PREFACE

To reflect the findings in research, new products or changes in design concepts, revised portions of the manual, will be issued to maintain a current Bridge Design Manual. The agency or its representative using this manual is responsible for compliance with not only the Manual but any revisions.

The Office of Structural Engineering offers a web page, the Manual and revisions at:

http://www.dot.state.oh.us/se/

Any comments or suggestions you may have to better the manual should be addressed to the Ohio Department of Transportation, Office of Structural Engineering, 1980 W. Broad Street, Columbus, Ohio 43223.

In this manual “Department” shall refer to the “Office of Structural Engineering” or “its representatives” including any district office production Administrator responsible for plan review or any consultant contracted by the Department to perform bridge plan review. Where the “Office of Structural Engineering” is specifically referenced in the manual, only the ODOT Office of Structural Engineering shall be the governing authority. The user of this manual (i.e. consulting engineers, ODOT District Personnel, County and other governmental agencies) will be referred to as the “Design Agency”. The owner of the project will be referred to as the “Appointing Authority”.

Updates and Revisions to this manual will be released on a quarterly basis as needed. The revisions will be available in Adobe Acrobat Reader (.pdf) format from the Department’s publication website:

http://www.dot.state.oh.us/drrc/

Subscribers to the ODOT Bridge Design Manual from this website will receive an email notification when revisions are released.

The format of the revisions will be in the form of replacement or insert pages to the existing manual. The replacement pages will include the date of the revision in the upper right corner of the page. The revised or new text will be marked with a single vertical line in the right hand margin of the page.
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101 INTRODUCTION

This manual shall be utilized for the design and rating of new non-buried bridges; the rating of new buried bridges; and the design of new retaining walls and noise walls.

The ODOT Bridge Design Manual, January 2004 shall be utilized for the analysis and rating of existing non-buried bridges; the rating of existing buried bridges; the analysis of existing retaining walls and noise walls; and the design of rehabilitations for existing non-buried bridges, retaining walls and noise walls.

The ODOT Location & Design Manual, Volume 2 shall be utilized for the design of structure types consisting of Type A and Type B conduit as defined in the CMS Item 603.

The ODOT Traffic Engineering Manual shall be utilized for the design of sign supports.

101.1 GENERAL CONSIDERATIONS

The design values, policies, practices, etc. established by this Manual promote uniform, safe and sound designs for bridges and structures in the State of Ohio. During the normal staged review process, the appointing authority may require the design agency to justify or otherwise seek recommendation from the Office of Structural Engineering when deviation from the design values, policies, practices, etc. is necessary.

The user of this Manual should be fully familiar with the AASHTO LRFD Bridge Design Specifications including all issued Interim Specifications, the ODOT Construction and Material Specifications, and Office of Structural Engineering Standard Bridge Drawings and Design Data Sheets, along with the contents of this Manual.

The practicability of construction should be considered with reference to each detail of design. This applies particularly as new ideas are considered.

Where complete description or instruction is not provided in the Construction and Material Specifications, the description or instruction should be shown on the plans, but care should be taken to insure clarity both from a structural and contractual viewpoint.

101.2 TABLE OF ORGANIZATION

An organizational chart for the various sections in the Office of Structural Engineering and a list of bridge contacts is available on our website at http://www.dot.state.oh.us/se/.
101.3 **DEFINITIONS**

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  
CMS  ODOT Construction and Material Specifications  
CVN  Denotes material meets Charpy V-Notch impact testing requirements  
FHWA  Federal Highway Administration  
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.  
LRFD  Load and Resistance Factor Design. When used in combination with AASHTO, this refers to the current edition of the AASHTO LRFD Bridge Design Specifications.  
MSE  Mechanically Stabilized Earth. Typically used as earth retaining wall systems.  
NCHRP  National Cooperative Highway Research Program  
NDT  Non-Destructive Testing  
NFIP  National Flood Insurance Program  
ODOT  Ohio Department of Transportation  
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.  
USGS  United States Geological Survey  

102 **PREPARATION OF PLANS**

Drawings should be so planned that all details will fall within the prescribed borderlines. All detail views should be carefully drawn to a scale large enough to be easily read when reduced to half size. Views should not be crowded on the sheet.

The scale of the views on the drawings should not be stated because in making reproductions of the drawing the prints may be either the same size as the drawing or half-size.
A North Arrow symbol should be placed on the Site Plan, General Plan and all plan views.

Elevation views of piers and the forward abutment should be shown looking forward along the stationing of the project. The rear abutment should be viewed in the reverse direction. Rear and forward abutments should be detailed 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 the centerline of construction (survey) and stationing should be used as a basis for these directions and locations. Elements are located either left or right of the centerline and to the rear and forward with respect to station progression. [e.g. rear abutment; forward pier; left side; right railing; left forward corner] Sheets in the bridge plans should be numbered in accordance with Figure 102.5-3.

For each substructure unit, the skew angle should be shown with respect to the centerline of construction or, for curved structures, to a reference chord. The skew angle is the angle of deviation of the substructure unit from perpendicular to the centerline of construction or reference chord. The angle shall be measured from the centerline of construction or reference chord to a line perpendicular to the centerline of the substructure unit or from a line perpendicular to the centerline of construction or reference chord to the centerline of the substructure unit.

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

In general, the designer should avoid showing a detail or dimension in more than one place on the plans. Such duplication is usually unnecessary and always 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, both sheets should be clearly cross-referenced.

Abbreviation of words generally should be avoided. Abbreviations, unless they are in common use, may cause delay and uncertainty in interpreting the drawings. If abbreviations are used, a legend should be provided to explain the abbreviation.

Plan sheet size to be used is 22" x 34" [559 mm x 864 mm]. Margins shall be 2" [50 mm] on the left edge and 1/2" [15 to 20 mm] on all other edges.

Where a project includes more than one bridge, plan preparation economies may be obtained by coordination of the individual plans. Where general notes are numerous and extensive, time can be saved by using a sheet of notes common to all bridges, or by including all of the common notes on one bridge plan and referring to them on the other bridge plans. The same applies to common details.
102.1 BRIDGE DESIGN, CHECK AND REVIEW REQUIREMENTS

The Department requires bridge design computations and bridge plans be made and prepared by an experienced bridge design engineer, the designer; all bridge computations be independently verified by an experienced engineer, the checker; and all bridge plans be reviewed by an experienced engineer, the reviewer. The design agency shall perform the required checks and reviews prior to submitting prints to the Department for review.

All outside agencies performing bridge design work for the Department shall be pre-qualified according to the requirements contained in the Departmental document “Consultant Prequalification Requirements and Procedures” which is on file with the Office of Contracts. Those individuals upon whose experience the classification level of the design agency is based shall complete work. The initials of these same individuals shall be placed in the appropriate spaces in the title block signifying that they performed the work.

The designer shall be responsible for preparing a design that follows sound engineering practice and conforms to ODOT manuals and AASHTO guides and specifications. The designer shall also be responsible for preparing an accurate and complete set of final bridge construction plans.

The checker shall be responsible for ensuring correctness, constructability and completeness of the plans and calculations and adherence to pertinent specifications and manuals. The checker shall perform and prepare a set of separate, independent calculations verifying all stations, dimensions, elevations and estimated quantities.

The checker shall independently check all structural calculations to assure that the structural theory, design formulae and mathematics used by the designer are correct. The intent is not to produce two separate sets of structural calculations. However, for atypical designs, fracture critical components, and situations where the designer’s theory is unclear or questionable, the checker shall perform and prepare a set of separate, independent calculations. The checker and designer shall resolve all discrepancies and the final product shall reflect mutual agreement that the design is correct.

The checker shall verify all structural calculations performed by computer analysis by preparing independent input for comparison with the designer’s input. The checker shall perform an independent analysis of the output and agree with the designer on the final design.

The design agency’s reviewer is responsible for the overall evaluation of the plans for completeness, consistency, continuity, constructability, general design logic and quality.

Design and check computations shall be kept neat and orderly so they may be easily followed and understood by a person other than the preparer.
102.2 MANUAL DRAFTING STANDARDS

102.2.1 GENERAL

A. All lines and lettering shall be dark and opaque. All lines and lettering shall be on the front face of the drawing, whether original or reproduced.

B. Plan sheets submitted to the Department shall be of extremely good quality on reproducible mylar.

102.2.2 LETTERING STANDARDS

A. All lettering shall be Braddock No. 5 size (upper case 5/32" [4 mm] in height), or larger.

B. Lettering within lined areas, such as quantity box, should at no time come in contact with any of these lines.

C. Letters should be properly spaced so that a crowded condition does not exist.

102.2.3 MANUAL DRAFTING LINE STANDARDS

A. "0" (Rapidograph pen size) (decimal width of 0.4 mm) is minimum and can only be used for dimension lines, X-hatching and index map.

B. All other lines and lettering shall be a minimum of "1" (Rapidograph pen size) (decimal width of 0.5 mm).

C. Individual lines shall be of uniform weight and density.

D. 1/16" [1.5 mm] is the minimum distance between two or more adjacent lines, even though an out of scale condition might exist.

102.3 COMPUTER AIDED DRAFTING STANDARDS

Refer to the Ohio Department of Transportation CADD – Engineering Standards for all CADD drafting standards.

102.4 DESIGNER, CHECKER, REVIEWER INITIALS BLOCK

The design agency's designer, checker and reviewer's initials and the date of the final review shall be shown in the title block of each sheet.

102.5 TITLE BLOCK

See Figure 102.5-1 for example Site Plan sheets. See Figure 102.5-2 for example Detail Plan
sheet. See Figure 102.5-3 for example title blocks for 22" x 34" [559 mm x 864 mm] sheets.

Straight Line Mileage (SLM) shall be shown to the nearest 1/100 of a mile. (Example: MER-707-16.92)

Straight Line Kilometers (SLK) shall be shown to the nearest 1/1000 of a kilometer (nearest 10 meters). (Example: MER-707-27.310)

A bridge number is the SLM of the structure written without the decimal point. (Example: MER-707-1692)

A bridge number is SLK of the structure written without the decimal point. (Example: MER-707-27310)

A Station is defined as 100 feet. Stations shall be shown to the nearest 1/100 of a foot. (Example: Sta 895+08.75)

A Station is a kilometer. Roadway stations are shown to the nearest 1/1000 of a meter. (Example: Sta 8+282.273)

The correct Structure File Number (SFN) shall be shown in the Existing Structure Block and Title blocks for the existing and proposed structures respectively. The SFN for the existing structure should be included in the Scope of Services. If not, it may be obtained from the responsible office.

If a new SFN is required for a proposed structure, contact the appropriate office as follows:

A. Contact the Office of Structural Engineering, Inventory Section, for structures on the State System or special systems statewide.

B. Contact County Bridge Engineers for SFN’s to structures within their jurisdiction.

C. Contact the responsible District Office for SFN’s to structures in municipalities within the District’s jurisdiction.

It is the Designer’s responsibility to contact and confirm the correct SFN with the appropriate office. For more information on the Structure File Number, refer to the Structure Inventory website at http://www.dot.state.oh.us/sfn/inventory.

Provide the Project Identification Number (PID) below the project County, Route and Section number in the Title Block.

The wording “State of Ohio, Department of Transportation”, “Office of Structural Engineering” is used on plans prepared by the Office of Structural Engineering. The name and address of the consulting firm should replace “State of Ohio, Department of Transportation, Office of Structural Engineering” in the title block.
102.6 ESTIMATED QUANTITIES

Plan quantities shall be listed separately for each bridge structure. Incorporating common bid items between multiple bridge structures in a project is not acceptable. Summation of common items cannot be done due to computer tracking of quantities based on Structural File Number.

In order to avoid the re-calculation of pay quantities by the construction forces, a copy of the quantity calculations made, as the basis for the quantities shown on the plans, shall be furnished to the District Production Office. Therefore, it is important that the quantity calculations be accurate and complete and prepared neatly on standard computation sheets. They should be arranged in an orderly fashion so that a person examining them will be able to follow the calculation sequence. Someone other than the original designer should independently check the calculations. The designer and checker shall each prepare separate sets of figures to minimize risk of error. The two sets of calculations then shall be reconciled, and one set (either the designer's or the checker's) shall be selected as the official set. The calculations shall be initialed and dated on each sheet by both the designer and checker. The results of this official set shall correspond to the quantities shown on the plans.

Each sheet of computations, notes, estimated quantities and steel list shall be marked with the Bridge Number, Structure File Number, the date, the writer's initials, and subject. Sheets that are accidentally misplaced are sometimes very difficult to identify if they are unmarked.

102.7 STANDARD BRIDGE DRAWINGS

Current standard bridge drawings should be followed and used whenever practicable. Reference to standard bridge drawings should be made by stating the Drawing Number and latest date of revision or the approval date if there has been no revision. If reference is made to a standard drawing, details shown on such standard drawing generally should not be duplicated on the project plans. The designer shall be familiar with the standards and know if they are adequate for the particular design situation being addressed. If they are not, then standards shall be modified as necessary by supplying pertinent details, dimensions or material specifications in the plans.

The designer shall assure that standard drawings referenced on the General Notes sheet are also transferred to the Project Plans Title Sheet.

A standard drawing should not be referenced if only one or two small details on the standard are applicable. Details should be copied on the project plans. In general the call out of more than one standard drawing for a particular bridge component should be avoided.

The quantities if shown on standard drawings are based upon average conditions and are only approximate. The quantities shall be computed from the actual plan dimensions.

In accordance with ODOT Policy 16-004(P), effective July 1, 2005, Standard Drawings will no longer be available for purchase through the Department’s Office of Contracts.
Drawings can be downloaded through the Office of Structural Engineering’s web page:

http://www.dot.state.oh.us/se/

102.8 SUPPLEMENTAL SPECIFICATIONS

The Department has many Supplemental Specifications that the designer needs to be familiar with and should use rather than developing their own individual specifications in the form of plan notes or Special bid items.

Supplemental Specifications may be obtained on the internet by accessing the Design Reference Resource Center (DRRC) home page from the Department website.

The designer shall not modify Supplemental Specifications.

Supplemental Specifications, like standard bridge drawings are to be listed on the General Notes plan sheet. (See Section 600). The designer shall assure that standard drawings referenced on the General Notes sheet are also transferred to the Project Plans Title Sheet.

102.9 PROPOSAL NOTES

Proposal notes are similar to Supplemental Specifications. 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, like Supplemental Specifications, are not to be revised by the designer.

The designer not only needs to know what bid item the Proposal Note applies to but also understand the Proposal Note so it is only applied where applicable.

Proposal Notes can also be obtained at the DRRC from the Department website.

Proposal Notes are referenced on the bridge plan by adding a note to the end of the applicable bid item (See Proposal Note). If multiple Proposal Notes are being used with different bridge plan bid items a footnote method may be used at the end of each bid item with the footnote saying - “See Proposal Note”. Do not refer to Proposal Notes by number.

103 COMPUTER PROGRAMS

The following is a list of computer programs used by the Department. The Design Agency may want to consider these programs or other programs not listed. The Design Agency is responsible for obtaining any programs. It is the choice of the Design Agency as to which computer programs it uses.

Note: (MF) denotes a mainframe program
(PC) denotes a personal computer program

103.1 GEOMETRIC PROGRAMS
A. COGO - Coordinate Geometry (MF)
B. GEOPAK COGO - Roadway Design Software (PC)

103.2 DESIGN PROGRAMS
A. GAD - Girder Automated Design (MF)
B. BDS - Bridge Design Systems (MF) & (PC)
C. BRASS - Bridge Rating and Analysis of Structural Systems (MF) & (PC)
D. BOXCAR - Box Culvert Structural Analysis (PC)
E. MERLIN DASH - Beam and Girder Analysis and Design (PC)
F. DESCUS I - Curved Girder (PC)
G. CONSPAN – Prestressed Concrete Beams (PC)
H. PCA COLUMN - concrete column design (PC)
I. SIMON SYSTEMS (PC)
J. RISA3D - Structural Analysis (PC)
K. VANCK - curved steel bridge structures (PC)
L. RC-PIER - Concrete Substructure Analysis and Design (PC)
M. OPIS - Beam and Girder Analysis and Design (PC)
N. PSBEAM – Prestressed Concrete Beams (PC)

103.3 HYDRAULIC ENGINEERING PROGRAMS
A. HEC-2 or HEC-RAS - Computations of Water Surface Profiles in Open Channels (PC)
B. HY7 - (WSPRO) Water Surface Profiles (PC)
C. HY8 - Culvert Hydraulics (PC)
D. HEC-12 - Pavement Drainage (PC)
E. HYDRA V3.2 (PC)
   1. Universal Culvert
   2. Special Culvert
   3. Long Span Culvert
4. Storm Sewer Design
5. Inlet Spacing
6. Ditch Analysis

103.4 GEOTECHNICAL ENGINEERING PROGRAMS

A. PICAP - Pile Capacity (PC)
B. SHAFT - Drilled Shafts (PC)
C. COM624P - Lateral Loading of Piles and Drilled Shafts (PC)
D. WEAP - Wave Equation Analysis of Pile Driving (PC)
E. STABL - Slope Stability Analysis (PC)
F. SPW911 - Sheet Pile Design and Analysis (PC)
G. Driven - Pile Capacity (PC)

103.5 BRIDGE RATING PROGRAM

Refer to Section 900 for Bridge Rating Programs.

104 OHIO REVISED CODE SUBMITTALS

The Ohio Revised Code has been changed so that Section 5543.02 no longer requires county financed bridge projects to be submitted to the Department for approval. The Code does require the Department to review and comment on the plans for conformance with State and Federal requirements if requested to do so by the County.

105 BRIDGE PLAN SHEET ORDER

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

A. Site Plan
B. General Plan & General Notes
C. Estimated Quantities Phase Construction Details
D. Abutments
E. Piers
F. Superstructure
G. Railing Details
H. Expansion device details
I. Non-standard approach slab details

J. Reinforcing Steel List

The General Plan sheet does not require an elevation view. The General Plan sheet is only required for:

A. New bridge of variable width or curved alignment.
B. New bridge requiring staged construction

If no General Plan sheet is furnished, the bridge plans may require a line diagram to show stationing and bridge layout dimensions that would not be practical to show on the site plan due to the site plan’s scale. Other details may be required to adequately present information needed to construct the bridge.
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STANDARD TITLE BLOCKS

TX = 0.125 IN., WT = 1 (CADD) OR NO. (0) RAPIDOGRAPH (HAND)

Example Site Plan Title Block

Example Plan Sheet Title Block

TX = 0.175 IN., WT = 2 (CADD) OR NO. (1) RAPIDOGRAPH (HAND)

This block should be filled in with the name of the actual design agency.
SECTION 200 - PRELIMINARY DESIGN

201 STRUCTURE TYPE STUDY

201.1 GENERAL

In conformance with Section 1400 of the ODOT Location and Design Manual, Volume Three, a Structure Type Study shall be included in the Preferred Alternative Verification Review Submission for a Major Project or in the Minor Project Preliminary Engineering Study Review Submission. The Structure Type Study is not required for projects classified as Minimal.

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

201.2 STRUCTURE TYPE STUDY SUBMISSION REQUIREMENTS

A Structure Type Study submission should include the following:

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

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

Additional structural related items that are required at this stage of the review process include:

A. Retaining Wall Justification (L&D Section 1404.2)
B. Noise Wall Justification (ODOT Policy #21-001 and Procedure #417-001)
C. Pedestrian Overpass Justification (L&D Section 1404.4)
201.2.1 PROFILE FOR EACH BRIDGE ALTERNATIVE

The profile for each bridge alternative considered shall generally be drawn to a scale of 1"=20’ [1 to 200] and shall generally be taken along the proposed centerline of survey for the full length of the bridge. The profiles shall include: the existing and proposed profile grade lines; existing ground line; the cross-section of channel; an outline of the structure; highest known high water mark; normal water elevation; flow line elevation (thalweg); design and 100 year highwater elevations including backwater; overtopping flood elevation and frequency; existing and proposed profile grade elevations at 25 ft [10 m] increments; and minimum and required vertical and horizontal clearances. Note: normal water elevation is the water elevation in the stream which has not been affected by a recent heavy rain runoff and could be found in the stream most of the year.

201.2.2 PRELIMINARY STRUCTURE SITE PLAN

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

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

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

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

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

C. Horizontal and vertical curve data.

D. Size of drainage area. The elevation, discharge and stream velocity through the structure and the backwater elevation for the 100-year frequency base flood, the design year flood and if necessary the overtopping flood. The clearance from the lowest elevation of the bottom of the superstructure to the design year water surface elevation (freeboard) should be provided.
E. In the existing structure block, provide a brief description of existing bridge. This should include type, length of spans and how measured (c/c of bearings, t/f of abutments), roadway width (t/t of barrier, t/t of curb, or f/f of railing), skew angle, original design loading or upgraded loading, type of deck and type of substructure, date when built, Structure File Number (SFN), approach slabs and wearing surface.

F. In the proposed structure block provide a brief description of proposed bridge. This should include type, length of spans and how measured (c/c of bearings), roadway width (t/t of barrier, t/t of curb, or f/f of railing), width of sidewalks, design loading, future wearing surface loading, skew angle, wearing surface, approach slabs, alignment, superelevation or crown and latitude and longitude bridge coordinates.

G. A cross section of the proposed superstructure, including an elevation of the proposed pier type(s) if applicable.

H. The design and current average daily traffic (ADT) and the design average daily truck traffic (ADTT).

I. For each substructure unit where a bearing is to be used, the bearing condition (fixed or expansion) shall be designated in the profile view (FIX or EXP). Semi-integral substructures shall be designated as expansion (EXP) and integral shall be designated as integral (INT).

J. Horizontal and vertical clearances and their locations shall be provided for navigable waterway crossings.

K. A cross section sketch at the abutments shall be submitted to provide information to help verify bridge limits.

For all waterway crossings, Designers shall reference Supplemental Specification 832 and determine the anticipated area impacted by the potential placement of temporary construction access fills (i.e. temporary causeways or workpads) into the “Waters of the United States”. “Waters of the United States” are defined by ODOT CMS 101.03 as waters that are under the jurisdiction of the Corps of Engineers and include: rivers, streams, lakes and wetlands. For rivers and streams, the jurisdiction begins below the Ordinary High Water Mark (OHWM). The OHWM is defined as the elevation on the shore established by the fluctuations of water and indicated by physical characteristics such as a clear, natural line impressed on the bank, shelving, changes in the character of soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means that consider the characteristics of the surrounding areas. Placement of fill material into the “Waters of the U.S.” will require a 404 permit from the U.S Army Corps of Engineers and a 401 permit from the Ohio EPA.

To facilitate the permit process, the Preliminary Structure Site Plan shall show the OHWM and shall include quantities for: (1) the plan area representing the footprint of the temporary fill and (2) the total volume of temporary fill material placed in the waterway below OHWM. To determine these quantities, the Designer should first calculate the surface area of the waterway bounded by the proposed right-of-way lines on each side of the bridge and the OHWM on each bank. (Use existing ROW lines where no right-of-way will be purchased.)

A. If the resulting area does not exceed 1/3 acres [1350 m²], then include the result as the quantity for (1) above and calculate the total fill volume assuming the flow line as the bottom
elevation and the OHWM as the top elevation with 1.5 to 1 (Horz. to Vert.) side slopes. Refer to Figure 201.2.2-1 for more information.

B. If the resulting area exceeds 1/3 acres [1350 m²], then the site plan quantities should represent a causeway from bank to bank and independent workpads for each pier located in the “Waters of the U.S.”. Assume the causeway to be bounded by the flow line at the bottom and the OHWM at the top; and to be trapezoidal in shape, 20’-0” [6 m] wide at the top with 1.5 to 1 (Horz. to Vert.) side slopes. Assume the pier workpads to be bounded by the flow line at the bottom and the OHWM at the top; and to be trapezoidal in shape with at least 10’-0” [3 m] of work area on all sides of the pier with 1.5 to 1 (Horz. to Vert.) side slopes. Refer to Figures 201.2.2-2 for more information.

Unless environmental documentation provides specific areas where placement of temporary construction fills is prohibited, the project plans shall not include a prescribed location for causeways and workpads. Figures 201.2.2-1 and 201.2.2-2 are only to aid in the determination of temporary fill quantities. The actual location, shape and size of causeways and pier workpads may differ.

201.2.3 HYDRAULIC REPORT

The Structure Type Study shall include a Hydraulic Report that includes the following information:

A. Supplemental Site Plan showing information necessary for the determination of the waterway opening. Information shown on the Supplemental Site Plan should not be repeated on the Structure Site Plan. The following information should be include on the Supplemental Site Plan:

1. A small scale area plan showing: approximate location of all stream cross sections used for the hydraulic analysis; an accurate waterway alignment at least 500 feet [150 meters] each way from the structure; and the alignment of the proposed and present highways, taken from actual surveys. Note location of dams or other regulatory work on the waterway above the site, and the pool level, if the bridge is in a pool area above a dam.

2. A stream profile at least 500 feet [150 meters] each way from the bridge showing waterway flow line elevations and low water profile (where materially different) and high water profile if such is obtainable. If a high water profile cannot be obtained, high water elevations, with their locations marked or described, should be shown both above and below the bridge. Show high water elevations with dates and location of reading with relation to the existing structure. The source of high water data should be noted on the Supplemental Site Plan. High water data should preferably be collected from at least two locations and preferably verified by interviewing two local residents.

3. A profile along the centerline of highway so that the overflow section may be computed. This profile should extend along the approach fill to an elevation well above high water. If there are bridges or large culverts located within 1000 feet [300 meters] upstream or downstream from the proposed bridge, show stream cross sections including the structure...
and roadway profiles of the overflow sections of the structures. These may be used as a
guide in establishing the waterway requirements of the proposed structure.

4. The nature of the waterway should be described as to condition of channel, banks, drift,
   ice conditions, flow of channel during low or high water, etc.

5. In the areas where agricultural or other drainage improvements are proposed, local
   authorities should be consulted as to the nature of the improvement and the probability of
   future lowering of the flow line, and appropriate provisions should be made.

B. Hydraulic Analysis that includes:
   1. Drainage area determination in mi² [km²].
   2. One set of computations for the design year and 100 year frequency discharges, including
      calculations used for the watershed’s main channel slope.
   3. Stream cross-section plots representing the natural stream conditions. The cross-section
      plots should indicate the Manning’s “n” values chosen for the channel sub-sections.
   4. One set of color photographs of the upstream channel, downstream channel and bridge
      opening should be included as assistance in determining the Manning’s “n” values.
   5. A stream cross-section at the existing bridge, including the bridge structure, for bridge
      replacement projects. All computer input shall be substantiated by existing ground
      contours or additional cross sections.
   6. Hydraulic calculations for the computation of backwater and mean velocities at the
      proposed bridge for both the design year and 100 year frequency discharges. All
      computer input data should be provided on a disk and included with the submission.
      HEC-RAS submissions shall conform to the file structure shown on Figure 201.2.3-1.
      a. For bridge replacement projects, the computations should be made for both the
         existing and proposed bridges, using like analysis methods.
      b. If the proposed roadway is overtopped by a discharge less than the 100 year
         frequency discharge, the elevation and approximate frequency of the overtopping
         discharge shall be shown on the Preliminary Structure Site Plan.

C. Bridge Scour Analysis that includes: (See Section 203.3 for specific scour information)
   1. A narrative of findings and recommended scour counter-measures. Include a statement
      regarding the susceptibility of the stream banks and flow line to scour, and also the
      susceptibility of the piers and abutments to scour.
   2. A copy of the scour calculations (if necessary). These calculations shall be provided to
      the ODOT District Engineer responsible for bridge inspection.

D. Hydraulic Narrative that briefly discusses the hydraulic effects of the bridge. The narrative
   should include:
   1. A discussion of the hydraulic adequacy for both the design year and 100 year frequency
      discharges.
2. High water data from local residents and observed high water marks including their locations.

3. A description of the bridge deck drainage collection system. Indicate how the surface water will be collected and discharged and include proposed scupper (if necessary) and catch basin locations. Refer to Section 209.3 for more information.

E. Flood Hazard Evaluation. This is a condition statement regarding the nature of the upstream area, the extent of upstream flooding and whether buildings are in the 100 year frequency flood plain.

201.2.4 NARRATIVE OF BRIDGE ALTERNATIVES

The Structure Type Study shall include a brief narrative identifying the structure alternatives and their costs. The narrative should 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.

201.2.5 COST ANALYSIS

The Structure Type Study shall include a cost analysis comparing alternative structures shall be performed, unless the site conditions discourage the use of all but one type of structure. The cost analysis should include the initial construction cost and all major future rehabilitation and maintenance costs, converted to present dollars. Sufficient preliminary design must be performed for an accurate cost estimate. Cost data information may be obtained from “Summary of Contracts Awarded”. This publication is available from the Office of Contracts.

201.2.6 FOUNDATION RECOMMENDATION

The Structure Type Study shall include a Foundation Recommendation that consists of:

A. General foundation type (i.e. Drilled Shafts, Friction Piles, Bearing Piles or Spread Footings)

B. Preliminary estimates of nominal and factored resistances

C. Boring plan and boring logs

The boring plan and boring logs should be prepared in accordance with the Specifications for Geotechnical Explorations.

For the scour evaluation, Section 203.3(D), provide D_{50} values from the particle size analysis.
When the foundation recommendation for the preferred alternative includes MSE wall supported abutments, the Designer shall provide estimates for factored bearing pressure and factored bearing resistance for the in-situ material below the MSE wall and an estimate for settlement of the MSE wall. Refer to Section 204.4 for additional considerations.

When unique subsurface conditions arise, include a brief narrative in the Foundation Recommendation for justification to obtain extra soils borings.

201.2.7 PRELIMINARY MAINTENANCE OF TRAFFIC PLAN

The various components of the bridge stage 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).

The Preliminary Maintenance of Traffic Plan shall include a transverse section(s) defining all stages of removal and construction. The following information should be provided:

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, shall be shown. A temporary single lane width of 11’-0” [3350 mm] or greater is preferred; 10’-0” [3000 mm] is the minimum allowable. Minimum preferred lateral clearance from edge of lane to barrier is 1’-6” [500 mm] (ODOT’s Traffic Engineering Manual, Section 640-2) but Section 605-11.2 of the Traffic Engineering Manual, allows this lateral distance to be amended for specific sites and conditions. The designer should ensure that lane and lateral clearance requirements are evaluated versus effects of phased construction on a bridge structure.

D. Location of cut lines. The existing structure should be evaluated to determine where the cut-line can be made to provide structural adequacy. Cut lines through stone substructures should be carefully evaluated to maintain structural integrity through staged removals. Temporary shoring may be required and should be considered.

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. When determining the closure pour width (see Section 300 of this Manual), the designer should investigate the economics of using the lap splices versus using mechanical connectors. Any necessary structure modifications should be included in the cost estimate. Lap splices are preferred and recommended. A reduced closure width may cause transition problems in the finishing of the bridge deck surface when bringing the various phases of construction together.

G. Profile grade, alignment, approximate location and width of temporary structures

H. Location of temporary shoring
201.3 UTILITIES

All utilities should be accurately located and identified on the Preliminary Structure Site Plan. A note should state whether they are to remain in place, be relocated or be removed, and for the latter two, by whom.

Utilities should not be placed on bridges whenever possible.

The type of superstructure selected for a site may be dependent upon the number of utilities supported on the bridge. The request to allow utilities on the bridge shall be made through the ODOT District Utilities Coordinator. Refer to the ODOT Utilities Manual. Utilities shall be installed in substantial ducts or enclosures adequate to protect the lines from future bridge repair and maintenance operations. Utilities shall not be placed inside of prestressed concrete box beams. For some specific detail issues with utilities on bridges refer to Section 300 of this Manual.

202 BRIDGE PRELIMINARY DESIGN REPORT

202.1 GENERAL

In conformance with Section 1400 of the ODOT Location and Design Manual, Volume Three, a Bridge Preliminary Design Report shall be included in the Stage 1 Detailed Design Review Submission.

202.2 BRIDGE PRELIMINARY DESIGN REPORT SUBMISSION REQUIREMENTS

The Bridge Preliminary Design Report submission should contain the following:

A. Final Structure Site Plan .................................................................Section 202.2.1
B. Final Maintenance of Traffic Plan ................................................Section 202.2.2
C. Foundation Report ........................................................................Section 202.2.3
D. Supplemental Site Plan for Railway Crossings ..............................Section 202.2.4

202.2.1 FINAL STRUCTURE SITE PLAN

In addition to the Preliminary Structure Site Plan requirements of Section 201.2.2, the Final Structure Site Plan should show the following information in plan view: bridge width and approach pavement widths, showing curb or parapet lines and outer limits of the superstructure and substructure units; skew with respect to the centerline of a substructure unit (not to centerline of stream or centerline of tracks); lateral clearances (both the minimum required and the actual) with respect to railroad tracks or highways under the proposed structure; location of minimum
vertical clearance; treatment of slopes around the ends and under the bridge; channel changes; soil boring locations; centerline of temporary structure and temporary approach pavement; stationing of bridge limits (i.e. the bridge ends of approach slabs); limits of channel excavation shown by crosshatching with a description provided in a legend; the location and description of bench marks; the following earthwork note: “EARTHWORK limits shown are approximate. Actual slopes shall conform to plan cross sections.”; and guardrail stationing.

When providing guardrail stationing: for bridges with twin steel tube bridge railing, station the first top mounted post on the bridge, for bridges with deep beam railing and concrete barrier railing, station the first guardrail post off the bridge. Typically stationing should be given to the nearest 1/100th of a foot [mm]. Guardrail stationing may be changed during the detail design phase and then revised on the Site Plan.

In addition to the requirements of Section 201.2.2, the Final Structure Site Plan should show the following information in the profile view: profile gradient percent; embankment slopes and top of slope elevations; proposed footing elevations; type of foundations; top of bedrock elevations at each boring location; and shaded areas of the bridge that represent the new bridge components.

For geometrical clarification: spans on straight (tangent) alignments should be measured from center to center of bearings along the centerline of survey; spans on curved alignments should be measured along a reference line, a chord drawn from centerline to centerline of abutment bearings at the centerline of survey or extended tangent along the centerline of survey; the centerline of the abutment bearings for concrete slab bridges is assumed \(7 \frac{1}{2}”\) [190 mm] behind the face of abutment substructure or breastwall; skews should be given with respect to the centerline for straight alignments or to reference lines (chords or tangents) for curved alignments; for straight alignments, the bearing of the centerline of survey should be shown; for curved alignments, the bearing of the reference line (chord or tangent) should be shown; and a superelevation transition table or diagram similar to Figure 209.1.1-1, should be provided if the bridge crown (superelevation) changes across the structure, reference should be made to the table or diagram when detailing the typical bridge transverse section.

Descriptive data for the proposed structure should be shown in a “Proposed Structure” block. The “Proposed Structure” block should be placed in the lower right hand corner for the 22” x 34” [559 x 864 mm] sheet size. An “Existing Structure” block should be shown on the Site Plan if applicable and be placed above the “Proposed Structure” block. Structure blocks should be approximately \(6 \frac{1}{2}”\) [165 mm] wide for 22” x 34” [559 x 864 mm] sheet size.

202.2.2 FINAL MAINTENANCE OF TRAFFIC PLAN

In addition to the Preliminary Maintenance of Traffic Plan requirements of Section 201.2.7, the Final Maintenance of Traffic Plan should include the following information:

A. Plan views and preliminary working drawings to ensure constructability
B. Temporary barrier anchorage details and requirements
C. Location and type of temporary shoring (See Section 208)
D. Location of structural members that require strengthening
E. Temporary structure design information (See Section 500)
F. Additional notes and/or details necessary

For concrete slabs, 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 has been revised over the years as new standard drawings were introduced. When considering staged construction requirements and the orientation of the cutline, screen existing concrete single span slab bridges according to the following criteria:

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 skew angle. Existing exposed reinforcing steel may be used to confirm the direction of the reinforcing steel.

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 or if possible locate the cutline parallel to the reinforcing if sufficient room exists. 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.

When utilizing semi-integral construction, the stability of the new part-width superstructure is to be considered. There exists the potential of the superstructure to move laterally either from the effects of the traffic using the new deck or the lateral earth pressure against the approach slab. See Standard Bridge Drawing “SEMI-INTEGRAL CONSTRUCTION DETAILS” for more information.

**202.2.3 FOUNDATION REPORT**

The Bridge Preliminary Design Report shall include a Foundation Report in accordance with the ODOT Specifications for Geotechnical Explorations. The Foundation Report shall include:

A. Investigational Findings
B. Analyses and Recommendations
C. Boring Logs
D. Test Data

Where the scour evaluation has identified a potential problem, the probable scour depths,
calculated in accordance with Section 203.3(D), should be considered in the design of the substructures; the location of the bottom of footings; the minimum tip elevations for piles and drilled shafts; and the factored side resistance of piles and drilled shafts. See BDM Section 202.2.3.2.h for more information.

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

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

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

202.2.3.1 SPREAD FOOTINGS

The use of spread footings shall be based on an assessment of the following: design loads; depth of suitable bearing materials; ease of construction; effects of flooding and scour analysis; liquefaction and swelling potential of the soils; frost depth; and amount of predicted settlement versus tolerable structure movement.

Spread footings shall be designed in accordance with LRFD 10.6.

Elevations for the bottom of the footing shall be shown on the Final Structure Site Plan. The estimated size of the footing; estimated settlements; and the factored bearing resistances shall be provided for review with the Foundation Report.

Adjust the footing size, the amount of predicted settlement and the factored bearing resistance during detail design as the design loads for the Service, Strength and Extreme Event Limit State are refined.

All spread footings at all substructure units, not founded on bedrock, are to have elevation reference monuments constructed in the footings. This is for the purpose of measuring footing elevations during and after construction for the purpose of documenting the performance of the spread footings, both short term and long term. See Section 600 for notes and additional guidance.

202.2.3.2 PILE FOUNDATIONS

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

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

202.2.3.2.a  PILES DRIVEN TO REFUSAL ON BEDROCK

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

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

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

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

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

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

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

In order to protect the tips of the steel “H” piling, steel pile points shall be used when piles are driven to refusal onto strong bedrock. When the depth of overburden is more than 50 feet [15 meters] and the soils are cohesive in nature, piles driven to strong bedrock generally should not
have steel points. Steel points should not be used when the piles are driven to bear on shale.

For projects where steel points are to be used, include the plan note entitled “Item 507, Steel Points, As Per Plan” with the Structure General Notes (Section 600 of this Manual).

### 202.2.3.2.b PILES NOT DRIVEN TO REFUSAL ON BEDROCK

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. The preferred type of friction piles are cast-in-place reinforced concrete piles.

Cast-in-place reinforced concrete piles are closed end steel pipes that are filled with concrete after the piles have been driven into the ground to the required ultimate bearing value. Except for capped pile piers as mentioned below, the steel pipe provides sufficient reinforcement and no additional reinforcing steel is required.

The Ultimate Bearing Value ($R_{ndr}$) is the nominal driving resistance required to support the total factored load ($\sum \gamma_i Q_i$) applied to a pile. Every pile in a single substructure unit shall be driven to the same Ultimate Bearing Value as the pile with the largest factored load in the unit. The Ultimate Bearing Value for each pile shall be provided in the structure general notes. A sample note is provided in BDM Section 600. The Ultimate Bearing Value may need to be adjusted during detail design as the design loads for the Service, Strength and Extreme Event Limit States are refined.

The Ultimate Bearing Value for each pile to be shown in the plans shall be determined as follows:

$$R_{ndr} = \frac{\sum \gamma_i Q_i}{\phi_{DYN}}$$

Where:

- $R_{ndr} = \text{Ultimate Bearing Value (Kips)}$
- $\sum \gamma_i Q_i = \text{Total factored load for highest loaded pile at each substructure unit (Kips)}$
- $N_{DYN} = \text{Resistance factor for driven piles}$
- $N_{DYN} = 0.70$ for piles installed according to CMS 507 and CMS 523.

Determine the estimated length for friction piles using static analysis methods to calculate the length of pile necessary to develop the Ultimate Bearing Value as follows:

$$R_{ndr} = R_S + R_P$$

Where:

- $R_S = \text{Nominal resistance (i.e. Unfactored resistance) provided along the side of the pile,}$
**LRFD 10.7.3.8.6**

\[ R_p = \text{Nominal resistance (i.e. Unfactored resistance) provided at the pile tip, LRFD 10.7.3.8.6} \]

The commonly used pipe pile sizes and the maximum Ultimate Bearing Value allowed for each are listed below:

<table>
<thead>
<tr>
<th>Pipe Pile Diameter</th>
<th>Maximum R\text{ndr}</th>
<th>Pipe Pile Diameter</th>
<th>Maximum R\text{ndr}</th>
</tr>
</thead>
<tbody>
<tr>
<td>12 inch</td>
<td>330 kips</td>
<td>12 inch</td>
<td>330 kips</td>
</tr>
<tr>
<td>14 inch</td>
<td>390 kips</td>
<td>14 inch</td>
<td>390 kips</td>
</tr>
<tr>
<td>16 inch</td>
<td>450 kips</td>
<td>16 inch</td>
<td>450 kips</td>
</tr>
</tbody>
</table>

When H-piles are used as friction piles, the maximum Ultimate Bearing Value for commonly used H-pile sizes are listed below:

<table>
<thead>
<tr>
<th>H-Pile Size</th>
<th>Maximum R\text{ndr}</th>
<th>H-Pile Size</th>
<th>Maximum R\text{ndr}</th>
</tr>
</thead>
<tbody>
<tr>
<td>HP10x42</td>
<td>350 kips</td>
<td>HP10x42</td>
<td>350 kips</td>
</tr>
<tr>
<td>HP12x53</td>
<td>380 kips</td>
<td>HP12x53</td>
<td>380 kips</td>
</tr>
<tr>
<td>HP14x73</td>
<td>440 kips</td>
<td>HP14x73</td>
<td>440 kips</td>
</tr>
</tbody>
</table>

For pipe piles, the Maximum R\text{ndr} values listed in the table above are in accordance with the CMS 507.06 requirements assuming typical pile wall thicknesses available for each pile diameter. For H-piles, the Maximum R\text{ndr} values listed in the tables above are based on pile drivability using commonly available pile hammers. For both pile types, these values may be significantly less than the structural capacity of a pile in axial compression. Because there is a large degree of variability inherent in the estimation of these maximum values, these tables should be used as a guide. Designers shall perform a Drivability Analysis in accordance with LRFD 10.7.8 to determine the plan specified Ultimate Bearing Value. Consult the Office of Structural Engineering prior to specifying Ultimate Bearing Values in excess of those shown above.

For capped-pile piers with cast-in-place piles, 16 inch [400 mm] diameter piles shall be used. 16 inch [400 mm] diameter piles with additional reinforcing steel are preferred because the need for pile encasement is eliminated. Additional reinforcing steel which consists of 8 - #6 [19M] epoxy coated reinforcing bars with #4 [13M] spiral at 12 inch [300 mm] pitch should be provided for 16 inch [400 mm] diameter piles. Reinforcing steel shall be detailed on the plans, included in the
reinforcing steel list, and be paid for under Item 507, 16 Inch [400 mm] Cast-In-Place Piles Furnished, As Per Plan. The reinforcing steel cage should extend 15 feet [5 meters] below the flow line and into the pier cap. Pile encasement is not used when additional reinforcement is provided. Painting of the cast-in-place reinforced concrete pile is not required.

For capped-pile piers where the exposed length of the piles is more than 20 feet [6 meters], 18 inch [450 mm] diameter piles can be used. Consult the Office of Structural Engineering before recommending the use of 18 inch [450 mm] diameter piles. Refer to BDM Section 204.5 for additional information.

