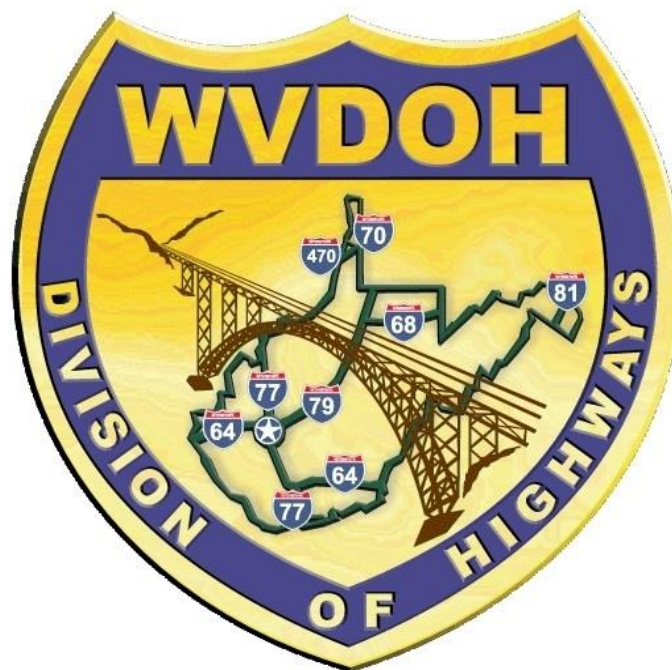


STRUCTURE DIRECTIVES



WEST VIRGINIA DEPARTMENT OF
TRANSPORTATION
DIVISION OF HIGHWAYS

With updates through August 2025

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**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1010
PROJECT DESIGN CRITERIA**

*January 25, 2023
Supersedes May 4, 2022*

All designs shall be in accordance with the latest edition of the *AASHTO LRFD Bridge Design Specifications* (Governing Specifications), including all interim specifications and the *West Virginia Division of Highways Standard Specifications, Roads and Bridges* (Standard Specifications) including the latest supplemental specifications.

See Design Directive (DD) 600 for applicable roadway design criteria associated with bridge planning. Reference is also made to DD 202, which contains the Bridge Submission Checklists for each phase of the project.

1010.1-TYPICAL DECK TRANSVERSE SECTION

The typical deck transverse section shall be determined by the Project Manager. Generally, the bridge width shall not be less than that of the approach roadway section and barriers shall be provided in accordance with the Governing Specifications.

1010.2-LINE AND GRADE GEOMETRICS

The line and grade shall be approved by the Project Manager. The Bridge Designer shall coordinate with the Project Manager to establish line and grade that can accommodate the proposed structural system. See SD 1040 for more information.

1010.3-EXISTING PROJECT RELATED INFORMATION

Early in the project, the Bridge Designer should gather as much existing information about the project as possible. This information could prove to be extremely useful during the planning phase of the project. Available information could consist of inspection reports, bridge replacement studies, as-built plans on the existing bridge and roadway, and photographs, among other items.

1010.4-HIGHWAY DRAINAGE, HYDROLOGY, HYDRAULICS AND SCOUR ANALYSIS

The WVDOH has developed a comprehensive Drainage Manual that shall be utilized in establishing design frequencies for Highway Drainage and Hydrology and Hydraulics on new and replacement structures. See also DD 501 and Governing Specifications Section 2.6. A scour analysis shall be performed on all waterway or stream/river crossings.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1020
BRIDGE LAYOUT CRITERIA**

*May 4, 2022
First Edition*

1020.1-GEOMETRIC GUIDELINES

The following are guidelines in the geometric layout of new or replacement structures:

- A. The desirable berm width in front of an abutment shall be as follows:
 - 1. A minimum berm width of three (3) feet shall be used.
 - 2. A berm width of five (5) feet is preferred when scour countermeasures are not used or when the berm is inundated at the elevation corresponding to the design storm.
 - 3. When very steep terrain is encountered, a berm width of ten (10) feet is desirable to facilitate safe construction practices.
- B. The berm shall be at an elevation below the bridge seat that will allow access to the bridge seat for future maintenance. See Figure 2020.A.
 - 1. A minimum 1.5 feet clearance between the berm and superstructure is required except that a minimum of three (3.0) feet clearance between the bottom of the deck slab or the bottom of adjacent box beams and berm is required, whichever is greater. However, if the berm width is greater than ten (10) feet a minimum three (3) feet clearance between the berm and superstructure should be used to provide clearance for ventilation and access.
 - 2. Where conditions warrant (e.g., steep terrain or where additional construction clearance is required) a three (3) feet minimum clearance is preferred.
- C. The maximum desirable skew is 30°, radial to centerline; however, elimination of skew is preferable.
- D. The maximum skew for the ends of box beams is 30°. When the bridge is skewed greater than 30°, additional bridge seat width may be required along with a stepped backwall to compensate for the difference in skew angles.
- E. Substructure units that are either parallel to one another or radial to the roadway curvature are desirable. The number of substructure units is determined by cost comparisons of various span arrangements and the topography of the site.
- F. All horizontal and vertical clearances for roadways, railroads, navigable waterways, or any adjacent features, that require a clear zone, shall be maintained. If they cannot be maintained, appropriate measures shall be taken to protect the public and the structure.
- G. The Bridge Designer shall consider the location of environmental features during the bridge layout phase.
- H. The maximum side slope of embankments is generally 2:1. Flatter slopes may be warranted by the existing topography, aesthetics, or slope stability concerns. However, steeper slopes up to 1 ½:1 may be utilized if soil/rock conditions permit

and a geotechnical stability analysis is performed resulting in a minimum factor of safety of 1.5 under the substructure.

