES DESIGNED UNDER VALUE ENGINEERING CHANGE PROPOSAL

For bridges re-designed under a Value Engineering Change Proposal (VECP), perform the design check and the load rating of the altered bridge design, and submit to the District Construction Engineer (DCE) with the Final VECP submission. The DCE will supply this information to the District Bridge Engineer.

926 LOAD RATING OF EXISTING BRIDGES**926.1 LOADS TO BE USED FOR LOAD RATING**

- A. Existing bridges shall be load rated at inventory and operating rating by either LFR or LRFR method with the prior approval of the bridge owner. When the LFR method is used, the load for inventory and operating rating shall be AASHTO HS20. When the LRFR method is used, the load for inventory and operating rating shall be AASHTO HL93. The default load rating method shall be LRFR starting June 13, 2019.
- B. Existing bridges shall also be load rated for all the legal loads at the operating level. Use the same method (LFR or LRFR) consistent with the A above.
- C. Existing timber bridges may be load rated using the ASR method.
- D. All legal loads used for analysis shall have transverse spacing, between the centerline of wheels or wheel-groups, of 6-ft.
- E. For long span bridges, refer to BDM Section 917.
- F. The inventory and operating ratings for the AASHTO HL93 or HS20 loading shall be expressed in terms of rating factors, rounded to the nearest three decimal points.
- G. The ratings for each of the legal loads shall be expressed in terms of rating factors of respective legal load, rounded to the nearest third decimal point. The controlling legal load rating factor shall be the smallest rating factor of all legal loads. The Ohio percent of legal shall be rounded off to the nearest 5 (i.e., smallest RF multiplied by 100).
- H. The Bridge Owner may also require load rating to be done for special loads in addition to those specified above. The Owner shall include full configurations of the special load used in the analysis, including but not limited to, axle weights and spacing, number of tires on each axle, tire gauges and overall dimensions of the load. All special loads are to be analyzed by the LRFR method of analysis at the operating level as per BDM Section 917 unless specified otherwise by the Owner.

926.2 LOAD RATING OF BRIDGES TO BE REHABILITATED**926.2.1 HOW THE LOAD RATING SHALL BE DONE**

- A. The designer shall analyze and load rate all spans that are designed to carry vehicular traffic.
- B. The load rating analysis shall be based on the final design or as-built plans.
- C. Future wearing surface dead loads shall be applied in load rating calculations for design checks and should not be included in the final load ratings.
- D. All members shall have actual net section and current conditions incorporated into the members' analysis. Any known section losses, defects or damage to the existing structural members shall be considered in the rating analysis.
- E. Bridge members designed as non-composite with the deck slab should be analyzed as non-composite.
- F. Structures to be rehabilitated shall be analyzed using the original design plans, the actual field conditions, and all major changes in the final rehabilitation plans.
- G. A complete review of all the available inspection information, as well as a thorough site inspection of the existing bridge, must be performed to establish the current conditions prior to proceeding with the analysis.
- H. The total thickness of the composite concrete slab shall be used in load rating for the calculations of section properties. Do not subtract for the monolithic wearing surface.
- I. Live load distribution factors, in accordance with the governing AASHTO specifications, shall be used.
- J. For existing bridges, the rater should review the original design plans as the first source of information for material strengths and stresses. If the material strengths are not explicitly stated on the design plans, ODOT Construction and Material Specifications ([C&MS](#)) applicable at the time of the bridge construction shall be reviewed. This may require investigations into old ASTM or AASHTO Material Specifications active at the time of construction.
- K. Ultimate or yield strengths of materials shall be as specified on the original design plans unless it is required in the scope of services to conduct specific tests to determine the material strengths.

- L. General information about allowable stresses in bending/shear and material strengths based on the year of construction is provided in BDM Table 926.2.1-1 and BDM Table 926.2.1-2. These are very conservative values and recommended to be used when the material strengths cannot be determined from the design plans, shop drawings, or C&MS applicable at the time of construction. Any material stresses and strengths specified on the design plans or in the C&MS applicable at the time of construction shall supersede the values given in BDM Table 926.2.1-1, and BDM Table 926.2.1-2 and BDM Table 926.2.1-3.
- M. The rater is cautioned to pay extra attention to the design plans and the year of construction when determining material strengths for structures built during the transition years of BDM Table 926.2.1-1, BDM Table 926.2.1-2 and BDM Table 926.2.1-3 (e.g. for member type SS, years 1964-68, or 1988-93, etc.), as materials of the newer (or older) specifications may have been substituted.

Table 926.2.1-1: Custom Allowable Stresses in Bending

Material of Construction	Year of Construction	Type of Rating							
		Fy / Fc' (ksi)	Fy / Fc' (MPa)	Inventory (ksi)	Inventory (MPa)	Operating (ksi)	Operating (MPa)	Posting (ksi)	Posting (MPa)
Structural Steel (SS),(CSC)	< 1900	26.00	179	14.00	97	19.00	131	19.00	131
	1901 To 1930	30.00	207	16.00	110	22.00	152	22.00	152
	1931 To 1965	33.00	228	18.00	124	25.00	172	25.00	172
	1966 To 1990	36.00	248	20.00	138	27.00	186	27.00	186
	1991 To Date	50.00	345	27.00	186	37.50	259	37.50	259
Reinforcing Steel (RC)	< 1935	32.00	221	16.00	110	24.00	165	24.00	165
	1936 To 1950	36.00	248	18.00	124	27.00	186	27.00	186
	1951 To 1983	40.00	276	20.00	138	30.00	207	30.00	207
	1984 To Date	60.00	414	24.00	165	36.00	248	36.00	248
Prestress. Strands (Fs') (CPS),(PSC)	All Years	270.0	1862	-	-	-	-	-	-
Cast-in-Place Reinf. Conc. (Compression in Bending) (RC),(CSC)	< 1930	2.00	14	0.70	5	1.30	9	1.30	9
	931 To 1950	3.00	21	1.00	7	1.50	10	1.50	10
	1951 To 1980	4.00	28	1.30	9	2.00	14	2.00	14
	1981 To Date	4.50	31	1.50	10	2.20	15	2.20	15
Prestressed Concrete (Fc') (PSC),(CPS)	All Years	5.50	38	-	-	-	-	-	-
Cast-in-Place Comp. Slab for Prestress. Conc. (Fc') (CPS)	All Years	4.00	28	-	-	-	-	-	-
Timber (fb) (TMB)	All Years	-	-	1.6	11	2.128	15	2.128	15
Cast-in-Place Slab for Composite Reinforced Concrete	< 1930	2.00	14	0.70	5	1.30	9	1.30	9
	1931 To 1950	3.00	21	1.00	7	1.50	10	1.50	10
	1951 To 1980	4.00	28	1.30	9	2.00	14	2.00	14
	1981 To Date	4.50	31	1.50	10	2.20	15	2.20	15

Table 926.2.1-2: Custom Allowable Stresses in Shear

Material of Construction	Year of Construction	Type of Rating							
		Fy / Fc' (ksi)	Fy / Fc' (MPa)	Inventory (ksi)	Inventory (MPa)	Operating (ksi)	Operating (MPa)	Posting (ksi)	Posting (MPa)
Structural Steel (SS),(CSC)	< 1900	26.00	179	8.50	59	11.50	79	11.50	79
	1901 To 1930	30.00	207	9.50	66	13.50	93	13.50	93
	1931 To 1965	33.00	228	11.00	76	15.00	103	15.00	103
	1966 To 1990	36.00	248	12.00	83	16.00	110	16.00	110
	1991 To Date	50.00	345	17.00	117	22.50	155	22.50	155
Reinforcing Steel (RC)	< 1935	32.00	221	16.00	110	24.00	165	24.00	165
	1936 To 1950	36.00	248	18.00	124	27.00	186	27.00	186
	1951 To 1983	40.00	276	20.00	138	30.00	207	30.00	207
	1984 To Date	60.00	414	24.00	165	36.00	248	36.00	248
Cast-in-Place Reinforced Conc. (RC),(CSC)	< 1930	2.00	14	0.70	5	1.30	9	1.30	9
	1931 To 1950	3.00	21	1.00	7	1.50	10	1.50	10
	1951 To 1980	4.00	28	1.30	9	2.00	14	2.00	14
	1981 To Date	4.50	31	1.50	10	2.20	15	2.20	15
Prestressed Concrete (Fc') (PSC),(CPS)	All Years	5.50	38						
Timber (Horizontal Shear Stress) (fb) (TMB)	All Years	-	-	0.09	1	0.12	1	0.12	1