202.2.3.2.c DOWNDRAG FORCES ON PILES

When a significant height of new embankment is constructed over a compressible layer of soil and long term settlement is anticipated, the possibility of downdrag loads on the piles should be considered. The potential downdrag load should be computed according to LRFD 3.11.8. Use either the traditional method or neutral plane method for calculating downdrag. The traditional method is described in LRFD 3.11.8 and in “Design and Construction of Driven Pile Foundations” (1996), FHWA-HI-97-013. The neutral plane method is discussed in NCHRP Report 393 (Briaud & Tucker, 1997). Both methods are described in “Design and Construction of Driven Pile Foundations” (2006), FHWA-NHI-05-042.

If using the traditional method to calculate the downdrag load, consider the transient loads concurrent with the downdrag load and permanent load. If using the neutral plane method, use the permanent loads plus the downdrag load or the permanent loads plus the transient loads, whichever is greater. For the neutral plane method, do not consider the transient loads concurrent with the downdrag load.

For piles driven to refusal on bedrock, include the factored downdrag load in the total factored load provided in the structure general notes.

For friction piles, the geotechnical resistance required to support the structure loads plus downdrag is only provided by the soil below the lowest soil layer contributing to downdrag. However, during pile installation, the soil within the downdrag zone provides positive side resistance. With time, the soil settles and this positive side resistance reverses direction and causes the downdrag load. Therefore, the Ultimate Bearing Value must include the downdrag loads and the positive side resistance within the downdrag zone. Determine the Ultimate Bearing Value (R_{ndr}) using the procedure shown below (also see LRFD C10.7.3.7 for more explanation). A plan note is available in BDM Section 600.

\[
R_{ndr} = \frac{\sum \eta_i \gamma_i Q_i}{\phi_{DYN}} + \frac{\eta_i \gamma_p DD}{\phi_{DYN}} + R_{Sdd}
\]

Where:

- \( R_{ndr} = \) Ultimate Bearing Value (kips)
- \( \sum \eta_i \gamma_i Q_i = \) Total factored load per pile, not including downdrag load (kips)
\[ N_{\text{DYN}} = \text{Resistance factor for driven piles. (See BDM Section 202.2.3.2.b.):} \]
\[ \eta \gamma_p DD = \text{Factored downdrag load per pile (kips)} \]
\[ R_{\text{Sdd}} = \text{Additional amount of Ultimate Bearing Value to account for side friction that must be overcome during driving through the downdrag zone. This value is equal to the nominal (i.e. unfactored) side resistance for the soil in the downdrag zone and shall be calculated using static analysis methods, LRFD 10.7.3.8.6. (kips)} \]

If using the traditional method to calculate the downdrag load, include permanent and transient loads in the total factored load given in the equation above. If using the neutral plane method, include only the permanent loads in the total factored load given in the equation above. For the neutral plane method, also ensure that the Ultimate Bearing Value is greater than the factored permanent and transient loads (do not include downdrag) divided by \( N_{\text{DYN}} \) (R\(_{\text{ndr}}\) \( \geq \) \( \Sigma \eta \gamma_i Q_i \) / \( N_{\text{DYN}} \)).

Determine the estimated length for friction piles using static analysis methods to calculate the length of pile necessary to develop the required Ultimate Bearing Value as described in BDM Section 202.2.3.2.b.

To resist downdrag forces, consider using larger H-piles or increasing the number of piles and reducing the applied load per pile. To reduce or eliminate downdrag loads, consider preloading the soil so settlement occurs before pile installation. Also consider installing wick drains to decrease the amount of time required for settlement to occur.

### 202.2.3.2.d PILE WALL THICKNESS

Minimum pipe pile wall thicknesses are specified by a formula in CMS 507.

### 202.2.3.2.e PILE HAMMER SIZE

According to Item 507, the contractor will select a pile hammer large enough to achieve the specified Ultimate Bearing Value and perform a dynamic load test to verify that the Ultimate Bearing Value is achieved. Refer to Section 303.4.2 for specific pile testing requirements.

### 202.2.3.2.f CONSTRUCTION CONSTRAINTS

For construction constraints regarding pile installation and embankment construction, see Section 600 of this Manual.

### 202.2.3.2.g PREBORED HOLES

The Designer shall specify prebored holes, CMS 507.11, for each pile on a project to be driven through 15 ft [4.5 m] or more of new embankment. 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. The length shall be the height of the new embankment at each pile location.

### 202.2.3.2.h SCOUR CONSIDERATIONS

Where the scour evaluation has identified a potential problem, any pile resistance provided by soil in the scour zone shall be neglected. The depth of scour resulting from the design flood shall be considered at the Strength and Service Limit States. The depth of scour resulting from the check flood shall be considered at the Extreme Event II Limit State.

For friction piles, the soil within the scour zone provides side resistance during pile installation. Therefore, the Ultimate Bearing Value must also include the side resistance from the soil within the scour zone. Determine this larger Ultimate Bearing Value using the procedure shown below. A plan note is available in BDM Section 600.

\[
R_{\text{ndr}} = \frac{\sum \eta_i \gamma_i Q_i}{\phi_{\text{DYN}}} + R_{\text{Src}}
\]

Where:
- \( R_{\text{ndr}} \) = Ultimate Bearing Value (Kips)
- \( \sum \eta_i \gamma_i Q_i \) = Total factored load for highest loaded pile at each substructure unit (kips)
- \( N_{\text{DYN}} \) = Resistance factor for driven piles. (See BDM Section 202.2.3.2.b)
- \( R_{\text{Src}} \) = Additional amount of Ultimate Bearing Value to account for side friction that must be overcome during driving through the scour zone. This value is equal to the nominal (i.e. unfactored) side resistance for the soil in the scour zone and shall be calculated using static analysis methods, LRFD 10.7.3.8.6. (kips)

Determine the estimated length for friction piles using static analysis methods to calculate the length of pile necessary to develop the larger Ultimate Bearing Value as described in BDM Section 202.2.3.2.b.

Because the pile will lose support along the scour depth, the Designer should investigate the structural capacity of the pile considering the depth of the scour as an unbraced length. The maximum factored structural resistances listed in BDM Section 202.2.3.2.a do not apply.

### 202.2.3.2.i UPLIFT RESISTANCE OF PILES

When a pile must resist uplift loads, the uplift resistance shall be calculated in accordance with LRFD 10.7.3.10. Use static analysis methods (LRFD 10.7.3.8.6) to determine the nominal uplift resistance due to side resistance.

Where the estimated pile length is controlled by the required uplift resistance, specify a minimum penetration pile tip elevation. A plan note is available in BDM Section 600.
The Ultimate Bearing Value is not shown on the plans for piles driven to a tip elevation, so the plans must specify the minimum pile wall thickness for cast-in-place reinforced concrete piles. Perform a drivability analysis to estimate the nominal driving resistance at the required tip elevation. Calculate the minimum pile wall thickness using the formula in CMS 507.06, with the Ultimate Bearing Value equal to the nominal driving resistance.

202.2.3.3 DRILLED_shafts

Drilled shafts should be considered when their use would:

A. Prevent the need of cofferdams
B. Become economically viable due to high design loads (eliminates the need of large quantities of pile)
C. Provide protection against scour
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 pile driving process (vibrations, interference due to battered piles, etc.).

Drilled shafts shall be designed in accordance with LRFD 10.8 and constructed in accordance with CMS 524.

Drilled shafts that support pier columns shall be 6 in. [150 mm] larger in diameter than the pier column diameter. The minimum diameter for drilled shafts that support pier columns shall be 42 in. [1065 mm]. The minimum diameter for all other drilled shafts shall be 36 in. [915 mm]. Drilled shaft diameters of less than 36 in. [915 mm] are not recommended.

Underreams or belled shafts should not be used. Belled shafts are difficult to construct under water or slurry and the bell will collapse in non-cohesive soils. Cleaning and inspecting the base of the drilled shaft within the bell are also very difficult.

Drilled shaft diameters shall be shown on the Final Structure Site Plan. For drilled shafts with friction type design, the tip elevation shall also be shown. For drilled shafts supported on bedrock, the tip elevation should not be given. Instead, the approximate top of the bedrock elevation and the length of the bedrock socket shall be shown in the profile view on the Final Structure Site Plan.

Designers should neglect the contribution to skin friction provided by the top 2 ft. [610 mm] of the rock socket.

The Foundation Report shall include the following drilled shaft information:
A. Unfactored unit tip resistance, $q_p$ (ksf)

B. Unfactored unit side resistance, $q_s$ for each soil layer contributing to the nominal shaft side resistance (ksf)
(THIS PAGE INTENTIONALLY LEFT BLANK)
C. Design methodologies used to determine unit tip and unit side resistances

D. Resistance factor from *LRFD Table 10.5.5.2.4-1* for each calculated unit resistance

At the detailed design stage, the factored resistance for each drilled shaft shall be provided in the structure general notes. A sample note is provided in BDM Section 600. The factored resistance may need to be adjusted during detail design as the design loads for the Service, Strength and Extreme Event Limit States are refined.

Consult the Office of Structural Engineering before recommending friction type drilled shafts. When drilled shafts with friction type design are used, a minimum of three (3) shafts per pier are recommended.

When lateral loads are controlling the design of drilled shafts, consult the Office of Structural Engineering to determine if lateral load testing should be specified.

The Design Agency should have the Department review any special proposed drilled shaft plan notes during the Stage 1 Review Submission. If casing is to be specified as to be left in place, a plan note will need to be added.

202.2.4 SUPPLEMENTAL SITE PLAN FOR RAILWAY CROSSINGS

For Railway-Highway grade separation structures, a Supplemental Site Plan is required. The Supplemental Site Plan should be completed and submitted with the Final Site Plan. The reproduced tracing of this plan should have the title block deleted so that the railroad can use the plan to show force account work necessary to complete the highway project.

This plan shall show information necessary for the determination of slope lines, probable property requirements, sight distance and other items involved in determining the type of separation. The following information should be shown:

A. A 1” = 100’ [1 to 1000] scale plan of the alignment of the railroad and the highway extended at least 1000 feet [300 meters] each way from the proposed point of intersection, taken from actual surveys.

B. Profile of top of rails of all railroads, extending at least 1000 feet [300 meters] each way from the proposed intersection.

C. Sufficient cross sections along the railroad and highway to determine approximate earthwork limits and encroachment on railroad property.

D. In case a highway underpass type of separation is at all possible, the submitted information should show the line and profile of the nearest or best outlet for drainage.

E. Intersection angle between highway centerline and railroad centerline.

F. Highway stationing and railroad mile post stationing at intersection.

G. Railroad right-of-way lines.
H. Railroad pole lines, signal control boxes, communications relay houses, signal standards and drainage structures.

I. Centerlines of all tracks and location of switch points.

J. Location of buildings or other structures within the railroad right-of-way.

K. Railroad traffic counts including type of movements and speed.

L. Location of all utilities occupying railroad right-of-way and the names of the owners of these utilities.

203 BRIDGE WATERWAY

203.1 HYDROLOGY


For urban drainage areas less than 4 square miles [10.4 km²] discharges shall be estimated by the method described in USGS Water-Resources Investigations Report 93-135, “Estimation of Peak Frequency Relations, Flood Hydrographs, and Volume-Duration-Frequency Relations of Ungaged Small Urban Streams in Ohio”.

B. Discharge estimates may be calculated by other methods for comparison with StreamStats against verified flood elevations and other known river data to ensure that the most realistic discharge for the area is used for the design of the waterway opening. Calculations and comparisons shall be submitted for review.

C. Federal Emergency Management Agency (FEMA), National Flood Insurance Program (NFIP) Flood Insurance Studies; U.S. Corps of Engineer Flood Studies; U.S. Soils Conservation Studies; U.S. Water Resources Data and other reliable sources may be used as reference information in estimating discharges and flood elevations. However, for waterway crossings located in a NFIP study area, the base discharge (Q₁₀₀) from the NFIP study takes precedence over all other calculated discharges.

D. Where a U.S. Geological Survey estimate is in conflict with that of another agency, the agencies should be contacted in order that the discrepancy can be resolved. In general, the U.S. Geological Survey estimate shall be given preference.

E. Proposed structures upstream or downstream from a flood control facility shall be designed for discharges as supplied by the U.S. Corps of Engineers, Ohio Department of Natural Resources or the agency responsible for the flood control facility.
203.2  HYDRAULIC ANALYSIS

A. The design flood frequency shall be based on the importance of the highway and the design
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average daily traffic (ADT) as follows:

1. Freeways or other multi-lane facilities with limited or controlled access ............... 50 years
2. Other Highways (2000 design ADT and over) and freeway ramps.............................. 25 years
3. Other Highways (under 2000 design ADT) ..................................................................... 10 years

B. The total backwater produced for the design flood should be calculated by WSPRO (HY-7), HEC-2, HEC-RAS or other comparable backwater calculation methods.

C. The allowable backwater depth shall generally be governed by the nature of the upstream area at the structure location and/or the induced mean velocity through the structure.

D. Local Flood Plain Coordinators will need to be contacted so they may be made aware of planned waterway crossings and proposed backwater effects. A listing of Local Flood Plain Coordinators is maintained by the Ohio Department of Natural Resources (ODNR) and may be obtained by calling (614) 265-6750 or visiting ODNR’s, Division of Water website: http://www.dnr.state.oh.us/water/.

The Local Flood Plain Coordinator may require a permit for any proposed waterway crossing regardless of the drainage area size. The District Production Administrator should be contacted, by the responsible governmental agency which initiated the project, as to how they wish to coordinate the permit process. The granted permit becomes a record which is kept by ODOT, at the appropriate District office. The governmental agency will be required to make application for the permit and to secure a granted permit prior to the initiation of any detail plan preparation.

E. In areas where the topography is flat, backwater should not be permitted to flood unreasonably large areas of usable land, if possible.

F. In urban areas the waterway opening for proposed structures shall be designed so that the allowable backwater elevation corresponds with the backwater elevation which currently exists.

G. When a proposed structure is subject to the approval of a Conservancy District, the waterway shall be designed to comply with their regulations if more restrictive than ODOT’s.

H. The design of all highway encroachments on the 100 year flood plain shall comply with the regulations as stated in the Code of Federal Regulations (23 CFR 650 A). Engineers responsible for bridge hydraulics should read these regulations to become familiar with their contents. When a highway encroachment is located in a detailed NFIP study area, create a duplicate of the original NFIP water surface model. Actual field survey data may be used to supplement the original NFIP data.

When making an encroachment, the proposed structure size submitted for preliminary design review shall be supported by an analysis of design alternatives with consideration given to capital costs and risk. “Risk” is defined as the consequences attributable to an encroachment. Risk includes the potential for property loss and hazard to life (A Flood Hazard Evaluation).

When making an encroachment on a NFIP designated flood plain in the floodway fringe, the
rise in the water surface is limited to one foot [0.3 meters] above the natural 100 year flood elevation as given by the NFIP study. No increase in the 100 year water surface is allowed when encroaching on a NFIP designated floodway (44 CFR 60.3(d)(3)). See Figure 203.2-1.

For bridges located outside NFIP jurisdiction regions, a limit of a one foot [0.3 meters] rise in the 100-year water surface elevation caused by any encroachment above the natural (no bridge, no roadway, no development, etc.) condition is mandated by the Ohio Revised Code, section 1521.13. There may be cases when this is too stringent of a criterion and may not be practical due to physical constraints or economic considerations. The Local Flood Plain Coordinator will need to be involved in any discussions/decisions when a new structure exceeds the one foot rise criteria. The responsible government agency which initiated the project will be responsible for contacting and coordinating with the Local Flood Plain Coordinator.

Longitudinal encroachments require alternative location studies to be summarized in the Conceptual Alternatives Study (L&D Section 1403.3). Evaluation of specific bridge hydraulics may not be necessary when alternative highway alignments are under consideration for the project. Refer to the Code of Federal Regulations (23 CFR 650 A) for more specific information.

I. It should not be assumed that an attempt should be made to lower existing high water elevations. However, for bridge replacement projects where the existing structure is severely hydraulically taxed, an effort should be made to improve the hydraulics for both the design and 100 year recurrence interval discharges, with consideration of the one foot [0.3 meters] rise criterion discussed in Section 203.2.h. No allowable backwater requirements are set by these criteria; rather the allowable backwater should be determined by good engineering judgment considering the area inundated and the mean velocities induced through the structure.

J. In general, the bridge should be designed to clear the design year frequency flood. This criterion may be waived because of roadway design constraints such as existing at-grade intersections, perpetuating existing profile grades, existing backwater elevations, presence of existing road overflow or other reasons.

K. Spill-thru type structures are generally preferred for cost effectiveness and hydraulic efficiency.

203.3 SCOUR

For bridges over waterways, armor the entire spill-through slope in front of the abutments and wingwalls, including the corner cones with Rock Channel Protection of the type determined from the following table. Rock Channel Protection requires the use of a filter. A 6 inch [150 mm] bed of crushed aggregate is allowed as an alternate in CMS 601.09 and should be specified when the rock is to be placed below water. The Item Master pay item descriptions allow for the differentiation between all options: with filter, with fabric filter or with aggregate filter.
The following table, relating bridge channel mean velocity of the design discharge versus rock type and thickness, shall apply as minimums. Special circumstances such as protection on the outside of curves or in northern regions of the state where ice flow is a concern may require greater rock thickness.

<table>
<thead>
<tr>
<th>Velocity (ft/s)</th>
<th>Type</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-8</td>
<td>C</td>
<td>2’-0”</td>
</tr>
<tr>
<td>8-10</td>
<td>B</td>
<td>2’-6”</td>
</tr>
<tr>
<td>above 10</td>
<td>A</td>
<td>3’-0”</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Velocity (m/s)</th>
<th>Type</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2.4</td>
<td>C</td>
<td>600 mm</td>
</tr>
<tr>
<td>2.4-3.0</td>
<td>B</td>
<td>750 mm</td>
</tr>
<tr>
<td>above 3.0</td>
<td>A</td>
<td>1000 mm</td>
</tr>
</tbody>
</table>

The locations, length, and the top of slope elevations for the rock channel protection should be shown on the Site Plan. The rock should be shown in greater detail in the roadway section in conjunction with the channel plans. It will generally be economical to provide bank protection during the initial construction in order to provide sufficient embankment protection to minimize future maintenance.

A. Excavation for stream channel work shall be limited to that portion of the channel one foot [300 mm] above normal water elevation in order to minimize intrusion and to preserve the natural low water channel. Where the spill-thru slope infringes upon the natural low water channel, excavation should be made for placement of the rock channel slope protection at the toe of the slope.

B. Substructures for bridges over waterways shall be supported by piling or drilled shaft foundations unless the footings can be founded on bedrock. Substructures for precast reinforced concrete three-sided flat-topped and arch culverts are addressed in the Location and Design Manual, Volume 2.

C. For bridges over waterways where bedrock is determined to be at or close to the flow line, spread footings or drilled shafts shall be used. Spread footings shall be embedded into the bedrock in accordance with the requirements of Section 204.1, except in laminated bedrock such as interbedded shale and limestone, in which case drilled shaft foundations with sufficient embedment into the bedrock are preferred.

D. A scour evaluation shall be performed for all bridges not founded on scour resistant shale or bedrock. When evaluating scour for a replacement structure, review all inspection reports for evidence of stream degradation (lowering of stream bed), scour or previous scour countermeasures. Scour depths are to be calculated with the equations in HEC-18 (Hydraulic Engineering Circular No. 18, Pub. No. FHWA NHI 01-001), “Evaluating Scour at Bridges”. The text of HEC-18 should be read in order to understand scour and river mechanics. The references cited in Chapter 3 of HEC-18 are also helpful in understanding the concepts of scour and river mechanics. Scour depths should be considered in the design of the substructures and the location of the bottom of footings and minimum tip elevations for piles and drilled shafts.
A value of Q500 should be used as the super flood is to be estimated by 1.3 x Q100.

**203.4 BRIDGE AND WATERWAY PERMITS**

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

A. U.S. Army Corps of Engineers ................................. 404 Permit and/or Section 10 Permit
B. U.S. Coast Guard .......................................................... Section 9 Bridge Permit
C. Ohio EPA .............................................................. 401 Certification and/or Isolated Wetland Permit

The designer and project manager shall coordinate with the ODOT District Environmental Coordinator and the ODOT Office of Environmental Services – Waterway Permits Unit throughout the permit determination 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 from the ODOT Office of Environmental Services.

Special Provisions are the method ODOT uses to attach the waterway permits and certifications to the project construction plans. The waterway permits Special Provisions Package (SPP) is prepared by the Office of Environmental Services – Waterway Permits Unit and may contain the following:

A. All pertinent waterway permits, certifications and related conditions
B. Drawings and/or mapping submitted with a permit application
C. Specialized plan notes associated with the waterway permits

The designer and the project manager shall confirm that the bridge design plans (e.g. the navigational clearances shown on the site plan, BDM Section 201.2.2.J; the amount of fill placed below ordinary high water, BDM Section 201.2.2; etc.) meet the requirements in the project waterway SPP (e.g. U.S. Coast Guard Section 9 Bridge Permit; U.S.A.C.E. 404 Permit; etc.) and shall ensure the project waterway SPP are submitted with the Final Plan Package.

**204 SUBSTRUCTURE INFORMATION**

**204.1 FOOTING ELEVATIONS**

Substructure footing elevations should be shown on the Final Structure Site Plan. The top of footing should be a minimum of one foot [0.3 meters] below the finished ground line. The top of footing should be at least one foot [0.3 meters] below the bottom of any adjacent drainage ditch. The bottom of footing shall not be less than four feet [1.2 meters] below and measured normal to the finished groundline.
Due to possible stream meander, pier footings for waterway crossings in the overflow section should not be higher than the footings within the stream unless the channel slopes are well protected against scour. Founding pier footings at or above the flow line elevation is strongly discouraged.

Where footings are founded on bedrock (note that undisturbed shale is bedrock) the minimum depth of the bottom of the footing below the stream bed, $D$, in feet [meters], shall be as computed by the following:

$$D = T + 0.50Y$$

Where:

- $T =$ Thickness of footing in feet [meters]
- $Y =$ distance from bottom of stream bed to surface of bedrock in feet [meters]

The footing depth from the above formula shall place the footing not less than 3 inches [75 mm] into the bedrock.

### 204.2 EARTH BENCHES AND SLOPES

A bench at the face of abutment shall not be used.

Spill thru slopes should be 2:1, except where soil analysis or existing slopes dictates flatter slopes. The slope is measured normal to the face of the abutment.

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 shall 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
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aesthetically pleasing structure.

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

204.3 **ABUTMENT TYPES**

Preference should be given to the use of spill-thru type abutments. Generally for stub abutments on piling or drilled shafts the shortest distance from the surface of the embankment to the bottom of the toe of the footing should be at least 4'-0" [1200 mm]. For stub abutments on spread footing on soil, the minimum dimension shall be 5'-0" [1525 mm]. For any type of abutment, integral design shall be used where possible, see Section 205.8 for additional information.

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

204.4 **ABUTMENTS SUPPORTED ON MSE WALLS**

When conditions are appropriate, the designer may consider stub type abutments with piling or spread footings supported on MSE walls. Use spread footings to support the abutment if the MSE wall is on bedrock or shale. If the MSE wall is on soil, then the selection of spread footings or piles to support the abutment should consider possible settlement of the MSE wall. Use piles to support the abutment if the bridge is a continuous multi-span structure, or if the bridge is constructed part width in phases. If the bridge is a single-span structure and is not constructed part width in phases, then either spread footings or piles may be used to support the abutment. Piles require a minimum 15-foot embedment below the MSE wall.

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

204.5 **PIER TYPES**

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

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

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

B. Cap-and-column type piers.

C. Solid wall or T-type piers.

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

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

204.6  RETAINING WALLS

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

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

Refer to SS840 for accredited MSE wall systems. Contact the Office of Structural Engineering for modular block wall systems. For wall systems that utilize geogrid reinforcements, the wall height shall be limited to 30 ft.
204.6.1 DESIGN CONSTRAINTS

Below are some design constraints to consider in the wall justification study to establish acceptable wall types:

A. Future use of the site (future excavations cannot be made in Mechanically Stabilized Embankments)
B. Deflection and/or differential settlements
C. Accessibility to the construction site
D. Aesthetics, including wall textures
E. Right-of-way (or other physical constraints)
F. Cost (approximate cost analysis)
G. Stage construction
H. Stability (long-term and during construction)
I. Railroad policies

204.6.2 STAGE 1 DETAIL DESIGN SUBMISSION FOR RETAINING WALLS

When a justification study has determined that a retaining wall is required, generally the wall will be a cast-in-place reinforced concrete wall or some type of proprietary wall system. The use of proprietary wall systems should be considered when the wall quantity for the project exceeds 5000 ft² [450 m²].

204.6.2.1 PROPRIETARY WALLS

If a proprietary wall is justified, the Design Agency shall include the following information in the Stage 1 Detailed Design Submission: wall alignment; footing elevations; factored bearing resistance at the leveling pad elevation; a global stability analysis; the effect of settlement and settlement calculations; and any construction constraints, such as soil improvement methods, that may be required. Refer to Section 303.5 for plan requirements for Detail Design.

The alignment of proprietary retaining walls should be straight and with as few corners or curves as is practical. When changes in wall alignment are required, use gradual curves or corners with an interior angle of at least 135 degrees whenever possible. Do not use corners with interior angles of less than 90 degrees (acute corners).

The design of the wall shall be in conformance with the 4th Edition of the AASHTO LRFD Bridge Design Specifications and the following:

A. Determine the height of the wall (h) for minimum soil reinforcement lengths as follows:
   1. When the surface of the retained soil is level, measure (h) from the top of the concrete
leveling pad to the top of the concrete coping.

2. When the surface of the retained soil is sloping, measure (h) as shown in LRFD Figure 11.10.7.1-1b.

3. If the wall will be located at an abutment, measure (h) from the top of the concrete leveling pad to the profile grade elevation at the face of the wall.

B. Determine the minimum soil reinforcement length to meet external stability requirements (sliding, bearing resistance, overturning, overall global stability). However, the minimum soil reinforcement length shall not be less than 70% of the wall height (h) or 8’-0” [2.5 m], whichever is greater. Generally, the soil reinforcement length should not be greater than 150% of the wall height (h).

C. The thickness of the unreinforced concrete leveling pad shall not be less than 6 inches [150 mm]. The minimum distance from the top of the leveling pad the ground surface at a point located 4’-0” [1.2 m] from the face of the wall shall be the larger of 3’-0” [900 mm] or the frost depth. Refer to Figure 303.5.1-4 for more information.

D. The minimum thickness of the precast reinforced concrete face panels may be assumed to be 5½ inches [140 mm].

E. The maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent. Provide slip joints if the estimated differential settlement is greater than one (1) percent.

F. Use the following soil parameters for design:

<table>
<thead>
<tr>
<th>Fill Zone</th>
<th>Type of Soil</th>
<th>Soil Unit Weight</th>
<th>Friction Angle</th>
<th>Cohesion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Zone</td>
<td>Select Granular Embankment Material</td>
<td>120 lb/ft³ [18.9 kN/m³]</td>
<td>34º</td>
<td>0</td>
</tr>
<tr>
<td>Retained Soil</td>
<td>On-site soil varying from sandy lean clay to silty sand</td>
<td>120 lb/ft³ [18.9 kN/m³]</td>
<td>30º</td>
<td>0</td>
</tr>
</tbody>
</table>

Determine soil parameters for the foundation soils based on the soils encountered by the soil borings.

G. Compute the coefficient of lateral earth pressure, $k_a$, using the Coulomb equation.

H. MSE walls located within 25’-0” [7.6 m] of the centerline of tracks, or other distance as specified by an individual railroad, shall be protected by a crash wall as specified in Section 209.8 and the AREMA Manual for Railway Engineering. The MSE wall system does not meet the definition of a crash wall as defined by the AREMA Manual for Railway Engineering.

I. For MSE walls supporting abutments on spread footings, the minimum distance between the back face of the MSE wall panels and the toe of the bridge abutment footing shall be 3’-0”
and the minimum distance between the back face of the MSE wall panels and the centerline of the abutment bearings shall be 5’-0” [1525 mm].

J. For MSE walls supporting abutments on piles, the minimum distance between the back face of the MSE wall panels and the toe of the bridge abutment footing shall be 1’-0” [305 mm] and the minimum distance between the back face of the MSE wall panels and the centerline of the closest row of piles shall be 3’-6” [1065 mm]. The distance between the centerlines of adjacent rows of piles shall be 3’-6” [1065 mm] to allow compaction of the fill between the pile sleeves.

K. Integral abutment designs placed on MSE wall embankments are prohibited. Semi-integral abutment designs are allowed.

L. The bearing pressure at the service limit state for a spread footing abutment placed on an MSE wall embankment shall be less than or equal to 4 ksf [190 kPa].

M. When detailing the pile layout and the design of the abutments and/or wingwalls, consider that 100% of the ground reinforcement shall be connect to the facing elements and the Department will not allow field cutting of reinforcement systems to avoid piles or other obstacles.

### 204.6.2.2 CAST-IN-PLACE WALLS

If a cast-in-place wall is justified, the design agency will be responsible for providing the complete wall design in the detail plans. The Stage 1 Detailed Design Submission shall include: footing elevations; allowable bearing pressures; a global stability analysis; settlement calculations, if necessary; and any construction constraints that may be required.

### 204.6.2.3 OTHER WALLS

The other wall types listed in Section 204.6 are for use with special project conditions such as top-down construction and other excavation methods. Contact the Office of Structural Engineering for recommendations when considering these other wall types. Typically only one wall type design shall be prepared for these methods.

### 205 SUPERSTRUCTURE INFORMATION

#### 205.1 TYPE OF STRUCTURES

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. Normally shallow abutments and spill-thru slopes will be used. The type of superstructure used should be selected on the basis of economy as well as appearance. For special conditions where other types of superstructures may be considered, consult the Office of Structural Engineering for recommendations prior to initiating the design.
205.2 SPAN ARRANGEMENTS

The length of a bridge will be determined by the requirements for horizontal clearance at grade
(highway or railway) separations or by the requirements for waterway opening at stream crossings. Typically for any given bridge, there are a number of combinations of spans and lengths of spans that can be utilized. Generally a preferred span arrangement that minimizes the number of substructure units should be used (i.e. fewer piers with longer spans).

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

For waterway crossings, one or three span bridges are typically used. This span arrangement is preferred so that a pier is not located in the middle of the waterway. If a series of precast, three-sided structures are used to produce a multiple span structure over a waterway, spread footings on soil shall not be used to support any of the precast structures.

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

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

205.3 CONCRETE SLABS

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

205.4 PRESTRESSED CONCRETE BOX BEAMS

The span limits for prestressed, side by side, concrete box beams generally range from 15 to 100 feet [5 to 30 meters]. These span limits are based on designs with 0.167 in² [108 mm²] low relaxation strands, a concrete 28-day compressive strength of 7000 psi [48.3 MPa], and a release strength of 5000 psi [34.5 MPa]. Prestressed box beams of up to 120 foot spans [36 meters] have been designed using 10,000 psi [68.9 MPa] concrete and larger diameter strands. Concrete compressive strengths should be limited to 5000 psi [34.5 Mpa] at release and 7000 psi [48.3 Mpa] at 28-days. Consult the Office of Structural Engineering for recommendations prior to designing a structure with higher compressive strengths.

The skew angle should be limited to a maximum of 30 degrees. Consult the Office of Structural Engineering for recommendations prior to designing a box beam structure with a higher degree
of skew. For all four lane divided highways or where the design ADTT (one way) is greater than 2500 prestressed box beam superstructures shall not be used. Box beams may be used on curved alignment where the mid-ordinate is 6 inches [150 mm] or less, as long as the required bridge width is provided. The maximum asphalt wearing surface thickness for a non-composite designed box beam bridge shall be 8 inches [200 mm]. For multiple span bridges, individual span lengths may vary but the proposed box beam depth should be constant.

The Designer shall consider the site limitations for practical hauling. 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. For beams 100 ft [30 m] long or more, the Designer should 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. The agreements should be included in the Structure Type Study, Narrative of Bridge Alternatives.

Non-composite boxbeam designs should be used where over the side drainage is provided and where the combined deck grade is less than 4 percent. The combined deck grade, \( C_g \), should be computed by the following equation:

\[
C_g = \left( \text{transverse deck grade}^2 + \text{roadway grade}^2 \right)^{1/2}
\]

For a normal transverse deck grade horizontal to vertical of 3/16 inch per foot [1 to 64 (1.56 percent)], the maximum roadway grade would be 3.68 percent or less for non-composite design. Where the combined deck grade is greater than 4 percent or the deck drainage is confined to the bridge deck by a parapet, curb, etc., a composite design should be used.

205.5  PRESTRESSED CONCRETE I-BEAMS

The span limits for prestressed concrete I-beams (AASHTO Type II, III, IV, and Modified Type IV) generally range from 60 to 125 feet [18 to 38 meters]. The shapes are to conform to Standard Bridge Drawing PSID-1-99. Consult the Office of Structural Engineering for recommendations prior to designing a structure with a non-standard shape. Cast-in-place concrete decks compositely designed shall be used. The transportation and weight requirements listed for box beams will also apply for I-beams.

Standard Bridge Drawing PSID-1-99 allows 28-day concrete strengths up to 7000 psi [48.3 MPa] and release strengths up to 5000 psi [34.5 MPa]. Consult the Office of Structural Engineering for recommendations prior to designing a structure with higher compressive strengths. Straight strand and draped strand designs are allowed. Refer to Section 300 of this Manual for the preferred methods to relieve excessive tensile stresses.

Prestressed I-beam highway bridges should have a minimum of 4 stringer lines.

Prestressed I-beam bridges that meet the vertical clearance specified in Section 207 are acceptable over highway crossings.
205.6 STEEL BEAMS AND GIRDER

For spans greater than 60 feet [18 meters], rolled beams, up to and including the 40 inch [1000 mm] depth, or welded plate girders should be considered. Continuous spans shall be used for multiple span bridges. 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 115 feet [35 meters]. For single span, composite designed, rolled beams, the maximum span is generally around 100 feet [30 meters].

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. Haunched girders over the intermediate substructure units should be considered for spans greater than 350'-0" [105 meters] or where economics warrant their use. Selections of any steel members should be based on an overall cost analysis of the structure.

<table>
<thead>
<tr>
<th>Stringer type</th>
<th>Span length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rolled beam</td>
<td>up to 115' [35 m]</td>
</tr>
<tr>
<td>Constant Depth Girder</td>
<td>100’ - 350’ [30 - 105 m]</td>
</tr>
<tr>
<td>Haunched Girder</td>
<td>&gt; 350’ [105 m]</td>
</tr>
</tbody>
</table>

Generally the minimum economical beam spacing for rolled beams is 8'-0" [2450 mm]. For plate girders a minimum spacing of 9’-0” [2750 mm] is generally recommended.

In order to facilitate forming, deck slab overhangs should not exceed 4’-0” [1200 mm]. On over the side drainage structures, the minimum overhang shall be 2’-3” [700 mm]. Where scuppers are required for bridge deck drainage, the overhang shall be 1’-6” [450 mm].

Steel rolled beam or girder highway bridges should have a minimum of 4 stringer lines.

ASTM A588[M]/A709[M] 50W should be selected wherever possible as it eliminates the need for a coating system and the maintenance associated with a coating system. See Section 300 of this Manual.

If a steel structure requires a coating system, the steel should be ASTM A572[M]/A709[M] 50. A coating shall be specified. See Section 300 of this Manual.

For more information on steel materials, see Section 300 of this Manual.

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.
205.7 **COMPOSITE DESIGN**

Composite design of concrete slab on steel beams or girders shall be used when the resulting design is more economical than a non-composite design. The preliminary designs shall be in sufficient detail to permit an adequate cost comparison for the Structure Type Study.

205.8 **INTEGRAL DESIGN**

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). These abutment designs are appropriate for bridge expansion lengths up to 250 feet [75 meters] (400 feet [125 meters] total length, assuming 2/3 movement could occur in one direction) and a maximum skew of 30 degrees. See Figure 205.8-1 for further criteria. The superstructure may be structural steel, cast-in-place concrete, prestressed concrete boxbeam or prestressed-I beams. Integral design shall be used where practical. This design should be used for uncurved (straight beams) structures and at sites where there are no concerns about settlement or differential settlement. See Figure 205.8-1 for additional limitations. An example of an integral design can be found in the figures portion of Section 300 of this Manual. There is a standard bridge drawing available that establishes details for integral abutment designs.

The limitations previously discussed are basically for steel superstructures. If a concrete superstructure is being proposed, longer structure lengths may be investigated. During preliminary design, consult the Office of Structural Engineering for recommendations on a specific site that exceeds the prescribed limits.

The expansion length, at the abutment, is considered to be two-thirds (2/3) of the total length of the structure. On new structures, all pier bearings should be expansion bearings. The pier expansion bearings are designed 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 (i.e. fixed bearings), regardless of structural rigidity, does not allow an increase in bridge length nor does it reduce the 2/3 movement assumption. Depending on its distance from the abutments, the pier will need to be designed for a portion of the movement from the superstructure.

205.9 **SEMI-INTEGRAL DESIGN**

Semi-integral design should be considered and is preferred to abutments with a deck joint. These abutment designs are appropriate for bridge expansion lengths up to 250 feet [75 meters] (400
feet [125 meters] total length, assuming 2/3 movement could occur in one direction). Generally there are no skew limitations. The foundation for these designs must be stable and fixed in position. These designs are not applicable when a single row of piles is used. The expansion and contraction movement of the bridge superstructure is accommodated between the end of the approach slab and the roadway. This design should be used for uncurved (straight beams) structures and at sites where there are no concerns about settlement or differential settlement. An example of a semi-integral design can be found in the figures portion of Section 300 of this Manual.

Spread footings may be appropriate for semi-integral abutments but settlement should be evaluated. Consult the Office of Structural Engineering for recommendations during preliminary design.

To utilize a semi-integral design, the geometry of the approach slab, the design of the wingwalls, and the transition parapets if any must be compatible with the freedom required for the integral (beams, deck, backwall and approach slab) connection to translate longitudinally. The expansion and contraction movements of the bridge superstructure will be transferred to the end of the approach slabs, see Section 209.6, Pressure Relief Joints.

There is a standard bridge drawing available that establishes details for semi-integral abutment designs.

The limitations previously discussed are basically for steel superstructures. If a concrete superstructure is being proposed, longer structure lengths may be investigated. During preliminary design, consult the Office of Structural Engineering for recommendations on a specific site that exceeds the prescribed limits.

The expansion length, at the abutment, is considered to be two-thirds (2/3) of the total length of the structure. On new structures, all pier bearings should be expansion bearings. The abutment bearings shall always be expansion bearings and be designed for the assumption that the 2/3 movement could occur at one of the abutments. The pier expansion bearings are designed proportionally (by distance) to the abutment design length.

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 (i.e. fixed bearings), regardless of structural rigidity, does not allow an increase in bridge length nor does it reduce the 2/3 movement assumption. Depending on its distance from the abutments, the pier will need to be designed for a portion of the movement from the superstructure.

206 MINIMAL BRIDGE PROJECTS

Minimal projects are defined in Section 1400 of the ODOT Location and Design Manual,
Volume Three, as projects that do not alter the basic highway cross section or geometry, require no additional right-of-way, are exempt from Categorical Exclusion documentation, and require little or no public involvement. Minimal project types include: bridge painting, deck overlays, scupper installations, barrier facings, concrete sealing, partial depth concrete repairs, etc. Minimal projects do not require a preliminary design submission.

207 BRIDGE GEOMETRICS

207.1 VERTICAL CLEARANCE

The “Required Minimum” and “Actual Minimum” Vertical Clearances and their locations shall be shown on the Preliminary Structure Site Plan, Section 201.2.2. The “Actual Minimum” Vertical Clearance is the minimum overhead clearance provided by the design plans. For new grade separation structures, the “Required Minimum” Vertical Clearance shall not be less than the preferred clearance specified in ODOT’s Location and Design Manual, Figure 302-1 unless otherwise specified in the scope of services. A “Required Minimum” Vertical Clearance less than the L&D Manual minimum clearance will require a Design Exception in accordance with Section 105 of the L&D Manual.

207.2 BRIDGE SUPERSTRUCTURE

Bridge superstructure widths shall be established in accordance with ODOT’s Location and Design Manual, Section 302, unless specified in the scope of services or other contract criteria.

207.3 LATERAL CLEARANCE

Divided highways having four or more lanes crossing under an intersecting highway shall be provided with a minimum lateral clearance of 30 feet [9000 mm] from the edge of traveled lane to the point where the 2:1 back slope intersects the radius at the toe of the 2:1 slope. Refer to ODOT’s Location and Design Manual, Figure 307-2. To satisfy cost considerations or in order to maintain the typical roadway section (including roadway ditch) of the underpass through the structure, for four or more lane highways, wall abutments or the 2:1 slope of typical two-span grade separation structures may be located farther than 30 feet [9000 mm] from the near edge of traveled lane.

Lateral clearances for other roadway classifications shall be established in accordance with ODOT's Location and Design Manual, Section 302, unless specified in the scope of services or other contract criteria.

207.4 INTERFERENCE DUE TO EXISTING SUBSTRUCTURE

Where a new pier or abutment is placed at the location of an existing pier or abutment the usual
“Removal” note (and also the text of CMS 202.03) calls for sufficient removal of the old pier or abutment to permit construction of the new. However, a new pier or abutment preferably should not be located at an existing pier or abutment where the existing masonry may extend appreciably below the bottom of the proposed footing, or appreciably below the ground in case of capped-pile construction. 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.

Where existing substructure units are shown on the Site Plan, the accuracy of the locations and extent should be carefully drawn. The existing substructure configuration should be shown based on existing plans or field verified dimensions, otherwise just a vertical line showing the approximate face of the abutment or pier widths should be shown. Misrepresentation of the location of the existing substructure units has resulted in expensive change orders during construction. Existing dimensions should be labeled as (+/-) plus or minus.

207.5 BRIDGE STRUCTURE, SKEW, CURVATURE AND SUPERELEVATION

During the Assessment of Feasible Alternatives, the location of the proposed structure should be studied to attempt to eliminate the presence of excessive skew, curves or extreme superelevation transitions within the actual bridge limits.

208 TEMPORARY SHORING

208.1 SUPPORT OF EXCAVATIONS

Whenever shoring is required to support a roadway where traffic is being maintained and the height of the retained earth will be over eight feet [2.5 meters], the Design Agency shall be required to provide a temporary shoring design with details provided in the plans and feasibility studied during the Structure Type Study.

For projects involving Railroads, the requirements will be different as each railroad company has their own specific requirements. The Design Agency is responsible for contacting the responsible railroad and obtaining the specific requirements for design and construction.

Following are some conceptual ideas for the design of temporary shoring:

A. A cantilever sheet pile wall should generally be used for excavation up to approximately 12 feet [3.5 meters] in height. Design computations are necessary.

B. For cuts greater than 12 feet [3.5 meters] in height, anchored or braced walls will generally be required.

C. For anchored walls, the use of deadmen is preferred. Braced walls using waler and struts can sometimes be braced against another rigid element on the excavated side. The use of soil or rock anchors(tiebacks) is generally the last option considered in the design of anchored walls.
D. The use of steel “H” piles with lagging is also a practical solution for some sites. Please note that some railroad companies allow only interlocking steel sheet piling adjacent to their tracks.