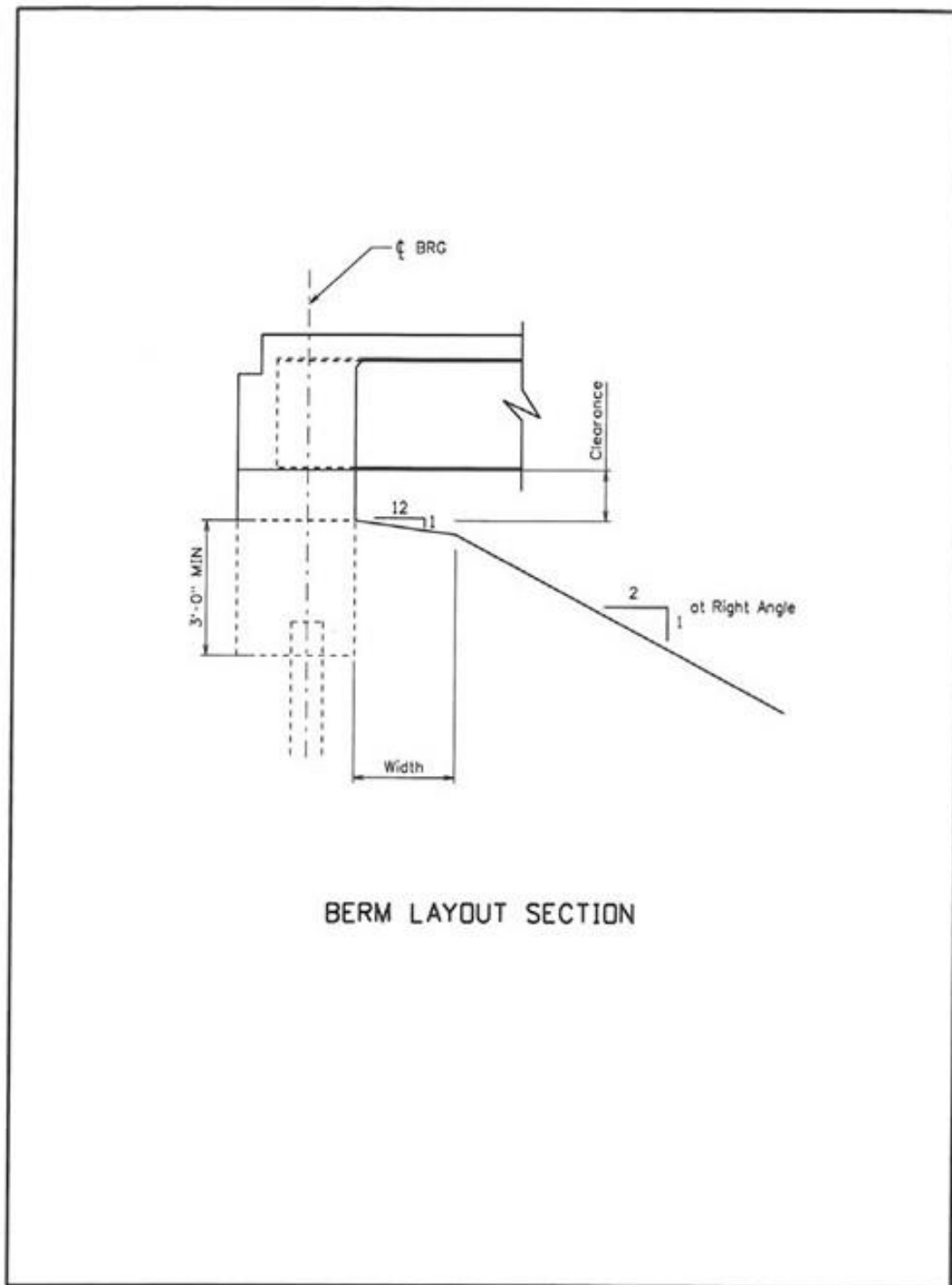


Figure 1020.A

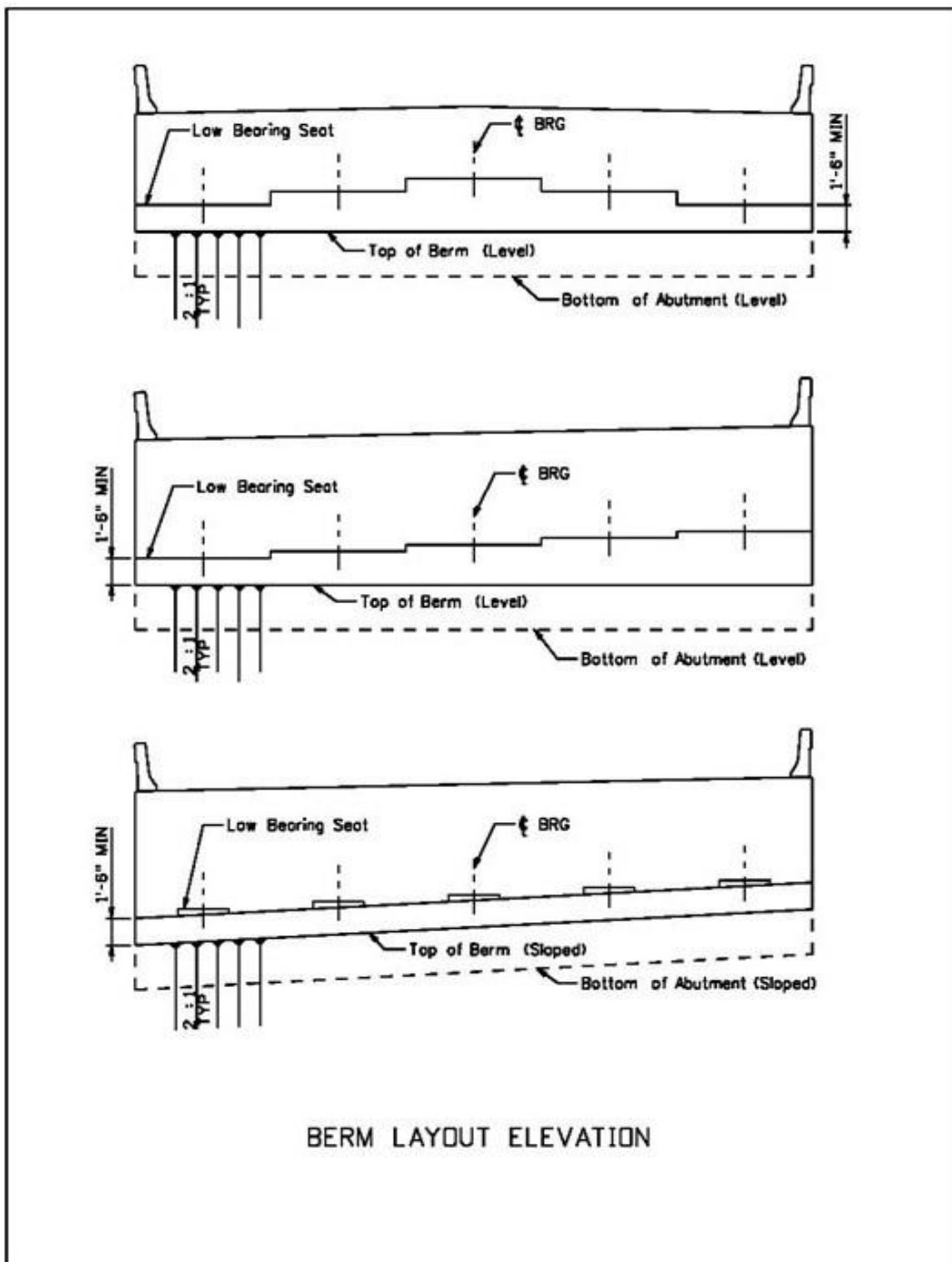


Figure 1020.B

1020.2-BRIDGE LENGTH

The length of the bridge is determined by the attributes of the features that they cross, such as streams, highways, railroads, and cultural and natural resources.

1020.2.1-Stream Crossings: Stream and floodplain crossings shall be designed to not make flooding or stream instability more severe. Refer to the WVDOH Drainage Manual for further guidance.

The geometric design of the bridge and approach roadways may be an iterative process requiring the cooperation of the structures, roadway, hydraulic and geotechnical engineers.

The Designer should avoid a span arrangement that places a pier in or near the center of the stream. It is preferable for pier columns to be located outside the normal flow.

1020.2.2-Highway Crossings: Bridge layouts for highway crossings are usually controlled by the cross section of the roadway. Minimum vertical under clearances, horizontal safety clearances and adequate sight distances will frequently control not only the overall length of the bridge, but the span arrangement as well.

Relatively extreme gradients at either roadway grade require careful consideration of the vertical clearances. The point of minimum under clearance can be beneath any of the superstructure members at any point in the traveled way below. The superelevation rates for both alignments must be evaluated throughout the layout process. The Designer should consider the effects of future widening, final grade, live load deflection, and probability of vehicle strikes to establish the minimum vertical clearance.

When possible, obstructions (abutments, piers, etc.) should be placed outside of the clear zone. If an obstruction is within the clear zone, appropriate safety measures shall be incorporated, such as (but not limited to), guardrails, crash walls, etc.

Table 1020.C shows horizontal and vertical clearances for highway crossings.

1020.2.3-Railroad Crossings: The two principal railroads currently operating in West Virginia are the Norfolk Southern Corporation (NS) and CSX Transportation, Inc. The proposed bridge length is determined from the embankment slopes and berm requirements similar to those for highway crossings. See SD 1100 for clearance and additional railroad requirements.

Horizontal and Vertical Clearances for Highway Crossings		
Classification*	Horizontal Clearance to Obstructions	Minimum Vertical Clearance
Local Roads	10 Feet from edge of traveled way.	14.5 Feet over the entire roadway. This value includes a 6 Inch future resurfacing allowance for new structures. **
Rural Collectors	Design speeds of 40 MPH and below - 10 Feet from edge of pavement. Design speeds of 50 MPH and above - see the current edition of the AASHTO Roadside Design Guide.	14.5 Feet over the entire roadway. This value includes a 6 Inch future resurfacing allowance for new structures. **
Two-Lane Arterial	See the current edition of the AASHTO Roadside Design Guide	16.5 Feet over the entire roadway and usable shoulder. This value includes a 6 Inch future resurfacing allowance for new structures.
Divided Arterial	See the current edition of the AASHTO Roadside Design Guide.	16.5 Feet over the entire roadway and usable shoulder. This value includes a 6 Inch future resurfacing allowance for new structures.
Freeway	See the current edition of the AASHTO Roadside Design Guide.	16.5 Feet over the entire roadway and usable shoulder. This value includes a 6 Inch future resurfacing allowance for new structures. A minimum of 17.5 Feet should be provided to pedestrian overpasses, sign trusses, and from the bridge deck to cross bracing on through trusses.

* The AASHTO functional classification system is to be used as a design type of highway for design purposes.

** Both Local Roads and Rural Collectors shall have a minimum vertical clearance of 16.5 Feet when passing under an Interstate Route adjacent to the Interstate Interchange.

Table 1020.C

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1030
GEOTECHNICAL INVESTIGATIONS**

*May 4, 2022
First Edition*

The purpose of this Directive is to provide Design Engineers a guide to the proper procedures in the performance of geotechnical investigations. Specifically, this Directive is intended to define the procedures that may be involved in performing a subsurface investigation and the various geotechnical aspects of the design and construction of bridges and approach embankments. For the purpose of preliminary foundation design, existing geotechnical data or presumptive values found in the Governing Specifications may be used at the service limit state. All new or modified substructures shall have borings drilled and the foundation soils and rock shall be evaluated. All foundations, including pile foundations, must be designed in accordance with the Governing Specifications.

Each project presents unique considerations and requires engineering judgment based on a thorough knowledge of the individual situation. This Directive is not intended to serve as the geotechnical scope of services for individual projects. The scope of services dictates the specific practices, which are to be used on a particular project. Additionally, the scope defines the required interaction between the Design Engineer and those performing the geotechnical work. For In-House designed bridges, and for District designed bridges, if needed, the bridge engineer shall determine the scope of the investigation with the Geotechnical Section of the Technical Support Division.

Details of coring requirements shall be contained in the Core Boring Contract Documents in the Span Arrangement Report. Core Boring Contract Documents are available from the Engineering Division and the WVDOH Web Site. Determination of soil and rock properties shall be in accordance with the Governing Specifications.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1031
STRUCTURE BORING REQUIREMENTS**

*May 4, 2022
First Edition*

1031.1-STRUCTURE BORING REQUIREMENTS

The purpose of structure borings is to provide sufficient information about the subsurface materials to permit proper design and construction of the structure foundations. All structure borings shall include Standard Penetration Testing (SPT) at five (5.0) feet intervals unless other sampling methods and/or in-situ testing are being performed. It is the Engineer's responsibility to assure that appropriate explorations are carried out for each specific project.

A Geotechnical Inspector shall, as a minimum, witness at least one boring drilled to completion on each bridge project. Color photographs are to be taken for each core box showing the boring number, depths, recovery, and Rock Quality Designation (RQD). Refer to DD 409 for the specific duties, qualifications, and definitions concerning Geotechnical Inspectors.

1031.1.1-Bridge Substructures: In most cases, two borings shall be obtained at each substructure unit. However, one boring may be adequate for smaller foundations, with the approval of the Bridge Project Manager. The hole pattern should be staggered so that borings occur at the opposite ends of adjacent piers or abutments. Pier foundations or abutments may require at least two borings, preferably at the extremities of the proposed substructure. For structure widening, the total number of borings may be reduced depending on the information available for the existing structure. Additional borings to define general site conditions may be needed. General site conditions will include construction and post construction effects on slope stability of stream banks, adjacent natural hillsides, adjacent cut slopes, approach embankments, erosion and scour potential, settlement of approach fill, mine voids and any other bridge related considerations requiring subsurface information. Unanticipated findings may require supplemental borings.

If pier locations are unknown, their approximate locations may be deduced based on experience and a preliminary design concept for the structure. Generally, place borings at no more than one hundred (100) feet intervals along the alignment if substructure locations cannot be deduced. Additionally, for projects with a pier in water, at least one boring should be located in the water when practical, depending on the width of the crossing. All bore holes shall be backfill in accordance with Section 19 of 47 CSR 60 Legislative Rules.

Continue the borings until all unsuitable foundation materials have been penetrated. For pile foundations, core a minimum of ten (10.0) feet of rock. For spread foundations, drill two times the estimated breadth of the foundation or a minimum of ten (10.0) feet into rock at the discretion of the Geotechnical Engineer. For rock socketed drilled shafts, core a minimum of three times the estimated shaft diameter into rock. If shaft locations are known, drill one boring at each shaft location. Coring beyond these limits for at least one boring shall be considered in locations of karst topography, coal seams, or other geologic features that may result in weak founding system. Scour, soft embankment foundation soils, and lateral squeeze must be taken into account when planning borings.

When using the Standard Penetration Test, split-spoon samples shall be obtained per AASHTO T206 Specifications.

When cohesive soils are encountered, undisturbed samples shall be obtained when N-Values of 4 or less are obtained. The N-Value is the sum of the last two consecutive blow counts when performing Standard Penetration Testing. Undisturbed samples shall be obtained from more than one boring where possible.

When rock is encountered, successive core runs shall be made with the objective of obtaining the best possible core recovery. Coring shall be initiated at N-Values of 50 blows per 6 inches but no deeper than 50 blows / 3 inches.

Corrosion tests are required on all new bridge projects where acidic soil or acid mine drainage is suspected.

When drilling for a bridge over a stream, bag samples of streambed materials shall be obtained for determination of the grain size distribution needed for scour analysis. See DD 409.

For projects where the potential for large ship impacts exists, or other critical lateral loads exists, the Geological Strength Index, per the Governing Specifications, is to be determined for rock mass deformation. A minimum of three unconfined compressive strength tests or three-point load testing groups shall be attempted per major rock type encountered.

The type of foundation selected for a substructure unit shall be based on the findings from the core borings obtained.

1031.1.2-Approach Embankments: At least one boring shall be taken at the point of highest fill; usually the borings taken for the bridge abutment will satisfy this purpose. If settlement or stability problems are anticipated, as may occur due to the height of the proposed embankment and/or the presence of poor foundation soils, additional borings shall be taken in the suspect area. Extend borings below any unsuitable founding material.

Sampling criteria is the same as for bridges; however, undisturbed sampling may be taken where N-Values are greater than 4 in suspect areas.

1031.1.3-Retaining Walls and Miscellaneous Structure: The maximum interval between borings is one hundred (100) feet, as close to the structure's foundation as possible. Borings shall be extended below the bottom of the foundation at least ten (10) feet into competent material and SPT sampling on 2.5 feet intervals shall be performed. This applies to proprietary systems, Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS), as well as precast and cast-in-place wall sections. Sampling and testing criteria are the same as for bridges.

1031.2-Slope Stability: Instability of approach embankments has been a very costly problem. A stability analysis shall be performed on approach embankments. The Geotechnical Engineer can waive the need for stability analysis for approach embankments ten (10) feet or less, above the surrounding ground, that are supported by firm ground. The minimum factor of safety of 1.5 shall be obtained for all failure surfaces that intersect a bridge substructure. Otherwise, the minimum factor of safety of 1.3 shall be used for embankments. Should the above factors of safety not be attainable by flattening the slope, then the presence of the piles may be used. Some concerns that shall be addressed during this geotechnical analysis are:

- A. Excess pore pressure during construction.
- B. Ground water seepage during wet weather on hillsides on which the approach embankments are founded.

- C. Where pre-existing slides in natural hillside slopes have occurred.
- D. Rapid draw down results in a factor of safety less than 1.1.