Table 926.2.1-3: Custom Allowable Stresses for Fasteners

Material	Constructed	LRFR	ASD		
		ΦF_v	Inventory	Operating	Posting
Riveted	<1939	18.0 ksi	9.5 ksi	12.5 ksi	12.5 ksi
	1936 to 1950	21.0 ksi	11.0 ksi	15.0 ksi	15.0 ksi
	1950 to Date	25.0 ksi	13.5 ksi	18.0 ksi	18.0 ksi
Bolted – Bearing	< 1965	17.0 ksi	11.0 ksi	15.0 ksi	15.0 ksi
	1965 to Date	36.5 ksi *	NA	NA	NA
		32.0 ksi **	NA	NA	NA
Bolted – Slip Critical	< 1965	NA	NA	NA	NA
	1965 to Date	17.0 ksi *	15.0 ksi *	20.0 ksi *	20.0 ksi *
		15.0 ksi **	13.0 ksi **	17.5 ksi **	17.5 ksi **

* Diameter $\leq 1"$.** Diameter $> 1"$.**926.2.2 WHEN THE BRIDGE LOAD RATING SHALL BE DONE**

The load rating of bridges to be rehabilitated shall be done as per following:

926.2.2.1 BRIDGES DESIGNED UNDER MAJOR OR MINOR PLAN DEVELOPMENT PROCESS

For bridges designed under the Major or Minor Plan Development Process (PDP), perform the load rating, and include the load rating report in the Stage 2 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 2 and prior to contract sale, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.

926.2.2.2 BRIDGES DESIGNED UNDER MINIMAL PLAN DEVELOPMENT PROCESS

For bridges designed under the Minimal Plan Development Process (PDP), perform the load rating, and include the load rating report in the Stage 3 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 3 and prior to contract sale, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.

926.2.2.3 BRIDGES DESIGNED UNDER DESIGN-BUILD PROCESS

Unless otherwise indicated in the project scope, include the load rating report for bridges designed as part of a Design Build project with the Stage 2 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 2, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.

926.2.2.4 BRIDGES DESIGNED UNDER VALUE ENGINEERING CHANGE PROPOSAL

For bridges re-designed under a Value Engineering Change Proposal (VECP), perform a load rating of the altered bridge design, and submit the load rating report to the District Construction Engineer (DCE) with the Final VECP submission. The DCE will supply this information to the District Bridge Engineer.

926.2.3 LOAD RATING OF EXISTING BURIED BRIDGES**926.2.3.1 CAST-IN-PLACE STRUCTURES**

- A. Cast-in-place bridges shall be load rated by the designer of the bridge.
- B. The AASHTO BrR program shall be used to load rate the structure.

926.2.3.2 PRECAST BOXES OF SPAN GREATER THAN 12-FT

- A. The design check and the load rating analysis will be performed by the OSE.
- B. The AASHTO BrR program shall be used to load rate the structure.

926.2.3.3 PRECAST BOXES OF SPAN EQUAL TO OR LESS THAN 12-FT

The load rating analysis will be performed by the OSE using AASHTO BrR program.

926.2.3.4 PRECAST FRAMES, ARCHES, AND CONSPANS & BEBO TYPE STRUCTURES

- A. The load rating analysis for any new or replacement precast sections will be performed by the manufacturer; otherwise, the load rating analysis will be performed as per the scope of services.
- B. The load rating report shall be submitted along with the shop drawings before the placement of the units.
- C. The design software shall be used to load rate the bridge.

926.2.3.5 ANALYSIS OF CONCRETE BOX SECTIONS & FRAMES

Unless more accurate soil data exists, calculate the rating based on a lateral pressure as specified in AASHTO.

Apply a live load surcharge according to AASHTO.

The effect of soil-structure interaction shall be considered according to AASHTO.

Assume hinged connections between the walls and the top and bottom slabs unless there is adequate concrete reinforcement continuous between the slab and the walls at the joint. In that case, continuity between the slab and the walls can be assumed.

926.2.4 LOAD RATING OF EXISTING NON-BURIED STRUCTURES

All structures, including flat slabs, arch structures, frames, box sections, etc., having a fill or pavement material of less than 2-ft on top shall be load rated according to the provisions of BDM Sections 912 through 924 (as applicable).

926.3 LOAD RATING OF EXISTING BRIDGES WITH NO REPAIR PLANS

926.3.1 HOW THE LOAD RATING SHALL BE DONE

- A. The rater shall analyze and load rate all spans that are designed to carry vehicular traffic.
- B. Existing structures shall be analyzed using the information from the original design plans and the actual field conditions.
- C. A complete review of all the available inspection information, as well as a thorough site inspection of the existing bridge, must be performed to establish the current conditions prior to proceeding with the analysis.
- D. The bridges rated using design plans shall be noted as such in the load rating report.
- E. Allowable stresses for the working stress and the ultimate or yield strengths of materials for Load Factor ratings shall be as specified on the original design plans unless it is required in the scope of services to conduct specific tests to determine the material strengths.
- F. For existing bridges, the rater should review the original design plans as the first source of information for material strengths and stresses. If the material strengths are not explicitly stated on the design plans, ODOT Construction and Material Specifications ([C&MS](#)) applicable at the time of the bridge construction shall be reviewed. This may require investigations into old ASTM or AASHTO Material Specifications active at the time of construction.
- G. The total thickness of the composite concrete slab shall be used in load rating for the calculations of section properties. Do not subtract for the monolithic wearing surface.
- H. General information about ODOT Allowable Stresses in bending and shear and material strengths based on the year of construction is provided in BDM Table 926.2.1-1 and BDM Table 926.2.1-2.
- I. The rater is cautioned to pay extra attention to the design plans and the year of construction, when determining material strengths for bridges built during transition years of BDM Table 926.2.1-1 and BDM Table 926.2.1-2 (e.g., for member type SS, years 1964-68, or 1988-93, etc.), as materials of the newer (or older) specification may have been substituted.
- J. Any material stresses and specifications specified on the design plans shall supersede the values given in BDM Table 926.2.1-1 and BDM Table 926.2.1-2.

926.3.2 WHEN THE BRIDGE LOAD RATING SHALL BE DONE

The load rating of existing bridges shall be done as per the Scope of Services.

926.3.3 LOAD RATING OF EXISTING BURIED BRIDGES

- A. The load rating analysis will be performed as per the Scope of Services.
- B. Unless specified otherwise, structures shall be load rated for the Loads as per BDM Section 926.2.3.

926.3.4 LOAD RATING OF EXISTING NON-BURIED STRUCTURES

All structures, including flat slabs, arch structures, frames, box sections, etc., having a fill or pavement material of less than 2-ft on top of the structures shall be load rated according to the provisions of BDM Sections 911 through 924 (as applicable).

927 LOAD RATING OF NON-ODOT BRIDGES

- A. Provisions of BDM Section 900 may also be applied to load rating of non-ODOT buried and non-buried bridges at the discretion of the respective bridge owners.

- B. The load rating files and reports of a non-ODOT bridge shall be submitted to the respective bridge owners. The bridge owner shall keep the bridge load rating report in the bridge file for future reference and use.
- C. Based on the field conditions and the load rating calculations, if the rating engineer determines a bridge should be posted for reduced load capacity, the engineer shall forward the recommendation to the respective bridge owner. Applicable portions of BDM Section 917, Bridge Posting for Reduced Load Limits may be followed.
- D. It is the responsibility of the respective bridge owner (or designated consultant/rating engineer) to ensure that the load rating information is finally updated in the ODOT AssetWise.

928 ODOT POLICY ON ASSIGNED LOAD RATINGS

ODOT policy on assigning load rating is in accordance with the AASHTO Manual of Bridge Evaluation, 3rd edition [MBE 6A.1.1] and FHWA Memorandum HIBT-30 dated September 29, 2011.