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

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

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

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

**208.2 SUPPORT OF EXISTING STRUCTURE**

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

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

**209 MISCELLANEOUS**

**209.1 TRANSVERSE DECK SECTION WITH SUPERELEVATION**

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

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

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

D. When the roadway break point is located at the edge of pavement (adjacent shoulder width is equal to or greater than eight feet [2.4 meters]), a five foot [1.5 meter] rounding distance
from the edge of pavement onto the shoulder is used to transition from the bridge cross slope to the 0.5 in. per ft. [0.04] shoulder cross slope. See “CASE d” in Figure 209.1-2.

The transition from the roadway approach transverse section to the bridge deck transverse section is to take place within the limits of the approach slab, whenever possible. On bridges with high skews, it may not be possible to do the transition within these limits and other alternatives should be considered during the Assessment of Feasible Alternatives.

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.

209.1.1 SUPERELEVATION TRANSITIONS

Because of the complexities associated with superelevation transitions on bridge superstructures (i.e. beam and girder cambering, crossframe fabrication, deck form construction, slip forming of parapets, etc.) all reasonable attempts should be made to keep such transitions off of bridge decks. Where transitions must be located on bridge decks, preferably, the transitions should be straight. An example of a transition diagram is shown in Figure 209.1.1-1. A table with the information shown in Figure 209.1.1-1 is also acceptable. Where this is not practicable, then transition's discontinuities should be smoothed by inserting 50 foot [15 meter] roundings at each discontinuity.

209.2 BRIDGE RAILINGS

All bridge structures on the National Highway System (NHS) or the State System require the use of crash tested railing meeting the loading requirements of TL-3 as defined by NCHRP report 350. The requirement for the NHS became effective October 1, 1998. For detailed information, refer to Section 304.

For structures with over the side drainage on the National Highway System, Twin Steel Tube Bridge Guardrail, Standard Bridge Drawing TST-1-99 should be used.

Over the side drainage shall not be used for bridges over highways and railroads. For four lane divided highways concrete deflector parapets shall be used. For bridges with heights of 25 feet [7.6 meters] or more above the lowest groundline or normal water, concrete deflector parapets should be used.

Refer to Section 305 of this Manual for vandal protection fencing requirements.

209.3 BRIDGE DECK DRAINAGE

The preferred minimum longitudinal grade of the bridge deck surface, when using concrete parapets, is 0.3 %, whenever possible.
The number of scuppers used for collecting the deck surface drainage should be minimized or eliminated if possible. The allowable spread of flow, which is used to help determine the need for scuppers, can be computed by the procedures as described in Section 1103 of the ODOT Location and Design Manual. Scuppers when provided, should preferably be located inside the fascia beam.

Drainage collection systems should be sloped as steeply as practical, generally not less than 15 degrees. The system should have a minimum bend radius of 18 inches [450 mm], no 90 degree bends, adequate pipe supports and cleanouts at the low ends of runs. The cleanout plugs should be easily and safely accessible. The necessary deck drainage outlet locations should be included in the Structure Type Study, Hydraulics Report.

Scuppers with drainage collection systems should be placed as closely as possible to the substructure unit which drains them. Uncollected scupper downspouts should be as far away from any part of the structure as possible.

When the deck drainage is to flow off the ends of the bridge, provisions must be made to collect and carry away this run-off. On bridges without MSE walls at the abutments and where the pavement flow from the deck is no more than 0.75 ft³/s [0.021 m³/s], a sodded flume, as shown on Standard Construction Drawing DM-4.1, should be provided. Six feet [2 meters] of excelsior matting shall be placed on each side of the flume. On grade separation structures with 2:1 approach embankment slopes and where the pavement flow from the deck exceeds 0.75 ft³/s [0.021 m³/s], an integral curb shall be provided on the approach slab with a standard catch basin located off the approach slab in lieu of the sodded flume. At the trailing end of bridge barriers, a bridge terminal assembly is required to protect this curb. The catch basin should be a Catch Basin No. 3A, as shown on Standard Construction Drawing CB-2.2. A properly sized conduit (Type F, 707.05 Type C) shall be used to provide an outlet down the embankment slope and the outlet shall be armored to prevent erosion.

Control of drainage is especially critical at abutments with MSE walls. On structures with MSE walls at the abutments, a barrier shall be provided on the approach slab with a standard catch basin to collect the drainage. Where possible, the catch basin shall be located at least 25 ft [7.6 m] beyond the limits of the MSE wall soil reinforcement. Continue the barrier 10 ft [3.0 m] past the catch basin. Use the same type of catch basin and conduit as described above.

For bridges that have deck joints consisting of finger joints or sliding plates with a trough collector system scuppers should be considered near the joint to minimize the amount of deck drainage flow across the joint.

For bridges that have over the side drainage a stainless steel drip strip should be provided to protect the deck edge and beam fascia from the deck surface run-off.

209.4 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, the slope under the structure shall be paved with Concrete slope protection, CMS 601.07. Consideration of slope protection should be given to all areas under freeway bridges over city streets not covered by pavement or sidewalk. Drainage discharge from the bridge should be checked to ensure that discharge is not crossing sidewalks, etc. so that ice, dirt and debris build-ups are prevented.

On spill-thru slopes under grade separation structures, areas that are not protected by concrete slope protection, shall be protected by crushed aggregate material as provided in CMS 601.06.

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 feet [1 meter] beyond the outer edges of the superstructure, except that at the acute corners of a skewed bridge the outside edge of the slope protection shall intersect the actual or projected face of the abutment 3 feet [1 meter] beyond the outer edge of the superstructure and shall extend down the slope, normal to the face of the abutment, to the toe of the slope. The base of the slope protection shall be toed in. Note that the natural vegetation on the slopes when shaded by a new structure will die out. For this case additional slope protection should be considered.

209.5 APPROACH SLABS

Approach slabs should be used for all ODOT bridges. Determine the length of the approach slab using the following formula:

\[ L = \left[ 1.5(H + h + 1.5) \right] \cos \theta \] #30 ft

\[ L = \left[ 1.5(H + h + 0.45) \right] \cos \theta \] #9.15 m

Where:  

- \( L \) = Length of the approach slab measured along the centerline of the roadway rounded up to the nearest 5 ft [1.5 m]
- \( H \) = Height of the embankment measured from the bottom of the footing to the bottom of the approach slab (ft) [m]
- \( h \) = Width of the footing heel (ft) [m]
- \( \theta \) = Skew angle

For four lane divided highways on new embankment, the minimum approach slab length shall be 25 ft [7.6 m] (measured along the roadway centerline). For structures with MSE walls at the abutments, the minimum approach slab length shall be 30 ft [9.1 m]. For all other structures the minimum length shall be 15 ft [4.6 m]. Refer to the approach slab standard bridge drawing for details.

Provide detail drawings for approach slabs which differ from the standard approach slabs. Examples include approach slabs that are a non-standard length, tapered, have a non-uniform width, or other such variation. When an approach slab falls within the limits of a vertical curve.
or superelevated section, the elevations at the edges of the approach slab shall be provided. Include these detail drawings in the structure plans for review during the detail design review stage. Approach slabs are paid for under Item 526.

For bridge replacement projects, when the existing approach slab is to be removed, the Designer shall include Item 202 - Approach Slab Removed in the structures estimated quantities.

209.6 PRESSURE RELIEF JOINTS

Type A pressure relief joints shall be specified when the approach roadway pavement is rigid concrete and shall be placed at the end of the approach slab.

The pressure relief joints are detailed on Standard Construction Drawing BP-2.3 (Revised 7/28/00), “Pressure Relief Joint Type A”.

209.7 AESTHETICS

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.

One of the most significant design factors contributing to the aesthetic quality of the structure is unity, consistency, or 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. As special situations arise preliminary concepts and details should be developed and coordinated with the Office of Structural Engineering.
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 $\frac{1}{4}''$ to $\frac{1}{2}''$ [6 to 13 mm],
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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 insure 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 Section 300 of this Manual.

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.

For additional guidance, refer to the Department's document entitled “Aesthetic Design Guidelines” available at the Design Reference Resource Center on the Department's website.

209.8 RAILWAY BRIDGES

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 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.

Consideration for providing future tracks and the possibility of track abandonment should be investigated. All submissions are to be made in accordance with the Department's review process. Railway submissions shall be made as directed by the District planning administrator. The guidelines of the individual railway company may be requested thru the District's designated rail transportation coordinator.

Generally if a steel superstructure is proposed over the railway the type of steel should be ASTM A588[M]/A709[M] 50W steel. Bridges located in urban areas or which have sidewalks located on the bridge should include protective fencing. Preferably drainage from the bridge should be collected in drain pipes and drained away from the railway right of way. No drains shall be allowed to drain on the railroad tracks or roadbed.

Where piers are located within 25'-0” [7.6 meters] of the centerline of tracks or if required by an
individual railroad, a crash wall shall be provided unless a T-type or wall type pier is used. Crash walls should have a minimum height of 10 feet [3.1 meters] above the top of rail, except where a pier is located within 12 feet [3.6 meters] of the centerline of tracks and in that instance the minimum height should be 12 feet [3.6 meters] above the top of rail. The crash wall shall be at least 2'-6” [760 mm] thick. For a cap and column pier the face of the wall shall extend 12 inches [300 mm] beyond the face of the columns on the track side. The designer should note that this requirement does not automatically require a crash wall thickness greater than the minimum. The crash wall should be anchored to the footings and columns.

When temporary shoring details are required for construction of substructure units adjacent to railway tracks, details shall be included 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. The design should be such that no settlement of the tracks is allowed. Interlocking sheet piling of cantilever design is preferred. It may be appropriate to leave the temporary shoring in place after construction.

The minimum vertical clearance from the top of rail should be 23’-0” [7.0 meters]. The point of minimum vertical clearance should be measured (calculated) from a point six feet [1.8 meters], measured horizontally, from the centerline of tracks measured level with the top of the high rail. The horizontal clearances vary between railway companies and need to be addressed for each specific site. Minimum construction clearances shall at least be 14’-0” [4.25 meters] horizontal, measured from centerline of tracks, and 22’-0” [6.7 meters] vertical, measured six feet [1.8 meters] from centerline of tracks, wherever possible.

209.9 BICYCLE BRIDGES

Reference should be made to ODOT’s most current design guidelines and Section 300 of this Manual. The current design guidelines can be found on ODOT’s Office of Local Projects web page, www.dot.state.oh.us/local/. For new structures generally the minimum bridge width should be the same as the width of the paved bicycle path and approach shoulders. A minimum transverse slope of 1/4 inch per foot [0.021] sloped in one direction should generally be used. Bicycle railings should be a minimum of 4’-6” [1370 mm] high. A smooth rub rail should be provided at a height of 3’-6” [1065 mm]. For the design of the railing refer to AASHTO LRFD Article 13.9. If an occasional maintenance vehicle is going to use the bridge, the railing should only be designed as a bicycle railing. The type of bridge deck joints used should be bicycle safe.

If a timber deck is used, a 1½ inch [38 mm] minimum thickness of Item 448, Asphalt Concrete Surface Course, Type 1, PG64-22, shall be applied in order to provide an abrasive skid resistant surface. Consult the Office of Structural Engineering for recommendations before specifying other alternative surfaces.
209.10 PEDESTRIAN BRIDGES

Pedestrian facilities shall meet the grade and cross slope requirements specified in Volume One, Section 306.2.5 of the ODOT Location & Design Manual. For pedestrian bridges over highways an additional one foot [300 mm] of vertical clearance shall be provided. Refer to BDM Section 301.4.1 for additional design guidance.

If a timber deck is used, a 1 1/2 inch [38 mm] minimum thickness of Item 448, Asphalt Concrete Surface Course, Type 1, PG64-22, shall be applied in order to provide an abrasive skid resistant surface. Other alternative surfaces may be used if approved by the Department.

209.11 SIDEWALKS ON BRIDGES

Sidewalks should be provided where significant pedestrian traffic is anticipated and/or the approach roadway has sidewalks or requires provisions for future sidewalks. Refer to Volume One, Section 306.4 of the ODOT Location & Design Manual for specific pedestrian traffic requirements. The width of the bridge sidewalk is generally the width of the approach sidewalk plus 12 inches [300 mm], with the widths typically between 5 and 6 feet [1500 and 1800 mm] wide.

An 1/4 inch per foot [0.02] cross slope should be provided to drain the sidewalk towards the curbline. The sidewalk height shall be 8 inches [203 mm] on the bridge, tapering down to the approach curb height within the length of the approach slab.

A detail of the standard curb (height, face slope, and corner rounding) should be given. Refer to Section 300 of this Manual for vandal protection fencing requirements.

209.12 MAINTENANCE AND INSPECTION ACCESS

Maintenance and inspection access requirements should be included in the Structure Type Study, Narrative of Bridge Alternatives. For multiple span bridges with 8 feet [2400 mm] or deeper girders, an inspection handrail located on the girders should be provided. Also catwalks should 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 fracture critical girders, cross girders and bents that cannot be inspected from a snooper. The use of fracture critical members is strongly discouraged. For these types of structures, consult the Office of Structural Engineering for details and recommendations. Additional information is provided in “FHWA Guidelines for Providing Access to Bridges for Inspections”, dated November 1985.

209.13 SIGN SUPPORTS

Research has shown that overhead sign supports located on bridges are highly susceptible to
fatigue damage. Every effort shall be made to locate overhead sign supports off of bridge structures. When this is not possible, only two locations on the structure are acceptable and are listed below in order of preference:

A. Mounted directly to the substructure unit.

B. Mounted to the superstructure directly over a substructure unit.

Sign supports attached to the fascia of overpass bridges, as shown on Standard Construction Drawings TC-18.24 and TC-18.26, should also be avoided. Consult with the District Bridge Engineer before specifying their use.
TOTAL FILL AREA = 10,860 FT² = 0.25 ACRES

PLAN

PROFILE

LEGEND

O.H.W. = ORDINARY HIGH WATER ELEV.
N.W. = NORMAL WATER ELEV.
F.L. = FLOW LINE ELEV.
- - - TEMPORARY ACCESS FILL

SECTION A-A

Ordinary High
Water Elev.

Flow Line
Elev.
HEC-RAS File Structure
LINE A - B IS THE FLOOD ELEVATION BEFORE ENCROACHMENT
LINE C - D IS THE FLOOD ELEVATION AFTER ENCROACHMENT
* SURCHARGE IS NOT TO EXCEED 1.0 FOOT OR THAT ALLOWED
BY OTHER REGULATORY AGENCIES IF MORE RESTRICTIVE.
SEMI-INTEGRAL/INTEGRAL ABUTMENT TYPE

SKEW VS. BRIDGE LENGTH LIMITATIONS (FT)

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SECTION 300 – DETAIL DESIGN

301 GENERAL

301.1 DESIGN PHILOSOPHY

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

301.2 DETAIL DESIGN REVIEW SUBMISSIONS

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

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

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

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

C. Noise Barrier Plans generally consisting of the following:
   1. General Notes
   2. Plan and Profile Views
3. Noise Barrier Details
4. Foundations Table
5. Subsurface Investigation Plan Sheets
6. Other Details as necessary

D. Special Provisions
E. Load Rating Reports for bridges (Major and Minor PDP)

The Stage 3 Detail Design plan submission should include an updated cost estimate and the following:

A. Stage 2 Detail Design plans in compliance with all Stage 2 review comments.
B. Completed Estimated Quantities Table
C. Completed Reinforcing Steel Schedule
D. Estimated Quantities calculations
E. Load Rating Reports for bridges (Minimal PDP only)

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

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

301.3 DESIGN METHODS

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

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

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

301.4.1 HIGHWAY BRIDGES

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

301.4.2 PEDESTRIAN AND BIKEWAY BRIDGES

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

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

301.4.3 RAILROAD BRIDGES

Bridges are to be designed in accordance with current AREMA specifications and the individual railway company's loading requirements. All other aspects of the structure design shall conform to the AASHTO LRFD Bridge Design Specifications and this Manual.

301.4.4 SEISMIC DESIGN

Seismic design shall be in accordance with the AASHTO LRFD Bridge Design Specifications and BDM Section 1000.

Abutment and pier designs with raised pedestal bearing seats shall not be used.

301.4.5 APPLICATION OF LONGITUDINAL FORCES

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

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

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

All reinforcing steel shall be epoxy coated.

301.5.1  MAXIMUM LENGTH

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

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

301.5.2  BAR MARKS

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

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

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

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

301.5.3  LAP SPLICES

Bar splice lengths shall be shown on the plans.

Development and splice lengths shall conform to AASHTO requirements.

Reinforcing steel at construction joints should extend into the next pour only by the required splice length.
Reinforcing steel shall not project through expansion and contraction joints.

In lieu of lap splices, mechanical splices in accordance with the requirements of CMS 509 may be used. When specifying mechanical splices in congested areas, the Designer should consider staggering splice locations in order to meet the minimum spacing of reinforcing steel according to LRFD 5.10.3.1.

Designers shall use only mechanical type splices for #14 [43M] and #18 [57M] bars.

Splicing of reinforcing by welding is not permitted.

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 tension splice lengths, see Figure 301.5.3-1.

For compression splice lengths, see Figure 301.5.3-2.

For development length requirements for reinforcing steel, see Figures 301.5.3-2, 301.5.3-3 & 301.5.3-4.

### 301.5.4  CALCULATING LENGTHS AND WEIGHTS OF REINFORCING

Reinforcing steel lengths shall be calculated to the nearest 1 inch [25 mm]. Standard bend lengths shall be based on criteria in CMS 509.

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.

The weight of the additional 1-1/2 coils of spiral required at the end by LRFD 5.10.6.2 shall be calculated and included in the estimated quantities. For one, #4 [13M] spiral with a 4½" pitch, the weight, including the 1-1/2 coils at each end, is given by the following formula:

\[
\text{Spiral Weight (lb)} = 0.148\pi H \left(\frac{4.5}{2}\right)^2 + (D - 0.5)^2 + 0.167\pi (D - 0.5)
\]

Where:
- \(D\) = Outside Diameter of the Spiral (in)
- \(H\) = Height or Length of the Spiral (ft)

\[
\text{Spiral Weight (kg)} = 8.643\pi H \left(\frac{0.115}{2}\right)^2 + (D - 0.013)^2 + 0.248\pi (D - 0.013)
\]

Where:
- \(D\) = Outside Diameter of the Spiral (m)
- \(H\) = Height or Length of the Spiral (m)
See Figure 301.5.4-1 for area, weight and diameter of standard reinforcing. See Figure 301.5.4-2 for bar bending data. See Figure 301.5.4-3 for standard bar length deductions of common bends.

### 301.5.5 BAR LIST

Bar lists should include the following:

A. Bar Mark
B. Number of bars required
C. Overall length required of the bar
D. Total Weight for each bar mark
E. Column for type of bar:
   1. "ST" for straight
   2. "Number" assigned to
   3. "Numbered Bent Bar Detail"
   4. "Number" and "Series" for series bars

Dimensions are defined by letters A through Z associated with the “Numbered Bent Bar Detail” showing position of letters.

Spiral reinforcing shall also be included in the detail plan's bar list. The following information shall be shown on the bar list:

A. Core diameter
B. Pitch
C. Mark
D. Number
E. Height
F. Weight
G. Plan note for spiral bars

A sample bar list is provided in Figure 301.5.2-1.

### 301.5.6 USE OF EPOXY COATED REINFORCING STEEL

All reinforcing steel shall be epoxy coated except as noted for prestressed box beams in Section 302.5.1.8.

All approach slabs shall have epoxy coated reinforcing steel.
### 301.5.7 MINIMUM CONCRETE COVER FOR REINFORCING

The minimum concrete cover shall be as follows:

A. Bridge decks*, slab bridges* and sidewalks (top surface) ................................... 2½ in. [65 mm]
B. Bridge decks and slab bridges (bottom surface) ............................................. 1½ in. [40 mm]
C. Footings (bottom surface) ................................................................................. 3 in. [75 mm]
D. Approach slabs (top & bottom surfaces) .......................................................... 3 in. [75 mm]
E. Column ties or spirals ......................................................................................... 3 in. [75 mm]
F. Drilled shaft ties or spirals (diameter ≤ 4.0 ft.) ................................................ 3 in. [75 mm]
G. Drilled shaft ties or spirals (diameter > 4.0 ft.) .................................................. 6 in. [150 mm]
H. Prestressed concrete I-beams & Box beams (side & bottom surfaces) .......... 1.25 in. [32 mm]
I. Prestressed concrete Box beams (inside surfaces) .......................................... 1 in. [25 mm]
J. All other concrete surfaces ............................................................................... 2 in. [50 mm]

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

Clearances not given in C&MS 509.04, C&MS 524 or referenced Standard Bridge Drawing shall be shown in the detail plans.

### 301.5.8 MINIMUM REINFORCING STEEL

Minimum reinforcing steel requirements shall conform to AASHTO requirements for shrinkage and temperature reinforcement. Reinforcement for shrinkage and temperature stresses shall be provided near exposed surfaces of walls and slabs not otherwise reinforced.

### 301.6 REFERENCE LINE

For structures on a horizontal curve a reference line, usually a chord of the curve shall be provided. This reference line should be shown on the General Plan/Site Plan view with a brief description, including, for example, “Reference Line (centerline bearing to bearing),” and the stations of the points where the reference line intersects the curve. Skews, dimensions of substructure elements and superstructure elements should be given from this Reference Line, both on the General Plan /Site Plan and on the individual detail sheets. Dimensions from the curve generally should be avoided. The distance between the curve and reference line should be dimensioned at the substructure units. In this manner a check is available to the contractor.

The reference tangent can be used if appropriate.
301.7 UTILITIES

Utilities should not be supported on the fascia of bridge decks.

Utilities, other than gas and water, may be run through sidewalk sections or parapets of bridges but shall be encased in a protective conduit.

Placing utilities through or underneath MSE walls should be avoided when possible. When it is necessary to place a utility through or beneath an MSE wall, it shall be encased in a protective conduit or casing pipe that extends ten feet [3.0 m] beyond the limits of the select granular backfill for the MSE wall. Placing pipe culverts through MSE walls should be avoided. Water and sewer lines within ten feet [3.0 m] of an MSE wall shall also be encased in a protective conduit or casing pipe.

Utility conduits embedded in concrete should be shown and dimensioned so as to clear construction joints by a minimum of one inch [25 mm] and other conduits by a minimum of 2 inches [50 mm].

No utilities shall be embedded in the actual vehicular traffic carrying section of a concrete deck.

Utilities should not be suspended below the bottom of the bridge superstructure.

For approval procedures for installation of utilities on bridges, please refer to ODOT’s “Utilities Manual.”

301.7.1 UTILITIES ATTACHED TO BEAMS AND GIRDERS

All utility lines placed between the stringers of grade separation structures should not be located in the floor panel behind the fascia stringer. This is to protect the lines from collisions.

Critical utility lines (gas, etc.) that could contribute to the severity of a collision should be located well above the bottom of the superstructure or be otherwise protected.

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 boxbeams and the deck.

No utilities shall be placed inside of box beams.

301.8 CONSTRUCTION JOINTS, NEW CONSTRUCTION

Construction joints should be anticipated and provided for in the detail plans. Joint locations should be selected such that they are aesthetically least objectionable, allow construction to be properly performed and are at locations of minimum stress. Construction joints shall be
designed to transfer all loads.

302 SUPERSTRUCTURE

302.1 GENERAL CONCRETE REQUIREMENTS

302.1.1 CONCRETE DESIGN STRENGTHS

A. Superstructure Concrete (Class S, HP or QSC2) ........................................ 4500 psi [31.0 MPa]
B. Substructure Concrete (Class C, HP or QSC1) ........................................... 4000 psi [27.5 MPa]
C. Drilled Shaft Concrete (Class S Modified) ................................................. 4000 psi [27.5 Mpa]

302.1.2 SUPERSTRUCTURE CONCRETE TYPES

302.1.2.1 CLASS S & HP CONCRETE, QC/QA CONCRETE FOR STRUCTURES & CONCRETE WITH WARRANTY

Class S Concrete is the Department’s traditional concrete mix design for superstructures.

Class HP (High Performance) Concrete mix designs are intended to give a highly dense, very impermeable concrete resulting in a longer structure life. When Class HP Concrete is specified, the Designer shall include the bid item for Class HP Concrete Test Slab. However, the bid item for Class HP Concrete Testing is no longer required because the Department has acquired sufficient test data since the inception of High Performance Concrete.

QC/QA Concrete for Structures, SS898, is a contractor designed mix that meets minimum requirements for strength, permeability and air content. QC/QA Concrete is divided into three classes: substructure (QSC1), superstructure (QSC2) and project specific (QSC3). The contractor assumes responsibility for quality control sampling and testing. Final payment for in-place concrete includes incentives for concrete meeting or exceeding minimum requirements and disincentives for concrete not meeting minimums. QC/QA concrete should not be considered for pay items with less than 100 yd³ [75 m³] of concrete.

Class S Concrete for New Bridge Decks with Warranty, SS893, and Class HP Concrete for New Bridge Decks with Warranty, SS894, are standard Class S and HP mix designs that warrant the concrete for a period of seven years against scaling, spalling and cracking. Remedial measures required during the warranty period are to be performed by the original Contractor.

The mix design, curing and placing requirements for both Class S and HP concretes are defined in the CMS.
302.1.2.2 SELECTION OF CONCRETE FOR BRIDGE STRUCTURES

The following concrete types may be specified for superstructure concrete:

A. Class S Concrete
B. Class HP Concrete
C. Class S Concrete for New Bridge Decks with Warranty
D. Class HP Concrete for New Bridge Decks with Warranty
E. QC/QA Concrete Class QSC2
F. QC/QA Concrete Class QSC3

The following concrete types may be specified for substructure concrete:

A. Class C Concrete
B. Class HP Concrete
C. QC/QA Concrete Class QSC1

Contact the District to confirm the selection of concrete type to be used for a specific structure.

High performance concrete shall not be used as a replacement for the drilled shaft concrete specified in 524.

302.1.3 WEARING SURFACE

302.1.3.1 TYPES

A. 1 inch [25 mm] monolithic concrete - defined as the top one inch [25 mm] of a concrete deck slab. This one inch [25 mm] thickness shall not be considered in the structural design of the deck slab or as part of the composite section.

B. 3 inches [75 mm] asphalt concrete - defined as the minimum asphaltic concrete wearing surface to be used on only non-composite prestressed box beams. The asphalt concrete wearing surface shall be composed as follows:
   1. 1½ inches [38 mm] of Item 448 Asphalt Concrete Surface Course, Type 1H.
   2. 1½ inches [38 mm] minimum thickness of Item 448 Asphalt Concrete Intermediate Course, Type 2, PG64-28.
   3. Two applications of Item 407 Tack Coat - one prior to placement of the intermediate course and one prior to placement of the surface course. Refer to the ODOT Pavement Design & Rehabilitation Manual, Section 404.11 for application rates.

C. 6 inches [155 mm] cast-in-place composite deck - defined as the minimum thickness of
concrete slab for composite prestressed box beams. The top 1 inch [25 mm] shall be
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considered monolithic as defined above. Also see Section 302.5.1.3.

302.1.3.2 FUTURE WEARING SURFACE

All bridges shall be designed for a future wearing surface (FWS) of 60 psf [2.87 kPa].

The future wearing surface is considered non-structural and shall not be used in design to increase the strength of the superstructure. The presence of a future wearing surface does not exclude the use of the 1 inch [25 mm] monolithic wearing surface as defined above.

302.1.4 CONCRETE DECK PROTECTION

302.1.4.1 TYPES

A. Epoxy Coated Reinforcing Steel - CMS 709.00
B. Minimum concrete cover of 2½ inches [65 mm]
C. Class S Concrete
D. Class HP Concrete
E. Drip Strips
F. CMS 512, Type D, Waterproofing or CMS 512 Type 3 Waterproofing
G. Asphaltic concrete wearing surface

302.1.4.2 WHEN TO USE

All reinforcing steel shall be epoxy coated.

All cast-in-place concrete decks shall have minimum concrete top cover of 2½ inches [65 mm].

A drip strip may be used on decks with over the side drainage.

Non-composite box beam bridges, with over the side drainage, shall have an asphalt concrete overlay. The overlay shall be placed over either Type D Waterproofing, CMS 512 or Type 3 Waterproofing, CMS 512. Minimum thickness of overlay is 3 inches [75 mm] - See Section 302.1.3.1.

302.1.4.3 SEALING OF CONCRETE SURFACES SUPERSTRUCTURE

Specifications for sealing material are defined in CMS 512. Concrete surfaces shall be sealed with an approved concrete sealer as follows: (See Figures 302.1.4.3-1 & 302.1.4.3-2)
A. Concrete slabs or concrete decks on steel superstructures with over-the-side drainage:

The exterior 9 inch [230 mm] width on the top of the deck, the deck fascia and a 6 inch [150 mm] (minimum) width under the deck shall be sealed 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:

A 9 inch [230 mm] width of the roadway along the curbline; the vertical face of curb; the top of the curb/sidewalk; the inside face, top and outside face of the parapet; the deck fascia; and a 6 inch [150 mm] (minimum) width under the deck shall be sealed with either an epoxy-urethane, or non-epoxy sealer.

C. Concrete slabs, composite prestressed box beam superstructures or concrete decks on steel superstructures with deflector parapets:

A 9 inch [230 mm] width of the roadway along the face of parapet; the inside face, top and outside face of parapet; the deck fascia; and a 6 inch [150 mm] (minimum) width under the deck shall be sealed with either an epoxy-urethane, or non-epoxy sealer.

D. Non-composite prestressed concrete box beam decks with over-the-side drainage:

The fascia of the outside beams and a minimum 6 inch [150 mm] width under the beam shall be sealed with an epoxy-urethane or a non-epoxy sealer.

E. Concrete decks on prestressed I-beam superstructures with over-the-side drainage:

The exterior 9 inch [230 mm] width on the top of the deck; 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 shall be sealed with an epoxy-urethane sealer.

F. Concrete decks on prestressed I-beam superstructures with sidewalks:

A 9 inch [230 mm] width of the roadway along the curbline; the vertical face of curb; the top of the curb/sidewalk; the inside face, top and outside face of the parapet; 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 shall be sealed with an epoxy-urethane sealer.

G. Concrete decks on prestressed I-beam superstructures with deflector parapets:

A 9 inch [230 mm] width of the roadway along the face of parapet; the inside face, top and outside face of parapet; 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 shall be sealed with either an epoxy-urethane sealer.
Concrete surfaces that include patches should be sealed with an epoxy-urethane sealer so the concrete color will remain uniform.

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

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

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

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

### 302.2 REINFORCED CONCRETE DECK ON LONGITUDINAL MEMBERS

#### 302.2.1 DECK THICKNESS

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

\[
T_{\text{min}} \text{ (inches)} = \frac{(S + 17)(12)}{36} \approx 0.858\text{ in.}
\]

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

Where: \( S \) is the effective span length in feet [millimeters] determined according to *LRFD 9.7.3.2*. \( T_{\text{min}} \) shall be rounded up to the nearest one-quarter inch [5 mm].

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

#### 302.2.2 CONCRETE DECK DESIGN

The concrete deck design shall be in conformance with the approximate elastic methods of analysis specified in the *AASHTO LRFD Bridge Design Specifications*, latest edition, and the additional requirements specified in this Manual. Refined methods of analysis and the empirical design method, *LRFD 9.7.2*, are prohibited. The design live load shall be HL-93 and the design dead load shall include an allowance for a future wearing surface equal to 0.06 k/ft\(^2\).
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 Figures 302.2.2-1, 302.2.2-2 and 302.2.2-3. These designs apply for the full length of the bridge and preclude the need for additional transverse reinforcement at supported deck ends. The design of overhang reinforcement is valid for BR-1 (36.0 & 42.0 in.), SBR-1-99, BR-2-98 and TST-1-99 barrier systems. A complete list of design assumptions is
provided with Figure 302.2.2-1.

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

302.2.3 DECK ELEVATION REQUIREMENTS

302.2.3.1 SCREED ELEVATIONS

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

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

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

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

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

302.2.3.2 TOP OF HAUNCH ELEVATIONS

Top of haunch elevations represent the theoretical bottom of deck elevation before the concrete deck is placed. Top of haunch elevations should be provided at the centerline of each girder at bearing points, quarter points, mid-span points, splice points and additional points to meet a
maximum spacing between points of 25'-0” [7.5 m]. The top of haunch elevation locations should be identified in a plan view and on the transverse section. Top of haunch elevations are not required for composite box beam bridges. Provide a plan note for a definition and description of the purpose for the top of haunch elevations (see BDM Section 700). Refer to Figure 302.2.3-1.

302.2.3.3 FINAL DECK SURFACE ELEVATIONS

Final deck surface elevations represent the position of the deck after all dead loads except future wearing surface have been applied. Final deck surface elevations shall be provided at bearing points, quarter points, mid-span points, splice points and additional points to meet a maximum spacing between points of 25'-0” [7.5 m] for each: girder centerline; curbline or deck edge; transverse grade-break line; and phased construction line. The final deck surface elevation locations should be identified in a plan view. Refer to Figure 302.2.3-1.

302.2.4 REINFORCEMENT

302.2.4.1 LONGITUDINAL

Secondary reinforcement in the top-reinforcing layer of a reinforced concrete deck on steel or concrete stringers shall be approximately 1/3 of the main reinforcement, uniformly spaced.

Research has shown that secondary bars in the top mat of reinforced concrete bridge decks on stringers should be small bars at close spacing. Therefore the required secondary bar size shall be a #4 [#13M]. The only exception to this requirement is if the bar spacing becomes less than 3 inches [75 mm].
For stringer type bridges with reinforced concrete decks, the secondary bars shall be placed above the top of deck primary bars. This helps in reducing shrinkage cracking and adds additional cover over the primary bars.

For reinforced concrete deck slabs on non-composite stringer type bridges, where the main reinforcement is transverse to the stringers, additional top longitudinal reinforcement shall be provided in the negative moment region over the piers. This additional secondary reinforcement shall be equal to the distributional reinforcement (1/3 of the main reinforcement). This additional reinforcement shall be uniformly spaced and furnished in length equal to the larger of: 40 percent of the length of the longer adjacent stringer span or a length that meets the requirements of LRFD 5.11.1.2.3.

For composite designs, the total longitudinal reinforcement over a pier shall meet the requirements of LRFD 6.10.1.7.

Additional negative moment reinforcement should be placed approximately symmetrical to the centerline of pier bearings but with every other reinforcing bar staggered 3 feet [1000 mm] longitudinally.

302.2.4.2 TRANSVERSE

To facilitate the placement of reinforcing steel and concrete in transversely reinforced deck slabs top and bottom main reinforcement shall be equally spaced and placed to coincide in a vertical plane.

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. Bridges with a skew greater than 15 degrees or where the transverse reinforcing will interfere with the shear studs should have the transverse reinforcement placed perpendicular to the centerline of the bridge. Refer to the appropriate Standard Bridge Drawing for the requirements on slab bridges.

For prestressed I-beams, transverse reinforcing shall be placed perpendicular to the centerline of the bridge.

For composite box beam decks, the transverse reinforcing steel may be placed parallel to the abutment.

For steel beam or girder bridges, the clearance of the bottom transverse bars over the top of any bolted beam splice plates or moment plates should be checked as reinforcing bars at a skew generally cannot be placed between bolt heads.

302.2.5 HAUNCHED DECK REQUIREMENTS

Concrete decks on steel beam, girder or prestressed I-beam structures shall have a concrete haunch to prevent a thinning of the deck slab as a result of unforeseen variations in beam
camber. At a minimum, the design haunch shall allow for 2 inches [50 mm] of excessive camber. For steel beam and girder structures, the haunch shall be tapered back to the original concrete deck thickness in a 9 inch [225 mm] length and the concrete haunch shall encase the edges of the top flange. See Figures 302.2.5-1 & 302.2.5-2.

302.2.6 STAY IN PLACE FORMS

Galvanized steel or any other material type, stay in place forms, shall not be used.

302.2.7 CONCRETE DECK PLACEMENT CONSIDERATIONS

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.

302.2.7.1 FINISHING MACHINES

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.
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° 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). In order 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.

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 in order 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 in order to line up with a pin connection. The figure below 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.

Grade break locations that move laterally along the length of the bridge cannot be paved in a single operation using a mechanized finishing machine and should therefore be avoided. Note that as the machine progresses forward, the truss hinge locations and the grade break locations no longer coincide. See the figure below.
302.2.7.2 SOURCES OF GIRDER TWIST

The interconnectivity between girders, intermediate crossframes/diaphragms and end crossframes/diaphragms is essential to a structure’s stability throughout the construction process. Therefore, it is of utmost importance to ensure that all crossframes/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.

302.2.7.2.a GLOBAL SUPERSTRUCTURE DISTORTION

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.
Transverse sections with more heavily loaded interior girders distort in a concave shape.

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 $N_g$) 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.

For bridges with tangent alignments and adjacent substructure skews that vary by no more than 15°, the magnitude of the girder twist can be reduced by utilizing transverse sections with balanced tributary deck loadings. For a new superstructure, the amount of girder twist due to global superstructure deformation can be neglected when the tributary deck load carried by the fascia girder does not exceed 110% of the average of the tributary deck load carried by the interior members for a given construction phase.
When the aforementioned tributary deck loading requirements of the fascia members cannot be met or, because of geometry, do not apply, the Designer shall perform a refined analysis of the superstructure system to determine the magnitude of fascia girder twist ($N_g$) 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 $N_g$ 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 crossframes/diaphragms; and increasing the stiffness of the girders. An increase in the crossframe 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.
302.2.7.2.b OIL-CANNING

Distortion due to oil-canning occurs when large lateral loads from the cantilevered deck slab falsework bracket deform the girder web.

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” of the top of the bottom flange. Given this requirement and the geometric capabilities of the falsework brackets, the magnitude of girder twist ($N_o$) resulting from oil-canning may be neglected for girder webs 84” deep or less.

For web depths greater than 84”, designers shall 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 600 for an example). The pay item for deck concrete shall be “as per plan”. Using the plan bracket location, designers shall determine $N_o$. Designers may assume the lowest location of the falsework bracket to be 76” 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. Any temporary bracing should be detailed in the plans.

The magnitude of girder twist resulting from oil-canning may be neglected for prestressed I-beam superstructures.

302.2.7.2.c GIRDER WARPING

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. crossframes). The bracket loads produce
twist between the crossframes due to a combination of girder warping and pure torsional distortion. The girder is restrained from warping at the crossframe 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.

For steel superstructures, Designers should calculate the magnitude of twist ($N_w$) due girder warping using the TAEG software developed by the Kansas Department of Transportation. TAEG (“Torsional Analysis of Exterior Girders”) is available at no cost and can be downloaded at: http://www.ksdot.org/kart/.

Since most of the data input in TAEG is dependent upon the contractor’s equipment and falsework design, designers should use conservative assumptions to accommodate most contractor resources. For design-build projects and value engineering change proposals (VECP’s), data input for TAEG shall represent the actual falsework and equipment to be used by the contractor. Designers may use the following assumptions in lieu of actual contractor supplied information:

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 crossframe 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; however, if required these should be detailed in the contract plans.
C. Permanent Lateral Support Data:

The default crossframe 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 crossframe, designers should input stiffener dimensions and select the “Diaphragms (Inputted Ix)” option. For ODOT Type 1 crossframes, designers should assume a fictitious stiffener of dimensions: 5” x 3/8”. Determine the diaphragm moment of inertia for all standard ODOT crossframes as follows:

\[ I_x = \frac{h^2 s}{4L_d^2 \left( \frac{1}{A_d L_d^2} + \frac{L_h}{A_h L_d^2 + A_d L_h^3} \right)} \]

Where:

- \( A_d \) = Area of the diagonal member (in²)
- \( A_h \) = Area of the horizontal member (in²)
- \( L_d = \sqrt{L_h^2 + h^2} \)

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.

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} lb/ft²
   \((t_{avg} = \text{Average thickness [ft.] of deck slab overhang})\)
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.
### Skew Angle Data

<table>
<thead>
<tr>
<th>Skew Angle</th>
<th>Rail-to-Rail Length, ft.</th>
<th>Extra End Length, ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.00 W</td>
<td>0.0</td>
</tr>
<tr>
<td>15</td>
<td>1.04 W</td>
<td>5.0</td>
</tr>
<tr>
<td>20</td>
<td>1.06 W</td>
<td>5.5</td>
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<tr>
<td>25</td>
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<td>9.0</td>
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<td>45</td>
<td>1.41 W</td>
<td>10.5</td>
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<tr>
<td>50</td>
<td>1.56 W</td>
<td>11.5</td>
</tr>
<tr>
<td>55</td>
<td>1.74 W</td>
<td>13.5</td>
</tr>
</tbody>
</table>

For total machine lengths of 36 ft. and less, assume a total machine weight of 7.6 kip. Add 0.09 kip for each additional foot of machine length required above 36 ft. The maximum total machine length shall not exceed 120 ft. If greater lengths are required, consult the Office of Structural Engineering for recommendations.

To determine the maximum wheel load, divide the total machine weight by 8.0.

### F. Bracket Data:

1. Refer to the following figure 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
For prestressed I-beam superstructures, Designers should verify that the intermediate crossframes/diaphragms in the exterior bay are capable of resisting the torsion caused by the cantilevered falsework.

**302.2.7.3 DETERMINING EFFECT OF GIRDER TWIST**

Once all sources of girder twist are quantified, Designers should determine the total effect that girder twist has on the finished deck surface. The primary effect of greatest concern is the loss of concrete cover over the top mat of deck reinforcing steel and the subsequent loss of deck thickness. The maximum loss due to twisting shall not exceed 0.5 in.

The total amount of girder twist at both the left and right screed rail should be determined as follows:

\[
\phi_{\text{left}} = \left(\phi_g + \phi_o + \phi_w\right)_{\text{left}} \quad \text{and} \quad \phi_{\text{right}} = \left(\phi_g + \phi_o + \phi_w\right)_{\text{right}}
\]

where:

- \(N_g\) = Girder twist due to global superstructure distortion (See BDM Section 302.2.7.2.a)
- \(N_o\) = Girder twist due to “oil-canning” (See BDM Section 302.2.7.2.b)
- \(N_w\) = Girder twist due to girder warping (See BDM Section 302.2.7.2.c)
The total amount of screed rail deflection at both the left and right screed rail should be determined as follows:

\[ \delta_{\text{left}} = \tan(\phi_{\text{left}}) \times L_b \] and \[ \delta_{\text{right}} = \tan(\phi_{\text{right}}) \times L_b \]

where:

* \( \phi_{\text{left}} \) = 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.)

The total loss of deck thickness should be determined as follows:

\[ \delta_{\text{Total}} = \frac{\delta_{\text{left}} + \delta_{\text{right}}}{2} \]

### 302.2.8 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. The slab depth dimension shall show this variation by giving the maximum and minimum depth dimensions with their respective location, over the piers, center of span, etc.

An alternate is to accommodate the differential depth by including it in the Camber Table as geometric camber.

### 302.2.9 STAGED CONSTRUCTION

For all bridge types, except non-composite concrete box beams, where the differential dead load deflection between adjacent beams, girders or structural slabs is greater than \( \frac{1}{4} \) inch [6 mm], a deck closure is required if the bridge is constructed in stages.

The closure pour between the stages shall be a minimum width of 30 inches [800 mm] but should be wide enough to accommodate the required reinforcing steel lap splices. In special cases, this distance may be reduced when mechanical reinforcing steel connectors are used (see Section 200). The mechanical connector system used shall be able to develop 125 percent of the full yield strength of the reinforcing steel as a minimum.