The hydraulic and scour effects on the stability of slopes adjacent to bridges are also a concern and shall be analyzed at the discretion of the Geotechnical Engineer and found to have a minimum factor of safety of 1.1.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 1033

SCOUR ANALYSIS

May 4, 2022

First Edition

A scour analysis will be performed on all waterway and stream/river crossings. The attached Form DS 34 shall be completed and included in project documents.

Revised May 2022

(1) Bridge Number _____ (2) BARS Number _____
 (3) Name _____
 (4) Route _____ (5) County _____ (6) District _____
 (7) Stream _____
 (8) Location _____ (9) Construction Date _____
 (10) Scour Evaluation Priority _____ (11) ADT _____ in _____

SCREENING

(12) Scour Potential (13) Screening Category
 Yes _____ No _____ (1A) Low Risk _____
 (1B) Scour Susceptible _____
 (1C) Unknown Foundation _____

(14) Summary of Screening Evaluation ^(See Note 1):

(15) Recommendations based on screening ^(See Note 2):

(16) Recommended DS 21 Code, Sheet 8, Item 19, Col. 3 _____

(17) Screening By: _____ Date: _____

Note 1 If structure does not have scour potential and scour potential is checked "No", then it cannot be classified as "Low Risk", "Scour Susceptible" or "Unknown Foundation". If structure is classified as having no scour potential or Low Risk, provide justification for classifying as no scour potential or assigning to Low Risk category. If unknown foundation, provide the action proposed to determine foundation details.

Note 2 If scour evaluation indicates a scour critical situation then a positive recommendation must be made for needed corrective action, as well as proposed method of implementing the corrective action. Scour critical situations must be documented and tracked as a critical deficiency per the requirements of BMD A-63 and BMD A-64. If scour susceptible, provide results of scour depth analysis and resultant evaluation. Provide additional sheets or details as necessary to adequately document the evaluation.

SCOUR EVALUATION

Page 2 of 2

(18) Blank (19) Scour Potential? (20) Scour Category?
Yes _____ No _____ 1A, Low Risk _____
1D, Scour Critical _____

(21) Summary of Scour Evaluation (See Note 2):

(22) Recommendations based on Scour Evaluation (See Note 2):

(23) Recommended code for DS21, Sheet 8, Item 19, Col.32: _____

(24) Evaluation By: _____ (25) Date _____

CORRECTIVE ACTION (26) Corrective Action Complete _____

(27) Recommended Revised Code for DS 21 Code, Sheet 8, Item 19, Col. 3 _____

(28) Reported By: _____ Date: _____

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1040
STRUCTURAL SYSTEM SELECTION**

January 17, 2023

Supersedes May 4, 2022

1040-STRUCTURAL SYSTEM SELECTION

The WVDOH encourages diversity in studying a wide range of bridge systems for each project. However, the number and complexity of the systems studied will vary for each specific site. A bridge structural system consists of a superstructure and substructure.

All feasible superstructure types must be considered in the preliminary phases of the project. Haul lengths and weight limits should be verified by the Designer by contacting suppliers in the area. Prior to the submission of the Span Arrangement, the Designer shall meet with the Bridge Project Manager to discuss the span arrangement alternatives that will be included in the submission. In the case of a bridge design by a consultant, this meeting is referred to as the Pre-Span Arrangement meeting. At this meeting, the Designer and the Bridge Project Manager will make decisions on what superstructure, abutment, pier types and span arrangements should be studied in the span arrangement phase of the project. All structures studied shall accommodate their anticipated movements. In this regard, jointless bridges are to be used whenever possible. However, for very long structures, the Bridge Designer shall minimize the number of intermediate expansion joints.

The substructure consists of abutments, piers and bents founded on various types of foundations.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1041
STEEL SUPERSTRUCTURE TYPES**

March 1, 2023

Supersedes May 4, 2022

Steel superstructures should be considered for any span length. ranging from twenty (20) feet to nine hundred (900) feet or more. Generally, Table 1041.A can be used as a guideline for selecting steel superstructure types.

SPAN LENGTH (Feet)	SUPERSTRUCTURE TYPE
20 to 100	Rolled Beams
60 to 130	Rolled Beams with Cover Plates
80 to 500	Welded Plate Girders
200 to 400	Box Girders
400 to 900	Truss
500+	Cable Stayed
650+	Tied Arch

Table 1041.A

The superstructure should be designed such that the structure has redundant load paths and does not contain Nonredundant Steel Tension Members (NSTM). Some designs, especially truss and tied arch designs, by their very nature, contain NSTM's. As defined by the National Bridge Inspection Standards (NBIS), a Nonredundant Steel Tension Member (NSTM) is "*A primary steel member fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse.*" The Designer is to declare at Span Arrangement and/or TS&L if the structure has NSTM's. Design calculations, welding procedures, and material specifications can be incorporated into the project to make the use of these superstructure types acceptable.

Unpainted weathering steel in bridge construction has been shown to be a cost-effective choice when the site conditions are appropriate for its use. The cost savings associated with the use of weathering steel is realized both in initial construction and in long-term maintenance of the structure. Unpainted weathering steel will be used for construction whenever appropriate. For a more detailed discussion, see SD 2039.

High performance steel should also be considered when determining viable superstructure alternatives. It has been found to not only provide cost savings but also increase the serviceability of a structure. For a more detailed discussion, see the WVDOH's policy on high performance steel, SD 2031.5.

Painted steel may be used where the use of weathering steel is not permitted. These locations include:

- A. Wet environments

- B. Industrial areas where concentrated chemical fumes may drift directly onto the structure
- C. Grade separations resulting in “tunnel-like” conditions
- D. Low level water crossings
- E. Other locations as determined by the Bridge Project Manager

The following section discusses the various types of steel superstructure types and guidelines for when to consider them.

1041.1-ROLLED BEAMS

Rolled beams should be considered for any span lengths up to one hundred (100) feet. With cover plates, the span range of rolled beams can be extended to 130 feet. However, only end bolted cover plates shall be used. See Figure 1041.B. The Designer shall determine the availability of any rolled section considered, including lengths and grade of steel.

The Designer should minimize the number of beam lines. Rolled beam bridges should have a minimum of three stringer lines, however four is desired.

Continuous spans shall be used for multi-span bridges. The ratio of the length of the end spans to the intermediate spans should preferably be 0.75. If the end span to intermediate span ratio is small, anchored end spans shall be used to eliminate any uplift problems at the abutments. Configurations subject to uplift shall be approved by the State Bridge Engineer.

1041.2-PLATE GIRDERS

Plate girders should be considered for any span lengths ranging from eighty (80) feet to 500 feet. The Designer shall carefully evaluate the bridge cross section to ensure appropriate girder spacing. Substantial cost savings may be realized early in the design process. The following shall be considered during the span arrangement study:

- A. Use of wider girder spacing to eliminate girder lines, in some cases, may increase the total weight of the steel. However, the savings realized through fabrication of fewer girders, fewer cross frames and bearings, as well as savings realized through shorter erection time will often offset an increase in raw steel cost. Three girder lines is the minimum, however four is desired.
- B. Consultation with fabricators and erectors is recommended to assess the fabrication and erection costs of the girders.

Generally, continuous spans shall be used for multi-span bridges. The ratio of the length of the end spans to the intermediate spans should preferably be 0.75. If the end span to intermediate span ratio is small, anchored end spans shall be used to eliminate any uplift problems at the abutments. Configurations subject to uplift shall be approved by the State Bridge Engineer. The Bridge Designer should also consider the economics of a system designed span by span (i.e., simply supported for dead load and continuous for live load).

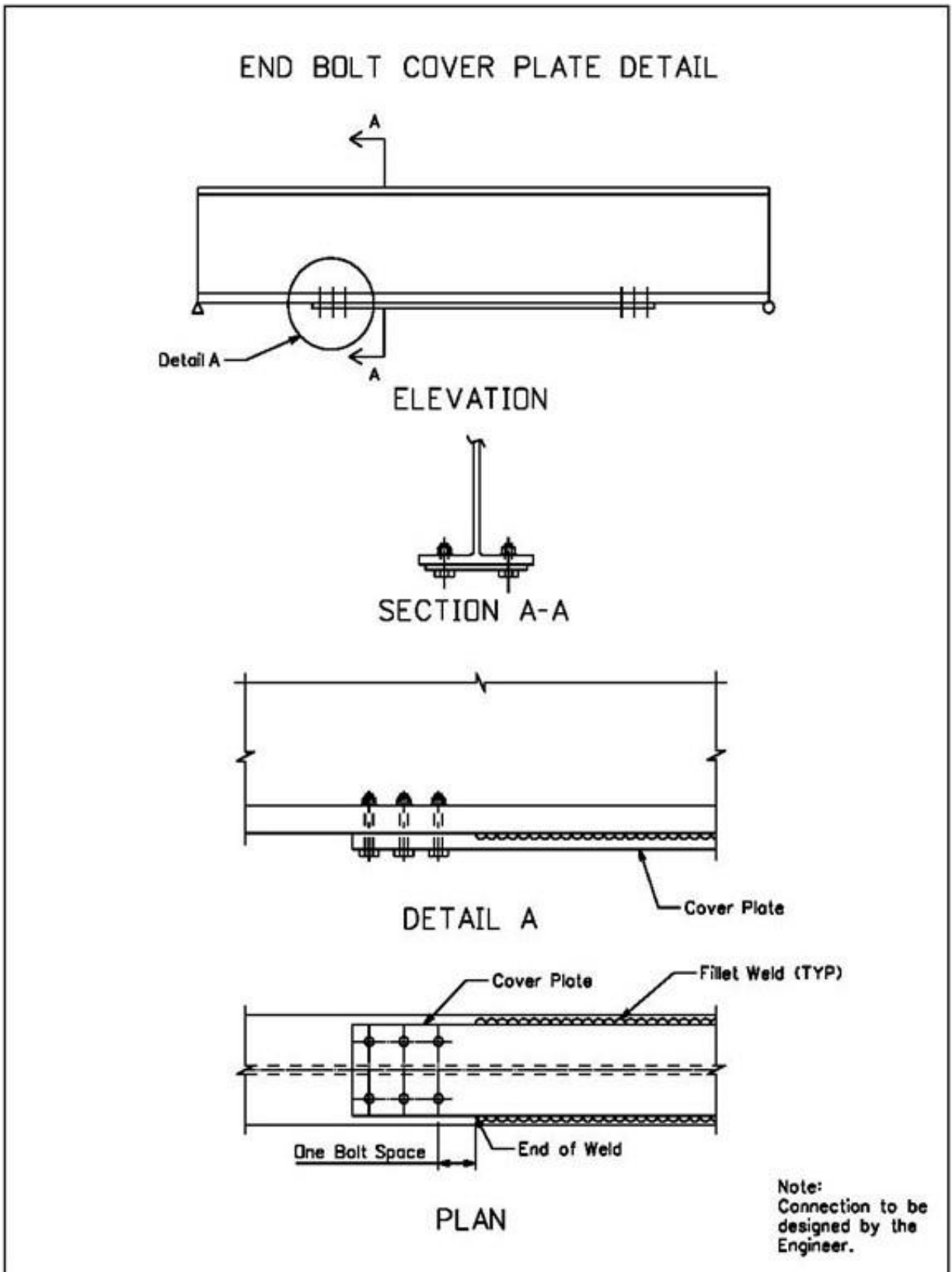


Figure 1041.B

Detailing interior and exterior girders the same are often desirable. Therefore, when designing tangent bridges, consider “balancing” the total factored design stress for interior and exterior girders to yield similar performance. Balancing factored design moments is accomplished by adjustment of girder spacing and overhang dimensions. This type of study may be efficiently performed using simple line girder analyses. Consult with fabricators to ascertain the least cost approach.