Wherever possible, ODOT recommends modeling and rate bridges using rating software preferred by ODOT, especially when overweight loads are being regularly permitted to travel on those bridges. This policy covers all current Ohio legal and AASHTO loads and shall be re-visited/revised, if needed.

This policy is applicable to ODOT structures, which otherwise cannot be modeled by BrR or other rating programs available to raters. For non-ODOT bridges, this policy can be applied at the discretion of the bridge owner.

928.1 STRUCTURES DESIGNED BY LFD FOR HS20 OR HIGHER HS LOADING

A structure designed and checked by the Load Factor Design (LFD) method may be assigned following rating factors without performing rating calculations, provided:

1. The bridge was designed and checked using AASHTO LFD method for HS20 or higher loading;
2. The bridge was built in accordance with the original design plans, standard drawings, and ODOT Construction & Material Specifications;
3. No changes to loading conditions or the structure conditions have occurred that could reduce the inventory rating below the design load level;
4. The checked design calculations and relevant computer input and output information are accessible and referenced or included in the individual bridge records. If complete design files have not been retained for a bridge, design plans that clearly identify the design loading and bear the stamp of an Ohio licensed PE may be used by the individual responsible for load rating under FHWA 23 CFR 650.309(d) as the basis for an assigned load rating;
5. A field evaluation has been completed and the bridge has not developed any cracks, excessive deflections, or signs of deterioration;
6. The main structural members of the bridge have not been damaged or repaired since it was originally built;
7. The bridge was appraised “Good” during the last bridge inspection and is neither posted nor closed.

Inventory rating factor for HS20 loading = 1.00

Operating rating factor for HS20 loading = 1.67

Ohio legal loads rating factor (all legal loads)= 1.25 (125%)

Rating factor for EV3 = 1.15

SNBI Item B.LR.04: Code AR

A summary report (BR100) of the assigned load rating, which states that the above specified conditions are met, is to be included in the bridge records and signed & stamped by an Ohio PE. If any of the above conditions cannot be met for a bridge at any point during its service life, load ratings cannot be assigned and must be determined by the methods defined in BDM Section 900.

928.2 STRUCTURES DESIGNED BY LRFD FOR HL93 LOADING

A structure designed and checked by the Load & Resistance Factor Design (LRFD) method may be assigned following rating factors without performing rating calculations, provided:

1. The bridge was designed and checked using AASHTO LRFD method for HL93 loading.
2. The bridge was built in accordance with the original design plans, standard drawings, and ODOT Construction & Material Supplemental specifications.
3. No changes to loading conditions or the structure conditions have occurred that could reduce the inventory rating below the design load level.
4. The checked design calculations and relevant computer input and output information are accessible and referenced or included in the individual bridge records. If complete design files have not been retained for a bridge, design plans that clearly identify the design loading and bear the stamp of an Ohio licensed PE may be used by the individual responsible for load rating under FHWA 23 CFR 650.309(d) as the basis for an assigned load rating.
5. A field evaluation has been completed and the bridge has not developed any cracks, excessive deflections, or signs of deterioration.
6. The main structural members of the bridge have not been hit or repaired since it was originally built.
7. The bridge was appraised “Good” during the last bridge inspection and is neither posted nor closed.

Inventory rating factor for HL93 loading = 1.00

Operating rating factor for HL93 loading = 1.30

Ohio legal loads rating factor (all legal loads) = 1.50 (150%)

Rating factor for EV3 = 1.30

SNBI Item B.LR.04: Code AR

A summary report (BR100) of the assigned load rating, which states that the above specified conditions are met, is to be included in the bridge records and signed & stamped by an Ohio PE. If any of the above conditions cannot be met for a bridge at any point during its service life, load ratings cannot be assigned and must be determined by the methods defined in BDM Section 900.

928.3 CULVERT TYPE BRIDGES DESIGNED USING TABLES OF ASTM C1577 (LRFD), C1433 (LFD), C789 (LFD) AND C850 (LFD)

A culvert type bridge designed for AASHTO design loading using Tables given in any of the above referenced ASTM Specifications may be assigned following rating factors provided:

1. The structure was designed and meets one of the above-mentioned ASTM Specifications.
2. The ASTM Specifications were referenced in the ODOT Construction & Material Specifications (CMS), and the CMS were utilized during construction.
3. No changes to loading conditions or the structure conditions have occurred that could reduce the load rating below the design load level.
4. The design plans and the referenced ASTM Specification are accessible to form a basis for assigned load rating under FHWA 23 CFR 650.309(d).
5. A field evaluation has been completed and the structure has not developed excessive cracks, deflections, or signs of deterioration.

The main structural members of the bridge have not been damaged or repaired since the structure was originally built.

The culvert was appraised “Good” during the last bridge inspection and is neither posted nor closed.

When Culvert is designed for HS20 loading

Inventory Rating Factor for HS20 loading = 1.00

Operating Rating Factor for HS20 loading = 1.67

Ohio legal loads rating factor (all legal loads) = 1.25 (125%)

Rating factor for EV3 = 1.15

SNBI Item B.LR.04: Code AR

When Culvert is designed for HL-93 loading

Inventory Rating Factor for HL93 loading = 1.00

Operating Rating Factor for HL93 loading = 1.30

Ohio legal loads rating factor (all legal loads) = 1.50 (150%)

Rating factor for EV3 = 1.30

SNBI Item B.LR.04: Code AR

A summary report (BR100) of the assigned load rating, which states that the above specified conditions are met, is to be included in the bridge records. BR100 shall be filled out and checked by two different engineers and one of them must be an Ohio PE. If any of the above conditions cannot be met for a bridge at any point during its service life, load ratings cannot be assigned and must be determined by the methods defined in BDM Section 900.

929 LOAD RATING OF BRIDGES WITH INSUFFICIENT INFORMATION

929.1 GENERAL

The purpose of this section is to specify ODOT policy on rating of bridges when complete bridge plans or sufficient information cannot be found in records or archives after all files and records are thoroughly searched. Wherever possible, ODOT recommends modeling and load rate bridges using load rating programs preferred by ODOT. This ODOT policy is applicable to ODOT structures. For non-ODOT bridges, this policy can be applied at the discretion of the bridge owner.

References shall be made to AASHTO Manual of Bridge Evaluation (MBE) Section 6.1.4, which addresses load rating of bridges with unknown structural components and National Cooperative Research Program (NCHRP) Synthesis 571, published in 2022, has stated common practices in the nation to load rate bridges and culverts with missing or incomplete as-built information.

The load rater will use one of the following methods or a combination of methods to load rate bridges without plans or with missing details.

When method A) or B) given in this policy is used to determine the load rating of a bridge, the rating information shall be coded as follows:

SNBI Coding:

Item B.LR.04: Code “EJ”

Item B.LR.05: Code inventory rating factor

Item B.LR.06: Code operating rating factor

Item B.LR.07: Code controlling legal load rating factor

929.2 METHOD A: LOAD RATING USING FIELD MEASUREMENTS AND HISTORICAL DATA (FH)

When bridge design plans or standard drawings used in design are not available or have missing information, determine if field measurements can be made to establish the overall dimensions of the bridge and the member cross-sections to create a bridge rating data model to analyze a bridge to calculate its load rating using one of the software programs preferred by ODOT. Material properties can be assumed based on the historical data and ODOT specifications applicable at the time of the bridge construction (see Bridge Design Manual, BDM Section 926.2). General information about the material strengths based on the ranges of years of construction is provided in BDM Tables 926-1 through 926-3. Extra attention should be paid when the year of construction is close to the start or end of a range as materials during construction might have been substituted. It is recommended to use conservative values.

This method is recommended to be used for most common steel and timber bridges.

929.3 METHOD B: LOAD RATING BASED ON THE DESIGN LOAD AND RATIONAL EVALUATION (DLRE)

When method A) cannot be applied, the load rating of a bridge with insufficient information may be determined using the design load and current Bridge Appraisal Rating (BAR) provided the bridge is in service for some time. Prior to determining the final rating by this method, inspection and maintenance records including historical performance, deterioration, modification, and repair records etc. of the bridge shall be thoroughly reviewed to ensure that structural material, geometry and superimposed dead load have not changed, the appraisal ratings reflect current conditions, and no localized deterioration is present that controls the rating of the entire structure.