Intermediate cross frames and diaphragms shall not be permanently attached in the closure pour.
location until the concrete pours on both sides of the closure pour location have been completed.

The two construction joints created by the concrete closure pour should be sealed with High Molecular Weight Methacrylate (HMWM), 705.15. The sealing width shown in the plans should be 2'-0" [600 mm], centered on the construction joints.

Placement of the staged construction joints above beam flanges is not recommended. The preferred location is the positive moment regions of the cast-in-place concrete deck slab.

The designer shall provide plan notes on the stage construction details sheet that detail the sequence of construction.
CONTINUOUS OR SINGLE SPAN CONCRETE SLAB BRIDGES

Continuous reinforced concrete slab bridge design shall be in accordance with LRFD 4.6.2.3.

For simple span reinforced concrete slab bridges cast in place directly on concrete substructures, the effective span length shall be considered equal to the clear span plus 15" [380 mm].

The Designer shall include a final deck surface elevation table. Elevations shall be shown for all profile grade lines, curblines, crownlines, and phased construction lines for the full length of the bridge. Bearing points, quarter-span points and mid-span points shall be detailed as well as any additional points required to meet a maximum spacing between points of 30'-0" [10 m].

STRUCTURAL STEEL

GENERAL

Structural steel should be designed utilizing a composite section. Refer to LRFD 6.10.10.1 and BDM Section 1006 for more information.

A non-composite design may be used only if the design is the most economical.

The laterally unsupported length of top flanges of beam and girder members with a concrete deck encasing the top flange or compositely designed with studs shall be considered to be zero. In the absence of such fastening or direct contact of an individual beam or girder member, the unsupported length shall be considered as the distance between the diaphragms, struts, bridging, or other bracing.

For designs that assume the unbraced length of the top flange to be zero as mentioned above, the designer shall investigate the strength of the non-composite section during steel erection, deck slab construction, etc. using laterally unsupported lengths that reflect actual bracing conditions.

MATERIAL REQUIREMENTS

Types of steel to be selected for use in the design and construction of bridges is as follows:

A. ASTM A709[M] grade 50W shall be specified for an un-coated weathering steel bridge.
B. ASTM A709[M] grade 50 shall be specified for a coated steel bridge.
C. ASTM A709[M] grade 36 is not recommended and is being discontinued by the steel mills.
D. High Performance Steel (HPS), A709[M] grade 70W, un-coated weathering steel 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 provided in the appendix.
There are several systems available for coating steel bridges. These coatings are specified in CMS 514 or by the plan notes provided in the appendix. See Section 302.4.1.5 for guidelines on selecting a coating system.

### 302.4.1.2 ATTACHMENTS

Detail plans of steel beam and girder bridges shall show where welded attachments are allowed for construction purposes.

Welding of attachments, either permanent or temporary, is not acceptable in tension areas. Welding is allowed in compression areas. Detail plans shall show the extent of compression and tension areas.

Welding of scuppers, down spouts or drainage supports should not be allowed in tension areas of main members.

### 302.4.1.3 STEEL FABRICATION QUALIFICATION

The Department’s requirements for steel fabricators are defined in CMS 513 and Supplement 1078. 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:

A. AISC Category Sbr - Fabricators qualified for single span rolled beam bridges
B. AISC Category Mbr - Fabricators qualified for all other bridge structures
C. AISC has also established a P and F endorsement for fabricators:
   1. P - Painting of steel structures endorsement
   2. F - Fracture Critical endorsement

### 302.4.1.4 MAXIMUM AVAILABLE LENGTH OF STEEL MEMBER

Mills can supply lengths up to maximum shipping limits, but extra charges may be added for lengths over 80'-0" [24 meters]. The designer should consider cost by providing for field splices and allowing for optional field splices. The National Steel Bridge Alliance (NSBA) and the American Iron and Steel Institute (AISI) are available to provide assistance with material sizes.

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'-0" [36 meters] is generally the maximum trucking length between splices, but girder lengths of 160'-0" [49 meters] and greater have been transported to project sites.
302.4.1.5 STRUCTURAL STEEL COATINGS

This section shall serve as a guide in selecting corrosion control systems.

302.4.1.5.a PRIMARY COATING SYSTEMS

The Department’s primary system is un-coated weathering steel. Weathering steel reduces the initial cost of the structure by approximately 3 dollars per square foot [30 dollars per square meter] of steel surface area. It may also eliminate future maintenance coatings. However, weathering steel structures should be monitored on a five year cycle to determine if section loss justifies a partial or total structure maintenance coating. Contact the Office of Structural Engineering for further guidance on field monitoring.

If a site-specific study finds that un-coated weathering steel should not be used, specify a coated non-weathering grade of steel.

Site-specific studies should consider the following:

A. Salt usage tons per lane mile
B. Site condition potential for tunnel like conditions and salt spray from under passing traffic.
   These are structures or zones that receive deposits of salt and high humidity or long term wet conditions.

Additional resources available to aid in the site studies include:

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

Un-coated weathering steel shall have a protective coating applied to a 10'-0" [3 meter] length of beam or girder adjacent to abutments with expansion joints and on both sides of intermediate expansion joints. All cross frames, end frames or other steel in this 10'-0" [3 meter] section shall also be coated. The top coat shall be tinted to a color closely matching Federal Standard No. 595B-20045 or 20059, the color of weathering steel.

Un-coated bridges with integral or semi-integral abutments shall not have this protective coating.

Aesthetics may dictate coating the fascia lines of an otherwise un-coated weathering steel bridge. If this treatment is selected, the use of darker fascia colors (forest green, medium to dark browns,
dark blues, rustic reds) may provide a more homogenous look than the light neutral colors.

New steel structures, including the aforementioned ends of weathering steel beams and girders, shall receive a three-coat paint system consisting of an inorganic zinc prime coat, an epoxy intermediate coat and a urethane finish coat (formerly called system IZEU). The top coat shall be tinted to a color closely matching Federal Standard No. 595B-20045 or 20059, the color of weathering steel. The inorganic prime coat is shop applied while the intermediate and top coats are field applied. This system has proven to have a life span of up to 30 years.

302.4.1.5.b ALTERNATIVE COATING SYSTEMS

Special conditions or member size may warrant the use of other coating systems.

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.

The IZEU three coat shop applied system may provide a better protective coating than the standard IZEU 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 IZEU 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.

Required plan or proposal notes for the IZEU three-coat shop applied systems are located in the appendix.

A galvanized coating system is an alternative for new steel when the requirements addressed in Section 302.4.2.1 are met.

Galvanized systems are proven durable up to 40 years. Additional information is also available from the American Galvanizers Association.

Required plan or proposal notes for the galvanized coating system are located in the appendix.

A shop applied metallizing system is another alternative coating system, but currently 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.

Required plan or proposal notes for the metallized coating system are located in the appendix.

302.4.1.6 STEEL PIER CAP

Steel pier caps are non-redundant, fracture critical members. As specified in Section 301.2, these structure types require a concurrent detail design review to be performed by the Office of Structural Engineering. In general, structure designs that require stringers to be continuous through, and in the same plane with a steel pier cap or cross beam, should be avoided if at all possible.

302.4.1.7 OUTSIDE MEMBER CONSIDERATIONS

The designer is to evaluate the actual loads for outside main members. 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.").

In order to facilitate forming, deck slab overhang should not exceed 4'-0" [1200 mm]. On over the side drainage structures the minimum overhang shall be 2'-3" [700 mm]. Where scuppers are required for bridge deck drainage the overhang shall be 1'-6" [450 mm].

302.4.1.8 CAMBER AND DEFLECTIONS

When establishing dead load deflection for determining the required shop camber of non-composite steel beam or girder bridges with concrete deck slabs and determining deck screed elevations, the weight of curbs, railings, parapets and separate wearing surface, may be equally distributed to all beams. Future wearing surfaces shall not be included in determining required camber. This weight may be assumed (for dead load deflection only) to be supported by the beams acting compositely, based on a moment of inertia approximately twice that of the beam. Therefore, deflection due to dead loads above the deck slab may be based on one-half of the weight distributed to each beam, using the beam moment of inertia.

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, railings, parapets and separate wearing surface may be equally distributed to all beams. Future wearing surfaces shall not be included in determining required camber.

The deflection and camber table in the design plans shall detail all points for each beam or girder line for the full length of the bridge. Bearing points, quarter-span points, mid-span points and splice points shall be detailed and any additional points required to meet a maximum spacing
between points of 30'-0" [10 meters].

In cases of special geometry, i.e. spirals, horizontal or vertical curves, superelevation transitions, etc., additional points are to be detailed in the deflection and camber table if the normally required points do not adequately define a beam or girder required curvature.

The required shop camber shall in all cases be the algebraic sum of the computed deflections, vertical curve adjustment, horizontal curve adjustment and adjustment due to heat curving. Camber shall be measured to a chord between adjacent bearing points.

A camber diagram shall be provided showing the location of the points developed above and giving vertical offset dimensions at the bearing points from a “Base” or “Work” line between abutment bearings.

302.4.1.9 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 negative moment regions should not exceed the nominal fatigue resistance for Detail Category C.

302.4.1.10 TOUGHNESS TESTS

On steel structures, main load carrying members, such as beams, moment plates, bolted joint splice plates (excluding fill plates) require Charpy V Notch Testing. These components shall be identified on the detail plans by placing “(CVN)” after the component's description.

Example: W36x150 (CVN) [W920 x 223 (CVN)]

The web and all flanges of plate girders shall be CVN material.

Cross frame members, cross frame connection stiffeners and any steel connecting these elements on horizontally curved beam or girder structures are considered main members and shall require and be identified on the detail plans as CVN.

302.4.1.11 STANDARD END CROSS FRAMES

End cross frames for needed support and reduction of deflection of expansion devices should be designed to provide support at intervals not exceeding 4'-0" [1200 mm]. Standard expansion joints have designs already established as part of the standard drawings. For suggested details of special conditions review existing expansion joint Standard Bridge Drawings.
302.4.1.12  BASELINE REQUIREMENTS FOR CURVED AND DOG-LEGGED STEEL STRUCTURES

CMS 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 feet [3000 mm] of length. The designer shall provide this baseline in the plans along with enough information for the fabricator to be able to readily calculate the required offsets. The requirement for this information is especially critical on structures located on a curve or spiral or having other complex geometry.

302.4.1.13  INTERMEDIATE EXPANSION DEVICES

Intermediate expansion devices for a structure, if required, shall be located over a pier and the structural members shall be designed to be discontinuous at that pier.

302.4.1.14  BOLTED SPLICES

For galvanized structures, the bolt hole size requires a 1/16 inch [1.5 mm] increase over a standard hole size to allow for the additional thickness of the zinc coating. This increase in hole size decreases the splice capacity.

Bolt allowable stresses for painted surfaces or unpainted weathering steel surfaces shall be based on AASHTO's values for Class A, Contact Surface, Standard Hole Type.

Bolt allowable stressed for metallized surfaces shall be based on AASHTO’s values for Class C, Contact Surface, Standard Hole Type.

Bolt allowable stressed for galvanized surfaces shall be based on AASHTO’s values for Class C, Contact Surface, Oversized Hole Type.

Beams having bolted splices at bend points shall have additional details incorporated in the plans to completely detail the joint requirements. The minimum edge distances specified in AASHTO shall be provided at the edges of all main members and splice plates.

For splices at bend points the lines of holes in the beam or girder flanges should be parallel to the centerline of the web. If the bend angle is small enough use rectangular splice plates (splice plates should not overhang flange by more than ½ inch [13 mm] and inside splice plates should not have to be trimmed to clear web or web to flange radius). When the angle is too large to allow rectangular splice plates the plates should be trimmed to align with the flange edges. In either case minimum edge distances shall be met.

Bolted compression splices, such as in a column, while designed as a friction type connection, also require the ends of the spliced members to be in full bearing by milling of the ends. For compression splice members with milled ends the requirements of LRFD 6.13.6.1.3 shall be met.
The designer should recognize that “FULL BEARING” of beams and girders is not defined by AASHTO. “FULL BEARING” has been generally defined by ODOT as 75 percent of the bearing surface in contact and the other 25 percent with no gap greater than 1/32 inch [0.8 mm]. The designer shall specify the required fit definition when designing in conformance to the AASHTO design requirements for bolted splices in compression members.

Refer to Figure 302.4.1.14-1 for additional bolted splice details.

302.4.1.14.a BOLTS

Field splices in beams and girders shall be bolted connections using high strength bolts, ASTM A325[M].

The designer shall specify the diameter of the bolts and check that the type (Type I for Galvanized or Type III for Weathering) of A325[M] bolts is described in the coating notes or bolt material specifications.

Coating systems that are zinc based, such as OZEU, IZEU, Galvanizing or Metallizing require galvanized Type I bolts.

Un-coated weathering steel structures shall have A325[M], Type III bolts. If the faying surfaces under both the head and nut of every bolt of a weathering steel member are coated, specify galvanized A325[M] Type I bolts. Otherwise, specify A325[M], Type III bolts.

Generally, bolted splices should be designed using 1 inch [25 mm] or 1½ inch [29 mm] diameter bolts. No metric bolts or studs are available in the small quantities required for bridges.

302.4.1.14.b EDGE DISTANCES

The edge distances provided in LRFD 6.13.2.6.6 are absolute minimums allowed during fabrication. For design and detailing purposes, 0.25 in. shall be added to the minimum edge distances listed in Table 6.13.2.6.6-1.

This increase will allow for fabrication tolerances when drilling bolt holes in splice plates, especially the inside flange splice plates.

302.4.1.14.c LOCATION OF FIELD SPLICES

Generally bolted splices should be located at points of dead load contraflexure on a continuous structure. Splices may also be supplied to help meet shipping and handling limitations. Plans should show optional field splice locations.
302.4.1.15 SHEAR CONNECTORS

Design shear connectors in accordance with *LRFD 6.10.10*.

Shear connectors shall be automatic end-welded stud-type. The use of channel sections is not
allowed. 7/8 inch [22 mm] diameter studs are recommended as a standard diameter. The length of stud specified should be checked with manufacturers as to availability.

The Department’s policy of using a 2 inch [50 mm] deep haunch over the top flange will have an effect on the length of shear studs.

Shear studs shall be field installed. In the case of galvanized structures, the design plans shall allow shop installation of studs prior to galvanizing or field installation after removing the coating by grinding at each stud location. If the studs are shop installed, the Contractor will be responsible for meeting all applicable OSHA requirements. A Detail note is available in Section 700.

### 302.4.2 ROLLED BEAMS

Effective in January 2006, the producers of rolled beams implemented changes to the physical dimensions of the W36X16 group of shapes (i.e. beams with 16” and wider flanges). The traditional W36X16 series of shape sizes will no longer be available from the producers. Below is a complete list for the new W36X16 group of shapes.

<table>
<thead>
<tr>
<th>Designation</th>
<th>Area, $A$ (in$^2$)</th>
<th>Depth, $d$ (in)</th>
<th>Flange Width, $b_f$ (in)</th>
<th>Thickness, $t_f$ (in)</th>
<th>Web Thickness, $t_w$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
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### 302.4.2.1 GALVANIZED BEAM STRUCTURES

If a galvanized bridge structure is the selected structure type, the following problems should be recognized and dealt with by the designer.

Galvanizing tanks are shallow and normally not longer than 45 feet [13.7 meters] in length. Therefore, beam lengths should not be longer than 60 feet [18.5 meters]. Before a design is
completed, the designer should confirm with local galvanizers if a local plant can galvanize the structural members detailed.

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

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

302.4.2.2 STIFFENERS

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

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

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

302.4.2.3 INTERMEDIATE CROSS FRAMES

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

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

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

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

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

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

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

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

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

The design plans shall show:

A. The maximum cross frame spacing for each region along the length of the stringer. Actual spacing of the cross frames should be left to the steel fabricator’s detailer.

B. The typical cross frame details or reference to the General Steel Details Standard Bridge Drawing for standard cross frame configurations. If a design requires a specific location of cross frames, clearly show the cross frame locations that cannot be adjusted.

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

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

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

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

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

Cross frames for curved stringers may be one of the types shown on the Standard Bridge Drawing with an additional top strut. The designer shall confirm that the standard cross frames and their connections meet the additional loading developed in a curved member design. Since
cross frame components in a curved structure share the live loading, Charpy V-notch (CVN) testing shall be specified. If specially designed cross frames are used, they should be bolted to stiffeners with oversized holes. The designer shall recognize the reduction in allowable capacity associated with oversized holes. If the capacity reduction is too much to allow for oversized holes and standard holes are required, the designer shall denote on the plans that shop assembly of the specially designed cross frames and adjacent curved member is required.

Both doglegged stringer cross frames at the dogleg or curved stringer cross frames shall be connected to the main member by use of welded stiffeners.

302.4.2.4 WELDS

CMS 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.

The designer should 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. The designer should not select the joint configuration to be used for a full penetration weld. This should be left to the fabricator and the welding code.

302.4.2.4.a MINIMUM SIZE OF FILLET WELD

Fillet welds shall be designed for required stresses but should 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 inch [6 mm] leg for up to 3/4 inch [19 mm] thick material.
C. 5/16 inch [8 mm] leg for greater than 3/4 inch [19 mm] material.

302.4.2.4.b NON-DESTRUCTIVE INSPECTION OF WELDS

Non-destructive testing (NDT) of welds is defined in CMS 513. The designer should be familiar with and understand these NDT requirements and their application.

For any special NDT inspection of unique or special welded joints, the designer should clarify
the NDT requirements with the Structural Steel Section of the Office of Structural Engineering. A plan note defining any special requirements is required.

302.4.2.5 MOMENT PLATES

Fully welded moment plates shall not be used in areas of tensile stress due to the poor fatigue characteristics. 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. Welded moment plates shall not extend into a zone where the calculated total stresses are tensile. Designers should consult the Office of Structural Engineering for recommended moment plate details.

302.4.3 GIRDERS

302.4.3.1 GENERAL

Multiple designs should be investigated to determine the most economical. Often a design with an unstiffened web, eliminating transverse stiffeners, is the most economical. A design with a thicker 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. Fabrication costs should be reviewed with the Department.

Longitudinal stiffeners shall not be used.

For haunched girders the corner between the flat bottom flange bearing seat area and the curved section of the bottom flange should be detailed as two plates with a full penetration weld. The fabricator shall be given 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 Section 700.

In applying the above practices, consideration should also be given to the availability of plate lengths. Plates should not be extended beyond the lengths that can be furnished by the rolling mills.

302.4.3.2 FRACTURE CRITICAL

This section is not intended to recommend fracture critical designs. The designer should make all efforts to not develop a structure design that requires fracture critical members. As specified in Section 301.2, structures with fracture critical details require a concurrent detail design review to be performed by the Office of Structural Engineering.

Fracture critical members are defined in Section 2, Definitions, of the AASHTO/AWS D1.5,
chapter 12 Fracture Control Plan.

If a bridge design includes any members or their components that are fracture critical, those members and components should be clearly identified as FRACTURE CRITICAL MEMBERS (FCM) in the plans. Fracture critical welds shall also be designated FCM in the plans. Include the detail note provided in Section 700 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. The designer shall designate the tension and compression zones in the fracture critical members.

302.4.3.3 WIDTH & THICKNESS REQUIREMENTS

302.4.3.3.a FLANGES

In addition to design limitations of width to thickness, flanges shall be wide enough such that the girder will have the necessary lateral strength for handling and erection. The following two empirical rules shall apply:

A. \( b_f = \frac{d_w}{6} + 2.5 \) $12''$

B. \( b_{fc} \geq \frac{L}{85} \)

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.)

Whenever possible, use constant flange widths throughout the length of the girder. The minimum thickness for any girder flange shall be 7/8 inch [22 mm]. Generally, selection of flange thicknesses should conform to the following:

A. For material 7/8" [22 mm] to 3" [76 mm] thick, specify thickness in 1/8" [2 mm] increments.
B. For material greater than 3" [76 mm] thick, specify thickness in 1/4" [5 mm] increments.

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[M] grade 36 steel:
   \[300 \text{ lb} + (25 \text{ lb} \times \text{cross sectional area, in}^2, \text{of the lighter flange plate})\]
   \[135 \text{ kg} + (0.0175 \text{ kg} \times \text{cross sectional area, in mm}^2, \text{of the lighter flange plate})\]

B. For A709[M] grade 50 & 50W steel, the cutoff point shall be 85 percent of the value for grade 36 material.

302.4.3.3.b WEBS

The minimum web thickness shall be 3/8 inch [10 mm].

See Section 302.4.3.1 for recommendations on use of unstiffened web designs.

302.4.3.4 INTERMEDIATE STIFFENERS

Intermediate web stiffeners shall be a minimum 3/8 inch [10 mm] thickness. Stiffeners that extend beyond the edge of flange shall be clipped at a 45° angle. All intermediate stiffeners should be the same size.

Where intermediate stiffeners are to be used for the purpose of stiffening the web, it is preferable to use single stiffeners on alternate sides of the web of interior girders and only the inside of the web for fascia girders. These stiffeners shall be welded to the web and the compression flange. The tension flange shall be a tight fit.

Stiffeners shall be provided for the attachment of cross frames and shall be welded to the web and both flanges to help eliminate cracking of the web due to out of plane bending. The designer shall investigate that the fatigue criteria is met in these areas.

Stitch welding or single sided welding is not acceptable.

Stiffener plates shall have corners in contact with both web and flange clipped. The clip dimensions shall be 1 inch [25 mm] horizontally and 2½ inches [65 mm] vertically.

For details of stiffeners refer to the General Steel Details Standard Bridge Drawing.
302.4.3.5 INTERMEDIATE CROSS FRAMES

Cross frames for girders shall be connected to intermediate web stiffeners as shown in the Standard Bridge Drawing for General Steel Details.

For plate girder bridges, erection bolts shall be provided for the connections of cross frames to girder stiffeners. Erection bolts are normally 5/8 inch [16 mm] 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 Section 302.4.2.3

302.4.3.6 WELDS

CMS 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 advantages. Therefore, the designer should not specify the process.

The designer should 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. The designer should not select the joint configuration to be used for a full penetration weld. This should be left to the fabricator and the welding code.

For full penetration welds splicing flange materials or web materials a plan note should be added requiring removal of the weld reinforcement by grinding in the direction of the main stresses. The removal of reinforcement improves fatigue characteristics and makes NDT interpretation easier.

302.4.3.6.a TYPES

There are generally two (2) types of welds acceptable for bridge fabrication, fillet and complete penetration welds.

302.4.3.6.b MINIMUM SIZE OF FILLET AND COMPLETE PENETRATION WELDS, PLAN REQUIREMENTS

Fillets welds shall be designed for required stresses but should 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 inch [6 mm] leg for up to 3/4 inch [19 mm] thick material.
C. 5/16 inch [8 mm] leg for greater than 3/4 inch [19 mm] material.

Complete or full Penetration welds are by definition welded through the full section of the plates to be joined. No partial penetration welds are acceptable for use except in secondary members not subject to tension or reversal stresses.

The designer should specify either fillet weld leg size, in the case of fillet welds, or CP (complete penetration) for complete joint penetration groove welds. The designer should 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, detail plans for steel girders should designate all flange butt welds that are subjected to compressive stresses only. This designation should be made by placing the letters “CS” next to full penetration welds shown on detail drawings. The following explanatory legend should be placed on the same detail sheet:

CS - indicates butt weld subject to compressive stresses only.

302.4.3.6.c INSPECTION OF WELDS, WHAT TO SHOW ON PLANS

Non-destructive testing (NDT) of welds is defined in CMS 513. The designer should be familiar with and understand these NDT requirements and their application.

For any special NDT inspection of unique or special welded joints, the designer should clarify the NDT requirements with the Structural Steel Section of the Office of Structural Engineering. A plan note for any special requirements shall be necessary in the design plans.

On railroad bridges, when full penetration web to flange welds are specified, the designer should add a note requiring 10 percent ultrasonic inspection. (The designer should check the AREMA specifications and with the actual railroad to confirm the individual railroad's requirements for NDT of welds.)

302.4.3.7 CURVED GIRDER DESIGN REQUIREMENTS

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 according to Section 302.4.2.3. The Designer shall 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. Such 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.

The Designer shall supply 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.

302.5  PRESTRESSED CONCRETE BEAMS

Model multi-span, non-composite members as simple-span for all loading conditions. The live load and future wearing surface shall be as defined in Section 301.4.

Model multi-span, composite members as simple-span for non-composite dead loads and continuous span for live load and composite dead loads. The live load and future wearing surface shall be as defined in Section 301.4.

302.5.1  BOX BEAMS

Physical dimensions and section properties of box beam cross sections shall be as shown on the Prestressed Concrete Box Beam Bridge Details, Standard Bridge Drawing.

Box beams should be limited to a maximum skew of 30 degrees.

Multiple span box beam bridges shall be joined over the piers with a T-joint as shown in the Standard Bridge Drawing. Structurally, non-composite beams shall be designed as simple spans. Composite beams shall be designed as simple span for non-composite dead loads and continuous for live loads and composite dead loads.

Expansion at the piers shall be accommodated by elastomeric expansion bearings or by flexibility of the piers for integral designs.

The length of abutment seats of prestressed concrete box beam bridges should be long enough to accommodate the total width out-to-out of all beams including a fit-up allowance of ½ inch [12 mm] per joint between beams.

In order to keep the beam seat from extending beyond the fascia of any pier of a box beam bridge, the length of the pier seat should only include a fit-up allowance for the joints between
the beams of 1/4 inch [6 mm] per joint.

For box beam bridges that have skew combined with grade or which have variable superelevation, beam seats shall be designed and dimensioned 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, the difference should be evenly divided so that the box beam seat cross slopes at both bearings are made to be the same. This adjustment gives the box beam full support at the seat without creating any twist or torsion on the box beam. Any elevation differences created by this beam seat adjustment should be adjusted for in the overlay, whether asphaltic or concrete.

Prestressed box beam members shall be supported by two bearings at each support.

Abutment wingwalls above the bridge seat and backwalls should not be cast until after box beams have been erected. The cast in place wingwall and box beam should normally be separated by one inch [25 mm] joint filler, CMS 705.03. The designer should show both requirements in the plans. 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.

For box beam bridges with steel railing, the post spacing and position of post anchorage shall be detailed on the plans. The designer shall check that the post anchor spacing does not interfere with tierod locations or the "T" joint over the pier. The designer should confirm that post anchors at the ends of skewed box beams have both adequate concrete cover and do not interfere with the tierods. If the designer finds that no post spacing option can comply with the above requirements, the option of relocating the tie rods may be chosen. See standard drawings for maximum allowable spacing of tie rods.

When the box beam ends are not completely encased in concrete, the Standard Bridge Drawing requires Type B waterproofing on the ends. When required, Designers shall include a pay item for Item 512, Type B Waterproofing, in the estimated quantities.

302.5.1.1 DESIGN REQUIREMENTS

In order to prevent fabrication mistakes for beam length, the effect that the longitudinal grade has on dimensions measured along a beam’s length should be addressed in the plans. When the beam length measured along the grade differs from the beam length measured horizontally by more than 3/8" [10 mm], all affected dimensions measured along the length of the beam should be clearly labeled so that the fabricator can make the necessary allowances in the shop drawings. A Typical Detail note is available in Section 700.
302.5.1.2 STRANDES

Debonding of strands, by an approved plastic sheath, shall be done to control stresses at the ends of the beams. Refer to Section 302.5.2.2.d for debonding limits.

Deflecting of strands in box beams to limit stresses shall not be allowed.

The designer shall show on the plans the number, spacing and length of debonding. The box beam fabricator may have the option to change the position of debonding as long as the change is still symmetrical.

All strands extended from a beam to develop positive moment resistance shall not be debonded strands.

302.5.1.2.a TYPE, SIZE OF STRANDES

A. Low-relaxation ½ inch diameter \((A_s = 0.153 \text{ in}^2)\) seven wire uncoated strands, ASTM A416, Grade 270.
   
   Low-relaxation 12.7 mm diameter \((A_s = 99 \text{ mm}^2)\) seven wire uncoated strands, ASTM A416M, Grade 270.

B. Low-relaxation ½ inch diameter \((A_s = 0.167 \text{ in}^2)\) seven wire uncoated strands, ASTM A416, Grade 270.
   
   Low-relaxation 12.7 mm diameter \((A_s = 108 \text{ mm}^2)\) seven wire uncoated strands, ASTM A416M, Grade 270.

C. Low-relaxation 0.6 inch diameter \((A_s = 0.217 \text{ in}^2)\) seven wire uncoated strands, ASTM A416, Grade 270.
   
   Low-relaxation 15.24 mm diameter \((A_s = 140 \text{ mm}^2)\) seven wire uncoated strands, ASTM A416M, Grade 270.

Consult the Office of Structural Engineering and the Ohio/Indiana/Kentucky Prestressed Concrete Institute prior to specifying 0.6 inch [15.244 mm] diameter or larger strand sizes.

302.5.1.2.b SPACING

Strands shall be spaced at increments or multiples of 2 inches [50 mm].

The location of the centerline of the first row of strands shall be 2 inches [50 mm] from the bottom of the beam. If possible, all strands shall be completely enclosed by the #4 [#13M] stirrup bars. For designs that cannot meet this requirement, the minimum distance from the side of the beam to the centerline of the first strand shall be 2 inches [50 mm]. Strands near the top flange shall be placed below all transverse and longitudinal reinforcing steel and to the left and right of the void.
302.5.1.2.c STRESSES

Initial prestressing loads for low-relaxation strand shall be according to AASHTO requirements and shall be detailed on the plans.

Initial stress \( \leq 0.75 f_p = 202,500 \text{ psi} \)

Initial tension load \( = 30,982 \text{ lb/strand} \) \( (A_S = 0.153 \text{ in}^2) \)

\( = 33,818 \text{ lb/strand} \) \( (A_S = 0.167 \text{ in}^2) \)

Initial stress \( \leq 0.75 f_p = 1400 \text{ Mpa} \)

Initial tension load \( = 138,600 \text{ N/strand} \) \( (A_S = 99 \text{ mm}^2) \)

\( = 151,200 \text{ N/strand} \) \( (A_S = 108 \text{ mm}^2) \)

302.5.1.3 COMPOSITE

Composite reinforced deck slabs on prestressed box beams shall be a minimum of 6 inches [155 mm] thick and shall be reinforced with #6 [#19M] bars. The longitudinal bars shall be spaced at 18" [450 mm] and the transverse bars spaced at 9" [225 mm]. 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.

On multiple span composite box beam bridges additional longitudinal reinforcing steel over the piers is required. The additional bars shall be alternately spaced with the standard longitudinal reinforcement and the pier bar's length shall be equal to the larger of: 40 percent of the length of the longer adjacent stringer span or a length that meets the requirements of LRFD 5.11.1.2.3. The pier bars should be placed longitudinally and approximately centered on the pier but with a 3 foot [1000 mm] stagger.

In the negative moment regions of structures made continuous over the piers, the tensile stresses in the precast section shall not exceed the AASHTO allowable stresses for members with bonded reinforcement. Unless a more precise method of analysis is performed, the composite structure shall be conservatively modeled as a continuous beam on a single support centered on the pier.

Composite box beam structures with concrete parapets or sidewalks should not incorporate fit-up tolerances in the finished roadway width. To compensate for fit-up tolerances the composite deck and barrier and/or sidewalk should be designed to cantilever or overhang the boxbeam units by 2" [50 mm] to 8" [200 mm] each side with the fit-up being absorbed in the overhang. A mixture of 48" [1220 mm] and 36" [915 mm] boxbeam units may be necessary to meet this requirement.

See Figure 302.5.1.3-1 for a sketch of the cross-section of the composite deck superstructure.
302.5.1.4 NON-COMPOSITE WEARING SURFACE

Non-composite box beam bridges with asphalt overlays shall have either Type D Waterproofing or Type 3 Waterproofing as specified in CMS 512 placed on the boxes before the 1½ inch [38 mm] minimum layers of CMS type 448 asphaltic concrete is applied. See section 302.1.3.1. The Type 3 Waterproofing is preferred.

Non-composite box beam bridges with asphalt overlays shall be limited to a 4 percent combined grade. Combined grades greater than 4 percent require a composite deck design. Combined grade includes both the longitudinal and transverse structure grades calculated as follows:

\[
\text{Combined Grade (Cg)} = \left( \text{deck slope}^2 + \text{transverse grade}^2 \right)^{\frac{1}{2}}
\]

302.5.1.5 CAMBER

In establishing bridge seat elevations and assuring a minimum design slab or overlay thickness, allowance shall be made for camber due to prestressing according to the following:

\[
\begin{align*}
A &= \text{Minimum topping thickness} \\
B &= \text{Anticipated total mid-span camber due to the design prestressing force at time of release} \\
C &= \text{Mid-span deflection due to the self weight of the beam (including diaphragms)} \\
D &= \text{Mid-span deflection due to dead load of the topping and other non-composite loads} \\
E &= \text{Mid-span deflection due to dead load of railing, sidewalk and other composite dead loads not including future wearing surface} \\
F &= \text{Adjustment for vertical curve (Positive for crest vertical curves)} \\
G &= \text{Total topping thickness at beam bearings} = A + 1.8B - 1.85C - D - E - F. \text{ If } F > 1.8B - 1.85C - (D + E) \text{ then } G = A. \\
H &= \text{Total topping thickness at mid-span} = A. \text{ If } F > 1.8B - 1.85C - (D + E) \text{ then } H = A - (1.8B - 1.85C) + D + E + F.
\end{align*}
\]

Use the gross moment of inertia for the non-composite beam to calculate the camber and deflection values B, C, and D. For E, use the moment of inertia for the composite section when designing a composite box beam otherwise use the non-composite section. Note that with the exception of when F > 1.8B - 1.85C - (D + E), the dead load deflection adjustment (D + E) is made by adjusting the beam seat elevations upward.

The designer shall provide the camber at the time of release (B–C), the camber at the time of erection (1.8B – 1.85C), long term camber (2.45B – 2.40C), and a longitudinal superstructure cross section in the plans. For non-composite beams, show the thickness of the Item 448 Intermediate course and the Item 448 surface course at each centerline of bearing and at mid-span points. For composite beams, show the total topping thickness at each centerline of bearing and at mid-span points and provide a screed elevation table.
302.5.1.6 ANCHORAGE

In a box beam design, all beams shall be anchored at abutments and piers. The anchor shall be in the center of the cross section of the box beam and shall conform to details presented in the Standard Bridge Drawing.

Fixed end anchor dowels shall be installed with a non-shrinking grout (mortar). Expansion end anchor dowel holes shall be filled with joint sealer, CMS 705.04.

Preformed expansion joint filler, 705.03, the same thickness as the elastomeric bearing, shall be installed under the box beam, around the anchor dowel, to halt the grout or sealer from leaking through to the beam seat.

302.5.1.7 CONCRETE MATERIALS FOR BOX BEAMS

The designer has a choice of 28-day compressive strengths ranging from 5500 psi [38 Mpa] to 7000 psi [48 Mpa]. The 28-day compressive strength chosen for design shall be listed in the contract plan General Notes.

The designer has a choice of compressive strength at the time of release ranging from 4000 psi [27.5 Mpa] to 5000 psi [34.5 Mpa]. The release strength chosen for design shall be listed in the contract plan General Notes.

Cast-in-place concrete for composite decks, pier “T” sections, etc., shall be Class S or HP superstructure concrete - 4500 psi [31.0 MPa] at 28 days.

For concrete in composite decks see Section 302.1.2.

302.5.1.8 REINFORCING

Epoxy coated reinforcing steel shall be used in composite deck slabs and shall be Grade 60 [420], Fy = 60 ksi [420 MPa].

Reinforcing steel used in the standard design box beams is Grade 60 [420], Fy = 60 ksi [420 MPa].

The fabricator, by specification, is required to use a corrosion-inhibiting admixture in the concrete. Reinforcing bars projecting from the prestressed members shall be epoxy coated.

302.5.1.9 TIE RODS

Tie rods shall be provided and installed according to the Prestressed Concrete Box Beam Bridge standard bridge drawing.

Diaphragms and transverse tie rods for prestressed concrete box beam spans shall be provided at
mid-span for spans up to 50 feet [15 000 mm], at third points for spans from 50 feet [15 000 mm] to 75 feet [23 000 mm] and at quarter points for spans greater than 75 feet [23 000 mm].

302.5.2 I-BEAMS

AASHTO standard prestressed I-beam shapes, type II through type IV and modified type IV, as shown in standard bridge drawing PSID-1-99, shall be used.

Consult the Office of Structural Engineering and the Ohio/Indiana/Kentucky Prestressed Concrete Institute to review details and assess the constructability of I-beam cross-sections other than those shown on Standard Bridge Drawing PSID-1-99.

In designing prestressed I-beams, the non-composite section shall be used for computing stresses due to the beam and deck slab. The composite section shall be used for computing stresses due to the superimposed dead, railing and live loads. For multiple span continuous structures, the non-composite loadings shall be applied to the I-beam modeled as a simple span, and the composite loadings applied to the beam modeled as a continuous structure. Unless a more precise method of analysis is performed, the composite structure shall be conservatively modeled as a continuous beam on a single support centered on the pier.

302.5.2.1 DESIGN REQUIREMENTS

In order to prevent fabrication mistakes for beam length, the effect that the longitudinal grade has on dimensions measured along a beam’s length should be addressed in the plans. When the beam length measured along the grade differs from the beam length measured horizontally by more than 3/8" [10 mm], all affected dimensions measured along the length of the beam should be clearly labeled so that the fabricator can make the necessary allowances in the shop drawings. A Typical Detail note is available in Section 700.

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.

302.5.2.2 STRANDS

Prestressed I-beam designs shall follow the criteria established in standard bridge drawing PSID-1-99. 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. Straight strands fully bonded with additional strands placed above the neutral axis, debonded at midspan if necessary. The additional strands shall be placed inside the web and top flange reinforcing steel and shall be symmetrical about the centerline of the beam.
B. Draped strands.
C. Combination of draped strands and bottom flange strands debonded at the beam ends.

Transforming strand area in order to increase section properties is not allowed.

302.5.2.2.a TYPE, SIZE

A. Low-relaxation ½ inch diameter ($A_S = 0.153$ in$^2$) seven wire uncoated strands, ASTM A416, Grade 270.

Low-relaxation 12.7 mm diameter ($A_S = 99$ mm$^2$) seven wire uncoated strands, ASTM A416M, Grade 270.

B. Low-relaxation ½ inch diameter ($A_S = 0.167$ in$^2$) seven wire uncoated strands, ASTM A416, Grade 270.

Low-relaxation 12.7 mm diameter ($A_S = 108$ mm$^2$) seven wire uncoated strands, ASTM A416M, Grade 270.

C. Low-relaxation 0.6 inch diameter ($A_S = 0.217$ in$^2$) seven wire uncoated strands, ASTM A416, Grade 270.

Low-relaxation 15.24 mm diameter ($A_S = 140$ mm$^2$) seven wire uncoated strands, ASTM A416M, Grade 270.

Consult the Office of Structural Engineering and the Ohio/Indiana/Kentucky Prestressed Concrete Institute prior to specifying 0.6 inch [15.244 mm] diameter or larger strand sizes.

The prestressing strand type and size selected for the I-beam design shall be listed in the contract plan General Notes.

302.5.2.2.b SPACING

Strands shall be spaced at increments of 2 inches [50 mm].

A minimum 2 inch [50 mm] dimension from bottom of beam to centerline of the first row of strands and any exterior beam surface shall also be maintained.

302.5.2.2.c STRESSES

Initial prestressing loads for low-relaxation strand shall be as per AASHTO requirements and shall be detailed on the plans.

Initial stress ........................................................................................................... 0.75 $f_N = 202,500$ psi
BDM SECTION 300 DETAIL DESIGN  July 2007

Initial tension load........................................................................................................ 30,982 lb/strand (A_s = 0.153 in^2)
33,818 lb/strand (A_s = 0.167 in^2)

Initial stress........................................................................................................................................... 0.75 f_N = 1400 MPa
Initial tension load.................................................................................................................... 138 600 N/strand (A_s = 99 mm^2)
151 200 N/strand (A_s = 108 mm^2)

302.5.2.2.d  DEBONDING

Debonding or shielding of the strands, with an approved plastic sheath, may be done at the beam ends to relieve excessive stresses. The following guidelines shall be followed for debonded strand designs:

A. The maximum debonded length at each end shall not be greater than 0.16L - 40" [0.16L - 1000]. Where L equals the span length in inches [millimeters].

B. A minimum of one-half the number of debonded strands shall have a debonded length equal to one-half times the maximum debonded length.

C. No more than 25% of the total number of strands in the I-beam shall be debonded.

D. No more than 40% of the strands in any row shall be debonded.

E. Debonded strands shall be symmetrical about the centerline of the beam.

F. Strands extended from a beam to develop positive moment resistance at pier locations shall not be debonded strands.

G. Locate debonded strands as high as possible in the bottom flange to aid in the placement of the sheath during fabrication.

The designer shall show on the detail plans the number, spacing and the length of required debonding per strand.

302.5.2.2.e  DRAPING

Draping or harping of the strands may be done to relieve excessive stresses at the beam ends. The hold down point shall be located at least 5'-0" [1.5 m] on each side of the midspan of the beam using increments of 6" [150 mm]. The Designer shall calculate the vertical uplift force, P_U, at each hold-down location to ensure the following limits are not exceeded:
<table>
<thead>
<tr>
<th>No. of Draped Strands per Row</th>
<th>( P_U/\text{Strand (lb)} ) [kN]</th>
<th>( P_U/\text{Unit (lb)} ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6000 [26.6]</td>
<td>48,000 [213.5]</td>
</tr>
<tr>
<td>2</td>
<td>4000 [17.7]</td>
<td>48,000 [213.5]</td>
</tr>
<tr>
<td>3</td>
<td>4000 [17.7]</td>
<td>48,000 [213.5]</td>
</tr>
</tbody>
</table>

Where:

\[
P_U/\text{Strand} = (1.05)^{0.75} f_N A_{PS} \tan \psi
\]

\[
P_U/\text{Strand} = (1.05)^{0.00075} f_N A_{PS} \tan \psi
\]

\[
P_U/\text{Unit} = \sum_{i=1}^{n} (P_U/\text{Strand}_i)
\]

And

\[
f_N = 270,000 \text{ psi [1860 MPa]}
\]

\[
A_{PS} = \text{Area of single strand (in}^2\text{) [mm}^2\text{]}
\]

\[
n = \text{no. of strands}
\]

\[
\psi = \text{Angle of strand inclination measured from horizontal}
\]

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. It is not necessary for the angle of inclination for each row of draped strands to be the same. The height of draped strands at beam ends and at midspan shall be multiples of 2" [50 mm]. Do not place straight strands above draped strands in the same vertical column.

### 302.5.2.3 CAMBER

A variable depth haunch shall be required to account for the effects of camber due to the design prestressing. This haunch depth shall include an additional 2 inches [50 mm] that may be sacrificed to account for differences between actual and design camber. This sacrificial depth shall not be included when determining the composite section properties; however, its weight shall be included in the dead load of the slab. Haunch concrete shall be included in the total volume of superstructure concrete for payment.

In establishing bridge seat elevations and assuring a minimum design slab thickness, allowance shall be made for camber due to prestressing as per the following:

\[
A = \text{Design slab thickness}
\]

\[
B = \text{Anticipated total mid-span camber due to the design prestressing force at time of release.}
\]

\[
C = \text{Mid-span deflection due to the self-weight of the beam.}
\]
D = Mid-span deflection due to dead load of the slab, diaphragms and other non-composite loads.