Limit girder spacing to fifteen (15) feet for typical girder structures. For girder/sub-stringer framing arrangements, the main girders may be efficiently spaced at twenty (20) feet to 22 feet. Large girder spacings may cause an increase in the structural thickness of the deck slab. Therefore, evaluation of larger girder spacings must be accompanied by an evaluation and cost analysis of the deck slab. Steel fabrication and erection savings may be partially offset by an increase in deck cost.

Optimize the girder weight by investigating various web depths.

The minimum web thickness for plate girders is 7/16 inches. Increment the web thickness by a minimum of 1/16 inch. It is generally more economical to maintain a constant web thickness throughout a project. However, the web thickness may be varied at field splices, or less desirable, at shop splices. The Designer shall consult with a steel fabricator to determine the most economical location of a splice, and whether or not the added cost of additional web thickness will be offset by changing the web thickness.

1041.3-BOX GIRDERS

Steel box girders can be considered as an alternate for steel plate girders for span length ranging from two hundred (200) feet to 400 feet.

A box girder has two or more vertical or inclined webs, a continuous bottom flange plate connecting the webs, and narrow top flange plates on each web. The box girder cross-section having a hollow rectangular or trapezoidal section is a suitable candidate in an urban setting where aesthetics play an important role in bridge type selection. The closed section of a box girder has high torsional resistance, which makes them economical for curved bridges.

1041.4-TRUSSES

Trusses can be used for bridges over navigable river crossings with spans from four hundred (400) feet to 900 feet or where aesthetics play an important part in the bridge type. The main structural elements of a typical bridge truss consist of stringers, floor beams, top chord, bottom chord, vertical and diagonal members of the main longitudinal trusses, lateral bracings, and sway bracings. Chord members carry the bending moment while the diagonals carry the shear. Axial loads are the predominant forces in all truss members.

Based on aesthetics and the object of reducing the total truss weight, it is preferable to use a curved chord truss rather than a truss with parallel chords. Truss bridges can be designed as simple or continuous spans. Simple span trusses for multi span bridges are recommended only when problems due to excessive foundation settlement is anticipated. For a continuous truss bridge with three or more spans, a common method of construction utilizing cantilevered end spans that support the central suspended span can be used.

The stringers can be designed similar to steel rolled beam bridge members. The floor beams are generally plate girders with variable plate sizes. Generally, the truss members are composite box sections made of welded plates and the bracing members are rolled W, T, or channel

shapes. The use of high-performance steel shall be investigated in the span arrangement study for main truss members, stringers, and floor beams.

1041.5-CABLE STAYED

Cable-stayed bridges are competitive for medium and long spans (500 FT to 1500 FT). The superstructure, consisting of a concrete deck on steel girders, is supported at several intermediate points by cables radiating from one or more towers. Generally, a cable stayed bridge system consists of a three-span structure with a long main span and two smaller end spans.

1041.6-TIED ARCH

Tied arch bridges can also be used for medium and long spans (650 FT to 1700 FT). A tied arch may also be used as a center span in conjunction with plate girder approach spans. The high horizontal reactions induced in large span arches are carried by the tie-girder, which is essentially a tension member connecting both ends of the arch itself. The rib of an arch bridge can be either a girder member or a truss.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1042
CONCRETE SUPERSTRUCTURE TYPES**

March 1, 2023

Supersedes May 4, 2022

Concrete superstructure types should be considered for any span length ranging from twenty (20) feet to seven hundred (700) feet or more. Generally, the following table, Table 1042.A can be used as a guide for selecting concrete superstructure types.

SPAN LENGTH (Feet)	SUPERSTRUCTURE TYPE
up to 30	Slab Bridges
20 to 100	Box Beams
35 to 165	I - Girders
165 to 300	Post Tensioned I - Girders (Drop-In)
100 to 180	Segmental Concrete Boxes (Span-By-Span)
150 to 450	Segmental Concrete Boxes (Precast)
450 to 700	Segmental Concrete Boxes (Cast-In-Place)
500+	Cable Stayed

Table 1042.A

The possible exceptions to the use of precast concrete beams are structures with severe horizontal curvature, vertical curvature, limitations on structure depth, skew greater than acceptable limits, and restrictions on transportation.

Concrete compressive strengths for commonly used precast beams shall be no less than 6,000 PSI (5,500 PSI for WVDOH Standard Box Beams) at release (f'_{ci}) with a minimum final compressive strength of 8,000 PSI (f'_c).

High Strength Concrete (HSC) should also be considered when determining possible concrete superstructure alternatives. Precast beams may be designed using high strength concrete with a final compressive strength of up to 10,000 PSI and a release strength of up to 9,000 PSI. HSC allows engineers to design structures with smaller beams when clearance criteria needs to be met, reduce dead loads for more cost efficient substructures, and increase span lengths over conventional concrete. The Designer should consult with the fabricator to determine the most cost-effective solution when HSC is being considered.

1042.1-SLAB BRIDGES

This superstructure type consists of a reinforced concrete slab with the main reinforcing parallel to the direction of traffic. This type of structure may be economical for very short span bridges, generally less than thirty (30) feet in length.

1042.2-BOX BEAMS

For short span bridges of one hundred (100) feet or less, prestressed concrete box beams may be considered an economical solution.

Three basic cross-sectional configurations are commonly used. They are:

- A. Adjacent box beams with or without a hot-laid bituminous concrete (HLBC) wearing surface.
- B. Adjacent box beams with a composite reinforced concrete deck.
- C. Spread box beams with a composite reinforced concrete deck.

NOTE: All bridges, including adjacent box beam bridges, on routes designated as coal haul roads and/or subject to heavily loaded trucks shall have composite reinforced concrete decks unless otherwise approved by the State Bridge Engineer or designee.

Factors involved in the choice of box beam configuration design should include but are not limited to economics, traffic type and volume, time constraints, and method of construction (whether by contract or state construction crews which generally have limited construction capabilities). The Bridge Designer should verify capabilities with the District prior to designing a structure that will be built with state forces.

1042.3-PRESTRESSED CONCRETE BEAMS

Prestressed concrete beams should be considered for bridges with spans from twenty-five (25) feet to 160 feet. The maximum span length is based on the haul capacity and availability for a particular project site and shall be verified with a prestressed concrete beam supplier familiar with the project location. For continuous spans, the bridge system shall be designed simply supported for dead load and continuous for live load and superimposed dead load only. The same prestressed concrete beam type is desired for all spans. The Designer should minimize the number of beam lines. Prestressed concrete beam bridges should have a minimum of three stringer lines.

The design of all structures that utilize prestressed concrete I-beam sections will be accomplished using beam sections available within WVDOH Standard Details Volume 3 unless otherwise approved by the Project Manager.

Prestressed concrete beams shall be spaced to optimize girder size and strand usage. Examples of beam types, spacings and span lengths are shown in Table 1042.B.

Approximate Maximum Span Lengths (Feet)

		Beam Spacing (Feet)				
		14	12	10	8	6
AASHTO Type	I	25	30	35	40	45
	II	40	45	50	55	60
	III	60	65	70	75	85
	IV	75	85	90	95	105
	V	95	100	110	120	125
	VI	105	115	120	130	135
Type IV Modified	60 IN	85	95	100	110	120
	66 IN	95	100	110	120	125
	72 IN	100	110	120	125	135
	78 IN	110	115	125	130	140
	84 IN	115	125	130	135	145

NOTE: These values are approximate and should be used for preliminary design purposes only. These values shall not be used for final design. The designs were based on single span (simply supported) bridges with 32 Inch Type F barriers, no sidewalks and utilizing concrete with a release strength (f'_{ci}) of 6,000 PSI and a final strength (f'_c) of 8,000 PSI.

Table 1042.B

1042.4-POST-TENSIONED I-BEAMS (DROP-IN)

Using post-tensioned drop-in spans can increase span lengths for prestressed concrete beams up to 300 feet. The drop-in segments will be field spliced and beam post-tensioned as specified in the contract plans. At the field splice locations, temporary shoring towers or strongbacks may be required. Horizontal and vertical curvature may be better accommodated with post-tensioned drop-in spans.

1042.5-SEGMENTAL CONCRETE BOXES

Segmental concrete boxes are an economical solution for bridges with span lengths over 300 feet and where repetition of the box fabrication can be achieved. There are three methods of construction for segmental concrete: span-by-span, balanced cantilever, and cast-in-place. Each offers advantages in different situations.

1042.6-CABLE STAYED

Cable-stayed bridges are competitive for medium and long spans (500 feet to 1,500 feet). The superstructure, consisting of a concrete deck on prestressed concrete beams, is supported at several intermediate points by cables radiating from one or more towers.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 1043

ABUTMENT TYPES

March 1, 2023

Supersedes May 4, 2022

Abutments are structures positioned at the beginning and end of a bridge, which support the superstructure and approach roadway and retains the earth embankment. Abutments can be classified into the following five types:

1. Wall Type Abutment.
2. Pedestals.
3. Stub Abutment.
4. Integral Abutment.
5. Semi-Integral Abutment.
6. Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS).

1043.1-WALL ABUTMENT

This type of abutment, also known as a full height abutment, may be used when right-of-way is critical, or the site does not permit a longer bridge with sloping embankments. Span lengths can be reduced using a wall type abutment. The footing may transfer loads by direct bearing (spread footing) or it may be supported on piles or rock socketed drilled shaft.

The maximum exposed face should generally be thirty (30) feet, measured from gutter line to ground line in the profile view. Taller heights may be permitted, with permission of the Bridge Project Manager, when the negative effects of a tall structure on the traveling public are not a governing factor. Otherwise, where walls greater than thirty (30) feet are required, a stepped (terraced) wall configuration shall be used.

1043.2-PEDESTALS

The beam seat is supported on columns/drilled shafts or pedestals resting on individual footings. This configuration is useful for meeting unique construction problems, e.g., widely varying elevations of competent rock.

1043.3-STUB ABUTMENT

Stub abutments are relatively short abutments that resemble wall type abutments. These abutments are generally placed on the approach embankment and are supported on rock, piles or rock socketed drilled shafts.

1043.4-INTEGRAL ABUTMENT

Integral abutments are generally short abutments supported on a single row of piling. These abutments, like stub abutments, are generally placed on approach embankments and are well suited for bridges with limited thermal movements. The ends of the bridge beams are cast directly into the abutments, thereby eliminating the need for bridge deck expansion devices.

This abutment type can be used in combination with Mechanically Stabilized Earth (MSE) walls to provide the benefits of a wall type abutment while satisfying the preference for using jointless bridges.

See SD 2090 for limitations on the use of integral abutments.

1043.5-SEMI-INTEGRAL ABUTMENT

Semi-integral abutments can be either wall or stub type abutments. The difference between a semi-integral and an integral abutment is that for semi-integral abutments, the beams are cast in a closure diaphragm that is structurally independent from the stem. This type also eliminates the need for bridge deck expansion devices.

See SD 2090 for limitations on the use of semi-integral abutments.

1043.6-GEOSYNTHETIC REINFORCED SOIL-INTEGRATED BRIDGE SYSTEM ABUTMENT (GRS-IBS)

GRS-IBS Abutments were initially developed by FHWA and can provide an economic alternative to other abutment types especially where adjacent box beams are used, and scour is not considered to affect the foundations. The GRS-IBS abutment type consists of high-performance woven geotextile and open graded stone such as #8 crushed stone. For low abutment heights, this abutment type can save time since concrete curing time is eliminated. The integrated approaches provide the reinforced backfill required for bridges and can eliminate the need for approach and sleeper slabs on low ADT bridges. Since the bridge is supported on the layers of GRS and no deep foundations are needed, “the bump at the end of the bridge” is eliminated. Standard eight (8) inch split face masonry block should be used as the facing.