Initial rating is based on the assumptions that the bridge is in good condition, has known (or assumed) design loading which was in use when the bridge was built, and has the inventory rating (the capacity minus dead load effect) equivalent to its design loading demand. The operating rating is then calculated from the inventory rating ($OR=1.67*IR$). The inventory rating factor is based on a HS20 truck assuming the design truck produces demand (moment and shear) in proportion to demand of a HS20 truck. The legal and EV rating factors are based on experience and rational evaluation.

*Assuming a bridge has inventory rating equal to GVW of design vehicle

Design Loading	GVW of truck (ton)	Initial Ratings (SNBI)			
		Inventory Rating (RF)*	Operating Rating (RF)*	Legal RF	EV3 RF
Unknown		0.28	0.46	0.350	0.320
H10	10.0	0.28	0.46	0.350	0.320
H15	15.0	0.42	0.70	0.520	0.480
HS15	27.0	0.75	1.25	0.940	0.860
H20	20.0	0.56	0.93	0.690	0.640
HS25	45.0	1.25	2.09	1.500	1.440
HS20	36.0	1.00	1.67	1.250	1.150

* Based on HS20 truck GVW=36 ton

Modify the initial rating by multiplying with Bridge Appraisal Factor (BAF) based on the bridge appraisal rating (BAR).

Modified Rating = Initial rating * BAF

Bridge Appraisal Rating (BAR)	1	2	3	4	5	6	7	8	9
Bridge Appraisal Factor (BAF)	0.00	0.30	0.55	0.85	1.00	1.00	1.00	1.00	1.00

BAR is the lowest of the Deck, Superstructure, Substructure or Culvert appraisal ratings.

When the modified legal RF is determined to be below the posting threshold, the bridge shall be posted for the reduced legal loads determined by multiplying the GVW of each load by the modified legal RF.

When ratings are determined by this method, they shall not be used to issue permits to overweight loads to go over the bridge.

This method is recommended for common types of concrete and masonry bridges.

An example of load rating of a bridge designed for H15 loading, and BAR = 5, based on the DLRE method.

		Initial Ratings (NBI)			
Design Loading	GVW of truck (ton)	Inventory Rating (ton)	Operating Rating (ton)	Legal RF	EV3 RF
Unknown		10.0	16.7	0.350	0.320
H10	10.0	10.0	16.7	0.350	0.320
H15	15.0	15.0	25.1	0.520	0.480
HS15	27.0	27.0	45.1	0.940	0.860
H20	20.0	20.0	33.4	0.690	0.640
HS25	45.0	45.0	75.2	1.500	1.440
HS20	36.0	36.0	60.1	1.250	1.150

Modify the initial rating by multiplying it by BAF based on BAR, given below:

Modified Rating = Initial rating * BAF

BAR	1	2	3	4	5	6	7	8	9
BAF	0.00	0.30	0.55	0.85	1.00	1.00	1.00	1.00	1.00

BAR = 5; CF = 1.00

The initial ratings are then modified by multiplying with BAF.

Rating truck	Inventory Rating (ton)	Operating Rating (ton)	Legal RF	EV3 RF	Posting
HS20	0.42*1.00=0.42	0.70*1.00=0.70	0.52*1.00=0.52	0.48*1.00=0.48	Yes

929.4 METHOD C: LOAD RATING BASED ON THE FIELD TESTING (FT)

When it is not possible to perform load rating of a bridge due to insufficient plans, design information, or when the methods A) and B) do not yield results conforming to the observed performance, non-destructive load testing shall be performed to obtain desired load rating results. MBE, Section 8.2 describes two types of tests for bridge evaluation: diagnostic tests and proof tests. Diagnostic tests are generally performed to validate analytical procedures. ODOT recommends use of static proof tests to establish the maximum safe load capacity of the bridge, where behavior of the bridge is within the linear-elastic range, following MBE, Section 8.8.3.

A load test is not recommended (MBE, Section 8.6) to be performed when:

- The cost of testing reaches or exceeds the cost of bridge strengthening or bridge replacement.
- Pretest evaluation shows the load test is unlikely to show the prospect of improvement in load carrying capacity.
- According to calculations, the bridge cannot sustain even the lowest level of load test.
- There is a possibility of sudden failure (shear or fracture).
- Load tests may be impractical because of access difficulties or site traffic conditions.

Use the recommended load factor (X_p) value of 1.40, to be multiplied with the live load in the strength calculations. Following adjustments to be applied to X_p by adding all percentages applicable to obtain adjusted load factor, X_{pA} (not to be less than 1.30 or more than 2.20).

Considerations	Adjustment
One-lane Load Controls	+15%
Nonredundant Structure	+10%
NSTM Details Present	+10%
Bridges in Poor Condition	+10%
In-depth Inspection Performed	-5%
Rate-able, Existing RF ≥ 1.0	-5%
ADTT $\leq 1,000$	-10%
ADTT ≤ 100	-15%

MBE, Table 8.8.3.3.1-1

When method C) is used to determine the load rating of a bridge, the rating information shall be coded as follows:

SNBI Coding:

Item B.LR.04: Code “LT”

Item B.LR.05: Code inventory rating factor

Item B.LR.06: Code operating rating factor

Item B.LR.07: Code controlling legal load rating factor.

930 BRIDGE LOAD RATING QUALITY CONTROL AND QUALITY ASSURANCE GUIDELINES

930.1 GENERAL

Code of Federal Regulations 23CFR650.313(p), Federal Highway Administration (FHWA) and the American Association of State Highway & Transportation Officials (AASHTO) require using QC and QA processes to ensure and maintain high degree of accuracy and consistency in the load rating program and enhance safety. The FHWA and AASHTO recommendations were developed in response to the NTSB Recommendations No. 1 to AASHTO (H-08-20) and Recommendation No. 2 to FHWA (H-08-17).

The objectives of the QC/QA guidelines are to produce ratings representative of the bridge, data files appropriately created free of errors and omissions, clearly define sources of information, and contain as complete information as possible for future maintenance of data files. The information about the safe load carrying capacities of bridges obtained through the load ratings is used to determine the load posting needs of the bridges. Determination of safe load carrying capacity and load posting of a bridge are vital for the safety of the traveling public and ensuring the bridge will remain open and in service until it is strengthened, repaired, or replaced. Load rating and bridge posting information are also used in determining overall health of the system, & maintenance needs, prioritizing repairs, rehabilitation and replacements, allocation of resources and forecasting future needs.

930.2 QUALITY CONTROL (QC) REVIEW PROCESS

To ensure and maintain a high quality of bridge load rating analyses, the QC reviewer shall conduct a review before submitting the final rating report to the owner. The load rater must perform the load rating analyses, ensuring the assumptions, procedures, and data used are accurate and the most current deadload and inspection information are incorporated. A QC review is an internal process designed to verify the accuracy of data obtained from construction and repair and rehabilitation plans, field inspection reports, field measurements, as well as field and laboratory tests and other input parameters used in the analysis. QC reviews are generally conducted by a QC reviewer, who is not the rater, assigned from within the same agency, or company, or entity, which is performing the load rating analysis. The names of the engineers performing the bridge rating and reviewing must be documented in the final rating report (BR100).

The reviewer shall follow the following steps when performing bridge load rating QC review:

- The reviewer may use the same model that is created by the rater to check the analysis. In some instances, the reviewer may create his/her own independent model. If the same model that was generated by the rater is being used, the rater shall make a copy of the model and run it independently with the appropriate parameters to verify the load rating results obtained by the rater.
- The reviewer shall use original construction plans and all rehabilitated/rebuilt plans (if applicable) and other available documents to verify the input parameters for the rating analysis.
- The reviewer shall review the latest inspection report to see if there are any issues which may affect the rating analysis.
- The reviewer shall independently verify the parameters which may affect the load rating to ensure accuracy, and compliance with the policies and procedures defined in the BDM. The following list contains some of the parameters:
 - thickness of the existing wearing surface/overlay on the structure
 - span length
 - girder spacing
 - ultimate strength of structural material, such as concrete, steel or wood
 - allowable tension or compression of the structural material (timber)
 - section properties of the structural elements, such as girders, slabs, etc.
 - quantity of mild steel used in the analysis.
 - number of pre-stressing strands used in the analysis.