E = Mid-span deflection due to dead load of railing, sidewalk and other composite dead loads not including future wearing surface.

F = Adjustment for vertical curve. Positive for crest vertical curves.

G = Sacrificial haunch depth (2" [50 mm]).

H = Total topping thickness at beam bearings = A + 1.8B - 1.85C - D - E - F + G. If F > 1.8B - 1.85C - (D + E) then H = A + G

I = Total topping thickness at mid-span = A + G. If F > 1.8B - 1.85C - (D + E) then I = A - (1.8B - 1.85C) + D + E + F + G.

The gross moment of inertia for the non-composite beam shall be used to calculate the camber and deflection values for B, C and D. The moment of inertia for the composite section should be used to calculate value E.

The designer shall show a longitudinal superstructure cross section in the plans detailing the total topping thickness, including the design slab thickness and the haunch thickness at the centerline of spans and bearings. Provide the camber at the time of release (B-C), camber at the time of erection (1.8B – 1.85C), long term camber (2.45B – 2.40C), and a screed elevation table according to Section 302.2.3.

302.5.2.4 ANCHORAGE

One inch [25 mm] diameter anchors shall be provided at each fixed pier as shown on standard bridge drawing PSID-1-99.

Minimum number of anchors shall be 2 for each beam line.

The anchors shall be a minimum of 2'-0" [600 mm] long. Anchors shall be embedded a minimum of 1'-0" [300 mm] into the pier cap and 1'-0" [300 mm] into the field cast-in-place concrete pour which connects any two discontinuous prestressed I-beams in the same beam line into a continuous member. The anchors should be drilled in place at the centerline of the pier between the ends of adjoining prestressed I-beams. The designer should confirm the pier cap has reinforcing steel clearance to accept these anchors.

302.5.2.5 DECK SUPERSTRUCTURE AND PRECAST DECK PANEL

It is recommended that only cast-in-place concrete decks, Class S or HP Concrete be designed and used.

The precast panel alternative, previously used, has shown cracking problems at the joints between the panels and there are questions on the transfer of stresses in the finished deck.
sections.

302.5.2.6 DIAPHRAGMS

Maximum spacing of intermediate diaphragms shall be 40'-0" [12 000 mm].

Intermediate diaphragms may be either cast-in-place concrete or galvanized steel. The contractor shall choose the type. Details for each type are provided in the standard bridge drawing PSID-1-99. The design plans shall show the centerline location of each intermediate diaphragm. Payment for the intermediate diaphragms shall be made at the contract price for item 515, each, Intermediate Diaphragms.

Cast-in-place intermediate diaphragms should not make contact with the underside of the deck because they could act as a support to the deck, causing cracking and possible over stressing of the deck. The top of the cast-in-place intermediate diaphragm should start at the bottom vertical edge of the top flange and end at the top of the vertical edge of the bottom flange.

If the Standard Bridge Drawing for I-beams is not referenced by the contract plans, the designer shall 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.

Threaded inserts shall be used to connect the cast-in-place diaphragm reinforcing steel to the I-beam. Use of the inserts will ease installation of the diaphragms, allow transfer of load to the beam and help protect the diaphragms against cracking. The threaded inserts and the threaded rods shall be galvanized according to CMS 711.02.

End diaphragms shall be provided. Diaphragms shall be cast-in-place. The top of the end diaphragm shall make complete contact with the deck. The bottom of the end diaphragm shall end at the top vertical edge of the bottom flange. The bottom of the diaphragm shall not extend down to the bottom of the I-beam's bottom flange. Refer to standard bridge drawing PSID-1-99 for typical diaphragm details.

302.5.2.7 DECK POURING SEQUENCE

A deck pour sequence is required for all prestressed I-beam designs made continuous at pier locations. Standard bridge drawing PSID-1-99 establishes one sequence. The designer should either accept the standard drawing sequence or detail an alternative.

302.5.2.8 CONCRETE MATERIALS FOR I-BEAMS

The designer has a choice of 28-day compressive strengths ranging from 5500 psi [38 Mpa] to 7000 psi [48 Mpa]. The 28-day compressive strength chosen for design shall be listed in the contract plan General Notes.
The designer has a choice of compressive strength at the time of release ranging from 4000 psi [27.5 Mpa] to 5000 psi [34.5 Mpa]. The release strength chosen for design shall be listed in the contract plan General Notes.

Cast-in-place concrete, (composite decks, pier diaphragms, intermediate diaphragms, etc.) Shall be Class S or HP superstructure concrete - 4500 psi [31.0 MPa] at 28 days.

Consult the Office of Structural Engineering for recommendations prior to designing a structure with concrete strengths higher than those shown above.

**302.5.2.9 REINFORCING**

Unless otherwise specified all reinforcing steel used shall be epoxy coated, Grade 60 [420] $F_y = 60$ ksi [420 MPa].

The fabricator, by specification, is required to use a corrosion-inhibiting admixture to the concrete.

Reinforcing bars projecting from the prestressed members shall be epoxy coated.

Reinforcing steel stirrups shall completely enclose the strands for the entire length of the beam.

For composite designs the total amount of longitudinal reinforcing steel over the piers, for the deck slab shall be determined in accordance to AASHTO.

**302.5.2.10 TRANSPORTATION & HANDLING CONSIDERATIONS**

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.

**303 SUBSTRUCTURE**

**303.1 SEALING OF CONCRETE SURFACES, SUBSTRUCTURE**

Specifications for the sealer are defined in CMS 512. Concrete surfaces shall be sealed with a concrete sealer as follows:

A. The front face of abutment backwalls, from top to bridge seat, the bridge seat and the breastwall down to the groundline shall be sealed with an epoxy-urethane or non-epoxy sealer. (Note: Sealing of the backwall shall not be required on prestressed box beam bridges because the beams are installed before the backwall is placed.)
B. The exposed surfaces of all wingwalls and retaining walls, exclusive of abutment type, that are within 30 feet [10,000 mm] of any pavement edge shall be sealed with an epoxy-urethane sealer.

C. Ends and sides of piers exposed to traffic-induced deicer spray, from any direction, shall be sealed 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. The total vertical surface of piers which are adjacent to traffic lanes shall be sealed with either an epoxy-urethane or non-epoxy sealer. Structures with A588[M] weathering steel superstructures shall also have their piers sealed as stated above with either an epoxy-urethane or non-epoxy sealer.

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

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


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

303.2 ABUTMENTS

303.2.1 GENERAL

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.

For members designed to retain earth embankments and restrained from deflecting freely at their tops, the computed backfill pressure shall be determined by using at-rest pressure. Examples include: rigid frame bridges, abutment walls keyed to the superstructure, and some types of U-abutments.

For abutment walls of structures designed without provision for expansion between superstructure and substructure and where an appreciable amount of superstructure expansion is anticipated, passive earth pressure should be considered in the design.

To allow for slight tilting of wall type abutments after the backfill has been placed, batter the
front face 1/16" for each foot [5 mm for each 1000 mm] of abutment height. Height is measured from bottom of footing to the roadway surface.
(THIS PAGE INTENTIONALLY LEFT BLANK)
303.2.1.1 PRESSURE RELIEF JOINTS FOR RIGID PAVEMENT

If rigid concrete pavement or base is to be used adjacent to the structure, the designer shall confirm that the roadway plans require installation of type A pressure relief joints, as per Standard Construction Drawing BP-2.3.

Pressure relief joints are required to alleviate backwall pressures on abutments with expansion devices and to allow freedom of movement for integral and semi-integral abutments.

303.2.1.2 BEARING SEAT WIDTH

For all continuous slab bridges, the approach slab seat and the bridge slab seat should be placed at the same elevation by providing a haunch on the thinner slab.

For continuous or simple span beam or girder structures the abutment seat width should be 2'-0" [600 mm]. Centerline of bearing should be 1'-0" [300 mm] from face of backwall. Exceptions to this practice may be necessary for highly skewed structures.

The configuration for prestressed I-beam abutment bearing seat widths is similar to those for the steel beam or girder. The seat width will vary due to size of elastomeric bearing, prestressed I-beam flange width and the structure's skew.

Abutment bearing seat widths for prestressed box beams without expansion devices is normally 1'-4" [400 mm] with centerline of bearing 9" [225 mm] from face of breast wall. The seat width for prestressed box beams with expansion devices shall be increased but the centerline of bearing shall remain 9" [225 mm] from face of breast wall.

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 Section 301.4.4 of this Manual.

303.2.1.3 BEARING SEAT REINFORCEMENT

Bearing areas of abutments may require supplementary reinforcement to resist local compressive and shearing stresses.

The location and spacing of all reinforcing in bridge seats should be chosen to provide adequate clearance for bearing anchors whether pre-set or drilled in place. The designer should recognize that drilled in place anchors use larger holes than the actual anchor.

A note shall be provided on the substructure detail sheets cautioning the contractor to place the reinforcing to avoid interference with the anchor bolts. Also 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 shall be provided.
303.2.1.4 PHASED CONSTRUCTION JOINTS

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'-0" [915 mm] wide centered on the joint.

303.2.2 TYPES OF ABUTMENTS

303.2.2.1 FULL HEIGHT ABUTMENTS

If the computed horizontal forces at the bottom of the footing for full height abutments cannot be completely resisted by the friction of the subsoil, by the action of vertical and battered bearing piles, or drilled shafts, or by footing keys, steel sheet piling rigidly attached to the footing may be used to provide additional resistance. See Section 303.4.1.2.

The minimum projection of the steel sheet piling below the bottom of the footing shall be 5 feet [1.5 meters]. If the sheet piling is placed in front of battered bearing piles, it also should be specified to be battered.

Where these short lengths of steel sheet piling are used, the sheet piles should be anchored to the face of the toe of the footing by not less than two #6 [#19M] reinforcing bars attached near the top of each sheet pile and included with the sheet piling for payment. The #6 [#19M] bars shall be long enough to be fully developed in bond.

If a 5 foot [1500 mm] projection of sheet piling below the bottom of the footing is found to be sufficient, the piles should have a minimum section modulus of 7 in$^3$ per foot [375 000 mm$^3$ per meter] of wall. For other lengths of sheet piling the minimum required section modulus should be computed. The plans shall show the minimum required section modulus. See plan notes Section 700.

Vertical rustication grooves may be provided at 48 inch [1200 mm] centers to fit the width of standard plywood forms or liners.

An alternate to vertical rustication grooves is the use of formliners to provide the wall surface with an aesthetic appearance. While a variety of formliners are available the following criteria should be met:

A. Formliners should not be used when they will not be visible to the public. 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. Minimum cover requirements for reinforcing steel must be met. If a formliner is used minimum concrete cover shall not be violated by patterns or indents of the formliner. This will require additional concrete and in some cases dimensional changes.

C. The cost of formliners selected should add only minimal additional cost to the overall cost of
the concrete (1 to 3 percent per yard [meter] of the abutment, pier or wall)

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

303.2.2.1a  COUNTERFORTS FOR FULL HEIGHT ABUTMENTS

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

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

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

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

303.2.2.1b  SEALING STRIP FOR FULL HEIGHT ABUTMENTS

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

303.2.2.2  CONCRETE SLAB BRIDGES ON RIGID ABUTMENTS

For a continuous concrete slab bridge supported on rigid abutments, the joint between the deck slab and the top of the abutment shall be troweled smooth and a continuous strip of elastomeric material shall be recessed into the abutment seat before placement of the superstructure concrete.

The above bearing system for slabs on rigid abutments should conform to temperature movement and bearing design requirements of this Manual.

303.2.2.3  STUB ABUTMENTS WITH SPILL THRU SLOPES

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

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

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

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

303.2.2.4 CAPPED PILE STUB ABUTMENTS

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

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

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

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

303.2.2.5 SPREAD FOOTING TYPE ABUTMENTS

Where foundation conditions warrant the use of an abutment on a spread footing, the bottom of the footing should be at least 5'-0" [1525 mm] below the surface of the embankment.

The perpendicular distance from the surface of the embankment to the bottom of the toe of the footing should be at least 5'-0" [1525 mm].

In no case shall the top of the footing be less than 1'-0" [300 mm] below the surface of the embankment.

303.2.2.6 INTEGRAL ABUTMENTS

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

For an integral design to work properly, the geometry of the approach slab, the design of the wingwalls, (see section 303.2.4) and the transition parapets must be compatible with the freedom required for the integral (beams, deck, backwall, wingwalls and approach slab) connection to rotate and translate longitudinally.
See Figure 303.2.2.6-1.

The horizontal and vertical joint shall be sealed at the back face of the backwall by use of a 3'-0" [900 mm] wide sheet of nylon reinforced neoprene sheeting. The sheeting should only be attached on one side of the joint to allow for the anticipated movement of the integral section. A note for the neoprene sheeting is available in Section 600.

Integral abutments shall be supported on a single, row of parallel piles. If an integral abutment design uses steel H piles, they shall be driven so the pile's web is parallel to the centerline of bearing.

The expansion length at the abutment for an integral structure is considered to be two-thirds (2/3) of the total length of the structure.

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

Phased construction integral backwall details shall have a closure section detailed between sections of staged construction to allow for dead load rotation of the main beams or girders.

The standard bridge drawing shows details for integral abutments with a steel beam or girder superstructure. Cantilevered or turn-back wingwalls shall not be used with integral abutments.

303.2.2.7 SEMI-INTEGRAL ABUTMENTS

Semi-integral abutment use is limited as defined in Section 200 of this Manual. Semi-integral abutments require foundation types that are fixed in position (a single row of piles shall not be used). The expansion and contraction movement of the bridge superstructure is accommodated at the end of the approach slab. Semi-integral design should not be used with curved main members or main members that have bend points in any stringer line.

The expansion length at the abutment for a semi-integral structure is considered to be two-thirds (2/3) of the total length of the structure.

Semi-integral details can be used on wall type abutments, spill-thru type abutments on two or more rows of piles, spread footing type abutments or abutments on drilled shafts.

This design allows the superstructure and the approach slab to move together independent of the abutment. Therefore wingwalls should not be attached to the superstructure and the vertical joints between them should be parallel with the centerline of the roadway.

The joints between superstructure and wingwalls are normally filled with 2 inch [50 mm] performed expansion joint filler material, CMS 705.03.

The horizontal joint in the backwall created between the expansion section of the semi-integral abutment and the beam seat is filled with expanded polystyrene sheet or some equal material to
act as form work for the placement of the upper semi-integral abutment concrete.

Both the horizontal and vertical joints shall be sealed at the back face of the backwall by use of a 3 foot [900 mm] wide sheet of nylon reinforced neoprene sheeting. The sheeting should only be attached on one side of the joint to allow for the anticipated movement of the integral section.

A standard bridge drawing detailing semi-integral abutment is available.

See Figure 303.2.2.7-1.

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

Phased construction semi-integral backwall details shall have a closure section detailed between sections of staged construction to allow for dead load rotation of the main beams or girders.

303.2.3 ABUTMENT DRAINAGE

303.2.3.1 BACKWALL DRAINAGE

The porous backfill immediately behind abutments and retaining walls should be provided according to CMS 518. The porous backfill shall be effectively drained by the use of a corrosion resistant pipe system into which water can percolate. See Section 303.2.3.3 for possible exceptions.

Porous backfill shall be wrapped with filter fabric, CMS 712.09, Type A. The fabric shall cover the vertical face between the porous backfill and the excavation, the bottom of the porous backfill and the excavation and include a 6 inch [150 mm] vertical up turn between the porous backfill and the abutment backwall. The porous backfill excavation should extend up to the horizontal plane of the subgrade or 1'-0" [300 mm] below the embankment surface. The bottom of the porous backfill should extend to the bottom of the abutment footing except when the vertical backface of the abutment footing extends more than 1'-0" [300 mm] out from the vertical backface of the abutment backwall. Then the Porous backfill shall extend down only to the top of the abutment footing. Porous backfill should be 2'-0" [600 mm] thick for its full height behind the abutment and wingwalls except where the vertical backface of the abutment footing extends out 1'-0" [300 mm] or less. A pipe drainage system shall be placed at the bottom of the porous backfill and sloped to allow drainage.

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.

The pipe drainage system designs shall make use of standard corrugated plastic pipe segments, tees and elbows (either 90° or adjustable). Overlapping bands should connect pipe segments. Ends of runs, unless intended to function as outlets, should have end caps. While galvanized
corrugated pipe has been used for years, the inertness and life expectancy of smooth internal wall plastic corrugated pipe makes this the better material to specify. CMS 518 calls for 707.33, corrugated plastic pipe, if called for in the plans.

303.2.3.2 BRIDGE SEAT DRAINAGE

For full-height or spill-thru non-integral type abutments supporting steel beams, steel girders or prestressed I-beams, the drainage of the bearing seat shall be provided by sloping the bearing seat away from the backwall, except at the bearings.

303.2.3.3 WEEP HOLES IN WALL TYPE ABUTMENTS AND RETAINING WALLS

Positive drainage with a pipe system in porous backfill is preferred.

If a location demands the use of weep holes, the weep holes through the abutment and retaining walls should be 6 inches [150 mm] to 12 inches [300 mm] above normal water or ground line. The porous backfill with filter fabric behind the walls should be shown as extending at least 6 inches [150 mm] below the bottom of the weep holes.

Weep hole type drainage systems should not be used with concrete slope protection as the flow undermines the concrete protection, ultimately causing its failure.

Where sidewalks are located immediately adjacent to wall type abutments or retaining walls, some type of porous backfill collection and drainage system, with pipes if necessary, should be used in lieu of weep holes.

303.2.4 WINGWALLS

Wingwalls shall be of sufficient length to prevent the roadway embankment from encroaching on the stream channel or clear opening. Generally the slope of the fill shall be assumed as not less than 1 vertical to 2 horizontal, and wingwall lengths computed on this basis.

Wingwalls shall be designed as retaining walls.

Cantilevered wingwalls shall not be used with integral abutments, as the walls will create additional pressures due to superstructure movement.

303.2.5 EXPANSION AND CONTRACTION JOINTS

Expansion joints should generally be provided every 90 feet [30 000 mm] with the following exceptions:

A. When the total length of wingwalls and breastwall exceeds 90 feet [30 000 mm] in length,
vertical expansion joints should be provided just beyond each side of the superstructure.

B. When the length of a breastwall exceeds 90 feet [30 000 mm] in length, no expansion joint shall be placed under the superstructure. An expansion joint shall be positioned as described in the above paragraph.

An expansion joint shall be filled with preformed expansion joint material, CMS 705.03, or other suitable compressible material.

Expansion joints shall be waterproofed as described in Section 303.2.2.1.b of this Manual.

Contraction joints are not required for abutments.

Reinforcing steel shall not project through expansion or contraction joints.

303.2.6 REINFORCEMENT, “U” AND CANTILEVER WINGS

The minimum amount of reinforcing in the wings, their junctions with the backwall and their supports shall be #5 [#16M] bars on 1'-6" [450 mm] centers, both horizontally and vertically, in both faces.

If a secondary member, such as a short cantilevered turn-back wing, is attached to an abutment or other member, reinforcing steel shall be provided in the secondary member at its connection to the main member and in all parts of the main member stressed by the secondary member, even though small, with adequate lap or bond length at the junction between the several kinds of bars. The probable presence of some tensile stress at various locations, due to the secondary member, must be recognized.

303.2.7 FILLS AT ABUTMENTS

The requirements for fills at abutments, time of settlement, and what and when to use special notes to control field construction of fills are dealt with in earlier preliminary sections of this Manual. The designer should consult the Office of Structural Engineering for the recommended notes to use at a specific project site.

303.3 PIERS

303.3.1 GENERAL

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.

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

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

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

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

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

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

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

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

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

303.3.1 BEARING SEAT WIDTHS

Pier bearing seat widths for reinforced concrete slab bridges should conform to Standard Bridge Drawing CPP-2-94. Also see Section 303.3.2.5 of this Manual.

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

303.3.1.2 PIER PROTECTION IN WATERWAYS

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

303.3.2.1 CAP AND COLUMN PIERS

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

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

Longitudinal reinforcing shall conform to AASHTO. Round columns shall be reinforced with spiral reinforcing placed directly outside the longitudinal bars.

Round columns are preferred and normally should be 36" [915 mm] diameter.

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

Minimum column diameters of 36 in. [915 mm] are generally used with spiral reinforcing. Spirals are made up of #4 [#13M] bars at 4.5 in. [115 mm] c/c pitch with a 30 in. [765 mm] outside core diameter. Using the circumference of the spiral as the out-to-out of the reinforcing steel bar, this column size normally has a relatively large ratio of the column’s axial load capacity to the axial load (e.g. more than 1.5). Therefore, while this spiral reinforcement does not conform to LRFD 5.7.4.6 and 5.10.6.2 it is acceptable if the ratio of axial load to axial capacity is over 1.5.

For columns where the ratio of axial capacity to axial load is less than 1.5, the spiral reinforcing should conform to LRFD 5.7.4.6 and 5.10.6.2.

In no case shall column reinforcement not meet minimum cross section area, shrinkage and temperature requirements of AASHTO.

303.3.2.2 CAP AND COLUMN PIERS ON PILES

Piers supported on piles generally should have separate footings under each column.

Column piers shall have at least 4 piles per footing.

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

303.3.2.3 CAP AND COLUMN PIERS ON DRILLED SHAFTS

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

303.3.2.4 CAP AND COLUMN PIERS ON SPREAD FOOTINGS

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

For grade separation structures, the top of pier footings should be a minimum of 1'-6" [450 mm] below the level of the bottom of the adjacent ditch. This applies even though the pier is located in a raised earth median barrier. In no case should the bottom of the footings in existing soil or on embankment fills be above the frost line.

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

303.3.2.5 CAPPED PILE PIERS

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

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

The diameter of the exposed portions of cast-in-place reinforced concrete piles generally should be 16 inches [400 mm], but if exposed length, design load or other conditions make it necessary, larger diameter cast-in-place piles should be used. Cast-in-place piles shall be reinforced with a reinforcement cage composed of 8-#6 [#19M] reinforcing bars with a 12 inch [300 mm] outside diameter, #4 [#13M] spiral, with a 12 inch [300 mm] pitch. The cage length should extend from the finished top of the pile to 15 feet [5 meters] below ground level. The reinforcing steel shall be shown in the structure's reinforcing bar list and be included in item 507 for payment. The use of cast-in-place piles greater than 16 inches [400 mm] in diameter will require an increase in the
width of the cap of Standard Bridge Drawing CPP-2-94. See Section 303.4.2.3.

Exposed H piles and unreinforced concrete piles shall have pile protection. Refer to the description in Standard Bridge Drawing CPP-2-94. A plan note is also available. Also See Section 200 for a description of pile protection.

For pile embedment requirements into concrete, see Section 303.4.2.3.

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

The design of the cap for a capped pile pier supported on bearing piles should be based on the assumption that any one pile in any three consecutive piles does not have sufficient bearing to support axial loads. The cap design doesn’t need to assume the end piles cannot support axial loads.

Although actual performance of this type of pier indicates this condition to be rare, this conservatism is recommended.

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

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

303.3.2.6 STEEL CAP PIERS

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

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

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

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

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

303.3.2.7 POST-TENSIONED CONCRETE PIER CAPS

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

303.3.2.8 T-TYPE PIERS

In the cap of a T-type pier, the top layer of reinforcing bars shall extend the full length of the cap and be turned down at the end face the necessary development length. The second layer of reinforcing steel shall extend into the stem of the pier at least the necessary development length plus the depth of the cantilever at its connection to the stem.

Lateral ties are required for T-type and wall type piers. As a minimum, the lateral ties in redundant piers shall consist of #4 [#13M] bars spaced at 3'-0" [915 mm] centers both horizontally and vertically. The lateral ties in non-redundant piers shall be in accordance with LRFD 5.10.6.3.

303.3.2.9 PIER USE ON RAILWAY STRUCTURES

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

303.3.2.10 PIERS ON NAVIGABLE WATERWAYS

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

303.3.2.11 PIER CAP REINFORCING STEEL STIRRUPS

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

303.4 FOUNDATIONS

303.4.1 FOOTINGS

303.4.1.1 MINIMUM DEPTH OF FOOTINGS

Footings founded on bedrock, including unweathered shale, shall be keyed at least 3 in. into rock.

Footings, not exposed to the action of stream currents, should be founded as follows:

A. The top of footing shall be a minimum of 1 foot [300 mm] below the finished ground line. The top of footing should be at least 1 foot [300 mm] below the bottom of any adjacent drainage ditch.

B. The bottom of footing shall not be less than 4 feet [1200 mm] below, measured normal to, the finished groundline.

C. Due to the probability of stream meander, pier footings of waterway crossings in the overflow section should not be above channel bottom unless the channel slopes are well protected against scour. Founding pier footings at or above the flow line elevation is discouraged.

For footings exposed to the action of stream currents, the top of the footing shall be placed below the depth of contraction scour.

Adjustment may be made to the minimum depth of the bottom of a footing due to actual frost line at the structure site.

303.4.1.2 FOOTING RESISTANCE TO HORIZONTAL FORCES

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

A. Increase the footing width.

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

C. Use footing struts, sheeting or anchors.
For pile supported footings, if the methods presented in LRFD 10.7.2.4 and BDM Section 1010 do not furnish sufficient horizontal resistance, consult the Office of Structural Engineering.

303.4.1.3 REINFORCING STEEL IN FOOTINGS

Secondary reinforcing steel in a footing generally should be placed under the main steel.

For footings on piles the reinforcing bars shall be placed near the bottom of the footing rather than at the top of the piles.

If the footing dowels (footing to wall or column) are provided, a bent portion of the dowel should lie in the plane of the bottom footing bars.

For piers in embankment slopes the minimum dowel size and spacing should be #8 [# 25M] at 1'-0" [300 mm] centers. For full length wall type piers not in embankment slopes and without earth overturning forces the minimum dowel size and spacing should be #6 [# 19M] at 1'-0" [300 mm] centers.

Provide at least 0.20 in²/ft [420 mm²/m] of tensile reinforcement at the following footing locations:

A. The bottom of the toe and the top of the heel of a footing for a cantilever-type retaining wall
B. The bottom of the toe and the top of the heel of a footing for an abutment where the footing is thin in proportion to the toe and heel projections
C. The top of pier footings

303.4.2 PILE FOUNDATIONS

303.4.2.1 PILES, PLAN SHEET REQUIREMENTS

For record and project use, a unique number shall individually identify each pile for a structure. The designer may choose to number each pile on the individual substructure plan sheet or on a separate pile layout sheet.

Listed below are definitions to commonly specified pile lengths:

A. Estimated Length = Pile Cutoff Elevation - Pile Tip Elevation

   Round Estimated Length up to the nearest 5 ft [1 m]. Section 200 requires the Designer to provide the Estimated Length on the site plan.

B. Order Length = Estimated Length + 5 ft [1.5 m]

   The Designer shall provide the order length for each pile in the Structure General Notes.
Refer to Section 600.

C. Furnished Length = Order Length x No. of Piles

Include in the table of Estimated Quantities.

D. Driven Length = Estimated Length x No. of Piles

Include in the table of Estimated Quantities.

303.4.2.2 PILES, NUMBER & SPACING

The designer shall comply with the following maximum center-to-center spacing of piles:

A. In capped pile piers, 7.5 feet [2300 mm].
B. In capped pile abutments, 8 feet [2500 mm].
C. In stub abutments, front row, 8 feet [2500 mm].
D. In wall type abutments and retaining walls, front row, 7 feet [2100 mm].
E. Cap and column piers should have at least 4 piles per individual footing.

Maximum pile spacings should be utilized to minimize the number of piles.

303.4.2.3 PILE EMBEDMENT

Piles supporting capped pile piers shall be embedded 1'-6" [450 mm] into the concrete cap. Other substructure units on a single row of piles should have the piles embedded 2'-0" [600 mm] into the concrete. A 1'-0" [300 mm] embedment depth into the concrete footing is required for all other cases. In every case, there shall be at least 1'-6" [450 mm] 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'-6" [450 mm]. The distance from the edge of a concrete pier cap to the side of a pile shall be not less than 9 inches [230 mm].

303.4.2.4 BATTERED PILES

The path of battered piles should be checked to see that the piles remain within the right-of-way and do not interfere with piles from adjacent and existing substructure units nor conflict with portions of staged construction.

In general, a batter of 1:4 is considered desirable, but in cases where sufficient resistance is not otherwise attainable, a batter of 1:3 may be specified.

Piles should be battered to resist the stream forces. Battered piles also should be provided where
necessary to avoid settlement due to group action by increasing the periphery of the soil mass.

In order to avoid the effects of downdrag, no battered piles shall be driven into new embankments until a waiting period for in-situ soil consolidation has concluded. Consult the Office of Structural Engineering for more information.

Abutment piles should be battered normal to the centerline of bearings.

Piles less than 15 ft [5 m] in length and driven to refusal on bedrock should not be battered.

A plan note is available in BDM Section 600 to establish the driving criteria for battered friction piles.

303.4.2.5  PILE DESIGN LOADS

Refer to BDM Section 202.2.3.2 for specific plan requirements.

Factored pile loads approaching the maximum factored structural resistance as specified in BDM Sections 202.2.3.2.a or the maximum Ultimate Bearing Value as specified in BDM 202.2.3.2.b should be utilized to minimize the number of piles.

303.4.2.6  PILES, STATIC LOAD TEST

The Designer shall specify a Static Load Test when the total pile order length for an individual structure exceeds 10,000 ft [3000 m] for piling of the same size and Ultimate Bearing Value. Static load testing is not required for piling driven to refusal on bedrock.

The Designer shall specify one subsequent static load test for each additional 10,000 ft [3000 m] increment of pile order length. Each static load test requires two dynamic testing items and two restrike items. Restrikes are a useful tool to determine if a driven pile gains or loses capacity over time.

The results of both the static and dynamic testing shall be forwarded to the Office of Structural Engineering to the attention of the Foundations Engineer. Refer to Section 600 for a General Note to include in the plans.

303.4.2.7  PILES, DYNAMIC LOAD TEST

The Department now requires dynamic load testing to establish the driving criteria (i.e. blow count) for all piling not driven to refusal on bedrock. The dynamic testing and resulting wave analysis has replaced the Engineering News Record Formula, used in previous issues of the CMS.

For an individual structure, the Designer shall specify one dynamic load testing item for each
pile size. If multiple pile capacities are required for a given pile size, the Designer shall specify one testing item for each Ultimate Bearing Value. When static load tests are required, provide two dynamic load testing items and two restrike items for each static load test item.

The driving criteria for battered piles will be determined in the field as a function of a dynamically tested vertical pile of the same Ultimate Bearing Value. When battered piles are specified, refer to Section 600 for a General Note to include in the plans.

One dynamic load testing item consists of testing a minimum of 2 piles and performing a CAPWAP analysis on one of the two piles. One restrike item consists of performing dynamic testing on two piles and performing CAPWAP analysis on one of the two piles.

### 303.4.2.8 PILE FOUNDATION – DESIGN EXAMPLE

The following example for a 6-span bridge shall be used as a guide for specifying pile testing and estimated quantities for pile foundations.

**Rear Abutment ~**

30 - 12" C.I.P. Reinforced Concrete Piles  
20 piles installed vertical & 10 piles battered  
Ultimate Bearing Value = 152 kip  
Estimated Length = 65 ft  
Order Length = 70 ft (Total Length = 2100 ft)

Requires 1 dynamic load-testing item.

**Piers 1, 2, 3, & 4 ~**

80 - 14" C.I.P. Reinforced Concrete Piles at each pier  
56 piles installed vertical & 24 piles battered  
Ultimate Bearing Value = 250 kip  
Estimated Length = 70 ft  
Order Length = 75 ft (Total Length = 24,000 ft)

The total length (24,000 ft) requires 1 static load test item and 1 subsequent static load test. Each static load test requires 2 dynamic load testing items and 2 restrike items.
Pier 5 ~

52 - 14" C.I.P. Reinforced Concrete Piles
36 piles installed vertical & 16 piles battered
Ultimate Bearing Value = 270 kip
Estimated Length = 85 ft
Order Length = 90 ft (Total Length = 4680 ft)

The difference in Ultimate Bearing Value between piers 1, 2, 3 & 4 and pier 5 requires 1 dynamic testing item.

Forward Abutment ~

30 - 12" C.I.P. Reinforced Concrete Piles
20 piles installed vertical & 10 piles battered
Ultimate Bearing Value = 152 kip
Estimated Length = 75 ft
Order Length = 80 ft (Total Length = 2400 ft)

No additional dynamic load testing items are required.

For this example, the Designer should include notes [606.2-1], [606.2-4] and [606.2-5] from Section 606.2 in the General Notes. Note [606.2-1] should be modified as follows:

**PILE DESIGN LOADS (ULTIMATE BEARING VALUE):** The Ultimate Bearing Value is 152 kip per pile for the rear and forward abutment piles. The Ultimate Bearing Value is 250 kip per pile for Pier 1, 2, 3, and 4 piles and 270 kip per pile for Pier 5 piles.

Abutment Piles:

30 piles 70 ft long, order length (Rear)
30 piles 80 ft long, order length (Forward)
1 dynamic load testing item

Pier 1, 2, 3, and 4 Piles:

320 piles 75 ft long, order length
1 static load test item
1 subsequent static load test item
4 dynamic load-testing items
4 restrike items

Pier 5 Piles:

52 piles 90 ft long, order length
1 dynamic load testing item

The Designer should provide the following items in the Estimated Quantities:

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<th>Item</th>
<th>Extension</th>
<th>Total</th>
<th>Unit</th>
<th>Description</th>
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<td>11100</td>
<td>Lump</td>
<td>Sum</td>
<td>Static Load Test</td>
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<td>12200</td>
<td>1</td>
<td>Each</td>
<td>Subsequent Static Load Test</td>
</tr>
<tr>
<td>507</td>
<td>00500</td>
<td>4200</td>
<td>ft</td>
<td>12&quot; Cast-In-Place Reinforced Concrete Piles, Driven</td>
</tr>
<tr>
<td>507</td>
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<td>4500</td>
<td>ft</td>
<td>12&quot; Cast-In-Place Reinforced Concrete Piles, Furnished</td>
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<td>20000</td>
<td>6</td>
<td>Each</td>
<td>Dynamic Load Testing</td>
</tr>
<tr>
<td>523</td>
<td>20500</td>
<td>4</td>
<td>Each</td>
<td>Restrike</td>
</tr>
</tbody>
</table>

303.4.3  DRILLED SHAFTS

To allow for the misalignment of drilled shafts that support single pier columns, the shaft diameter shall be 6 in. [150 mm] larger than the column diameter. To allow for misalignment of shafts into footings, footing widths shall be at least 1'-0" [305 mm] larger than the shaft diameter.

The diameter of bedrock sockets for drilled shafts are generally 6 in. [150 mm] less than the diameter of the shaft above the bedrock elevation. This downsize provides sufficient room is the shaft for the rock core barrel. Reinforcing steel cages should be based on the bedrock socket diameter.

For uncased or temporarily cased drilled shafts, the spiral reinforcement should be a #4 [#13M] bar with a 4½ in. [115 mm] pitch. (Note: the above requirement shall be met even if the 4½ in. [115 mm] pitch may not meet the spiral requirements of LRFD 5.7.4.6) For shaft diameters 4.0 ft. and less, the out-to-out spiral diameter shall be 6 in. [150 mm] less than the rock socket diameter. For shaft diameters greater than 4.0 ft., the out-to-out spiral diameter shall be 12 in. [300 mm] less than the rock socket diameter. When steel casing is left in place, the spiral reinforcing pitch shall be 12 in. [300 mm].

The minimum clear distance between longitudinal reinforcement shall not be less than 3 times the bar diameter nor 3 times the maximum aggregate size. If bars are bundled in forming the reinforcing cage, the minimum clear distance between longitudinal reinforcement shall not be less than 3 times the diameter of the bundled bars. Where heavy reinforcement is required, consideration may be given to an inner and outer reinforcing cage.

For record and project use, each drilled shaft for a structure shall be individually identified by a unique number. The designer may choose to number the drilled shafts on the individual
substructure plan sheet or on a separate drilled shaft foundation layout sheet.

A construction joint between the top of a drilled shaft and the bottom of a column will be required. Therefore, the designer will need to specify reinforcing steel, incorporating the required lap splices, at the construction joint.

The designer should develop a lap splice that will allow both for required lap and minimum cover due to mis-alignment of the drilled shaft versus the column. Possible alternatives are two cages, one for the drilled shaft diameter and a second splice cage for the lap to the column.

When the exposed length of the pier columns is relatively short, one full length reinforcing steel cage, from the bottom of the drilled shaft up into the pier cap, should be designed. The steel cage should be designed to provide a 3 in. [75 mm] concrete cover within the pier column.

When the drilled shaft is socketed into bedrock, the quantity of the reinforcing steel in the drilled shaft, including the portion extending into the rock socket, should be included with Item 524 “Drilled Shaft, Above Bedrock” for payment. For drilled shafts with friction type design where the tip elevation is known, the reinforcing steel should be paid under Item 524, Drilled Shafts.

A general note as listed in Section 600 will be required.

The top of the drilled shaft shall be 1 ft. [0.3 meter] above normal water elevation, for piers in water, and 1 ft. [0.3 meter] below the ground surface for piers not in water.

303.5 DETAIL DESIGN REQUIREMENTS FOR PROPRIETARY RETAINING WALLS

Supplemental Specification 840 defines the requirements for construction and design for internal stability for Mechanically Stabilized Earth (MSE) walls. The project plans shall include a reference to SS 840 when MSE walls are shown. Special provisions are required for other types of proprietary walls.

303.5.1 WORK PERFORMED BY THE DESIGN AGENCY

The Design Agency is responsible for providing 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 wall after the project is awarded. Detail each wall on a project separately. As a
minimum, the project plans for each wall location shall provide the following:

A. Plan View of the wall showing: (Refer to Figure 303.5.1-1)
   1. Wall location with station and offset with respect to the centerline of construction for
each critical point
   2. All complex geometry information
   3. Limits of proprietary wall embankment
   4. North Arrow
   5. Locations of typical sections for (C.) below
   6. Locations of abutment footing, piles, utilities, catch basins, and other possible
obstructions (Refer to Section 209.3 for drainage and Section 301.7 for utility locations)
   7. Parapet/barrier locations
   8. Limits of proposed wall excavation
   9. Locations of sheeting and bracing
      If sheeting and bracing is required to support an excavation for undercut and backfill,
      show the location and provide a pay item for Item 503 – Cofferdams, Cribs and Sheeting.
      Refer to BDM Section 208 for more information.
   10. Backfill drainage locations
      Porous backfill with filter fabric and perforated plastic pipe, CMS 707.33, shall be
located as low as possible within the undercut for the wall while still providing positive
gravity flow in the pipe to an outlet. The porous backfill and pipe shall be located near
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. The
approximate location of the outlet shall be shown on the plan view. Drainage pipe
without perforations shall be used outside the limits of the select granular backfill. If the
proprietary wall supports an abutment, provide backfill drainage in accordance with
Section 303.2.3.1.

B. Elevation of the wall showing: (Refer to Figure 303.5.1-1)
   1. Station and elevation for each critical point on the wall
   2. Finished ground surface elevations for each critical point on the wall
   3. Leveling pad showing the minimum dimension from the finished ground line to the top of
the pad.
   4. Locations of abutment footing, piles, utilities, catch basins, and other possible
obstructions
   5. Backfill drainage
6. Approximate locations of slip joints

C. Typical Sections showing: (Refer to Figures 303.5.1-2, 303.5.1-3, 303.5.1-4 & 303.5.1-5)

1. Coping details
2. Parapet and sleeper slab details
3. Abutment footing details including the dimensions from the back of the proprietary wall to the centerline of bearing at the abutment, dimensions from the back of the proprietary wall to the toe of the abutment footing, and dimensions from the back of the proprietary wall to the centerline of the nearest row of piles.
4. Minimum clearance between the bottom of the footing/sleeper slab and the uppermost wall reinforcement strap. Six inches [150 mm] is preferred.
5. Locations of abutment footing, piles, utilities, catch basins, and other possible obstructions
6. Backfill drainage
7. Soil reinforcements attached to abutment (where required)
   Show soil reinforcements attached to the backside of abutments regardless of foundation types according to the following:
   a. For jointed structures, the soil reinforcement should be attached up to the level of the approach slab seat.
   b. For semi-integral structures, the soil reinforcement should be attached up to the level of the beam seat.
8. Limits of select granular material
   Show the limit of the select granular backfill extending from the bottom of the wall excavation sloping upwards at 45 degrees. Include the note that what is shown is the limit of the select granular backfill and not the slope of the required excavation. The upper limit of the select granular backfill shall be at least six inches [150 mm] above the uppermost layer of soil reinforcement, but not lower than six inches [150 mm] above the bottom of the abutment footing.
9. Limits of wall excavation
   Supplemental Specification 840 requires a minimum one foot [0.3 m] undercut for all MSE walls. If more undercut is required, show it on the plans. The backfill material is specified in SS 840.
10. Pay limits of proprietary wall
11. Pile sleeves (if required)
   Pile sleeves shall be shown 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 (i.e. show panel 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 reinforced soil mass
   2. The following factored loads applied to the reinforced soil mass from the bridge: vertical dead and live loads, horizontal loads and total bearing load.

Plan notes are provided in Section 600.

F. Final copy of the Special Provisions for proprietary wall types other than MSE walls.

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

   Include all pay items listed in SS 840. Also include as necessary; Item 203 – Embankment; Item 512 – Sealing of concrete surfaces (epoxy urethane); and Item 503 - Cofferdams, Cribs and Sheeteting.

303.5.2 WORK PERFORMED BY THE PROPRIETARY WALL COMPANIES

The proprietary wall companies will be responsible for designing the internal stability of the wall in accordance with the project plans and either Supplemental Specification 840 for MSE walls or the special provisions for other proprietary wall types.

304 RAILING

304.1 GENERAL

All bridge railing shall meet acceptance criteria contained in NCHRP Report 350 or its successor. The minimum acceptance level shall be TL-3 unless supported by a rational selection procedure described herein.

Bridge railings that have been found acceptable under the crash testing acceptance criteria defined in NCHRP Report 230 and the AASHTO Guide Specification for Bridge Railing, 1989 including all interims, will be considered as meeting the requirements of NCHRP Report 350 without further testing as indicated in the following table.
Bridge Railing Testing Criteria | Acceptance Equivalencies
--- | ---
NCHRP 350 | TL-1 | TL-2 | TL-3 | TL-4 | TL-5 | TL-6
NCHRP 230 | MSL-1 | MSL-2 | MSL-3
AASHTO Guide Specification | PL-1 | PL-2 | PL-3

The AASHTO Guide Specification provides a Performance Level Selection Criteria for bridge railings that is considered an acceptable alternative to NCHRP Report 350 and may be used for railing designs on any project defined above.

Section 304.2 of this Manual lists standard ODOT bridge railing types available along with the corresponding NCHRP 350 level of acceptability. For non-standard railing designs, a review submission, concurrent with the Structure Type Study, shall be made to the Office of Structural Engineering as stated in Section 201 of this Manual. The design of all non-standard railing systems shall be based on the NCHRP 350 level of acceptability. Designers may be required to submit actual crash test report data to verify the level of acceptability of a proposed design.