It is important to place GRS-IBS abutments adjacent to non-scourable streams (hard bedrock is exposed), or where the existing abutments can provide a scour wall, or where the Reinforced Soil Foundation (RSF) can be placed below the scour depth. All GRS-IBS bridges locations shall be approved by the State Bridge Engineer.

The design of GRS-IBS abutments is empirically based on a service limit bearing resistance of 4,000 PSF provided by the criteria presented in “Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide” (Publication No. FHWAHRT-11-026) is followed.

1043.7-WINGWALLS

Wingwalls are walls on either side of an abutment used to retain the roadway embankment. Wingwalls can be constructed of cast-in-place concrete or MSE walls and shall be designed as retaining walls. They shall be sufficiently sized to prevent the roadway embankment from spilling onto the abutment seats or into the clear area under the bridge.

U-shaped or turned-back wingwalls are commonly used in embankment situations and straight wings are used in cut sections. Flared wingwalls between these extremes can also be appropriate based on-site conditions. The Designer must study the existing and proposed surfaces to determine which type of wingwalls best fits the site. Wingwalls with a tapered bottom surface shall be avoided due to compaction difficulties beneath the wall. The top surface of wingwalls may be tapered to match the finished grade.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 1044

PIER TYPES

March 1, 2023

May 4, 2022

Piers are intermediate supports in a multi-span bridge system. All feasible pier types must be considered in the preliminary phases of the project.

1044.1-CAP-AND-COLUMN TYPE PIERS

Cap-and-column type piers have two or more circular or rectangular columns connected on top with a cap (a reinforced concrete beam that supports the superstructure). Generally, the pier cap ends will be cantilevered. For columns greater than one hundred (100) to 150 feet, the use of a compression strut at mid-height, similar to the pier cap, shall be investigated. Partial height walls may be necessary where vehicle or train collision are required in accordance with AASHTO. The individual columns will be supported on an appropriate foundation.

1044.2-T-TYPE OR HAMMERHEAD OR WALL TYPE PIERS

T-Type or Hammerhead piers have a deep rectangular tapered beam carrying the superstructure supported on a single wide rectangular or oval column in the middle. For wall type piers, the width of the rectangular column will be very close to the length of the pier cap. The single column will be supported on an appropriate foundation. In some situations, the feasibility of using a single large circular column instead of a wide rectangular or oval column should be investigated during the preliminary design phase of the project.

1044.3-POST-TENSIONED CONCRETE/INTEGRAL PIER CAPS

To satisfy the vertical clearance requirement beneath a pier cap, a post-tensioned or integral pier cap shall be investigated.

1044.4-STEEL PIER CAPS AND BENTS

Most steel pier caps and bents are Nonredundant Steel Tension Members (NSTM) and should be avoided. If used, the design shall allow for reasonable access to the interior for future maintenance, inspection, and repair. Steel pier caps and bents shall be designed for redundancy unless otherwise approved by the State Bridge Engineer.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 1045

FOUNDATION TYPES

September 19, 2024

Superseded May 4, 2022

All feasible foundation types must be considered in the preliminary phases of the project. The WVDOH's policy is to find all new bridge foundations on rock. However, bridges may be allowed to be supported on Intermediate Geomaterial (IGM) at the approval of the State Bridge Engineer.

1045.1-SPREAD FOOTING

Spread footings have been found to be economical for depths to twenty (20) feet. Preferably, spread footings should be founded on rock. However, spread footing foundations may be supported on Geosynthetic Reinforced Soil-Integrated Bridge Systems or MSE retaining wall backfill where allowed by the State Bridge Engineer.

In situations where a cofferdam may be required for the construction of a spread footing, the cost of the cofferdam shall be included when comparing foundation options. Spread footing foundations shall be placed below the scour depth. Other concerns to consider include the stability of approach embankments, differential settlement, etc.

1045.2-PILING

Piling must be designed for both axial and lateral loads as appropriate. As a minimum, piling shall be sized using a wave equation program such as GRLWEAP. Loads may include external (non-structure related) as well as structural loads. For example, pile foundations might be used to enhance stability of the approach embankment if the embankment factor of safety is questionable.

Piling to competent rock will normally be designed as end bearing and driven to refusal. Additional loading from negative skin friction (downdrag forces), resulting from embankment settlement, must be added to that from structural loads and any other external loads. Battered piles may be required to help resist lateral loads but shall be avoided wherever possible. Pile tips shall be used for refusal on rock. The cost for pile tips shall be included in the cost estimate for the pile foundation.

With permission of the State Bridge Engineer, friction piles and end bearing piles on non-competent rock strata may be considered when site-specific conditions warrant and when all other concerns (such as settlement or scour) are addressed.

The minimum piling length shall be ten (10) feet. See SD 2120.3 for further discussion.

For integral abutments, single line piling systems shall be used, predrilled fifteen (15.0) feet deep using one (1.0) foot diameter for soil or two (2.0) foot diameter for rock.

Foundations supported on piling should be placed below the scour depth. When the bridge scour computations indicate that the steel piling may be exposed due to scour, then the piling cap placement must be designed in accordance with SD 2120.3.

1045.3-ROCK SOCKETED DRILLED SHAFTS

Rock socketed drilled shafts provide superior scour protection versus traditional steel piling, greater resistance against high lateral and uplift loads, and accommodation of site concerns associated with the pile driving process (vibrations, interference due to battered piles, etc.), and in some cases exclude the need of cofferdams. In addition, rock socketed drilled shafts may eliminate the need of caisson caps, for certain configurations such as single or multiple column piers.

Rock socketed drilled shafts shall be designed using soil-structure interaction software such as LPILE. The rock socket length shall be determined as to the second node that crosses the zero-deflection line in the service limit state. For strong rock both end and side resistance can be added directly. For soft rock, such as claystone and soft siltstone, only end resistance shall be used.

Construction techniques shall be in accordance with the Standard Specifications. These include testing by the Division of: pre-installation core holes, wet or dry hole condition, plumbness, shaft sidewall and bottom cleanliness, and concrete inspection. Results from the testing may require remedial action from the Contractor.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1050
ADDITIONAL DESIGN CONSIDERATIONS**

August 13, 2025

Supersedes September 6, 2023

1050.1-CURVED BRIDGES

Bridge Designers are cautioned to design curved structures for all relevant forces as specified in the Governing Specifications, even if the girders are straight and the deck is curved.

For curved bridges, the use of straight steel girders or prestressed concrete beams satisfying the minimum and maximum deck overhang requirements shall be investigated. Investigation of curved bridges using straight steel girder or prestressed concrete beams may not be warranted when the central angle for any span exceeds 3.5 degrees as defined within AASHTO LRFD Bridge Design Specifications Chapter 4.

Slab bridges, cast-in-place and precast segmental bridges are other options for curved concrete bridges.

1050.2-AESTHETICS

The Bridge Designer shall determine and offer proposals of aesthetic treatment to the Bridge Project Manager in charge of the project.

For additional aesthetic requirements, see the AASHTO LRFD.

1050.3-APPROVAL OF VENDOR SUPPLIED PRODUCTS

All products designed and supplied by the Contractor or their Vendor shall be approved through DD 106, Proprietary Approval Process. These products include, but are not limited to culverts, proprietary retaining walls, bearings, expansion devices, inspection walkways, stay-in-place forms and any prefabricated item used on a structure other than those specifically designed and detailed in the contract plans.

If the product is not specified in the Standard Specifications, the Designer is responsible for writing a Special Provision to describe the product and the criteria it has to meet. This shall include directions to the Contractor as to the necessary information to submit for approval by the Engineer. The following is a partial list of items the Designer is to provide the Vendor: all applicable loads, critical dimensions, design method to be used (LRFD) and any additional information that may be required for the product to be designed and detailed.

Vendor submissions should include items such as design calculations, design method, detailed drawings, construction sequence, induced loads, etc. and shall be stamped by an Engineer registered in West Virginia.

When a proprietary item is proposed, the Designer shall justify the benefits of the product and show that no other alternate exists. Proprietary items may also be warranted when a new technology or product is being evaluated. The use of the proprietary item shall then be submitted for approval to the Bridge Project Manager. The Bridge Project Manager will then submit the request to the Deputy State Highway Engineer, of Development, who will then forward it to the Federal Highway Administration (FHWA) for approval.

1050.4-PROTECTIVE FENCING

It shall be the policy of the WVDOT to evaluate the need for screening on bridges and overpasses when:

- A. A new structure is being designed;
- B. An existing structure is being renovated; and
- C. A pattern of accidents or public complaints indicates that there is a problem with objects or debris being thrown or dropped from an existing overpass or viaduct.

The intent of this section is to assist the designer in identifying those bridges where the probability of occurrence is high or where problems are known to exist and to assure that reasonable protective measures are taken at those locations. It will be the responsibility of the Bridge Designer to document the decision to, or not to install screening, the factors influencing the decision and the reasons for the type of screening chosen. This decision shall be included in the TS&L Report for review by the Bridge Project Manager.

Factors that shall be considered in the decision to install, or not to install, screening shall be as follows:

- A. Presence of a sidewalk;
- B. If no sidewalk, proximity to a school, playground or neighborhood that may generate a significant number of children who may play on or around the structure. In addition to protecting those below, the screening may prevent children from climbing on the railing and falling off the structure;
- C. The presence of a transportation facility such as a roadway, a railroad, or a navigable waterway below the overpass or bridge. In the case of a railroad, or navigable waterway, the frequency and sensitivity of the traffic passing under the structure shall be taken into account;
- D. Requirements of the entity or agency over which the structure passes, such as a railroad, the Corps of Engineers or the National Park Service;
- E. Effect on aesthetics or on maintenance, including inspection, snow removal or ice control or navigation lights, and the maintenance of the screening itself; and
- F. Effects on the safety of vehicles crossing the structure, especially where speeds are high.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1060
MISCELLANEOUS STRUCTURES**

*May 4, 2022
First Edition*

1060.1-TIMBER STRUCTURES

Timber structures should not be considered as a superstructure type unless the project is designated by the WVDOH as such. Timber structures may be considered for pedestrian bridges.

1060.2-PEDESTRIAN STRUCTURES

Pedestrian bridges should undergo the same design evaluations as vehicular structures. All applicable superstructure types, including timber, shall be considered. In addition, pedestrian bridges should consider aesthetics, from both the user's standpoint as well as the view of the structure by motorists. See *AASHTO Guide Specifications for Design of Pedestrian Bridges*.

1060.3-BURIED STRUCTURES

Buried structures may be designed as precast or cast-in-place, or the contract plans may leave the option to the contractor.

All cast-in-place buried structure shall be fully designed and detailed in the contract plans. When precast products are specified, the Designer shall also specify all necessary design criteria including design method. Certified design calculations and drawings shall be submitted to the Engineer of Record for approval for precast products.

The Designer should make every effort to use single cell boxes; single cell openings with clear spans up to sixteen (16) feet are routinely used. Box culverts of three or more cells should be avoided, due to high construction and maintenance costs. However, high fill heights or other restraints can make three or more cell boxes cost effective.

Future maintenance of the boxes must be considered in the proposed layout. Such considerations should include a maintenance road to the inlet and outlet of the box and debris racks.

Buried structures with three (3) feet or less of fill shall be designed with epoxy coated reinforcing steel in the top slab of the structure.

Due to stream mitigation requirements, some box culvert bottom slabs may need to be buried and a natural streambed developed. The Bridge Designer should check to see if this is required on their project.