- strength and size (cross-sectional area) of the prestressing strands
 - location/placement of pre-stressing strands in the beam or girder
 - dead load and live load
 - proper application of composite and non-composite loads
 - legal vehicles and emergency vehicles used in analysis.
 - boundary condition, such as fix, pin, roller, and the values of the rotational springs, if used
- The reviewer shall verify the assumptions and calculations (if used in rating) or perform independent calculations.
 - The reviewer, if necessary, may ask for clarification from the rater, request additional calculations, and if unsure of any parameter/assumption, seek technical advice.
 - The reviewer shall verify the reasonableness of results.

930.3 FOLLOW-UP PROCEDURE FOR RESOLUTION

Any data issues identified during QC review, including errors, omissions, compatibility between items, changes, or any combination thereof, shall be resolved. The data issues, errors, omissions, etc. shall be communicated to the rater by the reviewer and shall be corrected by the rater. When an error in computer input, assumptions, or load calculations is found, consider what that error will do to the outcome. If the error has a negligible impact on the final rating, it may not be necessary to return the model back to the rater to revise and redo the rating.

When an error is found that will have significant impact on the remainder of the rating model and the results, return the rating model to the rater for correction prior to completing QC review. The rater may be required to resubmit the rating data files and reports after the resolutions of all review comments. Significant impact is defined as when the error is leading to posting a bridge or a change of controlling legal rating factor of 5 percent or more.

The data issues of minor nature (which may not significantly affect the rating of the bridge) may be corrected by the rater or the reviewer after mutual consultation.

ODOT also performs courtesy reviews of the load rating data files of non-ODOT bridges designed, and rated through Local-let program for completeness, accuracy, compliance with scope, and reasonableness of results. The reviewer of the rating is responsible for the complete QC review.

930.4 ACCEPTANCE OF RATING DATA FILES

ODOT will only accept the rating data files which have been created according to BDM Section 900, following ODOT policies, and are free of errors. The ODOT engineer from the OSE accepting and saving the rating data files of bridges which are created in BrR program is responsible to ensure the data files run error-free by both LFR and LRFR methods unless an exception of a method of analysis is approved by the BRE.

930.5 RETENTION POLICY

All load rating reports, and the bridge posting/rescinding information shall be maintained in the bridge files during the service life of the bridge. The most current rating values shall be coded in AssetWise.

930.6 QUALITY ASSURANCE (QA) PROCESS

To maintain high quality of load rating process and to ensure the QC procedures are being followed consistently, a QA review of the load ratings shall be performed by a qualified QA reviewer. The QA process varies depending on the type of bridge, method of load rating used and who performed the load rating.

QA review of ODOT bridges occurs in the Load Rating Section of the Office of Structural Engineering. Local agencies shall designate qualified internal or external QA reviewers to perform such reviews.

The QA review may consist of two parts.

Part I – An office review of a representative sample of documents, data in AssetWise, and inspection reports for evaluation of the organization's QC process, qualifications, resources, and documentation regarding the load rating for the bridges. Time required to post a bridge after determination of a bridge posting and evidence of presence of posting signs on bridges required to be posted shall also be reviewed.

Part II – If needed, review of a representative sample of the bridge files to ensure the bridge files are maintained correctly and current. This part is for county QA review.

930.7 QA REVIEW FREQUENCY

The QA period for reviews of the load rating shall be one year ($\pm$ 3 months from the previous QA review date).

930.8 QA REVIEW SAMPLING

The procedure, and sampling parameters for selecting bridges for independent review, shall be as follows:

1. All bridges posted for reduced loads during the QA period.
2. Bridge rating files may be selected by searching for known or suspected inaccuracies.
3. 10 percent of randomly selected load ratings (max = 40, min = 2) completed during the QA period.

930.9 RECOMMENDED FOLLOW-UP AND PROCEDURE FOR RESOLUTION

Results and recommendations based on the QA review are used to maintain the quality of the load rating program and make improvements where needed. Following a QA review the comments and recommendations may be communicated through emails, phone calls or other means of communication. Online meetings may also be held if needed. The resolution of the errors, omissions and/or comments shall be done in an expedited manner.

A. RATINGS PERFORMED IN-HOUSE BY ODOT

QA review findings and recommendations shall be communicated with the BRE, who will share them with the Load Rating Group and QC reviewers. The raters/reviewers may be required to fix errors, omissions, etc.

B. RATINGS PERFORMED BY CONSULTANTS FOR ODOT

QA review findings and recommendations shall be communicated to the Rater(s). The consultants in an active contract with ODOT at the time of their review, may be asked to fix the errors and omissions in the bridge rating models and

C. RATINGS PERFORMED FOR LOCAL ENTITY

QA review findings and recommendations shall be communicated with the Program Managers. The Program Managers/owners shall get the errors and omissions in the bridge rating models and reports fixed.

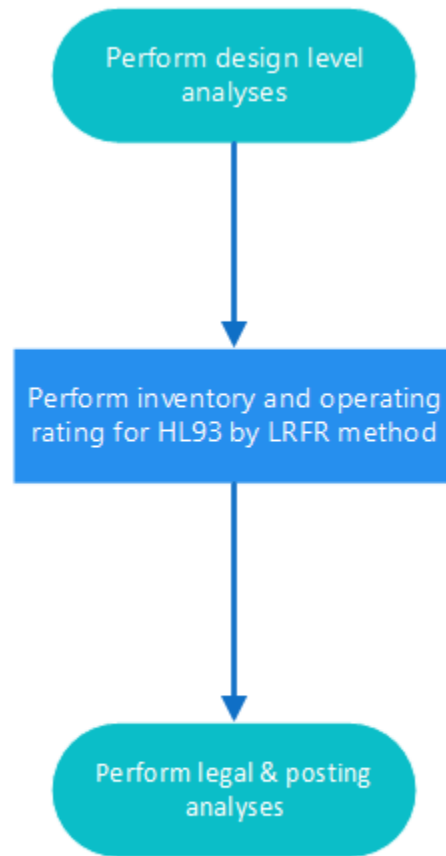
931 LOAD RATING FLOW CHART

Figure 931-1: Inventory and Operating Load Rating Flow Chart
(See Figure 931-2 for Legal and Posting Load Rating Flow Chart)