Modifications to the ODOT standard railing types or other NCHRP 350 approved railing system should be avoided. Additional structural steel tubing added to satisfy pedestrian concerns does not require additional crash testing provided these elements do not protrude nearer to the roadway than the rail elements on the tested design and they do not present any type of snagging potential to an impacting vehicle. If an accepted crash tested railing system is modified, the face geometry (i.e. offset, rail height, spacing, etc.) shall match the tested design and the static strength and deflections shall remain at least equal to the tested design. Include with the preliminary design submission to the Office of Structural Engineering, strength and deflection calculations to support these modifications. The calculations shall follow the procedure defined in the *AASHTO LRFD Bridge Design Specifications, Sections A13.1-3*. The intent of any modification shall be to maintain the original NCHRP 350 acceptability level.

All railing elements fabricated with ASTM A500 steel tubing shall specify a drop-weight tear test per CMS 707.10. Provisions shall be made at tube splices for expansion and contraction. Steel railing systems shall also allow for structural movement at expansion joints without adversely affecting the system’s level of acceptability.

Aesthetically pleasing railing systems have been successfully crash tested but are for use only where TL-2 acceptability requirements are allowed. These systems include the Texas Classic Traffic Railing, Type T411 with open windows, a smooth stone masonry barrier with reinforced concrete core wall and an artificial stone precast concrete barrier. Detailed information regarding the latter two systems may be found in FHWA Report No. FHWA-RD-90-087 “Guardrail Testing Program: Final Report”, June 1990 and FHWA Report No. FHWA-SA-91-051 “Summary Report on Selected Aesthetic Bridge Rails and Guardrails”, June 1992.
The recommended railing design for bridges with combination vehicular and pedestrian traffic is detailed in Standard Bridge Drawing BR-2-98. Other designs are allowed as previously mentioned above, provided the following requirements are met:

A. The curb height shall be 8".
B. The sidewalk width shall be 5'-0" or greater.

A pedestrian railing may be used in lieu of a crash tested barrier at the deck edge provided a crash tested barrier system meeting the minimum requirements for the specific location is used to separate the vehicular and pedestrian traffic. Pedestrian railing shall be designed in accordance with AASHTO.

### 304.2 STANDARD RAILING TYPES

<table>
<thead>
<tr>
<th>Drawing No.</th>
<th>Description</th>
<th>NCHRP Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>BR-1</td>
<td>36&quot; Deflector Parapet Type</td>
<td>TL-4</td>
</tr>
<tr>
<td>BR-1</td>
<td>42&quot; Deflector Parapet Type</td>
<td>TL-5</td>
</tr>
<tr>
<td>BR-2-98</td>
<td>Bridge Sidewalk Railing with Concrete Parapet</td>
<td>TL-4</td>
</tr>
<tr>
<td>DBR-2-73</td>
<td>Deep Beam Bridge Guardrail</td>
<td>TL-2</td>
</tr>
<tr>
<td>PCB-91</td>
<td>Portable Concrete Barrier (Fully Anchored)</td>
<td>TL-4</td>
</tr>
<tr>
<td>PCB-91</td>
<td>Portable Concrete Barrier (Unanchored)</td>
<td>TL-3</td>
</tr>
<tr>
<td>SBR-1-99</td>
<td>42&quot; Single Slope Parapet Type</td>
<td>TL-5</td>
</tr>
<tr>
<td>TST-1-99</td>
<td>Twin Steel Tube Bridge Railing</td>
<td>TL-4</td>
</tr>
</tbody>
</table>

### 304.3 WHEN TO USE

#### 304.3.1 PARAPET TYPE (BR-1 & SBR-1-99)

The department currently has three (3) standard concrete parapet type bridge railing systems: a 36" New Jersey shape, a 42" New Jersey shape and a 42" single slope shape. These systems are for use on roadway and railroad overpass structures with no sidewalks and structures where the finished deck surface is 25 ft [7.62 m] or more above the ground line or water surface. Details for these parapet types, including end transitions to terminal assemblies, are provided in the Standard Bridge Drawings. The transition section may be placed on a structure’s turned back wingwalls, widened approach slab or directly on the actual structure. If the transition section is placed directly on the structure, a curb is required for the full length of the approach slab.
The 36" barrier section is for use on structures located on two (2) lane routes with an ADTT in one direction less than 2500.

The 42" barrier sections are for use on structures located on interstates, divided highways of four (4) lanes or more, and two (2) lane routes with an ADTT in one direction of 2500 or more. Final decision of which section to use rests with the districts and should be finalized during the preliminary structural design review. The single slope barrier section is unaffected by the placement of future overlays, but weighs 23% more than the 42" New Jersey type parapet.

A 50" deflector type median barrier and a 57" single slope median barrier are for use on structures where protection against oncoming headlight glare is required. The structure’s barrier height and type shall match the design of the adjoining roadway median barrier.

For each of the above listed barrier types, designers are required to confirm the structural adequacy of the concrete deck slab as described in the “Concrete Deck Design” Section 302.2 of this manual.

All concrete parapet type barriers shall be designed and detailed as follows:

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

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

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

304.3.2 DEEP BEAM BRIDGE GUARDRAIL (DBR-2-73)

This railing configuration does not meet the Department’s minimum NCHRP 350 acceptance criteria (i.e. TL-3) for use on any project unless supported by the selection procedures described in Section 304.1 of this manual. In no case, shall this railing system be used on an overpass structure or a project where the finished deck surface is greater than 25 feet above the normal water surface elevation or final ground line.

The standard configuration for 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. A modified railing design meeting these requirements and using the Type 1 post design may be justified.
Use of Type A anchors, as detailed on the Standard Bridge Drawing, is not recommended. The Type B alternative is recommended because they are easier to install in a deck or box beam and easier to replace if damaged in a collision.

Designers should recognize that variable post lengths may be required along the length of a structure due to beam camber. A design data sheet is available from the Office of Structural Engineering to address these concerns.

304.3.3 TWIN STEEL TUBE BRIDGE RAILING (TST-1-99)

This railing configuration was developed as a replacement to the Deep Beam Bridge Guardrail system on projects requiring a higher NCHRP acceptance level. The Twin Steel Tube Bridge Railing is for use over rural stream crossings on two (2) lane routes with an ADTT in one direction less than 2500 where the finished deck surface is less than 25 feet above the normal water surface elevation or final ground line. This system shall not be used on an overpass structure.

The standard configuration for 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. A modified railing design meeting these requirements may be justified.

The required bridge terminal assembly section used to transition from Type 5 or 5A approach roadway guardrail to the bridge railing is detailed on Standard Construction Drawing GR-3.6.

The typical post spacing is 6'-3". The standard drawing enables the designer to reduce the first, last and one additional post spacing per span on each side of the bridge to account for construction clearances. The designer should carefully review the position of the posts that are near the corner of a structure for possible interference with wingwalls, tie rods, etc.

The site plan shall show the station of the center of the first inlet-mounted post on each corner of the bridge.

304.3.4 PORTABLE CONCRETE BARRIER (PCB-91)

This system is for use on construction projects to protect project personnel and to provide a temporary barrier system when a permanent bridge railing system does not exist. Application guidelines for PCB-91 are provided in Design Data Sheet, PCB-DD, available at the Office of Structural Engineering web site.

The designer is required to detail the installation requirements, including the number of anchor bolts per barrier, in the bridge plans. The pay item for this barrier system is Item 622 - Portable Concrete Barrier, 32 inch, Bridge Mounted. Although temporary railing is to be specified and completely described in the bridge plans, temporary railing is a roadway item and shall be included in the roadway quantities.
On projects where maintaining minimum lane widths during a construction phase is not possible due to limited bridge width, the use of a top mounted steel post and tubular steel rail system, similar to the Twin Steel Tube bridge guardrail, may be justified. The railing, post and anchorage designs of these systems are to be in accordance with the AASHTO LRFD Bridge Design Specifications, Sections A13.1-3.

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

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

Where Vandal Protection Fencing is required, the fencing shall be installed behind the steel tubing as shown in Figure 305.3-2. However, the steel tubing may be omitted if the concrete parapet height is 32" or greater. See Figure 305.3-1. If the tubing is omitted, the fencing should extend the full length of the concrete parapet and the additional 18" parapet height at each end, as detailed in the standard, is not required.

The concrete parapet shall be designed and detailed as follows:

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

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

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

### 305 FENCING

#### 305.1 GENERAL

The primary purposes of protective fencing are to provide for the security of pedestrians and to discourage the throwing or dropping of objects from bridges onto lower roadways, railroads, boat lanes or occupied property. In addition, fence may be needed on high level bridges where wind may threaten to blow pedestrians or occasional stranded motorists off the bridge and on bridges where there is a danger that the outside parapet may be mistaken for a median barrier, causing persons to jump over the parapet in emergency situations in periods of darkness. These situations should be treated on a case-by-case basis.
Since a falling object problem could occur at any bridge accessible to pedestrians, it is necessary to consider installation of protective fencing at such locations.

Generally, fencing attached to bridge structures for the protection of traffic and pedestrians should conform to the Vandal Protection Fencing Standard Bridge Drawing. The designer may need to enhance this standard to deal with requirements for the specific structure.

### 305.2 WHEN TO USE

Designers shall investigate the need for fencing on all new overpass bridges. Pedestrian Fencing may be required when a total of 10 points or greater is achieved for a structure due to the following criteria. The designer should use this procedure as a general guide as to the need for fencing. The affected district should also be consulted for their input. The list is not to be construed as all-inclusive. Other rationale may be used on a case-by-case basis. Similarly, retrofitting of bridges that qualify according to the total index number is not mandatory if adequate justification for not doing so can be documented.

<table>
<thead>
<tr>
<th>JUSTIFICATION ITEM</th>
<th>INDEX POINTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Overpass within an urbanized area of 50,000 or more population</td>
<td>2</td>
</tr>
<tr>
<td>B. Overpass with sidewalks but not in an urbanized area as defined in (A)</td>
<td>2</td>
</tr>
<tr>
<td>(&quot;Sidewalk&quot; does not include safety curbs 2'-3&quot; [685 mm] or less in width)</td>
<td>2</td>
</tr>
<tr>
<td>C. Overpass which is unlighted</td>
<td>2</td>
</tr>
<tr>
<td>D. Overpass not a main thoroughfare, i.e., on collectors or local streets</td>
<td>2</td>
</tr>
<tr>
<td>E. Overpass within ½ mile [0.8 km] of another overpass exclusive of pedestrian</td>
<td>2</td>
</tr>
<tr>
<td>bridges, having or requiring protection</td>
<td>2</td>
</tr>
<tr>
<td>F. Overpass within ½ mile [0.8 km] of another overpass having previous reports</td>
<td>4</td>
</tr>
<tr>
<td>of falling objects</td>
<td></td>
</tr>
<tr>
<td>G. Overpass within 1 mile [1.6 km] of a school, playground or other pedestrian</td>
<td>4</td>
</tr>
<tr>
<td>attraction</td>
<td></td>
</tr>
<tr>
<td>H. Bridges over any feature which has a high count of boat, rail, vehicular or</td>
<td>4</td>
</tr>
<tr>
<td>pedestrian traffic, or includes damage-sensitive property</td>
<td></td>
</tr>
<tr>
<td>I. Overpass which has had prior reported incident of falling objects</td>
<td>6</td>
</tr>
<tr>
<td>J. Overpass which is used exclusively by pedestrians</td>
<td>10</td>
</tr>
</tbody>
</table>

“OVERPASS” is a bridge over a highway or a railroad.

### 305.3 FENCING CONFIGURATIONS

For structures with sidewalks, the top of fence should be a minimum height of 8 feet [2450 mm] above the sidewalk. For a greater degree of protection against objects being thrown from the
bridge, the fence may be curved to overhang the sidewalk. For curved fence the posts should be vertical for approximately 8 feet [2450 mm] above the sidewalk before curving inward over the sidewalk. The overhang should be at least 1 foot [300 mm] less than the width of the sidewalk, with a maximum overhang of 3'-7" [1100 mm]. The slope of the straight overhanging portion should be 1 vertical to 4 horizontal. The radius of the connecting arc should be 32 inches [815 mm]. See Figures 305.3-1 & 305.3-2.

For narrow pedestrian bridges, bent pipe frames are generally used with pipe bend radii of 24" [600 mm] at the upper corners and the start of the radii about 8 feet [2450 mm] above the sidewalk surface. The fabric should start at the deck line, top of curb or parapet and may stop at the upper end of the bent portion of the frame.

Fabric on the top horizontal area of the frame is sometimes not installed because adventurous youngsters tend to walk on the top of the enclosure. See Figure 305.3-3 for an illustration of this configuration. To try to eliminate the adventurous youngster problem, some pedestrian bridges have used a frame design that comes to a peak at the center of the structure, similar to a house roofline.

Chain link fabric should not have an opening at the bottom through which large objects could be pushed. A detail to close the bottom of a fencing section is included on the standard bridge drawing. The closure plate detail is required for all fence configurations that have tension wire at the bottom of the fence fabric.

Posts and frames may be either plumb or perpendicular to the longitudinal grade of the bridge, subject to considerations of aesthetics or practicality of construction. Complete details of base plates, pipe inserts or other types of base anchorage shall be provided on the plans. If applicable to the specific project, details from the standard bridge drawing may be referred to in the project plans.

305.4 SPECIAL DESIGNS

The following information is given the designer as a basis for specialized designs. It is not intended for designers to develop their own requirements in lieu of the standard bridge drawing.

For fence installation projects on new structures, the installation of a traffic railing (steel tubing) is not required if the top concrete parapet or concrete wall is 32" [813 mm] above roadway for structures without sidewalks or 32" [813 mm] above the top of sidewalk for structures with sidewalks. See Figure 305.3-1.

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

Fencing mesh should consist of supported wire mesh of the chain-link variety with one inch [25 mm] diamonds. The core wire is to be 11 gage [3.05 mm] with a Polyvinyl chloride coating. (CMS 710.03)

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

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

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

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

305.5.1  WIND LOADS

The design wind pressure \( P \) in \( \text{lb/ft}^2 \) [kPa] shall be calculated using:

\[
P = 27.69C_h, \text{ derived from the formula:}
\]
\[
P = 0.00256(1.3V)^2C_sC_hC_i^{(1)}
\]

Where:
\[
V = 50 \text{ yr. mean wind vel.}^{(2)} = 80 \text{ mph}
\]
\[
1.3 = 30\% \text{ Wind Gust Factor}
\]
\[
C_s = \text{Shape Coefficient} = 1.0
\]
\[
C_h = \text{Height Coefficient (See table)}
\]
\[
C_i = \text{Ice Coefficient} = 1.0
\]

\[
P = 1.326C_h, \text{ derived from the formula:}
\]
\[
P = 0.0471(1.3V)^2C_sC_hC_i/1000^{(1)}
\]

Where:
\[
V = 50 \text{ yr. mean wind vel.}^{(2)} = 129 \text{ km/h}
\]
\[
1.3 = 30\% \text{ Wind Gust Factor}
\]
\[
C_s = \text{Shape Coefficient} = 1.0
\]
\[
C_h = \text{Height Coefficient (See table)}
\]
\[
C_i = \text{Ice Coefficient} = 1.0
\]
<table>
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<th>( C_h )</th>
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<td>50 - 100</td>
<td>15 000 - 30 000</td>
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<tr>
<td>1.40</td>
<td>100 - 150</td>
<td>30 000 - 46 000</td>
</tr>
<tr>
<td>1.50</td>
<td>150 - 200</td>
<td>46 000 - 61 000</td>
</tr>
</tbody>
</table>

The centroid of the horizontally projected area of the fence is to be used to determine the height above normal terrain and the value of \( C_h \).

The projected area for wind forces on 11 gage [3.05 mm] polyvinyl chloride coated one inch [25 mm] wire mesh shall be 20% of the gross horizontally projected area.

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

Ref. (1) Specifications for the Design and Construction of Structural Supports for Highway Signs, AASHTO.

Ref. (2) Isotach's of the U.S. The 80 mph [129 km/h] line covers the northwestern portion of Ohio and shall be used herein for all of Ohio.

### 306 EXPANSION DEVICES

#### 306.1 GENERAL

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

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

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

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

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

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

306.1.2 EXPANSION DEVICES WITH SIDEWALKS

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

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

306.1.3 EXPANSION DEVICES WITH STAGE CONSTRUCTION

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

306.2 EXPANSION DEVICE TYPES

306.2.1 ABUTMENT JOINTS IN BITUMINOUS CONCRETE, BOX BEAM BRIDGES

This poured joint seal system is capable of small expansion movements, up to 3/16" [5 mm]. A plan insert sheet, Abutment Joints in Bituminous Concrete Box Beam Bridges, is available through the Office of Structural Engineering’s web page. This device requires three bid items: Item Special - Sawing and Sealing Bituminous Concrete Joints; Item 516 - Joint Sealer, As Per Plan; and Item 516 - 1" Preformed Expansion Joint Filler.
306.2.2  ABUTMENT JOINTS AS PER AS-1-81

A group of no or small movement joints used for sealing and rotational purposes are detailed on Standard Bridge Drawing, AS-1-81.

306.2.3  EXPANSION JOINTS USING POLYMER MODIFIED ASPHALT BINDER

This device is generally for use on structures with concrete or asphalt overlays and where expected expansion is 0 to 1½" [40 mm]. A detail & plan note insert sheet, Polymer Modified Asphalt Expansion Joint System, is available through the Office of Structural Engineering’s web page. This item is bid as a special.

Thickness of the polymer-modified joint shall be a minimum of 2" [50 mm]. A thickness greater than 5" [125mm] should be avoided.

306.2.4  STRIP SEAL EXPANSION DEVICES

The seal size is limited to a 5" [125 mm] maximum. Unpainted A588[M] weathering steel should not be used in the manufacture of this type expansion device as A588[M] does not perform well in the atmospheric conditions an expansion device is subjected to. Standard Bridge Drawings, EXJ-4-87, EXJ-5-93 and EXJ-6-95, are available. The designer must ensure that all details are covered in the plans because the standard drawing is not inclusive for all structure types.

The strip seal shall be of one piece across the total width of the structure. No splices will be acceptable.

306.2.5  COMPRESSION SEAL EXPANSION DEVICES

Maximum allowable seal size is 4" [100 mm]. A 5" [125 mm] wide seal shall not be used since installation problems have been encountered. Compression seal expansion devices are limited to structures with a maximum skew of 15 degrees. Movement should be limited so that the seal is not compressed greater than 60 percent or less than 20 percent.

The compression seal shall be of one piece across the total width of the structure. No splices will be acceptable. Standard Bridge Drawings EXJ-2-81 & EXJ-3-82 give generally used details.

306.2.6  STEEL SLIDING PLATE ENDDAMS, RETIRED STANDARD DRAWING SD-1-69

In general steel sliding plate enddams are not recommended for new structures. This expansion device is limited to total movement of 4" [100 mm], including movement in both directions.
Sliding plates should be configured to prevent binding and bearing when the superstructure is supported on elastomeric bearings.

Unpainted A588[M]/A709[M] Grade 50W materials are not recommended for construction of this type of joint.

306.2.7 MODULAR EXPANSION DEVICES

Modular expansion devices may be required for structures when total required movements exceed movement capacity of a strip or compression seal. Consult the Office of Structural Engineering for recommendations prior to completion of the project plans.

Modular devices main load bearing beams, support beams and welds shall be designed for fatigue.

The manufacturer of the expansion device shall be required by plan note to submit design calculations showing that the device can meet the impact and fatigue design requirements.

Modular devices have been known to fail at connections due to welding and fatigue. Therefore it is recommended the following general requirements be included in any project plan notes:

A. Spacing of support beams shall be limited to 3'-0" [1000 mm] centers under main load bearing beams unless fatigue testing of the actual welding connection details has been performed to show that a greater spacing is acceptable. Fatigue resistance shall be determined according to *LRFD 6.6.1.2.5* and *BDM Section S 6.6.1.2.5*.

B. Shop or field welds splicing main beams, or connections to the main beams shall be full penetration welded and 100 percent non-destructively tested in accordance with *AWS D1.5 Bridge Welding Code*. Any required field splices or joints and non-destructive testing shall be located and defined in the plans.

C. Fabricators of modular devices shall be pre-qualified 513 Level UF fabricators. Review Section 302.4.1.3 and contact the Office of Structural Engineering for recommendations.

D. Approved manufacturer/fabricator shall supply a qualified technical representative to the jobsite during all installation procedures.

E. Seals shall be one continuous piece through the total length of the structure.

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.

Contact the Office of Structural Engineering for sample notes used on other projects.
306.2.8  TOOTH TYPE, FINGER TYPE OR NON-STANDARD SLIDING PLATE EXPANSION DEVICES

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. Consult the Office of Structural Engineering for recommendations prior to completion of the project plans. Example plan notes are provided in the appendix.

Use of a tooth type expansion device also requires neoprene drainage troughs and a suitable drainage system to carry away the water. Both the neoprene trough and downspout to drainage trough connection must be detailed completely. Special attention should be paid to developing a complete seal at the downspout to trough connection.

Vulcanization is recommended over adhesive for sealing.

Finger devices shall be designed for fatigue and conform to fracture critical requirements if the design has fracture critical components in it.

Fabricators of finger or sliding plate devices shall be pre-qualified 513 Level UF fabricators. Review Section 302.4.1.3 and contact the Office of Structural Engineering for recommendations.

306.3  EXPANSION DEVICE USES – BRIDGE OR ABUTMENT TYPE

306.3.1  INTEGRAL OR SEMI-INTEGRAL TYPE ABUTMENTS

No allowance for temperature need be made.

The vertical joint between abutment backwall and approach slab should be finished as per Standard Bridge Drawing AS-1-81, Detail B.

306.3.2  REINFORCED CONCRETE SLAB BRIDGES

The following table specifies joint requirements. 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.
### 306.3.3 STEEL STRINGER BRIDGES

The following table specifies joint requirements based on expansion length defined as the total length of structure, if no fixed bearing exists, or length from fixed bearing to proposed expansion device location if one exists.

<table>
<thead>
<tr>
<th>Expansion Length (ft)</th>
<th>Joint Required</th>
<th>Approach Slab Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 30</td>
<td>None</td>
<td>AS-1-81 detail B</td>
</tr>
<tr>
<td>30 - 125</td>
<td>PM (1) or EXJ-4-87</td>
<td>AS-1-81 detail B</td>
</tr>
<tr>
<td>125 - 400</td>
<td>EXJ-4-87</td>
<td>AS-1-81</td>
</tr>
<tr>
<td>400 +</td>
<td>TTED or MED</td>
<td>AS-1-81</td>
</tr>
</tbody>
</table>

PM = Polymer Modified Asphalt Joint

TTED = Tooth Type expansion device

MED = Modular Expansion Device

(1) = Stringer bridges with sidewalks should not use polymer modified expansion joint systems.

### 306.3.4 PRESTRESSED CONCRETE I-BEAM BRIDGES

The following table specifies joint requirements based on expansion length defined as the total length of structure, if no fixed bearing exists, or length from fixed bearing to proposed expansion device location if one exists.
### Expansion Length

<table>
<thead>
<tr>
<th>Expansion Length (ft)</th>
<th>Joint Required</th>
<th>Approach Slab Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 40</td>
<td>0 - 12</td>
<td>None, AS-1-81 detail B</td>
</tr>
<tr>
<td>40 - 225</td>
<td>12 - 65</td>
<td>PM (1) or EXJ-6-95, AS-1-81 detail C</td>
</tr>
<tr>
<td>225 - 500</td>
<td>65 - 150</td>
<td>EXJ-6-95, AS-1-81 detail C</td>
</tr>
<tr>
<td>500 +</td>
<td>150 +</td>
<td>TTED or MED</td>
</tr>
</tbody>
</table>

PM = Polymer Modified Asphalt Joint  
TTED = Tooth Type expansion device  
MED = Modular Expansion Device  
(1) = Stringer bridges with sidewalks should not use polymer modified expansion joint systems

### 306.3.5 NON-COMPOSITE PRESTRESSED BOX BEAM BRIDGES

The following table specifies joint requirements based on expansion length defined as the total length of structure, if no fixed bearing exists, or length from fixed bearing to proposed expansion device location if one exists.

<table>
<thead>
<tr>
<th>Expansion Length (ft)</th>
<th>Joint Required (2)</th>
<th>Approach Slab Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 40</td>
<td>PM</td>
<td></td>
</tr>
<tr>
<td>40 - 225</td>
<td>PM (1) or EXJ-5-93</td>
<td>AS-1-81 detail A,C,E</td>
</tr>
<tr>
<td>225 - 500</td>
<td>EXJ-5-93</td>
<td>AS-1-81 detail C</td>
</tr>
</tbody>
</table>

PM = Polymer Modified Asphalt Joint  
(1) = Bridges with sidewalks should not use polymer modified expansion joint systems  
(2) = Joint requirements are for rigid or fixed abutments. For flexible abutments requiring no expansion movement a PM joint is recommended except for (1).

### 306.3.6 COMPOSITE PRESTRESSED CONCRETE BOX BEAM BRIDGES

The following table specifies joint requirements based on expansion length defined as the total length of structure, if no fixed bearing exists, or length from fixed bearing to proposed expansion device location if one exists.
Expansion Length | Joint Required | Approach Slab Joint
---|---|---
| (ft) | (m) | (2) |
| 0 - 40 | 0 - 12 | PM |
| 40 - 225 | 12 - 65 | PM (1) or EXJ-5-93 | AS-1-81 detail C,D F |
| 225 - 500 | 65 - 150 | EXJ-5-93 | AS-1-81 detail C |

PM = Polymer Modified Asphalt Joint

(1) = Bridges with sidewalks should not use polymer modified expansion joint systems

(2) = Joint requirements are for rigid or fixed abutments. For flexible abutments requiring no expansion movement a PM joint is recommended except for (1).

306.3.7 **ALL TIMBER STRUCTURES**

No allowance for temperature need be made.

307 **BEARINGS**

307.1 **GENERAL**

The Department’s policy is, whenever possible, use laminated elastomeric bearings.

Justification, including design calculations showing elastomeric bearings will not be adequate for the structure, must be available.

When specialized bearings, such as pot, disc or spherical, are required, detail notes shall be included in the contract plans. A plan note for pot bearings is provided in the appendix and may require modification by the designer based on the specific structure. If a cost evaluation shows that either spherical or disc bearings could be competitive against pot bearings, those bearings should be included in the plans and special notes developed.

For specialized bearings, the designer's detail plan notes shall require the contractor to coordinate the required substructure bearing seat elevations or dimensions with the selected bearing manufacturer. A note is available in Section 700.

307.2 **BEARING TYPES**

307.2.1 **ELASTOMERIC BEARINGS**

Refer to SI4.7.5 for additional design requirements.
Non-laminated elastomeric bearings are only acceptable if actual design calculations support their use.

Elastomeric bearings should be designed based on a selected durometer of either 50 or 60.

Elastomeric bearings should generally be limited to a 5 inch [125 mm] maximum elastomeric height excluding internal laminates with a minimum total height of one inch [25 mm]. The designer should evaluate greater height elastomeric bearings, or elastomeric bearings with sliding surfaces, before arbitrarily selecting specialized, high priced pot spherical or Disc type bearings.

Elastomeric bearings for steel beam and girder bridges will require a load plate. Field welding of a beam or girder to the bearing load plate should be controlled so that the temperature of the elastomer is subjected to does not exceed $300\,^\circ F \left[150\,^\circ C\right]$. 

Elastomeric bearings with a load plate shall have the plate beveled if the rotation and or grade exceed the limitations of AASHTO Section 14. The load plate thickness required by design shall be the minimum thickness of the beveled plate. A nominal minimum thickness of 1½ inches [38 mm] is recommended but not mandatory.

Elastomeric bearings should not bear on unbonded steel surfaces. Therefore all steel plates in contact with an elastomeric bearing shall be vulcanized (bonded) to the bearing.

Vertical deformations of the bearings greater than 1/8" [3 mm] are to be compensated for in the elevations of the bridge bearing seats. A note shall be required in the design plans.

Detail plans shall include the unfactored dead load, live load (without impact) and total load reactions for each elastomeric bearing design.

### 307.2.2 SPECIALIZED BEARINGS

#### 307.2.2.1 POT TYPE BEARINGS

Pot type bearings are capable of sustaining high vertical loads and multi-directional rotations. Included with PTFE (Teflon) sliding surfaces, pot bearings are capable of large expansion movements. A plan note is provided in the appendix. The Designer may modify the note depending upon the specific structure.

AASHTO has both a design and construction section for pot bearings. The Designer should use these sections and this Manual as a guide in designing, selecting and specifying a pot bearing.

If requested, the Designer shall provide the Department justification for the use of pot bearings. This justification shall include calculations showing elastomeric bearings, with or without sliding surfaces, will not be adequate for the structure. Cost comparisons to other specialized bearing types shall also be included.
Pot bearings shall not be used with other bearing types.

Pot bearings are not considered proprietary and, therefore, alternate bearing designs are not required.

Design plans shall show design requirements for both vertical and horizontal loads, required movements, required rotations and maximum friction factor for the sliding surfaces.

The minimum vertical load on a pot bearing shall not be less than 20% of total vertical design load.

Plans should require anchors for bearings to be set by use of a steel template with a minimum thickness of 1/4 inch [6 mm].

To accommodate the required horizontal movements, this type of bearing utilizes PTFE (Teflon) to stainless steel sliding surfaces. The plan notes available in the Appendix include requirements for the sliding surfaces and materials. The Designer should be aware that Teflon to stainless steel friction factors vary with the applied load (i.e. the lower the load, the higher the friction factor).

307.2.2.2 DISC TYPE BEARINGS

Disc type bearings are capable of sustaining high vertical loads and multi-directional rotations. Included with PTFE (Teflon) sliding surfaces, disc bearings are capable of large expansion movements.

AASHTO has both a design and construction section for disc bearings. The Designer should use these sections and this Manual as a guide in designing, selecting and specifying a disc bearing. Generic plan notes for disc bearings are not available through the Office of Structural Engineering. The designer will need to develop specific notes based on the specific structure. Consult the Office of Structural Engineering for previously used notes.

If requested, the Designer shall provide the Department justification for the use of disc bearings. This justification shall include calculations showing elastomeric bearings, with or without sliding surfaces, will not be adequate for the structure. Cost comparisons to other specialized bearing types shall also be included.

Disc bearings shall not be used with other bearing types.

Disc bearings are no longer considered proprietary and, therefore, alternate bearing designs are not required.

Design plans shall show design requirements for both vertical and horizontal loads, required movements, required rotations and maximum friction factor for the sliding surfaces.

Plans should require anchors for bearings to be set by use of a steel template with a minimum thickness of 1/4 inch [6 mm].
thickness of 1/4 inch [6 mm].

To accommodate the required horizontal movements, this type of bearing utilizes PTFE (Teflon) to stainless steel sliding surfaces. The Designer should be aware that Teflon to stainless steel friction factors vary with the applied load (i.e. the lower the load, the higher the friction factor).

### 307.2.2.3 SPHERICAL TYPE BEARINGS

Spherical type bearings are capable of sustaining high vertical loads and large multi-directional rotations. Included with PTFE (Teflon) sliding surfaces, spherical bearings are capable of large expansion movements.

AASHTO has both a design and construction section for spherical bearings. The Designer should use these sections and this Manual as a guide in designing, selecting and specifying a spherical bearing. Generic plan notes for spherical bearings are not available through the Office of Structural Engineering. The designer will need to develop specific notes based on the specific structure. Consult the Office of Structural Engineering for previously used notes.

If requested, the Designer shall provide the Department justification for the use of spherical bearings. This justification shall include calculations showing elastomeric bearings, with or without sliding surfaces, will not be adequate for the structure. Cost comparisons to other specialized bearing types shall also be included.

Spherical bearings shall not be used with other bearing types.

Spherical bearings are not considered proprietary and, therefore, alternate bearing designs are not required.

Design plans shall show design requirements for both vertical and horizontal loads, required movements, required rotations and maximum friction factor for the sliding surfaces.

Plans should require anchors for bearings to be set by use of a steel template with a minimum thickness of 1/4 inch [6 mm].

To accommodate the required horizontal movements, this type of bearing utilizes PTFE (Teflon) to stainless steel sliding surfaces. The Designer should be aware that Teflon to stainless steel friction factors vary with the applied load (i.e. the lower the load, the higher the friction factor).
307.3 GUIDELINES FOR USE

307.3.1 FIXED BEARINGS

307.3.1.1 FIXED LAMINATED ELASTOMERIC BEARINGS FOR STEEL BEAM BRIDGES

Fixed laminated elastomeric bearings are recommended for use on new steel structures.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

Laminated elastomeric bearings will require analysis by the designer to fit the specific structure.

Laminated elastomeric bearings for steel beam and girder bridges shall be designed with a load plate.

For additional information, see Section 307.2.1 on elastomeric bearings.

307.3.1.2 FIXED LAMINATED ELASTOMERIC BEARINGS FOR PRESTRESSED BOX BEAMS

Laminated elastomeric bearings shall be used for prestressed concrete box beam bridges.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

A fixed bearing condition may be assumed to be obtained by the use of 1 inch [25 mm] thick laminated elastomeric bearing pads and the installation of anchor dowels with grout.

For additional information see Section 307.2.1 on elastomeric bearings.

307.3.1.3 FIXED LAMINATED ELASTOMERIC BEARINGS FOR PRESTRESSED I-BEAMS

Laminated elastomeric bearings shall be used for prestressed concrete I-beam bridges.

The Department has no standards for fixed or expansion bearings for prestressed I-beam superstructures; therefore, the designer is required to design the bearing.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

The designer should note that prestressed I-beam bridges, whether single or multiple continuous spans, will generally follow the same constraints as prestressed box beam bridges.

In designing the bearing for an I-beam bridges the designer should verify that the attachment of the bearing to the I-beam or any lateral restraining devices for the bearing do not interfere with
placement of the diaphragm. Interference of the bearing with the diaphragms may cause spalling of the diaphragm and future maintenance problems.

For additional information, see Section 307.2.1 on elastomeric bearings.

307.3.2 EXPANSION BEARINGS

307.3.2.1 EXPANSION ELASTOMERIC BEARINGS FOR BEAM AND GIRDER BRIDGES

This bearing type is recommended for use on new structures.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

Laminated elastomeric bearings will require analysis by the designer to fit the specific structure.

Laminated elastomeric bearings for steel beam and girder bridges shall be designed with a load plate.

When the decks of twin structures are being tied together, resulting in a total structure width in excess of 60 feet [20 meters] laminated elastomeric bearings shall be required.

307.3.2.2 EXPANSION ELASTOMERIC BEARINGS FOR PRESTRESSED BOX BEAMS

Box beam bridges shall have two elastomeric bearing pads at each end of each beam. At least 1 inch [25 mm] minimum thickness is required but the bearing shall be designed for the required movement and rotation.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

On skewed bridges, a 1/8" [3 mm] thick preformed bearing shim material, CMS 711.21, the same plan dimensions as the bearing, should be provided to accommodate any non-parallelism between bottom of beam and bridge seat. 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. The preformed bearing pads should be incorporated in an item 516 in the Estimated Quantities.

307.3.2.3 EXPANSION ELASTOMERIC BEARINGS FOR PRESTRESSED I-BEAMS

Unless special limitations exist, elastomeric bearings should be selected to handle load, expansion and rotation requirements for prestressed concrete I-beam bridges.
Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

In designing the bearing for an I-beam bridges the designer should verify that the attachment of the bearing to the I-beam or any lateral restraining devices for the bearing do not interfere with placement of the diaphragm. Interference of the bearing with the diaphragms may cause spalling of the diaphragm and future maintenance problems.

307.3.3 SPECIALIZED BEARINGS

Where specialized bearings, such as pot, disc or spherical, are required, plan notes are needed. A plan note for pot bearings is provided in the appendix. For spherical or disc bearings, plan notes will need to be developed. If a cost evaluation shows that either spherical or disc bearings could be competitive against pot bearings, those bearings should be included in the plans and special notes developed.

For specialized bearings, the designer’s detail plan notes must require the contractor to coordinate required substructure bearing seat elevations or dimensions with the selected bearing manufacturer. A note is available in Section 700.

307.3.3.1 POT BEARINGS

See Section 307.2.2.1 for specific requirements on Pot Bearings.

307.3.3.2 DISC TYPE BEARINGS

See Section 307.2.2.2 for specific requirements on Disc Bearings.

307.3.3.3 SPHERICAL BEARINGS

See Section 307.2.2.3 for specific requirements on Spherical Bearings.
Figure 301.4.2-1
<table>
<thead>
<tr>
<th>MARK</th>
<th>NUMBER</th>
<th>LENGTH</th>
<th>WEIGHT</th>
<th>TYPE</th>
<th>DIMENSIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>REAR</td>
<td>FWD</td>
<td>TOTAL</td>
<td>(LBS.)</td>
<td>A</td>
</tr>
</tbody>
</table>

**PIERS**

- SP401 1 1 28'-3" 373 12 2'-4" 0'-4½"
- SP402 1 1 13'-3" 175 12 2'-4" 0'-4½"
- P501 14 14 7'-6" 110 5 6'-4"
- P502 12 12 7'-9" 97 Str.
- P601 17 17 20'-0" 511 Str.
- P602 15 15 5'-10" 136 6 5'-2"
- 2 2 4 7'-6" 2'-5" 2'-5" 2'-5"
- P603 16 16 8'-4" 285 1 2'-11" 10 10 1"
- of 6 of 6 of 6 8'-4" 2'-10" 2'-10"
- 28 28 2'-10" 119 Str.
- DP602 16 16 3'-3" 78 Str.
- TOTAL 1879

**ABUTMENT**

- A401 8 8 8 16 10'-5" 112 1 0'-11" 4'-10½" 4'-10½"
- A402 4 4 8 10'-0" 53 Str.
- A501 12 12 12 24 12'-0" 300 Str.
- A502 7 7 10'-9" 76 6 10'-2"
- A601 16 16 28'-2" 677 Str.
- A602 16 16 8'-5" 203 1 1'-0" 5'-9" 2'-0"
- TOTAL 1423

**SUPERSTRUCTURE**

- S401 61 30'-0" 12224 Str.
- S402 61 13'-4" 543 Str.
- S501 530 41'-6" 22790 Str.
- S502 466 5'-10½" 2845 16 2'-2" 2'-5" 0'-1½" 0'-1½" 2'-8"
- S801 240 3'-8" 2150 17 0'-9" 0'-8½" 0'-9" 0'-6" 0'-10½"
- S802 16 8'-4" 356 1 1'-0" 5'-9" 2'-0"
- TOTAL 41259

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THE BAR SIZE NUMBER IS SPECIFIED ON THE PLANS IN THE BAR MARK COLUMN. THE FIRST DIGIT WHERE THREE DIGITS ARE USED, AND THE FIRST TWO DIGITS WHERE FOUR ARE USED, INDICATING THE BAR SIZE NUMBER. FOR EXAMPLE, P601 IS A NO. 6 BAR. BAR DIMENSIONS SHOWN ARE OUT TO OUT UNLESS OTHERWISE INDICATED. R INDICATES INSIDE RADIUS, UNLESS OTHERWISE NOTED. "STD." WRITTEN IN PLACE OF A DIMENSION INDICATES A STANDARD BEND AT THE END OF THE BAR.

ALL REINFORCING STEEL TO BE EPOXY COATED

**Figure 301.5.2-1**
### TENSION SPLICES (in.)

<table>
<thead>
<tr>
<th>BAR LOCATION</th>
<th>EPOXY</th>
<th>NON-EPOXY</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLR. SIZE</td>
<td>TOP 2</td>
<td>TOP 2</td>
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<tr>
<td>SIZE</td>
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<td>OTHER</td>
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<tr>
<td></td>
<td>162</td>
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</table>

**NOTES:**

1. FOR EPOXY BARS WITH COVER LESS THAN \(3d_b\) OR CLEAR SPACING BETWEEN BARS LESS THAN \(6d_b\) (LRFD 5.11.2.1.2)

2. TOP BARS REFERS TO HORIZONTAL BARS WITH 12.0 IN. OF FRESH CONCRETE CAST BELOW THE REINFORCEMENT.

3. FOR BARS SPACED LATERALLY AT LEAST 6 INCHES ON CENTER WITH AT LEAST 3 INCHES CLEAR COVER MEASURED IN THE DIRECTION OF THE SPACING, REDUCE VALUE BY 20% \((0.80)\) (LRFD 5.11.2.1.3), BUT NOT LESS THAN 12” PER LRFD 5.11.5.3.1.

4. VALUES SHOWN ARE FOR CLASS “C” LAP WITH \(f’c = 4,000\) P.S.I. AND \(f_y=60,000\) P.S.I. (LRFD 5.11.5.3.1)

- BAR DIAMETER

---

*Figure 301.5.3-1*
1. FOR NO. II BARS AND SMALLER WITH NOT LESS THAN 2 1/2 IN. COVER ON SIDE OF HOOK AND 2 IN. OVER END OF HOOK (LRFD 5.11.2.4.2)

2. FOR NO. II BARS AND SMALLER ENCLOSED WITHIN TIES OR STIRRUPS SPACED NO GREATER THAN 3db ALONG DEVELOPMENT LENGTH.

3. BRIDGE DESIGN MANUAL (303.3.2.I) CALLS FOR A 1/2" DIAMETER SPIRAL BAR WITH A 4 1/2" PITCH FOR 36" DIAMETER COLUMNS WITH LIMITED RATIO OF ACTUAL AXIAL LOAD TO AXIAL LOAD CAPACITY.

4. COMPRESSION LAP SPLICES WITHIN TIES MAY BE MULTIPLIED BY 0.83 PER LRFD 5.11.5.5.1, BUT IN NO CASE LESS THAN 12 INCHES.

5. FOR EPOXY COATED REINFORCING STEEL, VALUES SHALL BE MULTIPLIED BY 1.2.
## Development Length for Reinforcing Steel (1 in.)

<table>
<thead>
<tr>
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<th>Bar Type</th>
<th>Tension Reinforcement</th>
<th>Compression</th>
</tr>
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<td></td>
<td></td>
<td>Epoxy</td>
<td>Non-Epoxy</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Top Bars</td>
<td>Other Bars</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.4 (1.5) x 1.7</td>
<td>1.4 (1.2)</td>
</tr>
<tr>
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<tr>
<td>18</td>
<td>104</td>
<td>177</td>
<td>175</td>
</tr>
</tbody>
</table>

*(See Notes Fig. 301.5.3-4)*
NOTES:

1. FOR EPOXY COATED BARS WITH COVER LESS THAN $\times3d_b$ OR CLEAR SPACING BETWEEN BARS LESS THAN $\times6d_b$ (LRFD 5.II.2.I.2)

2. TOP BARS REFERS TO HORIZONTAL BARS WITH 12.0 IN. OF FRESH CONCRETE CAST BELOW THE REINFORCEMENT.

3. FOR BARS SPACED LATERALLY AT LEAST 6 INCHES ON CENTER WITH AT LEAST 3 INCHES CLEAR COVER MEASURED IN THE DIRECTION OF THE SPACING, REDUCE VALUE BY 20% ($\times0.80$) (LRFD 5.II.2.I.3), BUT NOT LESS THAN 12 INCHES PER (LRFD 5.II.2.I.1)

4. BRIDGE DESIGN MANUAL SECTION 303.3.2.I CALLS FOR A 1/2" DIAMETER SPIRAL BAR WITH A 4 1/2" PITCH FOR 36" DIAMETER COLUMNS WITH LIMITED RATIO OF ACTUAL AXIAL LOAD TO ALLOWABLE AXIAL LOAD CAPACITY.