Culverts shall be designed for discharges as required by the *WVDOH Drainage Manual* and DD 501.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1070
BRIDGE REHABILITATION**

*May 4, 2022
First Edition*

In the course of maintaining the State's highway system to accepted standards, it becomes necessary to rehabilitate or replace structures that have become either functionally or structurally obsolete or physically deteriorated to the point that structural capacity is significantly impaired. If rehabilitation is chosen, the durability of the repaired component shall be at least as durable as the original member. The Designer shall verify that the existing bridge components are not adversely affected by the new "global conditions" created from the rehabilitation. The extent of bridge rehabilitation is based on an assessment of current conditions, estimates of future travel demands, and anticipated capital and maintenance investments that will be realized through a specific year.

The District Bridge Engineer plays a key role in the selection of bridges for replacement or rehabilitation. Cost estimates should be done to help determine if either replacement or rehabilitation of a structure is needed. Inspection reports are a good basis for determining rehabilitation needs. These reports are available from the Operations Division. However, these reports should not be the sole source used by the Designer to develop the contract plans. Generally, a detailed inspection, by the Designer, is required along with material testing. This additional inspection work must be sufficient to detail and quantify the necessary repairs.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 1071

BRIDGE INSPECTION

May 4, 2022

First Edition

Bridges are inspected on an interval based on their condition or type. The current interval for a Routine Inspection required for all structures by the National Bridge Inspection Standards (NBIS) is 24 months. The WVDOT-Division of Highways (WVDOT-DOH) policy requires an In-Depth Routine Inspection at a 72-month interval. Additionally, WVDOT-DOH has a policy that has been approved by the FHWA that allows bridges that are in good condition and meet certain other criteria to have a Routine Inspection interval of 48 months and an In-Depth Routine Inspection interval of 96 months. Bridges that may warrant a specific concern may require a Special Inspection at an interval less than 24 months.

The various inspection types covered in the most current WVDOT-DOH *Bridge Inspection Manual* (WVBRIM).

- A. Inventory Inspection.
- B. In-Depth Routine Inspection.
- C. Routine Inspection.
- D. Special Inspection.
- E. Damage Inspection.
- F. Underwater Inspection.

This WVBRIM is available in the Bridge Evaluation Section of Operations Division and incorporated by reference various documents including the following:

- A. AASHTO Manual for Bridge Evaluation, 3rd edition or current FHWA adopted version.
- B. U. S. Department of Transportation Bridge Inspector's Reference Manual 2002 (2006 and 2012 Revisions)
- C. NBIS.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 1072

BRIDGE WIDENING

May 2022

First Edition

Widening is usually considered for deck bridges supported on steel or concrete beams or girders. The following items should be addressed in a widening project on all components:

Materials used in the construction of the widening shall preferably have the same thermal and elastic properties as the existing.

- A. The widening of the structure should be accomplished in a manner such that the new construction blends with the existing structure.
- B. The main load carrying members should be proportioned to provide similar longitudinal and transverse load distribution characteristics as the existing structure. The final design shall result in a structure with a uniform stiffness over the entire cross section.
- C. The construction sequence and degree of interaction between the widening and the existing structure after completion shall be fully considered in determining the distribution of the dead load for design of the widening and stress checks for the existing structure.
- D. The design of the widening shall conform to the Governing Specifications. This shall include the verification of all existing components using LRFD.
- E. The use of beams that are the same type as the existing beams are preferred.
- F. All dimensions that affect the details that the Designer is preparing shall be confirmed by the Designer or the Contractor. The dimensions that are to be checked by the Contractor must be called out for in the contract plans.
- G. All horizontal and vertical clearances shall be checked. For decks with normal crown or superelevation, the new fascia beam may become the lowest point on the deck, so vertical clearance must be checked.
- H. The widened deck section should be structurally attached to the existing deck and the transfer of moment and shear shall be provided using dowels with sufficient splice laps. A concrete shear key is not necessary, but a roughened construction joint should be used. A closure pour should be generally used when construction staging does not prohibit its use.
- I. A construction sequence detail, including maintenance of traffic details, shall be shown on the preliminary bridge plan submittal for all projects utilizing phased construction. In addition, the final plans shall include a complete outline of the order of construction.
- J. Changes to existing drainage must be investigated.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1073
REHABILITATION TECHNIQUES**

January 25, 2023

Supersedes May 4, 2022

This Directive describes various methods for repairing and rehabilitating bridges. These are in no way meant to limit the Designer to these methods but to give guidance in accepted procedures. All plans developed for rehabilitation shall include appropriate details to comply with AASHTO Standard Specifications for Seismic Design of Highway Bridges. The following shall be considered on all rehabilitation projects:

- A. Structural integrity and general acceptability of design.
- B. Future maintenance considerations.
- C. Hydraulic considerations (waterway opening, backwater effect, etc.).
- D. Geometric safety (roadway width, guardrail, etc.).
- E. Right of way clearance.
- F. DNR and Corps permit clearance.
- G. Erosion Control.
- H. Suitability of the sequence of construction required by the design.

All material used in any rehabilitation or repair project shall be in accordance with the Standard Specifications and supplemented by project specific special provisions, as necessary.

1073.1-STEEL

Repair of steel members may be necessary to correct deficiencies associated with cracking, corrosion, and fatigue. This includes cracking of joints and welded connections, partial length cover plates, and brackets. Nonredundant Steel Tension Members require special assessment because their failure would be expected to result in bridge collapse. All repairs shall consider the dead load that exists in original members and the original members shall not be stressed beyond their original allowable inventory stress level. All steel repairs shall be in accordance with the Steel Structures section of the Governing Specifications. All repairs to welds on steel members shall be in accordance with the AASHTO/AWS D1.5M/D1.5: current version, *Bridge Welding Code*.

1073.1.1-Cracks: One method for preventing crack propagation is by drilling holes at the ends of the crack. Consideration shall also be given to filling the hole with a tightened high strength bolt or crack compression bushing to aid in arresting further propagation. Non-Destructive Testing (NDT) is used to locate and determine the extent of surface cracks. The center of the drilled hole should be positioned so that the end of the crack is located within the hole. If the crack is visible on both sides of the plate, the position of the outside diameter of the hole is at the end of crack that has propagated farthest. NDT is again used to ensure that the crack did not propagate through the drilled hole. The FHWA has published guidelines on this procedure that are available at the WVDOH.

Welding can be used to repair typical cracks in flanges and webs of beams or girders. Welding in connection with crack repair shall be done in accordance with AWS and the Governing Specifications. The weldability of the bridge material must be assessed prior to the repair procedure to insure a successful weld repair. The risks associated with field weld repairs should be thoroughly evaluated before specifying said repairs.

Superficial nicks and gouges should be repaired by grinding rather than by welding repairs.

1073.1.2-Painting: Repair work for corrosion may include painting of the structure. This consists of surface preparation, prime coating, and finish coating and shall be in accordance with the Painting Steel Structures section of the Standard Specifications and SD 1074.

The Designer is responsible for determining the presence or absence of lead-based coatings by requesting that the Division of Highway's Materials Control, Soils and Testing Division conduct a field survey. If a lead-based coating is present, then the project plans shall contain a note as follows: "The contractor's attention is directed to the fact that the existing structure contains lead-based paint coatings".

1073.1.3-Fatigue: In zones of tension stress, when fatigue critical details exist, action should be taken to improve the expected fatigue life of the detail unless a cumulative damage fatigue analysis yields adequate life or the structure does not exhibit fatigue damage. The Designer should not use Category D, E or E' weld details for a repair or a new design. The fatigue life analysis shall be performed in accordance with the current version of the *AASHTO Guide Specifications for Fatigue Evaluation of Existing Steel Bridges*.

In designing a fatigue repair, an examination of the existing connections should be performed. The repair should be one that attempts to reduce the fatigue category of the existing connections. The Designer shall consult the Governing Specifications for common connection details and their fatigue category. Figures 1073.A and 1073.B illustrate two accepted fatigue repairs.

1073.1.4-Section Loss: Cover plates are an effective means for restoring section loss in a member. The member must be analyzed to ensure its original or target capacity can be attained with the addition of cover plates. Details of repairs are largely up to the Designer's creativity. The Designer must consider the fatigue characteristics of the repairs they design. If excessive deterioration exists, then replacement of the member may be required. The Designer must consider "locked-in" forces and differences in supplementary cover plate material properties.