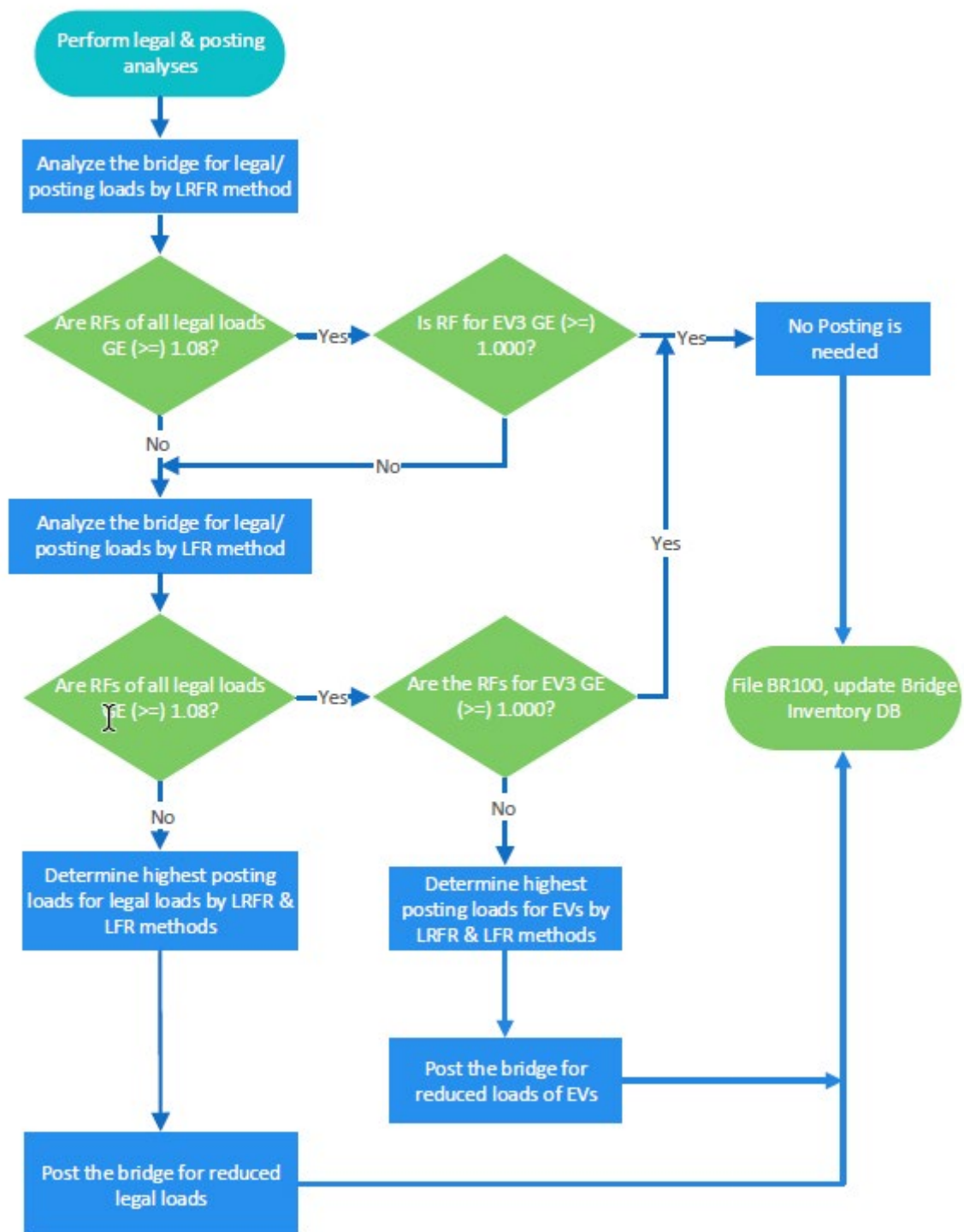


Figure 931-2: Legal and Posting Load Rating Flow Chart

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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 305)

1001 LRFD SECTION 1 - INTRODUCTION

1001.1 S1.3.3 DUCTILITY

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

1001.2 S1.3.4 REDUNDANCY

Non-redundant designs should be avoided.

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, cross-frames, 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.

1001.3 S1.3.5 OPERATIONAL IMPORTANCE

A bridge shall be considered a typical bridge with an operational importance load modifier (η_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 (η_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 (η_I) equal to 1.00 shall be used for all other limit states.

- A₁. Design ADT of 60,000 or greater, or
- B₁. Detour length of 50 miles or greater, or
- C₁. Any span length of 500-ft or greater.

For bridges meeting both of the following criteria, an operational importance load modifier (η_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 (η_I) equal to 1.00 shall be used for all other limit states.

A₂. Design ADT of 400 or less, and

B₂. 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

1002.1 S2.3.2.2.2 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 railing system. For routes with design speeds of 45 mph or lower, the Department requires a crash tested railing system to separate vehicle and pedestrian traffic when the pedestrian railing does not meet MASH or NCHRP 350 crash testing requirements. Refer to BDM Section 309.4 for more information.

1002.2 S2.3.3.2 HIGHWAY VERTICAL

The Department's requirements for vertical clearance for bridges and pedestrian structures are provided in the [L&D, Vol. 1](#). For the vertical clearance requirements for sign supports refer to the [TEM](#).

1002.3 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.

1002.4 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 309.1 for more information.

1002.5 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 concrete median bridge railings into the design of the composite section.

1002.6 S2.5.2.6.3 OPTIONAL CRITERIA FOR SPAN-TO-DEPTH RATIOS

Designers shall apply the minimum span-to-depth ratios shown in *Article 2.5.2.6.3*. See BDM Section 308.2.3.2 for span-to-depth ratios for adjacent box beam structures.

1002.7 S2.6.6.3 TYPE, SIZE AND NUMBER OF DRAINS

Refer to Section 1103 of the [L&D, Vol. 2](#) for ODOT's design criteria for deck drainage.

1002.8 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-1-19](#) for more information.

1003 LRFD SECTION 3 – LOADS AND LOAD FACTORS

1003.1 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.

1003.2 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-kef.

Design all bridges for a future wearing surface of 60-psf applied to the clear roadway width between curbs and/or bridge railings. Refer to BDM Section 303 for more information.

1003.3 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.

1003.4 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.

1003.5 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 309.3.2 for more information.

1003.6 S3.6.1.3.4 DECK OVERHANG LOAD

This article does not apply. Design deck overhangs in accordance with BDM Section 309.3.2.

1003.7 S3.6.1.4.2 FREQUENCY

The ADTT shall be estimated as follows:

$$ADTT = ADTT_{20} \times 4$$

Where:

ADTT = the number of trucks per day in one direction averaged over the design life

ADTT₂₀ = the number of trucks per day in one direction occurring in the design year (year 20)

1003.8 S3.6.1.6 PEDESTRIAN LOADS

For bridges that can accommodate service vehicles, refer to BDM Section 303.2 for loading requirements.

1003.9 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.

1003.10 S3.6.5.1 PROTECTION OF STRUCTURES

Use the flow chart provided in BDM Figure 1003-1 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.

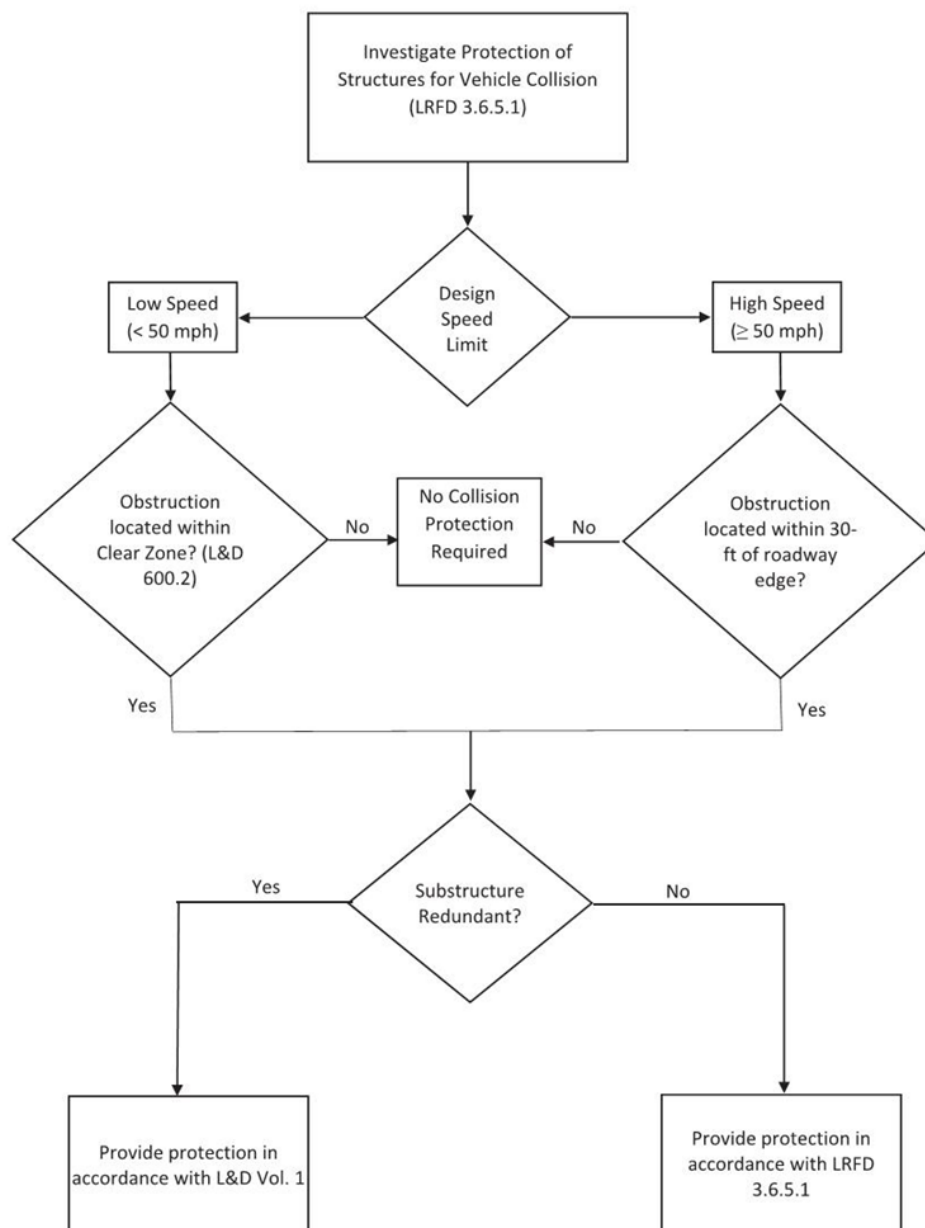


Figure 1003-1

1003.11 S3.10.2.1 GENERAL PROCEDURE

When required by BDM Section 303.1.4 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.