5. FOR BARS IN COMPRESSION MINIMUM DEVELOPMENT LENGTH SHALL BE $\geq 8"$ (LRFD 5.II.2.2)

6. VALUES SHOWN ARE FOR CLASS "C" LAP WITH $f'_c = 4,000$ P.S.I. AND $F_y=60,000$ P.S.I. (LRFD 5.II.5.3.1)

$\times$ BAR DIAMETER
### ASTM STANDARD
**REINFORCING BARS**

<table>
<thead>
<tr>
<th>BAR SIZE DESIGNATION</th>
<th>NOMINAL AREA SQ. INCHES</th>
<th>NOMINAL WEIGHT POUNDS PER FT.</th>
<th>NOMINAL DIAMETER INCHES</th>
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<td>0.11</td>
<td>0.376</td>
<td>0.375</td>
</tr>
<tr>
<td># 4</td>
<td>0.20</td>
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<td>0.500</td>
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<tr>
<td># 5</td>
<td>0.31</td>
<td>1.043</td>
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<td># 6</td>
<td>0.44</td>
<td>1.502</td>
<td>0.750</td>
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<td># 7</td>
<td>0.60</td>
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<td>1.27</td>
<td>4.303</td>
<td>1.270</td>
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<tr>
<td># 11</td>
<td>1.56</td>
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<td>1.693</td>
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<td>13.60</td>
<td>2.257</td>
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Figure 301.5.4-1
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<th>D</th>
<th>A</th>
<th>H</th>
<th>C</th>
<th>D</th>
<th>A</th>
<th>J</th>
<th>J-A</th>
<th>D</th>
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**BENDING TOLERANCES:** Refer to Section CMS 509
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<td>3/8&quot;</td>
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<td>24&quot;</td>
<td>8&quot;</td>
<td>16&quot;</td>
</tr>
</tbody>
</table>

**NOTE:**

"D" is the diameter of the bend per construction and material specifications item 509.05
CONCRETE DECKS WITH OVER THE SIDE DRAINAGE

CONCRETE DECKS WITH CURBS, SIDEWALKS AND PARAPET

CONCRETE DECK WITH DEFLECTOR PARAPET

PRESTRESSED BOX BEAM DECK WITH DEFLECTOR PARAPET

SEALING OF CONCRETE SURFACES, SUPERSTRUCTURE

Figure 302.1.4.3-1
CONCRETE DECKS WITH OVER THE SIDE DRAINAGE

CONCRETE DECKS WITH CURBS, SIDEWALKS AND PARAPET

CONCRETE DECK WITH DEFLECTOR PARAPET

PRESTRESSED BOX BEAM DECK WITH OVER THE SIDE DRAINAGE

SEALING OF CONCRETE SURFACES, SUPERSTRUCTURE

Figure 302.1.4.3-2
## Concrete Deck Design Aid

<table>
<thead>
<tr>
<th>Effective Span Length (ft.)</th>
<th>Deck Thickness (in.)</th>
<th>Overhang Deck Thickness (in.)</th>
<th>Transverse Steel</th>
<th>Longitudinal Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
<td>Top Bars</td>
<td>Bottom Bars</td>
</tr>
<tr>
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<td></td>
<td></td>
<td>Size</td>
<td>Spa. (in.)</td>
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<td>12.50</td>
<td>#6</td>
<td>5.75</td>
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</tbody>
</table>

### Notes:

1. Designs in accordance with AASHTO LRFD Bridge Design Specifications and the 2007 ODOT Bridge Design Manual
2. Design Assumptions:
   a. Four or more beam/girder lines
   b. Transverse steel is placed perpendicular to beam/girder lines
   c. Normal weight concrete with $f'_c = 4.5$ ksi
   d. Reinforcing steel with $f_y = 60$ ksi
   e. Monolithic Wearing Surface = 1.0 in.
   f. Future Wearing Surface = 0.06 ksf
   g. $LRFD$ 5.7.3.4 - Exposure Factor ($\gamma_e$) = 0.75
   h. Top cover = 2.50 in.; Bottom cover = 1.50 in.
   i. Maximum overhang width = 4.0 ft. (measured from cl. of fascia beam/girder to deck edge)
   j. Valid for BR-1 (36” & 42”), SBR-1-99, BR-2-98, and TST-1-99 barrier systems
3. 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 Section 302.2.1
5. Cutoff Length = length beyond the centerline of the fascia beam/girder where additional overhang bars are no longer required for strength.
6. Longitudinal bar spacing does not include additional reinforcing required for negative moments in accordance with $LRFD$ 5.7.3.2 (for prestressed beams) and $LRFD$ 6.10.1.7 (for steel beams/girders)
7. Refer to Figure 302.2.2-2 and Figure 302.2.2-3 for more information

---

**Figure 302.2.2-1**
**DECK SECTION**

- Additional overhang bars are to be bundled with each top transverse bar.
- Designer shall clearly show overhang deck thickness on plans. Minimum overhang thickness = 1 + 2".
- Cutoff length = length beyond the centerline of the fascia beam/girder where additional overhang bars are no longer required for strength. (See Figure 302.2.2-1)
- Dev'mt length = development length according to LRFD 5.11.2.

**OVERHANG PLAN VIEW**

- See Figure 302.2.2-1 for size of bar to be bundled with top interior bay bar.
- Toe of barrier.
- Exterior stringer.
- Interior stringer.
- See Figure 302.2.2-1 for size and spacing of interior bay bars.

**TYPICAL DECK DETAILS—ODOT LRFD STANDARD DECK DESIGN**

*Figure 302.2.2-3*
TYPICAL CONCRETE DECK HAUNCH DETAIL

Figure 302.2.5-2
The web splice plate shall not encroach upon the beam fillet.

The inside splice place shall not encroach upon the beam fillet.

$Fillet$ $k$

$Fillet$ $k1$ Maximum Inside Plate Width
PLAN VIEW

Crossframes for Dog-legged Splices

Figure 302.4.2.3-1
FIT-UP SHOULD NOT BE INCLUDED IN ESTABLISHING THE NUMBER OF BEAMS.
IN ACTUAL CONSTRUCTION FIT-UP WILL BE ABSORBED IN THE OVERHANG

DO - DESIGN OVERHANG (MINIMUM 2", MAXIMUM 8") BOX BEAM DESIGN
WIDTH SHOULD BE SELECTED SO DO STAYS WITHIN ACCEPTABLE RANGE.
ABUTMENT SEALING LIMITS (FOR STEEL BEAM BRIDGE)

ABUTMENT SEALING LIMITS (FOR PRESTRESSED BOX BEAM BRIDGE)

WINGWALL SEALING LIMITS (TURNBACK WALL ON U-TYPE ABUTMENT)

WINGWALL SEALING LIMITS (STRAIGHT WING ABUTMENT)

SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 303.1-1
Seal end of pier cap

Seal entire surface area of column

Ground Line

ELEVATION

SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

PIER SEALING LIMITS
(EXPOSED TO DEICER SPRAY)

SECTION A-A

Figure 303.1-2
ABUTMENT SEALING LIMITS
(FOR INTEGRAL ABUTMENT STEEL BEAM BRIDGE)

SEAL END OF WINGWALL ABOVE GROUND LINE.
CROSSHATCHED AREA REPRESENTS THE CONCRETE SEALING LIMITS.

SEAL ENTIRE SURFACE AREA
GROUND LINE

ABUTMENT SEALING LIMITS
(FOR SEMI-INTEGRAL ABUTMENT STEEL BEAM BRIDGE)

SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 303.1-3
INTEGRAL ABUTMENT

- CONCRETE DECK SLAB
- APPROACH SLAB
- CONSTR. JOINT
- STEEL STRINGER
- NEOPRENE WATERPROOFING
- CONSTR. JOINT
- POROUS BACKFILL WITH FABRIC
- PILES-PLACE PILE WEB PARALLEL TO PILE BEARING.

Figure 303.2.2.6-1
SAMPLE MSE WALL ABUTMENT PLAN
* - MEASURED TO FRONT FACE OF MSE WALL FACING PANELS

SEE FIGURE 330 FOR SECTION A-A (WITH ABUTMENT SUPPORTED ON SPREAD FOOTING AND ADDITIONAL WALL EXCAVATION)
SEE FIGURE 331 FOR SECTION A-A (WITH ABUTMENT SUPPORTED ON PILES)

SAMPLE MSE WALL ABUTMENT ELEVATION
* - MEASURED TO FRONT FACE OF MSE WALL FACING PANELS
NOTE: THE SLOPING LINE WHICH DEFINES THE LIMIT OF THE SELECT GRANULAR BACKFILL IS NOT AN ALLOWABLE SLOPE FOR EXCAVATION. CUT THE SIDES OF ALL EXCAVATIONS TO PREVENT CAVING, OR PROTECT THE EXCAVATION FROM CAVING.

* - TYPE D GRANULAR MATERIAL OR DUMPED ROCK FILL MAY ALSO BE USED BELOW THE GEOTEXTILE FABRIC. IF THESE MATERIALS WOULD PROVIDE BETTER SUPPORT TO BRIDGE SOFT FOUNDATION SOILS.
NOTE: THE SLOPING LINE WHICH DEFINES THE LIMIT OF THE SELECT GRANULAR BACKFILL IS NOT AN ALLOWABLE SLOPE FOR EXCAVATION. CUT THE SIDES OF ALL EXCAVATIONS TO PREVENT CAVING, OR PROTECT THE EXCAVATION FROM CAVING.
LIMITS OF SEALING CONCRETE SURFACES (EPOXY-URETHANE)

MIN. 5½' FACING PANEL THICKNESS, CENTERED ON LEVELING PAD. ADDITIONAL THICKNESS MAY BE REQUIRED FOR AESTHETIC SURFACE TREATMENT.

FINISHED GROUNDLINE

6" PLASTIC PIPE, CMS 707.33

POROUS BACKFILL WITH FILTER FABRIC (TYP.)
CAST-IN-PLACE LEVELING PAD

MSE WALL AND COPING DETAIL

COPING EXPANSION JOINTS

FRONT FACE OF MSE WALL FACING PANELS

MSE WALL COPING
ALL REINFORCING STEEL TO BE EPOXY COATED

FIGURE 303.5.1-4
**BRIDGE WITH SIDEWALK - VERTICAL FENCE**

1'-0" min. horizontal distance between face of curb and edge of fence.

1'-0" to 3'-7" max. overhang

**BRIDGE WITH SIDEWALK - CURVED FENCE**

Figure 305.3-1
BRIDGE WITH SIDEWALK - VERTICAL FENCE

1'-0" min.
horizontal distance between face of curb and edge of fence.

1'-0" to 3-7" max. overhang

BRIDGE WITH SIDEWALK - CURVED FENCE

Figure 305.3-2
PEDESTRIAN FENCING ON STRUCTURES

DEFLECTOR PARAPET WITH FENCING

SEE STD DRWG. VPF-1-90 FOR FENCING DETAILS

Optional Fence Fabric

Bent Pipe Frame

RAILING SHALL BE DESIGNED IN ACCORDANCE WITH THE AASHTO LRFD BRIDGE DESIGN SPECIFICATION FOR PEDESTRIAN/BICYCLE RAILING

PEDESTRIAN BRIDGE

8'-0" +

24"R (Typ.)

8" 42" Deflector Parapet

9" BR-1 & BR-1-67

2" BR-1

3" BR-1-67 or 42"
Deflector Parapet

8" 42" Deflector Parapets

BR-1 and BR-1-67

2'-8"

3'-6"

Figure 305.3-3
SECTION 400 – REHABILITATION & REPAIR

A project shall be considered rehabilitation when any portion of the existing structure is incorporated into the completed structure.

For rehabilitation and repair of existing bridges, refer to the 2004 ODOT Bridge Design Manual, Section 400.
SECTION 500 – TEMPORARY STRUCTURES ................................................................. 5-1
501 GENERAL ........................................................................................................... 5-1
502 PRELIMINARY DESIGN ..................................................................................... 5-1
  502.1 HYDRAULICS ............................................................................................... 5-1
503 DETAIL DESIGN .................................................................................................. 5-1
504 GENERAL NOTES .............................................................................................. 5-2
SECTION 500 – TEMPORARY STRUCTURES

501 GENERAL

This section is a supplement to CMS 502, Structures For Maintaining Traffic. All design guidelines of CMS 502 apply.

502 PRELIMINARY DESIGN

For the Structure Type Study, the Designer shall show the grade, alignment, approximate location and width of the temporary structure on the Preliminary Structure Site Plan.

For the Preliminary Design Report, the Designer shall show the grade and the alignment of the temporary structure on the Site Plan. The Designer shall also determine the roadway width, hydraulic design, clearance requirements, and all other design parameters in conjunction with the development of the preliminary design. 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 as detailed in Section 200. The Designer shall submit the preliminary design of the temporary structure concurrently with the Preliminary Design Report at the Stage 1 Detailed Design Review Submission for the permanent structure.

502.1 HYDRAULICS

The design year and other hydraulic requirements for temporary structures are defined in CMS 502.02. In addition to those requirements, scour depths for the design year discharge shall be calculated and accounted for in the design of the temporary bridge and its foundation.

With the owner’s approval, the design year may be reduced for low volume roads with an ADT less than 200.

The designer shall show the water surface elevation (“high water”) and velocity of the design year discharge on the temporary structure plans. The lowest portion of the superstructure of the temporary bridge shall clear the design year discharge.

Culvert pipes may be used in lieu of a bridge structure provided controls specified in Section 1006 of the ODOT Location and Design Manual are not exceeded for the design year discharge.

503 DETAIL DESIGN

The temporary structure detail plans shall be complete and independent of the permanent structure plans. The temporary structure detail plans shall include general plan and elevation views, general notes, a table of estimated quantities, a reinforcing steel bar list and all necessary detail plan sheets. The Designer should clearly indicate that the temporary structure will be paid
for under one Lump Sum bid item - Item 502, Structure for Maintaining Traffic, and the table provided for estimated quantities is “For Estimating Purposes Only”.

Temporary bridge structures shall be designed as permanent structures in accordance with the AASHTO LRFD Bridge Design Specifications and this Manual except that the design live loading, HL-93, may be reduced by 25 percent. The temporary bridge plans shall include Design Data in the General Notes as defined in BDM Section 600.

For ice pressure loads, the thickness of ice shall be assumed to be 6 inches [150 mm], with a 200 psi [1.4 MPa] effective ice strength. The force shall be assumed to act at the level of the design year highwater elevation.

The bridge railing for the temporary structures shall meet the requirements of Section 304 of this Manual. If the Designer elects to use standard Type 5 or 5A guardrail or standard portable concrete barrier, the Designer should account for the deflection characteristics of the barrier.

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. The allowable design unit stresses of the lumber used in the design shall be specified in the plans. State whether timber sizes are full sawn or standard dressed sizes.
C. The nominal thickness of wood plank or strip floor shall be 3 inches [75 mm] minimum.
D. Timber floors shall be securely fastened to the stringers and stringers shall be securely fastened to the pier and abutment caps.
E. When circumstances permit, all or part of the existing bridge may be used for the run-around.
F. Field welded connections shall require nondestructive testing as per 513. Bolted connections are preferred and generally are more economical.
G. Designs that minimize debris accumulation should be considered.
H. Shop drawings are not required. Adequate plan details need to be provided.
I. The road surface on the temporary structure shall have antiskid characteristics, crown, drainage and superelevation in accordance with the ODOT Location and Design Manual, Vol. 1 or the AASHTO publication A Policy on Geometric Design of Highways and Streets.

504 General Notes

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 502. Use only new bolts.

B. Structural steel need not be painted.

The following instructions are provided to assist in developing the necessary general notes.

When 513 Structural Steel is specified in the plans, only the following CMS descriptions shall apply:

A. Straightening.....................................................................................................................513.11
B. Holes for High Strength and Bearing Bolts......................................................................513.19
C. High Strength Steel Bolts, Nuts and Washers .................................................................513.20
D. Welding.............................................................................................................................513.21
E. Nondestructive Testing.....................................................................................................513.25
F. Shipping, Storage and Erection.........................................................................................513.26

When 511 Class "C" is specified in the plans, 511.18 shall be waived.

The following notes shall be included in the Structure General Notes. In the roadway plans the pay item description “614 Maintaining Traffic” shall include an “as per Plan.” Coordination with the roadway plans for this item is required.

A. MAINTENANCE: Maintain all portions of the temporary structure in good condition with regard to strength, safety and ridability. 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 109.05.

B. 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.
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 __________

________ 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 “LRFD Bridge Design Specifications” adopted by the American Association of State Highway
**NOTE TO DESIGNER:**

* Designer should fill-in current edition and latest interims.

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 S1.3.4.

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

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

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

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

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

For all bridges the following note shall be included:

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

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

602.2 DESIGN LOADING

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

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

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

[602.2-2] DESIGN LOADING: 0.085 kips/ft$^2$

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

[602.2-3] DESIGN LOADING: 0.085 kips/ft$^2$ and H15-44 vehicle

602.3 DESIGN STRESSES

A. General Design Data:

[602.3-1] DESIGN DATA:
Concrete Class (1) - compressive strength 4.5 ksi (superstructure)
Concrete Class (2) - compressive strength 4.0 ksi (substructure)
Concrete Class S Modified - compressive strength 4.0 ksi (drilled shaft)
Reinforcing steel - minimum yield strength 60 ksi
Structural Steel - ASTM A709 Grade (3) - yield strength (3)_ksi
Steel H-piles - ASTM A572 - yield strength 50 ksi

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

(1) Class S, Class HP or Class QSC2 Concrete for superstructure

(2) Class C, Class HP or Class QSC1 Concrete for substructure

(3) Grade 50 - yield strength 50 ksi, or
    Grade 50W - yield strength 50 ksi, or
    Grade HPS70W - yield strength 70 ksi, or
    Grade 36 - yield strength 36 ksi

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

B. Additional Design Data for Prestressed Concrete Members:

Provide the following note in addition to note [602.2-1].
[602.3-2] DESIGN DATA:

Concrete for prestressed beams:
Compressive Strength (final) - (1) ksi
Compressive Strength (release) - (2) ksi

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: 0.153 in², 0.167 in², or 0.217 in²

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:


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 and latest interims.

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 DECK PROTECTION METHOD

If any of the following deck protection methods have been specified in the plans, include the following note, modified as necessary for the specific structure, in the Design Data section of the Structure General Notes:

[602.5-1] DECK PROTECTION METHOD:

Epoxy coated reinforcing steel

2½” concrete cover

Superplasticized dense, Micro-silica, Epoxy, or Latex modified concrete overlay

Waterproofing and asphalt concrete overlay

Steel drip strip

Other (Specify)

602.6 MONOLITHIC WEARING SURFACE

Furnish the following note for concrete bridge decks.

[602.6-1] MONOLITHIC WEARING SURFACE is assumed, for design purposes, to be 1 inch thick.

602.7 SEALING OF CONCRETE SURFACES

Use the following notes when permanent anti-graffiti coatings are required:

[602.7-1] ITEM 512 SEALING OF CONCRETE SURFACES, AS PER PLAN, (PERMANENT GRAFFITI PROTECTION):
Apply a permanent graffiti coating qualified according to Supplement 1083 that is compatible with the concrete sealer over which it is applied. Apply the graffiti coating in accordance with the manufacturer’s printed instructions.

Use the following notes when sacrificial anti-graffiti coatings are required:

[602.7-2] ITEM 512 SEALING OF CONCRETE SURFACES, AS PER PLAN, (SACRIFICIAL GRAFFITI PROTECTION):
Apply a permanent graffiti coating qualified according to Supplement 1083 that is compatible with the concrete sealer over which it is applied. Apply the graffiti coating in accordance with the manufacturer’s printed instructions.
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 [300 mm] below ground line as specified in CMS 202 will not fill the specific requirements of the project.

[603.1-2] ITEM 202, PORTIONS OF STRUCTURE REMOVED, AS PER PLAN:
(THIS PAGE INTENTIONALLY LEFT BLANK)
Remove abutments to Elev. ____. Remove piers to Elev. ____.

**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 feet [7 meters], or if the use of the existing structure is part of the temporary road. See 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 Structural 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 * 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.

**NOTE TO DESIGNER:**

* Generally 200 feet [60 meters]. Optionally, this distance may be defined by station-to-station dimensions.

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.

NOTE TO DESIGNER:

(1) Generally 200 feet [60 meters]. Optionally, this distance may be defined by station-to-station dimensions.

(2) Identify specific piers.

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 feet behind the abutments. Do not begin the installation of the abutment piles until after the above required embankment has been constructed. 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.

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 at a 1:1 slope from the bottom of the heel of the footing to the subgrade elevation and for a minimum distance of 250 feet behind the abutments. 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.

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, 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 inch [150 mm] 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 feet behind each abutment

[605.3-2] ITEM 203 EMBANKMENT, AS PER PLAN: Place and compact embankment material in 6 inch 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 yd$^3$ [m$^3$] 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 yd$^3$ [m$^3$], 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 CMS 503.01. Generally, the basis of payment for Item 503 should be yd$^3$ [m$^3$]. 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 spread footings, provide the following note and table:

**[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 loads from the abutment provided in the table below. All loads in the table are nominal (i.e. unfactored) applied to the reinforced soil mass at the base of the concrete footing. Refer to AASHTO LRFD Bridge Design Specifications, Section 3, for load definitions.

<table>
<thead>
<tr>
<th>Wall Location</th>
<th>DC (k/ft)</th>
<th>DW (k/ft)</th>
<th>LL (k/ft)</th>
<th>PL (k/ft)</th>
<th>FR (k/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#2</td>
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</tr>
<tr>
<td>#3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For projects with proprietary retaining wall systems supporting bridge abutments on pile foundations, provide the following note:

**[605.5-2] 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.

**NOTE TO DESIGNER:** Both notes above apply 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 301.4.5). 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.

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 by penetrating weak bedrock for several inches to a minimum resistance of 20 blows per inch or by contacting strong bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing according to C&MS 523 to establish a driving criteria for each pile type and capacity. Establish the driving criteria to achieve an Ultimate Bearing Value that is 1.5 times the total factored load given below for the piles. Payment for dynamic load testing performed at the Contractor’s option is included in the unit price pay item for piles driven.

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

NOTE TO DESIGNER:

(1) Specify the total factored load according to BDM Section 202.2.3.2.a.
(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 inch diameter).
(4) Specify the order length according to BDM Section 202.2.3.2.a and 303.4.2.1.

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 by penetrating weak bedrock for several inches to a minimum resistance of 20 blows per inch or by contacting strong bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing according to C&MS 523 to establish a driving criteria for each pile type and capacity. Establish the driving criteria to achieve an Ultimate Bearing Value that is 1.5 times the total factored load given below for the piles. Payment for dynamic load testing performed at the Contractor’s option is included in the unit price pay item for piles driven.

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

**NOTE TO DESIGNER:**

(1) Specify the total factored load according to BDM Section 202.2.3.2.a.
(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 inch diameter).
(5) Specify the order length according to BDM Section 202.2.3.2.a and 303.4.2.1.

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 by penetrating weak bedrock for several inches to a minimum resistance of 20 blows per inch or by contacting strong bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing according to C&MS 523 to establish a driving criteria for each pile type and capacity. Payment for dynamic load testing performed at the Contractor’s option is included in the unit price pay item for piles driven.

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. If performing dynamic load testing to establish driving criteria, the Ultimate Bearing Value is \((4)\) kips per pile for the abutment piles and \((4)\) kips per pile for the pier piles.

Abutment piles:
\((5)\) piles \((6)\) feet long, order length

Pier piles:
\((5)\) piles \((6)\) feet long, order length

**NOTE TO DESIGNER:**

1. Specify the total factored load according to BDM Section 202.2.3.2.a.
2. Specify the location of piles for each total factored load.
3. Specify the anticipated factored downdrag loading.
(4) Specify the Ultimate Bearing Value for dynamic load testing, including downdrag.

(5) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).

(6) Specify the order length according to BDM Section 202.2.3.2.a and 303.4.2.1.

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

NOTE TO DESIGNER:

(1) Specify the Ultimate Bearing Value according to BDM Section 202.2.3.2.b.

(2) Specify the location of piles for each Ultimate Bearing Value.

(3) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).

(4) Specify the order length according to BDM Section 202.2.3.2.b and 303.4.2.1.

(5) Specify the number of dynamic load testing items according to BDM Section 303.4.2.7.

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 is \((1)\) kips per pile for the \((2)\) abutment piles. The Ultimate Bearing Value is \((1)\) kips per pile for the pier piles. The pier piles include an additional \((3)\) kips per pile of Ultimate Bearing Value due to the possibility of losing \((7)\) ft. of frictional resistance due to scour.

Abutment piles:
- \((4)\) piles \((5)\) feet long, order length
- \((6)\) Dynamic load testing items

6-13
Pier piles:

(4) piles (5) feet [meter] long, order length
(6) Dynamic load testing items

NOTE TO DESIGNER:

(1) Specify the Ultimate Bearing Value according to BDM Section 202.2.3.2.h.
(2) Specify the location of piles for each Ultimate Bearing Value.
(3) Specify the additional amount of Ultimate Bearing Value according to BDM Section 202.2.3.2.h.
(4) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).
(5) Specify the order length according to BDM Section 202.2.3.2.h and 303.4.2.1.
(6) Specify the number of dynamic load testing items according to BDM Section 303.4.2.7.
(7) Specify the scour depth.

The following note, modified to fit the conditions, will apply where downdrag loads on the piles are anticipated.

[606.2-3] 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. The addition of (3) kips of Ultimate Bearing Value per abutment pile is due to possible downdrag loads caused by settlement and to account for side friction within the downdrag zone that must be overcome during pile driving.

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

NOTE TO DESIGNER:

(1) Specify the Ultimate Bearing Value according to BDM Section 202.2.3.2.c.
(2) Specify the location of piles for each Ultimate Bearing Value.
(3) Specify the additional amount of Ultimate Bearing Value according to BDM Section 202.2.3.2.c. This amount includes the factored downdrag load and the unfactored side resistance from the soil in the downdrag zone.
(4) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).
(5) Specify the order length according to BDM Section 202.2.3.2.c and 303.4.2.1.

(6) Specify the number of dynamic load testing items according to BDM Section 303.4.2.7.

Provide the following note when Static Load Testing is required according to Section 303.4.2.5. Modify the note as necessary to fit the specific condition.

**[606.2-4]** 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 on piling represented by the static load testing for a minimum of 7 days. After the waiting period, perform pile restrikes on the four piles (two restrike test items). 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 Structural Engineering.

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.

When performing the restrike, if the pile has not reached the blow count determined for the plan specified Ultimate Bearing Value, continue driving the pile until this capacity is achieved.

Provide the following note when battered friction piles are specified.

**[606.2-5]** BATTERED PILES: The blow count for battered piles shall be the blow count determined for vertical piles of the same Ultimate Bearing Value divided by an efficiency factor \( D \). Compute the efficiency factor \( D \) as follows:

\[
D = \frac{1 - U G}{\sqrt{1 + G^2}}
\]

\( U = \) Coefficient of friction, which is estimated at 0.05 for double-acting air operated or diesel hammers; 0.1 for single-acting air operated or diesel hammers; and 0.2 for drop hammers.

\( G = \) Rate of batter (1/3, 1/4, etc.)

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.
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 inch diameter).

3. Specify the order length according to BDM Section 202.2.3.2.b and 303.4.2.1.

STEEL PILE POINTS

Use the following note where steel points are required, and see Section 202.2.3.2.a.

ITEM 507, STEEL POINTS, AS PER PLAN: Use steel pile points to protect the tips of the proposed steel “H” piling. Furnish steel points from the following manufacturers/suppliers: Associated Pile and Fitting Corporation, 262 Rutherford Blvd., Clifton, New Jersey 07014, phone: (973)773-8400, (800)526-9047, fax: (973)773-8442; International Construction Equipment, Inc., 301 Warehouse Drive, Matthews, North Carolina 28015, phone: (704)821-8200, (888)423-8721, fax: (704)821-8201; Dougherty Foundation Products, Inc., P.O. Box 688, Franklin Lakes, New Jersey 07417, phone: (201)337-5748, fax: (201)337-9022; Versa Steel Inc., 1618 N.E. First Ave., Portland, Oregon 97232, phone: (503)287-9822, (800)678-0814, fax: (503)287-7483; Versabite Piling Accessories, 1704 Tower Industrial Dr., Monroe, North Carolina 28110, phone: (800)280-9950, (704)225-1566, fax: (704)225-1567; or by a manufacturer that can furnish a steel point that is acceptable to Director. The material used for the manufacturing of pile points shall conform to ASTM A27/A27M 65/35 [450/240] – Class 2 – Heat Treated or AASHTO M103/M103M 65/35 [450/240] – Heat Treated. Weld the pile points to the pile in accordance with AWS D1.5 or the manufacturer’s written welding procedure supplied to the engineer before the welding is performed. Submit a notarized copy of the mill test report to the Engineer.

PILE SPLICES

Provide the following note when H-piles are specified.
PILE SPLICES: In lieu of using the full penetration butt welds specified in CMS 507.09 to splice steel H-piles, the Contractor may use a manufactured H-pile splicer. Furnish splicers from the following manufacturer:
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 Class C concrete. 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 (707.33), or PVC pipe (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.

In lieu of encasing the pile in concrete, galvanize the piles according to 711.02. The galvanizing shall be continuous from a minimum of 3 feet below the finish ground surface up to the concrete pier cap. The galvanized coating thickness shall be a minimum of 4 mils. Repair all gouges, scrapes, scratches or other surface imperfections caused by the handling or the driving of the pile to the satisfaction of the Engineer.

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 not pay for galvanizing provided beyond the project requirements. 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 or piers which are supported by spread footings.

[606.6-1] FOUNDATION BEARING RESISTANCE: (1) footings, as designed, produce
a maximum Service Load pressure of \((2)\) kips per square foot and a maximum Strength Load 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 202.2.3.1.

When abutments or piers are supported by spread footings on soil, 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.

**ITEM 511, CLASS \(*\) CONCRETE, \(*\), AS PER PLAN : \(*\)** In addition to the requirements of Item 511, 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.

The original completed tables will become part of the District’s project plan records. Send a copy of the completed tables to the Office of Structural Engineering.
<table>
<thead>
<tr>
<th>Project Number:</th>
<th>Maximum Factored Bearing Pressure: *</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Number: *</td>
<td>Structure File Number: *</td>
</tr>
<tr>
<td>Benchmark Location:</td>
<td></td>
</tr>
<tr>
<td>Footing Location: *</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Monitoring Period</th>
<th>Left monument</th>
<th>Right monument</th>
</tr>
</thead>
<tbody>
<tr>
<td>After footing concrete is placed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Before placement of superstructure members</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Before deck placement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After deck placement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Project completion</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTE TO DESIGNER:**

* The Designer shall modify items marked with an asterisk to describe the class of concrete, pier and/or abutment location, bridge number, SFN, maximum factored bearing pressure and to correctly describe the “As Per Plan” bid item.

### 606.7 FOOTINGS

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:

[606.7-1] 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.

[606.7-2] 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 inches 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 effect an appreciable saving:

[606.7-3] 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 Director, 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 inches 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.

606.8 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] 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 also by tip resistance. The factored resistance developed by side resistance is *** kips, assumed to act along the bottom *** feet of the bedrock socket for the abutments and *** feet of the bedrock socket for the piers. The factored resistance provided by the drilled shaft tip 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.

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 MISCELLANEOUS GENERAL NOTES

610.1 APPROACH SLABS

Item 526, Reinforced Concrete Approach Slabs was developed such that the concrete used in the superstructure would also be used for the approach slabs. The new supplemental specification for QC/QA concrete is not included in Item 526.

Provide both of the following notes on projects that specify SS898, QC/QA Concrete for Structures:

[610.1-1] ITEM 898 - QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (APPROACH SLAB), AS PER PLAN

Furnish approach slabs conforming to CMS 526 except concrete shall be in accordance with Supplemental Specification 898, QC/QA Concrete, Class QSC2. The accepted quantities shall include: concrete, curbs, reinforcing steel, joint fillers, joint sealers, joint seals, and waterproofing. The Department will measure approach slabs by the number of square yards. The Department will initially pay the full bid price to the Contractor upon completing the work. The Department will calculate the final adjusted payment according to 898.17 and include approach slab concrete and deck concrete in the same lot to determine final pay factors.

[610.1-2] ITEM 898 - QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (DECK), AS PER PLAN

The Department will calculate the final adjusted payment according to 898.17 and include approach slab concrete and deck concrete in the same lot to determine final pay factors.

610.2 INTEGRAL AND SEMI-INTEGRAL ABUTMENT EXPANSION JOINT SEALS

A neoprene sheet is required for waterproofing of the backside of the joint between the integral backwall and the bridge seat. Include the following note, which contains criteria for the installation of this seal, for all integral and semi-integral abutments. Plan details will be required to show location and dimensional position for installation.

[610.2-1] ITEM 516 SEMI-INTEGRAL ABUTMENT EXPANSION JOINT SEAL, AS PER PLAN: Install a 3 foot wide neoprene sheet at locations shown in the plans. Secure the neoprene sheeting to the concrete with 1 1/4" x #10 gage (length x shank diameter) galvanized button head spikes through a 1 inch outside diameter, #10 gage galvanized washer. Maximum fastener spacing is 9 inches. Use of other similar galvanized devices, which will not damage either the
neoprene or the concrete, will be subject to the approval of the Engineer.

Center the neoprene strips on all joints. For horizontal joints, secure the horizontal neoprene strip by using a single line of fasteners, starting at 6 inches, +/-, from the top of the neoprene strip. For the vertical joints secure the vertical neoprene strip by using a single vertical line of fasteners, starting at 6 inches, +/-, from the vertical edge of the neoprene strip nearest to the centerline of roadway. For vertical joints, install 2 additional fasteners at 6 inches, center to center, across the top of the neoprene strip on the same side of the vertical joint as the single vertical row of fasteners is located.

The vertical neoprene strips shall completely overlap the horizontal strips. Lap lengths of the horizontal strips that are not vulcanized or adhesive bonded, shall be at least 1 foot in length, or 6 inches in length if the lap is vulcanized or adhesive bonded. No laps are acceptable in vertically installed neoprene strips.

The neoprene sheeting shall be 3/32" thick general purpose, heavy-duty neoprene sheet with nylon fabric reinforcement. The sheeting shall be “Fairprene Number NN-0003”, by E. I. Dupont De Nemours and Company, Inc., “Wingprene” by the Goodyear Tire and Rubber Company, or an approved alternate. The neoprene sheeting shall conform to the following:

<table>
<thead>
<tr>
<th>Description of Test</th>
<th>ASTM</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness, inches</td>
<td>D751</td>
<td>0.094 ± 0.01</td>
</tr>
<tr>
<td>Breaking Strength, Grab, lbs, minimum</td>
<td>D751</td>
<td>700 x 700 (Long. x Trans.)</td>
</tr>
<tr>
<td>Adhesive Strip, 1&quot; wide x 2&quot; long, lbs, minimum</td>
<td>D751</td>
<td>9</td>
</tr>
<tr>
<td>Burst Strength, psi, minimum</td>
<td>D751</td>
<td>1400</td>
</tr>
<tr>
<td>Heat Aging, 70 Hr, 212 °F, 180° bend without cracking</td>
<td>D2136</td>
<td>No cracking of coating</td>
</tr>
<tr>
<td>Low temp. brittleness, 1 Hr, ! 40 °F, bend around 1/4” mandrel</td>
<td>D2136</td>
<td>No cracking of coating</td>
</tr>
</tbody>
</table>

METHOD OF MEASUREMENT: The Department will measure the total length of joint to be sealed by the number of feet.

BASIS OF PAYMENT: The Department will pay for accepted quantities at the contract price for Item 516, Semi-Integral Abutment Expansion Joint Seal, As Per Plan.

NOTE TO DESIGNER: Change “semi-integral” to “integral” as appropriate.
**610.3 CONCRETE PARAPET SAWCUT JOINTS**

Include the following note in the Structural General Notes when a concrete parapet or railing is used and standard drawings do not cover the below requirements.

[610.3-1] **CONCRETE PARAPETS:** As soon as a concrete saw can be operated without damaging the freshly placed concrete, sawcut 1 1/4" deep control joints into the perimeter of the concrete parapet starting and ending at the elevation of the concrete deck. Place the sawcuts at a minimum of 6 feet and a maximum of 10 feet centers. Use an edge guide, fence, or jig to ensure that the cut joint is straight, true, and aligned on all faces of the parapet. The joint width shall be the width of the saw blade, a nominal width of 1/4 inch. Seal the perimeter of the deflection control joint to a minimum depth of 1 inch with a polyurethane or polymeric material conforming to ASTM C920, Type S. Leave the bottom ½ inch of the inside and outside face unsealed to allow water to escape.

**610.4 BEARING PAD SHIMS, PRESTRESSED**

Add the following note to ensure proper seating of prestressed concrete box beams for skewed bridges.

[610.4-1] **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.

**610.5 CLEANING STEEL IN PATCHES**

Use this note with all concrete patching bid items that refer to the cleaning requirements specified in 519.04

[610.5-1] **ITEM 519 - PATCHING CONCRETE STRUCTURES, AS PER PLAN:** Prior to the surface cleaning specified in 519.04 and within 24 hours of placing patching material, blast clean all surfaces to be patched including the exposed reinforcing steel. Acceptable methods include high-pressure water blasting with or without abrasives in the water, abrasive blasting with containment, or vacuum abrasive blasting.
610.6  COFFERDAMS, CRIBS AND SHEETING

Use this note when the plans include detail designs for temporary shoring.

[610.6-1] ITEM 503, COFFERDAMS, CRIBS, AND SHEETING, 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, Cribs, and Sheeting. No additional payment will be made for providing an alternate design.

610.7  DECK PLACEMENT NOTES

610.7.1  FALSEWORK AND FORMS

Use the following note when web depths greater than 84 in. are specified.

[610.7.1-1] ITEM 511, CLASS HP CONCRETE, SUPERSTRUCTURE, AS PER PLAN *
Locate the lower contact point of the overhang falsework at least ** inches ± 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.

610.7.2  DECK PLACEMENT DESIGN ASSUMPTIONS

Use the following note on all projects requiring mechanized finishing machines to place deck concrete.

[610.7.2] 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 for a total machine 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 302.2.7.2.c for design information regarding finishing machine loads.
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SECTION 700 – TYPICAL DETAIL NOTES

701 SUBSTRUCTURE DETAILS

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 POROUS BACKFILL

Provide the following porous backfill note on the appropriate detail sheets.

[701.2-1] POROUS BACKFILL WITH FILTER FABRIC, 2 feet thick shall extend up to the plane of the subgrade, to 1 foot below the embankment surface, and laterally to the ends of the wingwalls.

For use when weep holes are specified:

[701.2-2] POROUS BACKFILL WITH FILTER FABRIC, 2 feet thick shall extend up to the plane of the subgrade, to 1 foot below the embankment surface, and laterally to the ends of the wingwalls. Place two cubic feet of bagged No. 3 aggregate at each weephole. The Department will include bagged aggregate with porous backfill for payment.

701.3 BRIDGE SEAT REINFORCING

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.)

[701.3-1] BRIDGE SEAT REINFORCING, SETTING ANCHORS: Accurately place reinforcing steel in the vicinity of the bridge seat to avoid interference with the drilling of bearing anchor holes or the pre-setting of bearing anchors.

[701.3-2] BRIDGE SEAT REINFORCING, SETTING ANCHORS: Accurately place reinforcing steel in the vicinity of the bridge seat to avoid interference with the
drilling of anchor bar holes.

701.4 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.

[701.4-1] BRIDGE SEAT ELEVATIONS have been adjusted upward _____ inches at abutments and _____ inches at piers to compensate for the vertical deformation of the bearings.

701.5 PROPER SEATING OF STEEL BEAMS AT ABUTMENTS

For a structure with concrete backwalls, deck joints and concrete decks supported on beams or girders, show an optional backwall construction joint at the level of the approach slab seat and provide the following note either on the appropriate abutment detail sheet or in the General Notes.

[701.5-1] BACKWALL CONCRETE: In addition to 511.10, do not place backwall concrete above the optional construction joint at the approach slab seat until after the deck concrete in the span adjacent to the abutment has been placed.

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:

[701.5-2] 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.

701.6 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:

[701.6-1] ABUTMENT CONCRETE: Do not place the abutment concrete above the bridge seat construction joint until the prestressed concrete box beams have been erected.
701.7 SEALING OF BEAM SEATS

Provide the following note when elastomeric bearings are to be placed on substructures with beam seats sealed with an epoxy or non-epoxy sealer:

[701.7-1] SEALING OF BEAM SEATS: If the beams seats are sealed with an epoxy or non-epoxy sealer prior to setting the bearings, do not apply sealer to the concrete surfaces under the proposed bearing locations. If these locations are sealed, remove the sealer to the satisfaction of the Engineer prior to setting the bearings. The Department will not pay for this removal.

702 SUPERSTRUCTURE DETAILS

702.1 STEEL BEAM DEFLECTION AND CAMBER

For steel beam or built-up girder bridges provide a table similar to Figure 702.1-1 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 Section 302.4.1.8. The table is to include bearing points, quarter points, center of span, splice points, and maximum 30 foot [10.0 meter] increments. Unique geometry may require an even closer spacing.

702.2 STEEL NOTCH TOUGHNESS REQUIREMENT (CHARPY V-NOTCH)

CVN material is a requirement to help assure fracture toughness of main material. Designers using this note should understand not only why CVN is specified but what is a main member. Section 302.4.1.10 helps with the definition of main members and specially highlights that crossframes of curved steel structures, because they are actual designed members carrying liveload forces, are also main members. Designers are reminded they must indicate specific pieces, members, shapes, etc. that are main 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 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  ELASTOMERIC BEARING LOAD PLATE

Where the load plate of an elastomeric bearing is to be connected to the structure by welding, provide the following note with the pertinent bearing details:

[702.4-1] WELDING: Control welding so that the plate temperature at the elastomer bonded surface does not exceed 300°F as determined by use of pyrometric sticks or other temperature monitoring devices.

702.5  BEARING REPOSITIONING

Where elastomeric bearing repositioning is required for a steel beam or girder superstructure, provide the following plan note.

[702.5-1] BEARING REPOSITIONING: If the steel is erected at an ambient temperature higher than 80°F or lower than 40°F and the bearing shear deflection exceeds 1/6 of the bearing height at 60°F (+/-) 10°F, raise the beams or girders to allow the bearings to return to their undeformed shape at 60°F (+/-) 10°F.

702.6  CONCRETE PLACEMENT SEQUENCE NOTES

Also see section 701.5 notes.

702.6.1  CONCRETE INTERMEDIATE DIAPHRAGM FOR PRESTRESSED CONCRETE I-BEAMS

If the design plans do not reference Standard Bridge Drawing PSID-1-99, provide the following note.

[702.6.1-1] 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 be Class S.

702.6.2  SEMI-INTEGRAL OR INTEGRAL ABUTMENT CONCRETE PLACEMENT FOR STEEL MEMBERS

The following notes may be needed depending on whether the bridge superstructure is steel or prestressed concrete; requires phased construction; is skewed a specific amount, or requires a closure pour.

Use the following note with steel superstructures. There is no skew limitation placed on the use of this note. Because steel members offer little resistance to torsional loadings, concrete diaphragms generally will not crack as a result of the placement of deck concrete on skewed steel
structures.

[702.6.2-1] ABUTMENT DIAPHRAGM CONCRETE, STEEL SUPERSTRUCTURE: Place the concrete encasing the structural steel members with the deck concrete or at least 48 hours before placement of the deck concrete.