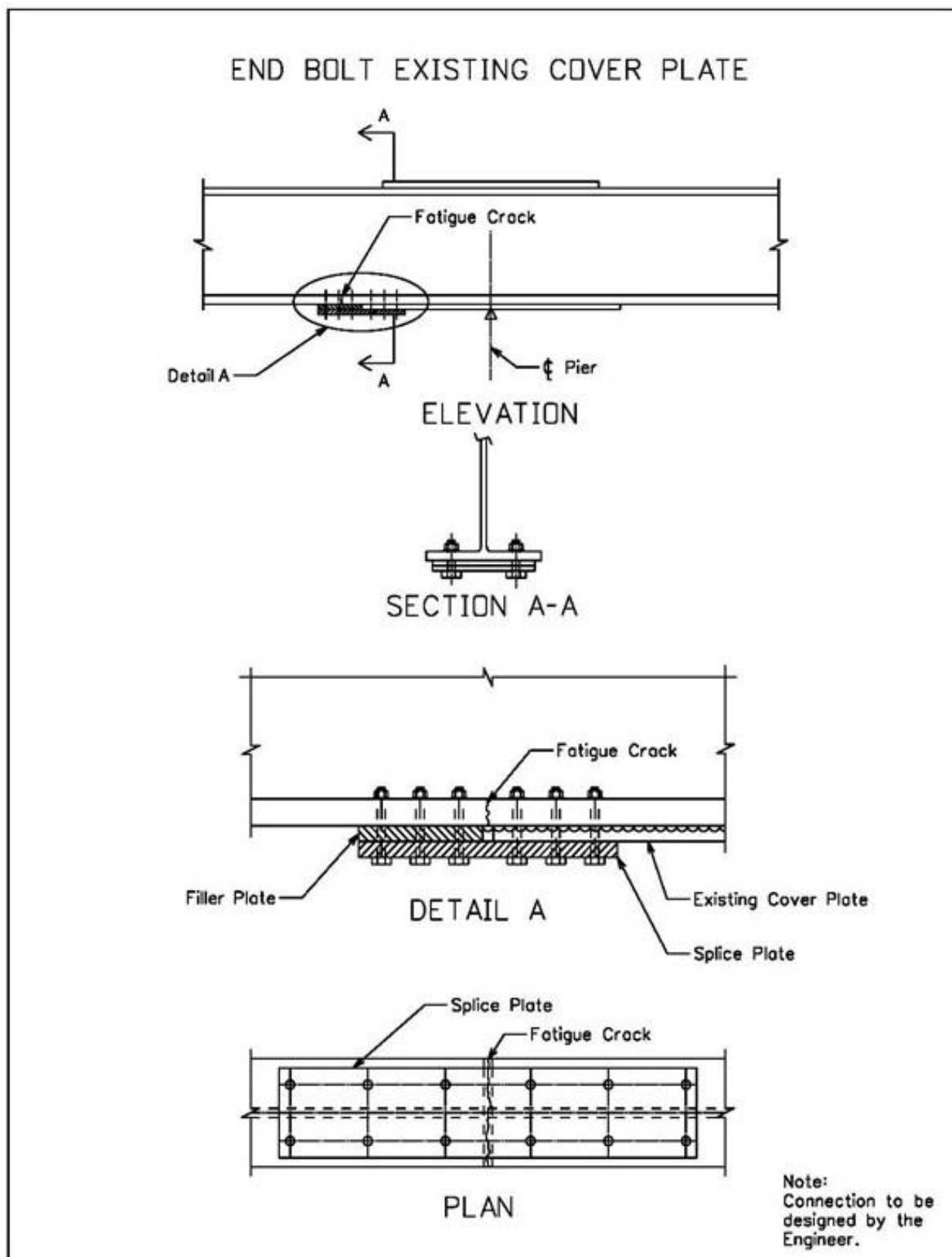
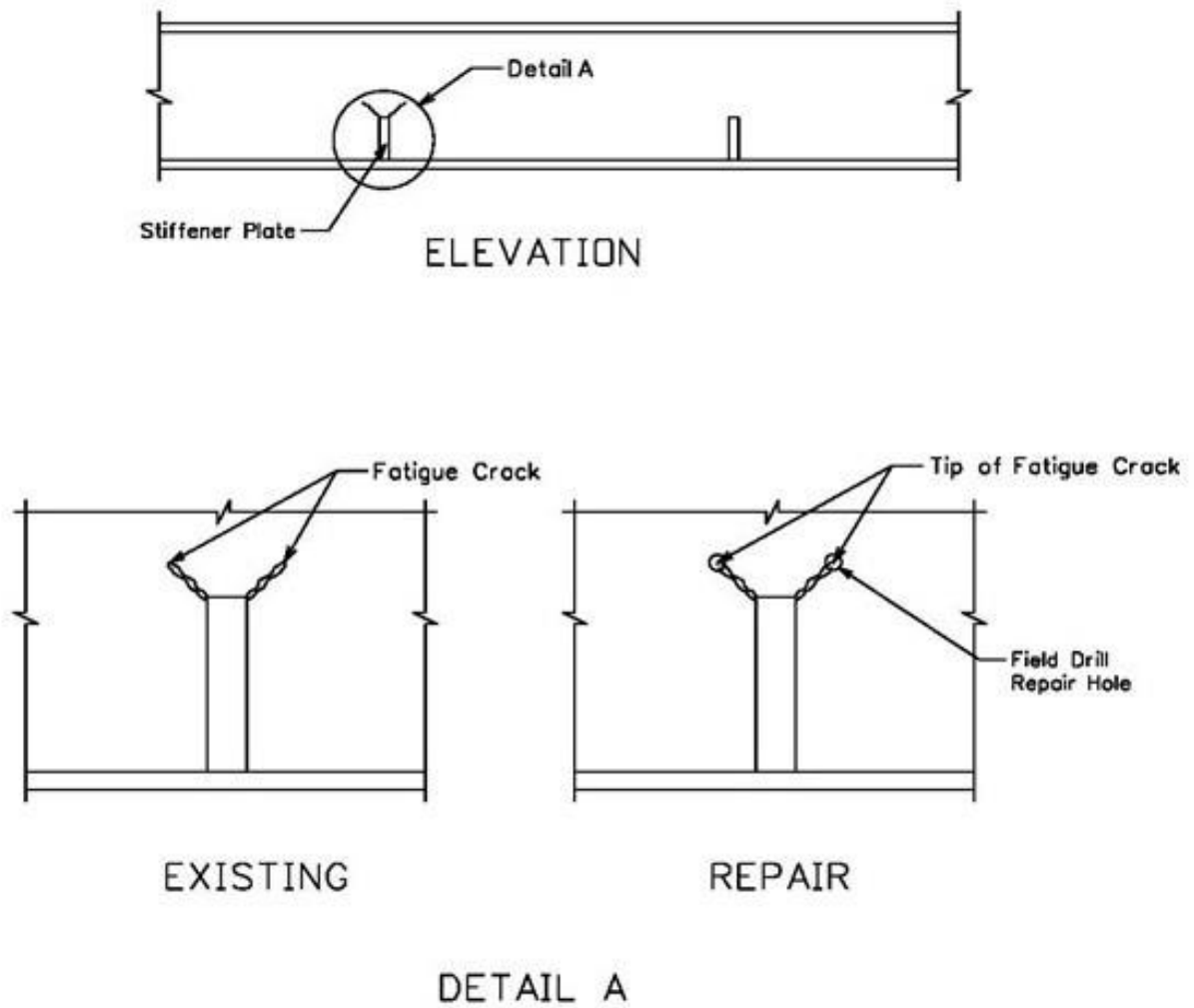


Figure 1073.A

TYPICAL WEB FATIGUE CRACK REPAIR DETAIL



Note:
Hole diameter to be
determined by the
Engineer.

Figure 1073.B

1073.2-CONCRETE

The intent of repairing concrete is to restore the structural integrity and function of the concrete. Typically, concrete repairs consist of removing deteriorated concrete and replacing it with cement mortar or another suitable material. Restoring proper cover, where existing cover is inadequate, is important in selecting repair materials. The following factors should be considered:

- A. Structural compatibility of the material and its expected performance with the original construction.
- B. Availability, cost, and anticipated life.
- C. Ease of construction and availability of qualified contractors in the area.

Initially, all exterior concrete surfaces should be thoroughly examined by means of soundings with hammers to determine loose or defective areas that may exist beyond the visual assessment of deficiencies and deterioration. Defective areas should be removed to a depth necessary to eliminate any loose and disintegrated materials. All exposed reinforcement should be cleaned, care being taken to not damage the steel. Loose reinforcement should be tied back into place and, where necessary, concrete adjacent to loose bars shall be carefully undercut to a depth that permits a minimum of one inch of new concrete around the reinforcement bars. Sections with deteriorated bars should be re-evaluated and capacities restored, when necessary. The area of concrete removal should be large enough to allow for adequate bar splicing. The exposed area of concrete should be cleaned. Where concrete deterioration requires substantial removal, consideration may be given to the replacement of the entire section in the deteriorated area.

A good bond between the repair material and existing concrete surfaces is essential in concrete repair. An epoxy-bonding coat applied just before the repair material can help to obtain a good bond. Dowel bars may be required in a section that is subjected to forces where the bond between the new concrete and the old section is not considered sufficient to transfer the loads. Dowels may consist of expansion anchors, grouted anchors, power-activated anchors, and epoxy and polymer grouts and resins. External or internal vibrators may be used for compaction. Proper curing is essential to ensure that excessive shrinkage will not occur.

Shotcrete can be used as a means for rebuilding an area where deteriorated concrete has been removed. Shotcrete applications are justified where conventional methods of forming and placing concrete are less suited to the damaged areas, such as vertical and overhead surfaces. Shotcrete application shall be in accordance with the Pneumatically Applied Mortar, Section 623, of the Specifications.

Cracks in concrete must be repaired to stop intrusion of water or chemicals into the concrete and restore the uniform appearance of the concrete surface. Epoxy grouts are typically used for crack repair. This involves injection of low viscosity material under pressure with the intent to seal the crack and restore structural continuity. Where active cracking conditions exists, it must be dealt with by addressing the cause directly.

Grouting can also be used for the repair of concrete substructures submerged in water. This type of repair may necessitate the use of pile jackets or formwork.

All concrete repairs shall be in accordance with the Governing Specifications.

1073.2.1-Concrete Decks: Most repairs needed in bridge decks are associated with increased traffic, heavier vehicles, deicing chemicals, and geometric deficiencies as a result of the initial construction. Common problems are cracking, spalling, chloride contamination, potholing, and delaminating. Cracking in the deck can be repaired as described in the previous section. Minor

spalling, potholes, etc. may be temporarily repaired with patches. Patches cannot be considered a permanent solution. Eventually, a bridge deck becomes a composition of patches with no end to the repair process. As the patching process is repeated to repair more damaged areas, an overlay will be needed to serve as a wearing surface and a moisture barrier.

When repairs on a concrete slab become too costly, partial, or complete replacement of the deck is needed. See SD 2020 for design details for concrete decks.

See SD 3000 for the Deck Removal-Grinding note to be included on the General Notes sheet for all projects requiring partial or complete deck removal on existing bridges.

1073.2.2-Deck Overlays: Conventional concrete removal, such as rotomilling and the use of pavement breakers shall not be utilized for slabs less than seven (7.0) inches thick. For slabs, 6.5 inches to 7.0 inches thick, special consideration must be given to methods of removal of the deteriorated concrete, such as hydro-demolition, so that damage of the remaining slab is minimized. A specialized concrete overlay will not be considered an acceptable method for deck retrofit for any bridge deck where the original slab thickness is less than 6.5 inches.

1073.3-ADDITIONAL REHABILITATION ISSUES

In past years, it was general practice in the steel bridge building industry to attach miscellaneous brackets, supports and details to the top flanges of stringers and floor beams by field welding. This work was not detailed on contract plans or steel fabrication drawings and was done to facilitate temporary support of various construction aids. The welding may have been performed under limited or no supervision, without proper preheat of the base material using electrodes of unknown quality and condition. Most of these welds were not removed prior to placing the deck.

The industry has since learned that these unauthorized welds are a potential source of fatigue cracking in the negative moment regions of the member flanges and should be removed during subsequent deck replacement. After removal, nondestructive testing is also appropriate to assure integrity of the member flange.

1073.4-TIMBER

Timber members may experience deterioration from decay, insect attacks, and mechanical damage.

Surface treatments or coatings are applied to existing bridge members to protect the wood. This is most effective when applied before decay begins and is used to treat splits, delaminations, mechanical damage or areas that were field fabricated during construction. Shallow penetration limits its effectiveness against established internal decay. Creosote is the preferred treatment. The wood surface should be thoroughly saturated with the treatment so that all cracks and crevices are coated. However, care must be exercised to prevent excessive amounts from spilling or running off the surface and contaminating water or soil. The effectiveness of surface treatments depends on the thoroughness of application, wood species, size, and moisture content at the time of treatment.

Mechanical repair methods use steel fasteners and additional wood or steel components to strengthen or reinforce members. These methods include splicing and stress laminating. Splicing is used to restore load transfer at a break, split, or other defect. Stress laminating may be used for the repair of nail-laminated decks.

Epoxy resins are used as a bonding agent in timber repairs. Epoxy seals the affected area, preventing water and other debris from entering. This should be limited to cosmetic repairs involving surface damage, not internal insect damage.

All timber repairs shall be in accordance with the Governing Specifications.

1073.5-DECK JOINTS

The following describes rehabilitation techniques associated with commonly used types of expansion joints. It is the WVDOT's policy to eliminate deck joints where practical. When replacing an expansion joint, the installation procedures shall be in accordance with the Governing Specifications and the Manufacturer's instructions.

1073.5.1-Open Joints: Finger joints are considered open joints. The major problems associated with finger joints are poor drainage, closed fingers, and loose attachments.

Improper drainage allows deicing chemicals, roadway grit, and gravel to collect on supporting beams and substructure units, causing accelerated rusting and concrete deterioration. Poor drainage can be corrected by first flushing the area to remove debris, then installing sheet metal deflectors or a neoprene trough to divert drainage and prevent the accumulation of debris. Future drainage problems can be prevented through frequent clearing of the drain troughs.

Finger joints that have become permanently closed can exert considerable forces on adjacent structural elements. Closed finger joints are a result of excessive movements of substructure units or insufficient allowances for roadway expansion. If roadway expansion is the cause of the joint closure, a pressure relief joint should be installed in the concrete approach pavement. When joints close due to excessive substructure movements, the unit that is causing the closure should be shifted to correct the problem. If the substructure unit is an abutment, the preferred solution, if practical, is to remove the joint and construct a semi-integral abutment. If the previously stated repairs are not economical, then the suggested means of relieving the pressure is to trim the expansion fingers or to remove and reinstall the entire joint system.

Structural components that have become loose, as a result of vehicular impact, can cause the joint to move in unanticipated ways and damage adjacent concrete. Excessive vertical movement may result in misalignment that can pose a roadway hazard. Finger bars that have broken loose at the welds should be repositioned and welded. Damaged curb plates, if still properly attached, should be straightened in place. Damaged concrete adjacent to the finger joint should be replaced.