1003.12 S3.10.2.2 SITE SPECIFIC PROCEDURE

This procedure is not required for Ohio.

1003.13 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 ratio of 60%, N_{60} as defined in the ODOT Specifications for Geotechnical Explorations for the average SPT blow count $\bar{N}$.

1003.14 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*.

1003.15 S3.10.6 SEISMIC PERFORMANCE ZONES

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

1003.16 S3.10.9.2 SEISMIC ZONE 1

For bridges assigned to Seismic Performance Zone 1, determine the minimum support length according to BDM Section 303.1.4.1.a and the horizontal design connection force according to *LRFD 3.10.9.2* and BDM Section 303.1.4.1.b.

1003.17 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 303.1.4.1.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 303.1.4.1.b.

1003.18 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.

1003.19 S3.11.6.5 REDUCTION OF SURCHARGE

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

1003.20 S3.11.8 DOWNDRAW

Refer to BDM Section 305.3.2.2 for more information.

1003.21 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.

1004 LRFD SECTION 4 – STRUCTURAL ANALYSIS AND EVALUATION**1004.1 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.

1004.2 S4.5.1 GENERAL

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

1004.3 S4.6.2.2.1 APPLICATION

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

Typical ODOT Bridge Type	Applicable <i>Table 4.6.2.2.1-1</i> Cross-section
Steel beam/girder	(a)
Concrete I-beam	(k)
Composite Box beam	(f)
Non-composite box beam	(g)*

* - 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).

1004.4 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.

1004.5 S4.6.3 REFINED METHODS OF ANALYSIS

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

1004.6 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 5.6.3.4* is not permitted.

1004.7 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 303.1.4.1.b, modal analysis shall be performed in accordance with *LRFD 4.7.4.3.2*.

1004.8 S4.7.4.4 MINIMUM SUPPORT LENGTH REQUIREMENTS

The minimum support length requirements shall be applied according to BDM Section 303.1.4.1.a.

1005 LRFD SECTION 5 – CONCRETE STRUCTURES

1005.1 S5.4.2.3.3 SHRINKAGE

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

1005.2 S5.4.3.3 SPECIAL APPLICATIONS

BDM Section 304.4 and [C&MS 509.02](#) specify all concrete reinforcement for structures to be epoxy coated unless otherwise specified in the Scope of Services.

1005.3 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.

1005.4 S5.5.3.1 GENERAL

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

1005.5 S5.6.3.4 MOMENT REDISTRIBUTION

Moment redistribution is not permitted.

1005.6 S5.6.4.6 SPIRALS AND TIES

The ratio of spiral reinforcement to total volume of the concrete core (ρ_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 (in²)

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)

1005.7 S5.6.7 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT

Unless otherwise noted below, the exposure factor (γ_e) for concrete reinforcement in cast-in-place concrete bridge decks and cast-in-place slab bridges shall be 0.75. The 1-in monolithic wearing surface, BDM Section 309.1, shall be deducted from both d_c and h .

1005.8 S5.7.4.4 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.

1005.9 S5.9.2.3.2b 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.

1005.10 S5.9.3.4 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 (ti)0.75 days
- B. Age at deck placement (td).....45 days
- C. Final age (tf) 10,000 days

1005.11 S5.9.4.1 PRETENSIONING STRAND

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

1005.12 S5.9.4.3.3 PARTIALLY DEBONDED STRANDS

Refer to BDM Section 308.2.3.4.a.2 for additional debonded strand requirements.

1005.13 S5.10.1 CONCRETE COVER

The minimum concrete cover for concrete reinforcement shall be provided according to BDM Section 304.4.8. No modification for W/C ratio shall be made.

1005.14 S5.10.2.2 SEISMIC HOOKS

Transverse reinforcement in plastic hinge zones of structures assigned to areas with $0.10 \leq S_{DI} \leq 0.15$ shall be detailed in accordance with *LRFD 5.11.4.1.4*. 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

1005.15 S5.10.3.1.1 CAST-IN-PLACE CONCRETE

The maximum nominal aggregate size permitted for structural concrete according to [C&MS 499](#) is 1-in. When a maximum nominal 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”.

1005.16 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 nominal aggregate size according to the No. 57 gradation shown in [C&MS Table 703.01](#).

When a maximum nominal 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”.

1005.17 S5.10.4.3 TIES

Ties are required for T-type and wall-type piers. Refer to BDM Section 306.3.3.3 for more information.

1005.18 S5.12.4 DIAPHRAGMS

Refer to BDM Section 308.2.3.4.d for additional information.

1005.19 S5.12.9.5.2 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 5.12.9.5.2*. Except as noted in BDM Section 306.3.3.2 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 5.12.9.5.2* do not apply to drilled shafts or piles for Capped Pile Piers. Refer to BDM Section 305.4.4.3 for concrete reinforcement requirements in drilled shafts.

1006 LRFD SECTION 6 – STEEL STRUCTURES**1006.1 S6.4.1 STRUCTURAL STEELS**

Refer to BDM Section 304.1 for steel selection criteria.

1006.2 S6.4.3.1 BOLTS

The use of ASTM A 490 bolts is prohibited.

1006.3 S6.6.1.2.3 DETAIL CATEGORIES

All components or details shall be designed for infinite life using the Fatigue I load combination. When a three-dimensions analysis per BDM 308.2.2.3.g has been conducted using the fatigue loading in *LRFD 3.6.1.4*, intermediate and end cross-frames and diaphragms may be designed for finite life using the Fatigue II load combination.

1006.4 S6.6.1.2.5 FATIGUE RESISTANCE

As noted in BDM Section 1006.3, *S6.6.1.2.3* above, all components and details shall be designed for infinite life using the Fatigue I load combination. The use of finite life and the Fatigue II load combination are permitted for intermediate and end cross-frames and diaphragms when a three-dimensions analysis per BDM 308.2.2.3.g has been conducted.

1006.5 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 is available upon request.

1006.6 S6.7.2 DEAD LOAD CAMBER

Design camber shall not include an allowance for deflections cause 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.

1006.7 S6.10.1.7 MINIMUM NEGATIVE FLEXURE CONCRETE DECK REINFORCEMENT

Refer to BDM Section 309.3.4.1 for negative moment deck reinforcement requirements on non-composite members.

1006.8 S6.10.3.4 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 308.2.2.3.c.1.

1006.9 S6.10.7.3 DUCTILITY REQUIREMENT

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

1006.10 S6.10.10.1 GENERAL

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

1006.11 S6.10.10.1.1 TYPES

The use of channel sections as shear connectors is not permitted. Refer to BDM Section 308.2.2.1.1 for more information.

1006.12 S6.10.10.1.4 COVER AND PENETRATION

A detail for deep haunches is provided in the BDM Figure 403-1.

1006.13 S6.10.11.1.1 GENERAL

Violation of the 6tw 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 308.2.2.2.a and 308.2.2.3.d for more information.

1006.14 S6.10.11.3 LONGITUDINAL STIFFENERS

BDM Section 308.2.2.3.a prohibits the use of longitudinal stiffeners.

1006.15 S6.13.2.4.1 TYPE

Refer to BDM Section 308.2.2.1.j.1 for hole requirements of galvanized members.

1006.16 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 308.2.2.1.j.2 in lieu of those provided in *Table 6.13.2.6.6-1*.

1006.17 S6.13.2.8 SLIP RESISTANCE

Refer to BDM Section 308.2.2.1.j for more information.

1006.18 S6.13.6.1.3a GENERAL

Holes larger than standard holes are required for galvanized members. Refer BDM Section 308.2.2.1.j for more information.