Prestressed I-beams have greater torsional rigidity than steel. When the diaphragms are pre-placed on I-beam bridges with skews greater than 10 degrees, the torsional rigidity of the beam combined with the torsional restraint of the pre-poured diaphragm can cause cracking of the beams and/or diaphragm when the beams deflect under the weight of the deck concrete. Use the following note for prestressed I-beam structures with skews greater than 10 degrees.

[702.6.2-2] ABUTMENT DIAPHRAGM, PRESTRESSED I-BEAM SUPERSTRUCTURE: Place the concrete encasing the prestressed I-beam structural members as part of the deck pour.

Use the following note for prestressed I-beam structures with skews of 10E or less.

[702.6.2-3] ABUTMENT DIAPHRAGM, PRESTRESSED I-BEAM SUPERSTRUCTURE: Place the concrete encasing the prestressed I-beam structural members with the deck concrete or at least 48 hours before placement of the deck concrete.

Phased construction of a bridge requires different procedures for placing concrete abutment diaphragms and bridges decks. For steel superstructures where a closure pour section is detailed between phases, use the following note.

[702.6.2-4] ABUTMENT DIAPHRAGM CONCRETE, STEEL SUPERSTRUCTURE, PHASED CONSTRUCTION: Place the concrete in the abutment diaphragm encasing structural steel members of an individual phase separately or with the deck concrete of that phase. If the diaphragm concrete is placed separately, allow at least 48 hours of set time before placing deck concrete. Locate the horizontal construction joint between the diaphragm and deck concrete at the approach slab seat.

For steel superstructures where no separate closure pour section between phases is detailed in the plans, use the following note.

[702.6.2-5] ABUTMENT DIAPHRAGM CONCRETE, STEEL SUPERSTRUCTURE, PHASED CONSTRUCTION: Place the concrete in the abutment diaphragm encasing structural steel members of an individual phase with the deck pour to allow for expected dead load rotation at the abutments.

For prestressed I-beam superstructures, whether a closure pour section is detailed between phases or not, use the following note.

[702.6.2-6] PHASED CONSTRUCTION ABUTMENT DIAPHRAGM CONCRETE,
PRESTRESSED I-BEAM SUPERSTRUCTURE  Place abutment diaphragm concrete encasing prestressed I-beam members at the same time as the deck concrete of an individual construction phase to allow for expected dead load rotation at the abutments.

702.7  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.

[702.7-1]  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 constant haunch width outside the edge of each beam/girder flange of 9 inches.  Deviate from this haunch thickness as necessary to place the deck surface at the finished grade.  The allowable tolerance for the haunch width outside the edge of each beam/girder flange is ± 3 inches.

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 511.24.

NOTE TO DESIGNER:  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.

702.8  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 as follows:

A = Design slab thickness.
B = Anticipated total midspan camber due to the design prestressing force at time of release
C = Deflection at midspan due to the self weight of the beam
D = Deflection at midspan due to dead load of the slab, diaphragms and other non-composite loads.
E = Deflection due at midspan to dead load of railing, sidewalk and other composite dead loads not including future wearing surface
F = Adjustment for vertical curve.  Positive for crest vertical curves
G = Sacrificial haunch depth (2" [50 mm])
H = Total Topping Thickness at beam bearings = A + 1.8B - 1.85C - D - E - F + G. (If F > 1.8B - 1.85C - (D+E) then H = A + G)

I = Total Topping Thickness at mid-span = A + G. If F > 1.8B - 1.85C - (D +E) then I = A - (1.8B - 1.85C) + D + E + F + G

Use the gross moment of inertia for the non-composite beam to calculate the camber and deflection values B, C, and D. For E, use the moment of inertia for the composite section.

Show a longitudinal superstructure cross section in the plans detailing the total Topping Thickness including the design slab thickness and the haunch thickness at the centerline of spans and bearings. Also show screed elevation tables similar to 702.16.2. Provide the following note in the plans:

[702.8-1] Calculated camber at the time of release is _____ inches.

Calculated camber at the time of erection is _____ inches.

Calculated long-term camber is _____ inches.

NOTE TO DESIGNER: The camber at the time of release is (B-C), the camber at the time of erection is (1.8B – 1.85C), and the long-term camber is (2.45B- 2.40C).

[702.8-2] DECK SLAB THICKNESS FOR CONCRETE QUANTITY: The Topping thicknesses shown from the top of the deck slab to the top of the top flange along the centerline of the I-beam are theoretical dimensions. The haunch depth is the Topping thickness minus the design slab thickness. The Department will pay for superstructure concrete based on the design slab thickness and the average of the theoretical haunch depths at mid-span and at each beam bearing even though deviation from the dimensions shown may be necessary to place the deck surface at the finished grade. Once all beams are set in their final position, the actual camber for each member will be the top of beam elevation at mid-span minus the average top of beam elevation at each bearing. The actual Topping thickness at mid-span will be the theoretical dimension plus or minus the difference between the actual and anticipated camber.

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" [10mm]:

[702.8-3] 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.
702.9 PRESTRESSED CONCRETE BOX BEAM BRIDGE

For prestressed concrete box beam bridges, the asphalt or concrete topping depth over the top of the beams shall be computed as follows:

A = Minimum topping thickness
B = Anticipated total midspan camber due to the design prestressing force at time of release
C = Deflection due to the self weight of the beam (including diaphragms)
D = Deflection due to dead load of the topping and other non-composite loads
E = Deflection due to dead load of railing, sidewalk and other composite dead loads not including future wearing surface
F = Adjustment for vertical curve. Positive for crest vertical curves
G = Total Topping Thickness at beam bearings = A + 1.8B - 1.85C - D - E - F. If F > 1.8B - 1.85C - (D + E) then G = A
H = Total Topping Thickness at mid-span = A
   If F > 1.8B - 1.85C - (D + E) then H = A - (1.8B - 1.85C) + D + E + F

Use the gross moment of inertia for the non-composite beam to calculate the camber and deflection values B, C, and D. For E, use the moment of inertia for the composite section when designing a composite box beam otherwise use the non-composite section. Note that with the exception of when F > 1.8B - 1.85C - (D + E), the dead load deflection adjustment (D + E) is made by adjusting the beam seat elevations upward.

For non-composite prestressed concrete box beam bridges with an asphaltic concrete surface course provide a note similar to [702.9-1]. For composite prestressed concrete box beam bridges with a concrete surface course provide a note similar to [702.9-2].

[702.9-1] Calculated camber at the time of release is ______ inches.

Calculated camber at time of paving is ______ inches.

Long term camber is ______ inches.

Calculated deflection due to dead load applied after the beams are set (weight of surface course, railings, sidewalks, etc.) is _____ inches.

The vertical curve adjustment to the topping thickness at midspan is _____ inches upward.

The vertical curve adjustment to the topping thickness at each bearing is inches upward/downward.

(1) The thickness of the intermediate asphalt course shall be 1½ inches. No variation
in thickness is required.

(2) The thickness of the intermediate asphalt course shall vary from 1½ inches at each centerline of beam bearing to _____ inches at midspan.

(3) The thickness of the intermediate asphalt course shall vary from _____ inches at each centerline of beam bearing to 1½ inches at midspan.

[702.9-2] Calculated camber at the time of release is _____ inches.

Calculated camber at time of paving is _____ inches.

Long term camber is _____ inches.

Calculated deflection due to dead load applied after the beams are set (weight of surface course, railings, sidewalks, etc.) is _____ inches.

The vertical curve adjustment to the topping thickness at midspan is _____ inches upward.

The vertical curve adjustment to the topping thickness at each bearing is _____ inches upward/downward.

(1) The concrete thickness shall be 6 inches. No variation in thickness of concrete is required.

(2) The concrete thickness shall vary from 6 inches at each centerline of beam bearing to _____ inches at midspan.

(3) The concrete thickness shall vary from _____ inches at each centerline of beam bearing to 6 inches at midspan.

NOTE TO DESIGNER: The calculated camber at the time of release is \((B - C)\), at the time of paving is \((1.8B - 1.85C)\), and long term is \((2.45B - 2.40C)\). The calculated deflection due to dead load applied after the beams are set is \((D + E)\). The vertical curve adjustment at midspan is \((F)\) when \(F > 1.8B - 1.85C - D - E\). The vertical curve adjustment at each bearing is \((F)\) when \(F < 1.8B - 1.85C - D - E\) and may be upward for sag curves or downward for crest curves. Remove the reference to the vertical curve adjustment that does not apply.

Conclude note [702.9-1] with note (1), (2) or (3) as appropriate. Note (1) should be used when after placement of the topping, the top surface of the beam parallels the profile grade. Note (2) should be used when \(F > 1.8B - 1.85C - D - E\). Note (3) should be used for all other cases.

For non-composite designs, include in the bridge plans a diagram similar to Figure 702.9-1 showing the thickness of the Item 448 Intermediate course and the Item 448 surface course at each centerline of bearing and at midspan.
For composite design, show a longitudinal superstructure cross section in the plans detailing the total Topping Thickness at each centerline of bearings and at midspan. Also show screed elevation tables similar to 0.

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" [10mm]:

[702.9-3] 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.

702.10 ASPHALT CONCRETE SURFACE COURSE

Place note [702.10-1] on the plans for prestressed concrete box beam bridges having an asphalt concrete surface course. If the nominal thickness of 448 varies from the 1½" [38 mm] shown, revise the note accordingly.

While this note specifies how to place only the two 448 bid items, the designer should recognize that two Item 407 tack coat items are also required. One tack coat is applied before the intermediate asphalt concrete course. The other tack coat is applied between the intermediate and surface course.

[702.10-1] ASPHALT CONCRETE SURFACE COURSE shall consist of a variable thickness of 448 asphalt concrete intermediate course, Type 2, PG64-28 and a 1½" thickness of 448 asphalt concrete surface course, Type 1H. Place the 448 intermediate 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 1 ½" below final pavement surface elevation.

702.11 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.

[702.11-1] 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).
702.12 ERECTION BOLTS

Where erection bolts are specified for attaching crossframes on steel girder or rolled beam bridges, and the expected dead load differential deflection at each end of the crossframes is less than or equal to ½" [13 mm] provide the following note. (Do not use the note if standard drawing GSD-1-96 is being referenced.)

702.12-1 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.

702.12-2 Note Retired – See Appendix

702.13 WELDED ATTACHMENTS

Provide the following note on plans for steel beam or girder bridges:

702.13-1 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.

702.14 DECK ELEVATION TABLES

702.14.1 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 302.2.3. Refer to Figure 0-1 for an example screed table for structural steel members and Figure 0-2 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 302.2.3.

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.

702.14.1 SCREED ELEVATIONS shown represent the theoretical deck surface location
prior to deflections caused by deck placement and other anticipated dead loads.

702.14.2 **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 302.2.3.

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.

[702.14.2-1] 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.

702.14.3 **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 302.2.3.

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.

[702.14.3-1] FINAL DECK SURFACE ELEVATIONS shown represent the deck surface location after all anticipated dead load deflections have occurred.

702.15 **ELASTOMERIC BEARING MATERIAL REQUIREMENTS**

Use the following note for elastomeric bearings designed in accordance with *LRFD 14.7.6* (i.e. Method A)

[702.15-1] 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)
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.

702.16 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.

[702.16-1] 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 reinforcing steel 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 reinforcing steel 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>
<thead>
<tr>
<th>Member Line 1</th>
<th>Rear Abutment</th>
<th>Pier No #</th>
<th>Forward Abutment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member Line 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Member Line 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Member Line 4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

702.17 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.

[702.17-1] 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.
702.18  **FRACTURE CRITICAL FABRICATION NOTE**  

For structures that contain fracture critical components and members, place the following note in the design plans.

[702.18-1]  
FCM: All items designated FCM (, including _______ )\(^*\) are Fracture Critical 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 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.

702.19  **WELDED SHEAR CONNECTORS ON GALVANIZED STRUCTURES**

For galvanized structures with welded shear connectors, place the following note on the same plan sheet as the shear connector spacing.

[702.19-1]  
WELDED SHEAR CONNECTORS: Install the welded shear connectors in the shop or in the field. If the connectors are shop installed prior to galvanizing, provide fall protection according to OSHA standards for all workers, including those engaged in connecting and in decking. If the connectors are field installed, remove the galvanic coating by grinding at each connector location prior to welding.

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 Section 201.2.2 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 Supplemental Specification 832.09 for the application requirements.

Plan Area of Temporary Fill Material = ____ acres

Total Volume of Temporary Fill Material = ____ yd\(^3\)
<table>
<thead>
<tr>
<th>Point (2)</th>
<th>Bearing Pt</th>
<th>1/4 Pt</th>
<th>Pt (3)</th>
<th>Mid Span</th>
<th>Pt (3)</th>
<th>1/4 Pt</th>
<th>Splice Pt</th>
<th>Bearing Pt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection due to weight of steel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deflection due to remaining deadload (4)</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
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<td></td>
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<tr>
<td>Adjustment required for vertical curve</td>
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<td></td>
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<td></td>
<td></td>
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<tr>
<td>Adjustment required for horizontal curve</td>
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<td></td>
</tr>
<tr>
<td>Adjustment required for heat curving (5)</td>
<td></td>
<td></td>
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<td></td>
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<td></td>
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<tr>
<td>Required Shop Camber</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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 30 feet. If the distance does exceed 30 feet locate the additional point midway between standard points.

4. Do not include a separate wearing surface that is not installed during the project.

5. For horizontally curved girders the designer is responsible for establishing the required additional camber to be included in the girder per LRFD 6.7.7.3. Include these values.
Variable Thickness 448 Intermediate course as required to match final elevation (Dimensions shall be detailed)

Proposed Roadway Surface

Excessive Curve

3"

Box Beam Member

Span

ASPHALT THICKNESS DIAGRAM
(Crest Vertical Curve)

1\(\frac{1}{2}\)" Uniform Thickness 448 Surface

1\(\frac{1}{2}\)" Minimum Uniform Thickness 448 Intermediate

Span

ASPHALT THICKNESS DIAGRAM
(Straight Grade or Sag Vertical Curve)

Variable Thickness 448 Intermediate as required to match final elevation (Dimensions shall be detailed)

1\(\frac{1}{2}\)" Uniform Thickness 448 surface

1\(\frac{1}{2}\)" Minimum Uniform Thickness 448 Intermediate

3"

Box Beam Member

Span

Figure 702.9-1
<table>
<thead>
<tr>
<th>Point (2)</th>
<th>SPAN NUMBER (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Station</td>
</tr>
<tr>
<td></td>
<td>Bearing Pt</td>
</tr>
<tr>
<td>Left Curb Line</td>
<td></td>
</tr>
<tr>
<td>Phased Const Line</td>
<td></td>
</tr>
<tr>
<td>Centerline</td>
<td></td>
</tr>
<tr>
<td>Right Curb Line</td>
<td></td>
</tr>
</tbody>
</table>

(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 feet. If the distance does exceed 30 feet, locate the additional point midway between standard points.
**COMPOSITE BOX BEAM SCREED TABLE**

<table>
<thead>
<tr>
<th>Point (2)</th>
<th>SPAN NUMBER (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Station</td>
</tr>
<tr>
<td>Bearing Pt</td>
<td>1/4 Pt</td>
</tr>
</tbody>
</table>

- 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 feet. If the distance does exceed 30 feet, locate the additional point midway between standard points.
SECTION 800 – NOISE BARRIERS

801   INTRODUCTION

According to Section 1400 of the ODOT Location and Design Manual, a Noise Wall Justification shall be included in the Preferred Alternative Verification Review Submission for Major Projects or in the Minor Project Preliminary Engineering Study Review Submission. When noise barriers are necessary, the Office of Environmental Services will furnish the required noise barrier height, length and location(s). The detail design for noise barriers shall be included in the Stage 2 Detailed Design Review Submission.

The Office of Structural Engineering maintains a list of approved noise barrier suppliers that are in conformance with this Manual and the Department’s “Noise Barrier Design” plan insert sheets. This list is shown on the plan insert sheets.

Noise barrier panel material types currently approved include: steel, fiberglass, concrete and brick or masonry. Department approval of other material types will proceed in accordance with the Department’s Standard Procedure 27-005(SP) for new products.

802   DESIGN CONSIDERATIONS

802.1   NOISE BARRIER FOUNDATIONS

The Design Agency shall perform a subsurface investigation at all noise barrier sites. The subsurface work shall be in accordance with the Specifications for Geotechnical Explorations.

802.2   NOISE BARRIER AESTHETICS

The Department limits standard aesthetic treatments for noise barriers. Currently aesthetic treatments standards are being evaluated and therefore the items listed below may be augmented or revised.

Aesthetic limitations include:

A. Concrete wall panels shall use an Ashlar stone pattern form liner, or other approved formliner. 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. Concrete posts shall be used except on bridge structures where steel posts are required.

C. Noise barrier panels and posts shall have a cap to create a shadow line. Caps shall be specifically designed for each project, should be of the same material as the panels and shall be integral with or mechanically fastened to the top panel.

D. General dimensions for the cap are:
1.  4 inches high

2.  4 inches wider than the post and panel selected and centered so the cap horizontally extends 2 inches beyond the panel or posts vertical surfaces. (Other options may be acceptable depending on limits of manufacturing process and final visual effect.)

E. The Department is standardizing color for panels and posts. Any color choices should be selected from the Department’s acceptable colors. The coating material used to produce the color is to be approved by the Office of Environmental Services.

Color alternative 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.

Aesthetic treatments beyond these limitations require review and approval by the Office of Environmental Services.

Due to aesthetic treatments each project will require special plan notes.

Those 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. The notes must assure the Contractor understands the requirements to assure the Project’s aesthetic objectives are met.

Any developed plan notes shall require the Contractor to submit to the Department, complete design and construction details of the walls and their aesthetic treatments, 30 days prior to the start of construction.

803 DETAIL DESIGN SUBMISSION REQUIREMENTS

Submit two sets of noise barrier plan sheets for Stage 2 Detailed Design Review that include the following:

A. All necessary dimensions, to adequately describe the required position of the noise barrier relative to the centerline of roadway

B. The length and height of barrier including beginning and ending stations.

C. Post spacing dimensions

D. Top elevation of barrier sound line. This elevation will normally be the same throughout the length of the wall. The Office of Environmental Services will specify the required elevation and elevation changes, if required.

E. The finished ground line and bottom of barrier elevations.

F. A noise barrier general summary containing all bid items and any required noise barrier material type alternate bids.
G. Plan notes and details to assure the aesthetic requirements of 802.2

H. Special plan details required to show features such as noise barriers on structures; locations of utilities, special access for fire hydrants, termination at structures, (such as bridges, culverts, overhead sign supports) and details not covered by the Department’s "Noise Barrier Design" plan insert sheets,

I. Subsurface investigation plan sheets including borings

J. The Department’s "Noise Barrier Design" plan insert sheets, revised as follows:
   1. List only the approved suppliers for the material noise barrier types authorized for the project and any alternate bid noise barrier material types authorized for the project.
   2. Clearly list the foundation depth for each drilled shaft throughout the total project length.
   3. Because Plan Insert Sheets are not standard drawings, the sheets shall be numbered as normal plan sheets for incorporation into final set of contract plans.

K. A copy of the Office of Environmental Services requirements for location and height of noise barrier walls.

L. The District Production Administrator should be contacted for the approved noise barrier material types, suppliers, alternate bid requirements, and special features in accordance with the Department’s Noise Wall Policy 417-001(P). A copy of the letter from the District Production Administrator stipulating the information in this paragraph should be part of the detailed design submission.

M. Show the location of all new and existing Reference Monuments in the noise wall plans. All new Reference Monuments, control points and benchmarks shall be located on the inside (i.e. roadway side) of noise walls. Provide a pay item to relocate every existing Reference Monument located on the outside of proposed noise walls.

804 NOISE BARRIERS – APPROVAL OF WALL DESIGNS

The Department does not have a standard design for noise barrier panels. The Department’s plan insert sheets do establish standard designs for posts and foundations.

Individual manufacturers submit panel designs and those designs, if approved, are added to the plan inserts sheets. A modification to an approved wall design requires a 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.

Environmental, structural and acoustic design for the walls shall meet the requirements of 805.1, 805.2 and 805.3 of this manual.

There are two types of noise barriers, reflective and absorptive. Noise barrier manufacturers interested in having their noise barrier wall approved should submit their proposed designs in accordance with 805.
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 ................................................................. Section 805.1
B. Structural ................................................................. Section 805.2
C. Material ................................................................. 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 20 working 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 Section 802.2 and the following Acoustic requirements:

A. Reflective noise barriers - Minimum TL (Transmission Loss) = 22 dBA
B. Absorptive noise barriers:
   1. Minimum TL (Transmission Loss) = 22 dBA
   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:

A. ASTM Standard Test Method for Sound Absorption and Sound Absorption Coefficients by Reverberation Room Method
B. ASTM C423 and E795 (Latest editions)
C. 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 4th Edition of the “AASHTO LRFD Bridge Design Specifications”, 2007; the AASHTO “Guide Specifications for Structural Design of Noise Barriers”, 1989, including all interims; the Department’s “Noise Barrier Details” plan insert sheets; 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.

B. Complete design calculations:
   1. Signed, sealed and dated by a registered professional engineer
   2. Using the following minimum wind pressures:
      a. Roadway barriers = 25 psf [1.2 kPa]
      b. Bridge barriers = 30 psf [1.4 kPa]
   3. Include all fabrication, shipping, handling and erection loads

C. Fabrication and construction drawings showing wall details, dimensions, connections and any other information required to define the wall system.

The following list provides the minimum design data for standard materials:

A. Reinforced Concrete ................................................................. \( f'c = 4000 \text{ psi} [27.5 \text{ MPa}] \)
B. Prestressed Concrete .............................................................. \( f'ci = 4000 \text{ psi} [27.5 \text{ MPa}] \)
   \( f'c = 5500 \text{ psi} [34.5 \text{ MPa}] \)
C. Structural Steel ........................................................................\( f_y = 50,000 \text{ psi} [345 \text{ MPa}] \)
D. Prestressing Strand (Low-relaxation) ........................................ \( f's = 270 \text{ ksi} [1860 \text{ MPa}] \)
E. Reinforcing Steel ................................................................. \( f_y = 60,000 \text{ psi} [413 \text{ MPa}] \)
F. Timber ................................................................................... \( f_b = 1500 \text{ psi} [10.3 \text{ MPa}] \)

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.
SECTION 900 – BRIDGE LOAD RATING

Refer to the January 2004 ODOT Bridge Design Manual for all bridge load rating requirements.
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<th>Title</th>
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SECTION 1000 - ODOT SUPPLEMENT TO THE
LRFD BRIDGE DESIGN SPECIFICATIONS

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

1001 LRFD SECTION 1 - INTRODUCTION

S1.3.3 DUCTILITY

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

S1.3.4 REDUNDANCY

Non-redundant designs should be avoided.

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

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

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

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

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

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

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

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

**S1.3.5 OPERATIONAL IMPORTANCE**

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

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

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

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

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

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

S2.3.2.2.2 PROTECTION OF USERS

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

S2.3.3.2 HIGHWAY VERTICAL

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

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

S2.5.2.4 RIDEABILITY

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

S2.5.2.6.2 CRITERIA FOR DEFLECTION

Designers shall apply the deflection limits shown. Do not include the stiffness contribution of railings, sidewalks and median barriers into the design of the composite section.

S2.5.2.6.3 OPTIONAL CRITERIA FOR SPAN-TO-DEPTH RATIOS

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

S2.6.6.3 TYPE, SIZE AND NUMBER OF DRAINS

Refer to Section 1103 of the Location and Design Manual, Volume 2 for ODOT’s design criteria for deck drainage.
S2.6.6.4   **DISCHARGE FROM DECK DRAINS**

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

S3.4.1 LOAD FACTORS AND LOAD COMBINATIONS
The load combinations and load factors specified in Table 3.4.1-1 shall apply. If a bridge design warrants the use of a special design vehicle analysis, the scope of services will provide the necessary information. Otherwise, the Department does not require an analysis using a special design vehicle, and the Strength II limit state will not apply.

S3.5.1 DEAD LOADS: DC, DW, AND EV
In lieu of more specific information, the assumed unit weight of normal weight reinforced concrete shall be 0.150 kcf.

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

S3.6.1.3.1 GENERAL
The investigation of load effects produced by two tandem vehicles spaced from 26.0 ft. to 40.0 ft as specified in Article C3.6.1.3.1 is not required.

S3.6.1.3.2 LOADING FOR OPTIONAL LIVE LOAD DEFLECTION EVALUATION
The live load deflection criteria specified in Article 2.5.2.6.2 applies.

S3.6.1.3.3 DESIGN LOADS FOR DECKS, DECK SYSTEMS, AND THE TOP SLABS OF BOX CULVERTS
Use the approximate strip method of analysis. Do not apply the Empirical Design Method specified in Article 9.7.2. Refer to BDM Section 302.2.2 for more information.

S3.6.1.3.4 DECK OVERHANG LOAD
This article does not apply. Design deck overhangs in accordance with BDM Section 302.2.2.

S3.6.1.4.2 FREQUENCY
The \( ADTT \) shall be estimated as follows:

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

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

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

**S3.6.1.6 PEDESTRIAN LOADS**

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

**S3.6.2.1 GENERAL**

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

**S3.6.5.1 PROTECTION OF STRUCTURES**

The provisions of Article 3.6.5.2 need not be considered for redundant substructures which are protected according to the ODOT Location and Design Manual, Volume One, Section 600.

The provisions of Article 3.6.5.2 need not be considered for non-redundant substructures which are protected by:

- An embankment;
- A structurally independent, ground-mounted 54.0-in. high TL-5 barrier, located within 10.0 ft. from the component being protected; or
- A 42.0-in. high TL-5 barrier located at more than 10.0 ft. from the component being protected.

**S3.6.5.2 VEHICLE AND RAILWAY COLLISION WITH STRUCTURES**

For bridges over roadways, unless protected as specified in Article 3.6.5.1, abutments and piers located within a distance of 30.0 ft. to the edge of the roadway shall be designed for an equivalent static force of 400 kip, which is assumed to act in any direction in a horizontal plane, at a distance of 4.0 ft. above ground.

For bridges over railways, refer to BDM Sections 204.5 and 209.8 for abutment and pier protection requirements.
S3.10.4  SEISMIC PERFORMANCE ZONES

All bridges in the state of Ohio fall within Seismic Zone 1.

S3.10.9.2  SEISMIC ZONE 1

The entire State of Ohio shall be assumed to have an acceleration coefficient above 0.025 but less than 0.09.

Design the connection of the superstructure to the substructure to resist a horizontal seismic force in the restrained direction equal to 0.2 times the vertical reaction due to the tributary permanent load. The tributary area refers to the uninterrupted segment of a superstructure contributing to the load on the seismic restraint. The restrained direction for an expansion bearing is typically transverse to the structure. The tributary permanent load shall include an allowance for future wearing surface.

Assume the Extreme Event I load factor for live load ($\gamma_{EQ}$) is equal to 0.0.

Standard integral and semi-integral abutment types do not require the addition of seismic restraints. The horizontal restraint provided by these abutment types is sufficient to resist the seismic force generated by the tributary area contributing dead load to the abutment. For multiple span structures with integral or semi-integral abutments, additional seismic restraints located at one or more piers are required to resist the remaining seismic force acting on the superstructure. Bearing guides, as detailed in the semi-integral abutment standard drawing, are required for skews of 30 degrees or more.

If seismic restraints are provided, design the substructure units for an earthquake force (EQ) at the Extreme Event I limit state equal to 0.2 times the tributary dead loads applied in the restrained direction resulting in the maximum load effect.

Refer to Article 4.7.4.4 for minimum seat width requirements.

S3.11.2  COMPACTION

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

S3.11.6.5  REDUCTION OF SURCHARGE

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

Refer to BDM Section 202.2.3.2.c for more information.

S3.12.2 **UNIFORM TEMPERATURE**

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

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

The base construction temperature assumed for design shall be 60°F.
1004      LRFD SECTION 4 – STRUCTURAL ANALYSIS AND EVALUATION

S4.4     ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS

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

Regardless of the analysis method utilized for design, specific superstructure types are required to be load rated using the AASHTO BARS-PC software in accordance with BDM Section 900. At the inventory level, the minimum rating factor for the HS20-44 loading shall be 1.25.

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

S4.5.1     GENERAL

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

S4.6.2.2.1     APPLICATION

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

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

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

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

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

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

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

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

S4.6.3 REFINED METHODS OF ANALYSIS

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

S4.6.4.3 APPROXIMATE PROCEDURE

The approximate procedure for moment distribution as described in Appendix B6 is permitted.

Moment redistribution as described in Article 5.7.3.5 is not permitted.
LRFD SECTION 5 – CONCRETE STRUCTURES

S5.4.2.3.3 SHRINKAGE
Designers shall assume the relative humidity to be 70% in the absence of more precise data.

S5.4.3.3 SPECIAL APPLICATIONS
BDM Section 301.5 and C&MS 509.02 specify all reinforcing steel for structures to be epoxy coated.

S5.4.4.2 MODULUS OF ELASTICITY
Designers shall assume the modulus of elasticity for prestressing strand to be 28,500 ksi in the absence of more precise data.

S5.5.3.1 GENERAL
Fatigue need not be investigated for the design of longitudinal edge beams of slab bridges.

S5.7.3.4 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT
Unless otherwise noted below, the exposure factor ($\gamma_e$) for reinforcing steel in cast-in-place concrete bridge decks and cast-in-place slab bridges shall be 0.75. The one inch monolithic wearing surface, BDM Section 302.1.3.1, shall be deducted from both $d_c$ and $h$.

S5.7.3.5 MOMENT REDISTRIBUTION
Moment redistribution is not permitted.

S5.7.4.6 SPIRALS AND TIES
This provision only applies to columns where the ratio of axial column capacity to axial column load at the Strength Limit State is less than 1.5. For all other column designs, spiral reinforcement shall be detailed as specified in BDM Section 303.3.2.1.

S5.8.4.2 COHESION AND FRICTION
The top surface of composite prestressed concrete beams produced under Item 515 is intentionally roughened to an amplitude of 0.25 in.
S5.9.4.2.2  **TENSION STRESSES**

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

S5.9.5.3  **APPROXIMATE ESTIMATE OF TIME-DEPENDENT LOSSES**

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

S5.9.5.4  **REFINED ESTIMATES OF TIME-DEPENDENT LOSSES**

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

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

A. Age at transfer \( (t_i) \) ................................................................. 0.75 days

B. Age at deck placement \( (t_d) \) ................................................. 45 days

C. Final age \( (t_f) \) ................................................................. 10,000 days

S5.10.3.1.1  **CAST-IN-PLACE CONCRETE**

The following two tables summarize the coarse aggregate gradations permitted for standard ODOT mixes and the maximum aggregate size corresponding to each gradation.

### Table S5.10.3.1.1-1

<table>
<thead>
<tr>
<th>ODOT Concrete Mix</th>
<th>Permitted Coarse Aggregate Gradations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class QSC1, QSC2, &amp; QSC3 (SS898)</td>
<td>No. 8, 78, 7, 67, or 57</td>
</tr>
<tr>
<td>Class C (C&amp;MS 499)</td>
<td>No. 8, 78, 7, 67, or 57</td>
</tr>
<tr>
<td>Class S (C&amp;MS 499)</td>
<td>No. 67 or 57</td>
</tr>
<tr>
<td>Class HP (C&amp;MS 499)</td>
<td>No. 8</td>
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</table>

### Table S5.10.3.1.1-2

<table>
<thead>
<tr>
<th>Gradation</th>
<th>Maximum Aggregate Size</th>
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<tbody>
<tr>
<td>No. 57</td>
<td>1.5 in.</td>
</tr>
<tr>
<td>No. 6, 67, 68</td>
<td>1.0 in.</td>
</tr>
<tr>
<td>No. 7, 78</td>
<td>3/4 in.</td>
</tr>
<tr>
<td>No. 8</td>
<td>1/2 in.</td>
</tr>
</tbody>
</table>
When a maximum aggregate size required for design purposes differs from the gradations specified in C&MS 499 or SS898, the Designer shall provide a plan note and specify the concrete pay item “As Per Plan”.

S5.10.3.1.2 PRECAST CONCRETE

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

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

S5.10.3.3.1 PRETENSIONING STRAND

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

S5.10.6.2 SPIRALS

If the ratio of axial column capacity to axial column load is 1.5 or greater at the Strength Limit States, the center-to-center spacing between spirals (i.e. pitch) shall be 4.5 in as specified in BDM Section 303.3.2.1. Otherwise, the spacing limitation in Article 5.10.6.2 applies.

S5.10.6.3 TIES

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

S5.11.4.3 PARTIALLY DEBONDED STRANDS

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

S5.12.3 CONCRETE COVER

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

Refer to BDM Section 302.5.2.6 for additional information.

S5.13.4.5.2  REINFORCING STEEL

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

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

S6.4.1 STRUCTURAL STEELS

Refer to BDM Section 302.4.1.1 for steel selection criteria.

S6.4.3.1 BOLTS

The use of ASTM A 490 bolts is prohibited.

S6.6.1.2.5 FATIGUE RESISTANCE

The nominal fatigue resistance range for base metal at details connected with transversely loaded fillet welds, where a discontinuous plate is loaded, shall be taken as specified in Article 6.6.1.2.5.

Otherwise, the nominal fatigue resistance, \((\Delta F)_n\), shall be taken as:

A. For bridges located on interstates and interstate look-alike routes:

\[
(\Delta F)_n = \frac{1}{2} (\Delta F)_{TH}
\]

B. For bridges located on all other routes:

\[
(\Delta F)_n = \left( \frac{A}{N} \right)^{\frac{1}{5}} \geq \frac{1}{2} (\Delta F)_{TH}
\]

in which:

\[N = (365)(75)n(ADTT)_{SL}\]

where:

\[A\] = constant taken from Table 6.6.1.2.5-1

\[n\] = number of stress range cycles per truck passage taken from Table 6.6.1.2.5-2

\[(ADTT)_{SL}\] = single-lane ADTT as specified in Article 3.6.1.4

\((\Delta F)_{TH}\) = constant-amplitude fatigue threshold taken from Table 6.6.1.2.5-3

S6.6.2 FRACTURE

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

The CVN requirements for HPS 70W steels are not provided in C&MS 711.01, but are included in BDM Appendix note AN-10.
S6.7.2   **DEAD LOAD CAMBER**

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

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

S6.7.4.1   **GENERAL**

Skewed crossframes at intermediate support points should be avoided. Refer to BDM Section 302.4.2.3 for additional crossframe design guidelines.

S6.7.4.2   **I-SECTION MEMBERS**

Crossframes shall be oriented perpendicular to the main steel members regardless of the structure’s skew angle.

S6.10.1.1.1b   **STRESSES FOR SECTIONS IN POSITIVE FLEXURE**

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

S6.10.1.7   **MINIMUM NEGATIVE FLEXURE CONCRETE DECK REINFORCEMENT**

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

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 302.4.3.3.a.

S6.10.7.3   **DUCTILITY REQUIREMENT**

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

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.
S6.10.10.1.1 TYPES

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

S6.10.10.1.4 COVER AND PENETRATION

A detail for deep haunches is provided in the 2004 ODOT BDM Section 400.

S6.10.11.1.1 GENERAL

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

Refer to BDM Sections 304.4.2.2 and 304.4.3.4 for more information.

S6.10.11.3 LONGITUDINAL STIFFENERS

BDM Section 302.4.3.1 prohibits the use of longitudinal stiffeners.

S6.13.2.4.1 TYPE

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

S6.13.2.6.6 EDGE DISTANCES

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

S6.13.2.8 SLIP RESISTANCE

Refer to BDM Section 302.4.1.14 for more information.

S6.13.6.1.4a GENERAL

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

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

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

S9.4.1 INTERFACE ACTION

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

S9.5.1 GENERAL

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

S9.6.1 METHODS OF ANALYSIS

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

S9.7.1.1 MINIMUM DEPTH AND COVER

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

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

S9.7.1.3 SKewed DECKS

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

S9.7.2 EMPIRICAL DESIGN

The Empirical methodology of concrete deck design is prohibited.

S9.7.4 STAY-IN-PLACE FORMWORK

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

S9.7.5.1 GENERAL

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

S10.4.2 SUBSURFACE EXPLORATION


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

S10.5.5.2.3 DRIVEN PILES

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

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

Refer to BDM Section 202.2.3.2 for more information.

S10.5.5.2.4 DRILLED SHAFTS

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

S10.5.5.3.2 SCOUR

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

S10.6.1.2 BEARING DEPTH

Minimum footing depth guidelines are provided in BDM Section 303.4.1.

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

Refer to BDM Section 303.4.2.2 for maximum pile spacings for typical ODOT substructure
types.

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

**S10.7.1.3 PILES THROUGH EMBANKMENT FILL**

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

**S10.7.1.4 BATTER PILES**

Refer to BDM Section 303.4.2.4 for more information.

**S10.7.2.4 HORIZONTAL PILE FOUNDATION MOVEMENT**

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

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

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

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

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

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

**S10.7.3.6 SCOUR**

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

**S10.7.3.7 DOWNDRAG**

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

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

**S10.7.9 TEST PILES**

Refer to BDM Sections 303.4.2.5 and 303.4.2.6 for specific ODOT pile testing requirements.

Refer to BDM Section 303.4.2.1 to determine pile order lengths.

**S10.8.1 GENERAL**

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

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

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

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

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

**S10.8.1.4 BATTERED SHAFTS**

The use of battered shafts is prohibited.
1011 LRFD SECTION 11 – ABUTMENTS, PIERS, AND WALLS

S11.5.1 GENERAL

The design life for MSE walls shall be 100 years.

S11.6.1.3 INTEGRAL ABUTMENTS

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

S11.6.1.6 EXPANSION AND CONTRACTION JOINTS

BDM Section 303.2.5 does not require contraction joints in abutments.

S11.7.2.2 COLLISION WALLS

Refer to BDM Section 209.8 for more information.

S11.10 MECHANICALLY STABILIZED EARTH WALLS

Refer to BDM Sections 204.6 and 303.5 for more information.

S11.10.2.1 MINIMUM LENGTH OF SOIL REINFORCEMENT

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

S11.10.2.2 MINIMUM FRONT FACE EMBEDMENT

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

S11.10.8 DRAINAGE

Impervious membranes shall not be used.
11.10.11 MSE ABUTMENTS

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

The bearing pressure at the service limit state for a spread footing abutment placed on an MSE wall embankment shall be less than or equal to 4 ksf [190 kPa].

Refer to BDM Sections 204.4 and 204.6.2.1 for additional information.
No ODOT comments have been made to this article. Refer to the ODOT Location and Design Manual, Volume Two.
1013 LRFD SECTION 13 – RAILINGS

S13.4 GENERAL

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

S13.7.2 TEST LEVEL SELECTION CRITERIA

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

S13.8.1 GEOMETRY

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

SA13.4.1 DESIGN CASES

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

SA13.4.2 DECKS SUPPORTING CONCRETE PARAPET RAILINGS

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

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

Where:

\[ R = \text{Barrier resistance to lateral impact force (kips)} \]

\[ H = \text{Height of the barrier (ft.)} \]

\[ L_c = \text{Critical length of yield line failure pattern (ft.)} \]

\[ X = \text{Lateral distance from toe of barrier to deck design section (ft.)} \]
Assume the barrier resistance (R) to be the lesser of:

A. 1.33 times the transverse force ($F_t$) specified in Table A13.2-1, or

B. The calculated parapet resistance specified in Article A13.3.1

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

<table>
<thead>
<tr>
<th>Barrier System</th>
<th>$L_C$ (ft.)</th>
<th>$R$ (kip)</th>
<th>$H$ (ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBR-1-99</td>
<td>12.7</td>
<td>165.0</td>
<td>3.5</td>
</tr>
<tr>
<td>42” BR-1</td>
<td>12.4</td>
<td>165.0</td>
<td>3.5</td>
</tr>
<tr>
<td>36” BR-1</td>
<td>8.8</td>
<td>72.0</td>
<td>3.0</td>
</tr>
<tr>
<td>BR-2-98</td>
<td>10.0</td>
<td>72.0</td>
<td>2.5 $^{(1)}$</td>
</tr>
</tbody>
</table>

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

SA13.4.3.1 OVERHANG DESIGN

Refer to BDM Figures 302.2.2-1 and 302.2.2-3 for the deck overhang reinforcement requirements for the TST-1-99 railing system. Alternative railing systems shall be considered for projects that do not meet the design assumptions for BDM Figure 302.2.2-1.
LRFD SECTION 14 – JOINTS AND BEARINGS

S14.5.6 CONSIDERATIONS FOR SPECIFIC JOINT TYPES

Refer to BDM Section 306 for standard ODOT joint types and applications.

S14.6 REQUIREMENTS FOR BEARINGS

Refer to BDM Section 307 for preferred ODOT bearing types and applications.

S14.6.3.2 MOMENT

For elastomeric bearings utilizing 4 anchor bolts connecting load plates to the bearing seat, $M_u$, shall be taken as specified by Eq. 14.6.3.2-3.

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

S14.7.5 STEEL-REINFORCED ELASTOMERIC BEARINGS – METHOD B

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

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

S14.7.5.3.2 COMPRESSIVE STRESS

The effect of impact shall be ignored.

S14.7.5.3.4 SHEAR DEFORMATION

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

S14.9 CORROSION PROTECTION

Refer to C&MS 516.03 for standard bearing corrosion protection requirements.
APPENDIX - MISC. BRIDGE INFORMATION

APPENDIX PURPOSE

The Bridge Design Manual’s appendix serves three purposes.

A. One is to serve as a repository for special plan notes that are infrequently used or subject to frequent revision. These notes are generally large and detailed documents. When a bridge design requires the use of appendix notes one of two methods should be used to incorporate the notes into the project plans. One, the designer transfers the notes to plan sheets for inclusion into the bridge plans. The second method is to treat the note as un-numbered proposal note. This method requires the designer to include with the bid item(s) a reference to the proposal and supply electronic versions, or typed hard copies, of the note with the final plan submission. If the proposal note method is used, the designer shall ensure the notes are presentable, that it is clear what notes are to be used as proposal notes, and that the agency receiving the completed plans understands the notes must be included in the project’s actual proposal. The choice of methods is the option of the owner.

B. The second purpose is to serve as a historical archive for old plan notes, old general notes or old proposal notes which are no longer active or not recommended for use.

C. The third purpose is to serve a repository for special bridge policy criteria and other items of similar concept.
ARN-1  RETIRED NOTE 702.12-2

If the differential dead load deflection at each end of the crossframes is greater than ½" [13 mm], provide the following note. (Note - if part of a structure’s crossframes have a differential deflection of greater than ½" [13 mm] and part of the structure does not, use the following ERECTION BOLT note.)

[702.12-2] ERECTION BOLTS AND CROSS FRAME FIELD WELDING: The hole diameter in the girder stiffeners shall be 3/16" [4 mm] larger than the diameter of the erection bolts. The cross frame members shall have slotted holes, 3/4" [19 mm] longer than the bolt diameter and 1/16" [2 mm] wider than the erection bolt diameter. The slot shall be parallel to the longitudinal dimension of the cross frame member. 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.

Do not weld the cross frame members to the stiffeners until the concrete deck has been placed.

HISTORY: Note [702.12-2] was retired in order to reduce the potential for unanticipated girder deflection during deck placement. All crossframes and lateral bracing shall be permanently fastened before deck placement begins.