1007 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.

1009 LRFD SECTION 9 – DECKS AND DECK SYSTEMS

1009.1 S9.4.1 INTERFACE ACTION

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

1009.2 S9.5.1 GENERAL

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

1009.3 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.

1009.4 S9.7.1.1 MINIMUM DEPTH AND COVER

The minimum depth of a concrete deck is 8.5-in as specified in BDM Section 309.3.1.

The minimum cover shall be in accordance with BDM Section 304.4.8.

1009.5 S9.7.1.3 SKEWED DECKS

BDM Section 309.3.4.2 reduces this skew limitation to 15-degrees. This allowance does not apply to all superstructure types. Refer to BDM Section 309.3.4.2 for more information.

1009.6 S9.7.2 EMPIRICAL DESIGN

The Empirical methodology of concrete deck design is prohibited.

1009.7 S9.7.4 STAY-IN-PLACE FORMWORK

BDM Section 309.3.7 prohibits the use of stay-in-place formwork.

1009.8 S9.7.5.1 GENERAL

If precast deck slabs are used, the minimum depth (BDM Section 309.3.1) and cover (BDM Section 309.3.4.2) requirements apply.

1010 LRFD SECTION 10 – FOUNDATIONS

1010.1 S10.4.2 SUBSURFACE EXPLORATION

Refer to ODOT Publication, “Specifications for Geotechnical Explorations”, available on the Department’s [Publication Gateway](#) web page.

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

1010.2 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 (ϕ_{dyn}) for a single driven pile in axial compression, installed according to [C&MS](#) 507 and 523 shall be 0.70.

Refer to BDM Section 305.3 for more information.

1010.3 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.

1010.4 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.

1010.5 S10.6.1.2 BEARING DEPTH

Minimum footing depth guidelines are provided in BDM Section 306.1.1.

1010.6 S10.7.1.2 MINIMUM PILE SPACING, CLEARANCE, AND EMBEDMENT INTO CAP

Refer to BDM Section 305.3.5.1 for specific pile spacing, clearance, and embedment requirements for typical ODOT substructure types.

1010.7 S10.7.1.3 PILES THROUGH EMBANKMENT FILL

Refer to BDM Section 305.3.5.7 for pre-drilling requirements.

1010.8 S10.7.1.4 BATTER PILES

Refer to BDM Section 305.3.5.8 for more information.

1010.9 S10.7.2.4 HORIZONTAL PILE FOUNDATION MOVEMENT

For drilled shafts and CFA piles, horizontal movement shall be determined according to *Article 10.7.2.4*. Battered drilled shafts and battered CFA piles are not permitted.

1010.10 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.

1010.11 S10.7.3.3 PILE LENGTH ESTIMATES FOR CONTRACT DOCUMENTS

Determine pile lengths for contract documents in accordance with BDM Section 305.3.5.2.

1010.12 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 305.3.2.1.

1010.13 S10.7.3.7 DOWNDRAG

Determine the required pile resistance and estimated lengths for piling subjected to downdrag loading in accordance with BDM Section 305.3.2.2.

1010.14 S10.7.3.8.2 STATIC LOAD TEST

Refer to [C&MS](#) 506 to determine the axial pile resistance for the test piles.

1010.15 S10.7.9 TEST PILES

Refer to BDM Section 305.7 for specific ODOT pile testing requirements.

Refer to BDM Section 305.3.5.2 to determine pile order lengths.

1010.16 S10.8.1 GENERAL

Refer to BDM Section 305.4 for specific ODOT drilled shaft considerations.

1010.17 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. These limits do not apply to drilled shaft walls or abutments where the drilled shaft resistance is lateral-load controlled.

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.

1010.18 S10.8.1.4 BATTERED SHAFTS

The use of battered shafts is prohibited.

1011 LRFD SECTION 11 – ABUTMENTS, PIERS, AND WALLS

1011.1 S11.5.1 GENERAL

The design life for MSE walls shall be 100 years.

1011.2 S11.6.1.3 INTEGRAL ABUTMENTS

The maximum structure length for integral abutments shall be in accordance with BDM Section 306.2.2.5.

1011.3 S11.6.1.6 EXPANSION AND CONTRACTION JOINTS

BDM Section 306.2.5 does not require contraction joints in abutments.

1011.4 S11.7.2.2 COLLISION WALLS

Refer to BDM Section 310.6 for more information.

1011.5 S11.10 MECHANICALLY STABILIZED EARTH WALLS

Refer to BDM Section 307.4 for more information.

1011.6 S11.10.2.1 MINIMUM LENGTH OF SOIL REINFORCEMENT

BDM Section 307.4 further defines the minimum length of soil reinforcement as the larger of: 70% of the wall height or 8-ft. The reinforcement length shall be uniform for the entire height of the wall facing.

1011.7 S11.10.2.2 MINIMUM FRONT FACE EMBEDMENT

The minimum depth shall be as defined in BDM Section 307.4. *Table C11.10.2.2-1* does not apply.

1011.8 S11.10.8 DRAINAGE

Impervious membranes shall not be used.

1011.9 S11.10.11 MSE ABUTMENTS

The minimum distances from facing panels to abutments and footings do not apply. Refer to BDM Section 307.4 for more information.

1012 LRFD SECTION 12 – BURIED STRUCTURES AND TUNNEL LINERS

No ODOT comments have been made to this article. Refer to the [L&D, Vol. 2](#).

1013 LRFD SECTION 13 – RAILINGS

1013.1 S13.4 GENERAL

Refer to BDM Section 309.4 for ODOT bridge railing warrants and railing design considerations.

1013.2 S13.7.2 TEST LEVEL SELECTION CRITERIA

Acceptance levels for standard ODOT bridge railings are listed in BDM Section 309.4.

1013.3 S13.8.1 GEOMETRY

If vandal protection fence is required, the fabric mesh shall have 1-in maximum openings. Refer to BDM Section 309.5 for more information.

1013.4 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 bridge railing and 1.0 for the wheel load furthest from the bridge railing. The position of the truck shall be in accordance with *Article 3.6.1.3.1*.

1013.5 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 = Railing resistance to lateral impact force (kips)
- H = Height of the railing (ft.)
- L_C = Critical length of yield line failure pattern (ft.)
- X = Lateral distance from toe of railing to deck design section (ft.)

Unless otherwise specified, assume the railing 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 railing resistance specified in *Article A13.3.1*

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

Bridge Railing System	L_c (ft.)	R (kip)	H (ft.)
SBR-1-20	9.2	124.0	3.5
SBR-2-20	10.0	124.0	4.75
SBR-3-20	9.2	80.0	3.0
42" BR-1-13	12.4	165.0	3.5
36" BR-1-13	8.8	72.0	3.0
BR-2-98	10.0	72.0	2.5 ⁽¹⁾

(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.

1013.6 S13.4.3.1 OVERHANG DESIGN

Refer to BDM Figure 309-3 and Figure 309-5 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 309-3.

1014 LRFD SECTION 14 – JOINTS AND BEARINGS

1014.1 S14.4.1 GENERAL

Irreparable damage in accordance with BDM Section 303.1.4.1.c is permitted.

1014.2 S14.5.6 CONSIDERATIONS FOR SPECIFIC JOINT TYPES

Refer to BDM Section 309.6 for standard ODOT joint types and applications.

1014.3 S14.6 REQUIREMENTS FOR BEARINGS

Refer to BDM Section 306.4 for preferred ODOT bearing types and applications.

1014.4 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 303.1.4.1.c is permitted.

1014.5 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.

1014.6 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 303.1.4.1.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.

1014.7 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.

1014.8 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.

1014.9 S14.7.5.3.2 COMPRESSIVE STRESS

The effect of impact shall be ignored.

1014.10 S14.7.5.3.4 SHEAR DEFORMATION

Designers shall assume the ambient temperature during setting is 60°F to calculate Δ_o .

1014.11 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 303.1.4.1.b. Elastomeric bearings, not otherwise requiring anchor bolts, do not require seismic anchoring and irreparable damage is permitted under BDM Section 303.1.4.1.c.

1014.12 S14.9 CORROSION PROTECTION

Refer to [C&MS](#) 516.03 for standard bearing corrosion protection requirements