1073.5.2-Closed Joints: Elastomeric expansion devices, compression seals, and strip seals are considered closed joints. Each type of closed joints has specific problems associated with them.

Elastomeric expansion devices are a sealed, waterproof joint consisting of steel plates and angles molded into a neoprene covering. Common joint failure occurs in the form of leaking, delamination, loosened or damaged anchor bolts, and damage caused by snowplows during snow removal. An elastomeric joint that shows signs of leaking can be repaired by resealing the joint. Where severe leakage has occurred, the entire section should be replaced. Elastomeric joints that have become delaminated should be replaced. Proper anchorage can be achieved by replacing loose or damaged anchor bolts with new bolts. A section of an elastomeric device that has been damaged by snowplows shall be replaced with a new elastomeric section.

Compression seals are extruded neoprene shapes that are chemically bonded to the adjacent structures. One common failure of compression seals is the loss of bond between the joint material

and the adjoining concrete or steel section. The neoprene can also become twisted if the concrete surrounding the joint armoring is not fully consolidated. An acceptable repair for these problems is a complete replacement of the compression seal with a two-part silicone sealant. However, this should only be performed if the concrete headers are found to be in satisfactory condition. If headers have failed, replace with an elastomeric expansion device. If it is practical, the desired repair for a compression seal is to replace the joint and convert the abutment into an integral or semi-integral abutment.

Strip seals consist of a heavy duty-neoprene gland, snaplocked into an extruded steel anchorage. Failures found in strip seals are similar to the ones associated with those of a compression seal, loss of anchorage and deformation of the neoprene gland. A common repair is to remove the damaged neoprene gland and spalling concrete, patch the concrete with an elastomeric concrete, then reinstall the neoprene gland after the concrete has cured. If it is practical, the desired repair for a strip seal is to replace the joint and convert the abutment into an integral or semi-integral abutment.

1073.6-BEARINGS

The following briefly discusses problems common to all types of bearings. This applies to expansion, fixed, pot, sliding and elastomeric bearings. The accumulation of debris on bridge seats attracts and retains moisture. This, combined with deicing chemicals, will cause corrosion of any steel member; particularly components subjected to movement and large forces. Any repairs shall be in accordance with the Governing Specifications.

The decision to repair or replace should be based on the ability of the device to transfer vertical loads and to accommodate superstructure movement. Deficiencies that in most cases warrant repair include the following:

- A. Heavy rust or surface scaling of non-contact surfaces.
- B. Significant tilting and displacement of bearing components at mild temperatures.
- C. Heavily rusted masonry and keeper plates.
- D. Heavily deteriorated anchor rods and nuts.

Bearings requiring replacement are ones that are severely deteriorated, suffered loss of function, and exhibit signs of imminent structural instability. The following can be used as a guideline in the choice of bearing replacement:

- A. The ability of the bearing to provide the same functions as the existing in terms of load transfer and movement.
- B. Compatibility with the environment.
- C. Dimensions of new bearing, particularly the height.
- D. Structural compatibility of the bearing with other bridge components.

1073.7-HISTORICAL STRUCTURES

Historic structures that are scheduled for rehabilitation shall adhere to the United States Department of Interior's *Standards for the Treatment of Historic Properties*. These standards can be obtained from the Technical Support Division, Environmental Section, of the WVDOH. The Designer shall work closely with the WVDOH on historic rehabilitation projects.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 1074

STEEL COATINGS

May 4, 2022

First Edition

1074.1-GENERAL

The steel coatings section is divided into two parts, coatings for new structures and previously painted existing structures. The reason for this separation is that two substantially different criteria exist with regard to coating new versus existing previously painted structures. Included in this section are an in-depth explanation of coating project containments, a discussion of problems encountered with removal of coatings containing hazardous toxic metal concentrations such as lead and a discussion of abrasive blasting. Also included in this Directive are methods of testing for toxic metal concentrations in the existing coatings and methods of testing for soluble salt contamination (chloride) on surfaces prior to and after cleaning.

1074.2-COATINGS FOR NEW STRUCTURES

1074.2.1-Types of Coatings Available: Uncoated weathering steel (Even though it is not a coating, weathering steel is included as an alternate to the standard coating materials. It is included in this section to give the Designer additional information during the decision-making process.

- A. Galvanizing.
- B. Metalizing (thermal spray applied).
- C. Zinc-rich primer without topcoats.
- D. Zinc-rich primer with topcoats.

1074.2.1.1-Uncoated Weathering Steel: SD 2039 lists areas of West Virginia where uncoated weathering steel should not be used. FHWA Technical Advisory; T5140.22, *Uncoated Weathering Steel in Structures* dated October 3, 1989, provides information on site locations where the use of uncoated weathering steel should be avoided. These include industrial areas with fallout from chemical plants, grade separations with “tunnel effects,” and low-level water crossings. While not included in the above advisory, the Designer should be aware that structures over deep valleys where high winds are common may be sites where uncoated weathering steel should not be used. The use of deicing salts and the resulting salt spray is blown underneath the structure and can contaminate the uncoated weathering steel. The salt spray contamination may attack or prohibit the formation of the protective patina layer on the weathering steel surface. If the patina layer is compromised, corrosion of the weathering steel can occur. Furthermore, structural details that prohibit the drying of the weathering steel should be avoided.

1074.2.1.2-Galvanizing: In this process, steel bridge members are properly prepared and dipped into a tank of molten zinc. Galvanizing of structural members is most often limited by the size of the dipping tank. At present, the maximum size that can be dipped is approximately seventy-five (75) feet in length. Care must be taken when galvanizing because the high temperature (750° F)

of the molten zinc bath can cause warping or distortion of the members. Differential steel thickness of the member can lead to inconsistent zinc thicknesses. There is also some concern that hydrogen embrittlement in weld areas can be caused by the high heat of the galvanizing bath. Steel chemistry can affect the efficiency of the galvanizing process. Advantages of galvanizing are long-term service life, galvanic corrosion protection properties and the ability to perform without any further finish coatings. If galvanizing is being considered, there are many excellent sources of information from the American Galvanizers Association.

1074.2.1.3-Metalizing (Thermal Spray Applied): Metalizing is the spray application of a coat of molten metal, usually a combination of zinc and aluminum, onto a prepared surface. The final product is similar to galvanizing without the heat-induced problems. Advantages are structural members that are too large to place in the galvanizing bath can be metalized, application temperature is lower than galvanizing and the heat generated by the thermal spray process is dissipated quickly. Clear or pigmented seal coats applied over metalizing are often used. Disadvantages are application rate is somewhat slower than conventional coatings; more labor intensive; and requires a higher degree of surface preparation than galvanizing.

1074.2.1.4-Zinc-Rich Primer without Topcoats (711.20): Inorganic zinc-rich primer without topcoats can be used in the same locations as uncoated weathering steel. Advantages are: the primer adds galvanic corrosion protection, can be used onto coated with no field painting (except for touch up); less costly than using topcoats; and shortens construction time. In mild environments, the life expectancy is 25+ years. Disadvantages are: requires a skilled applicator, prone to dry overspray, mudcracks, some that the concerns that the gray-green color may not be aesthetically pleasing.

1074.2.1.5-Zinc-Rich Primer with Topcoats (711.6, 711.20 and 711.22): Zinc-rich primer with topcoats is an all-purpose long-life system that may be used on any properly blast-cleaned structure. Advantages are galvanic corrosion protection, should perform for at least 25 years in harsh environments, choice of topcoat color, aesthetically pleasing and the topcoats protect the primer therefore providing a longer life. Disadvantages are high initial cost, additional construction time due to multiple coats of paint and structural repairs may not match in color due to color change resulting from weathering of the topcoat.

Complete shop painting (primer and topcoats applied in fabrication shop) should be considered for structures in industrial areas. This approach keeps the primer from becoming contaminated in the field prior to top coating. In addition, all the painting will be performed on the ground; application and inspection should be more efficient in the fabrication shop. In high traffic areas, such as Interstate overpass bridges, complete shop painting reduces the amount of time required for lane closures due to painting. Disadvantages are construction damage may require extensive costly repairs and the steel fabrication industry generally does not prefer to do complete shop painting.

1074.2.2-Items for Consideration by the Designer:

- A. Traffic control: How will the field coating operations affect traffic control?
- B. Public awareness: Is the structure located in an area where color is important to the citizens of the area?

- C. Structure location and design: If the structure is located in an industrial area, is the proposed design of the structure compatible with ease of painting? Open box members, seams, inaccessible areas etc. should be avoided.
- D. Slip critical surfaces: The slip coefficient of the coating must be taken into account in designing the connections. The cost factor of additional bolt holes for “slippery coatings” includes the cost of drilling additional holes, the cost of the fasteners and the time for installation.
- E. Demolition and toxic metals: Will the existing bridge be demolished? Does the existing bridge contain lead or other toxic metals? If the answers are yes, the plans should contain notes informing the Contractor.
- F. Steel Box beams: If using box beams, the inside of the boxes shall be coated with an inorganic zinc-rich primer without topcoats. Cover or screen box openings to keep birds and their pollution out of the inside of the boxes.
- G. Jointless structures: Jointless structures are coating friendly over the life of the structure. Generally, the area in the vicinity of joints is the first area to exhibit corrosion due to joints leaking onto the structure.
- H. Uncoated weathering steel: When using uncoated weathering steel at bridge joints, the steel within a distance of 1½ times the girder depth should be painted. Road salts (chlorides) can attack weathering steel. Will the fascia girders be painted? If fascia girders are painted, what color? Do concrete piers need to be protected from staining and chlorides?
- I. Environmental regulations: Is the bridge located in an area of the state that has restrictions on the Volatile Organic Content (VOC) of the coating? If a structure is being demolished, how will the existing coating affect the workers and the environment? For example, using a cutting torch on lead-based paint requires the worker to be protected from the resulting fumes.
- J. Containment: Some form of containment will most often be required for field painting and touch up. See SSPC’s *Guide for Containing Debris Generated during Paint Removal Operations* (Guide 6), SSPC’s *Guide for the Disposal of Lead Contaminated Surface Preparation Debris* (Guide 7) and MP688.03.20. Three parameters must be included in the plans: class of containment, method of assessment and method of assessment of quantity of emissions, SD 1074.3.3.3 below.

1074.3-COATINGS FOR EXISTING STRUCTURES

1074.3.1–Strategy:

- A. Spot or localized repair and overcoating: This strategy involves the spot preparation of localized areas or zones that are exhibiting deterioration (e.g., areas under leaking expansion dams), followed by the spot application of a primer to the repaired area and one or more finish coats to the repaired zone or to the entire structure. This approach has proven to be a cost-effective method for extending the life of paint systems provided the amount of surface preparation required is not extensive, and the coatings which are allowed to remain can satisfactorily withstand the curing stresses and weight of applying additional layers of paint. With this strategy, it is important that coatings compatible with the existing system be used. Risks associated with this strategy include: the potential for cohesive disbanding within the old paint system, adhesion failure between the new and the old

coating layers or total failure to the base metal substrate. Additionally, there is potential for lifting and corrosion at the typically more difficult to coat areas such as edges, connections, inside angles, welds, or even fractured edges of the old system at tie-in points. This strategy allows lead or other hazardous toxic metals in the existing coatings, if present, to remain for future remediation. This strategy may produce an aesthetically unpleasing patchwork appearance.

- B. Total coating removal and replacement: This strategy is initially the most expensive approach. However, the initial cost spread out over the expected service life makes this strategy less expensive in the long term. Costs may increase due to requirements for compliance with environmental and worker protection issues, the volume of waste to be handled and disposed of and the possible disruption of traffic services for extended periods of time. The risk of contaminating surrounding property can be significant. The major advantages of this strategy are the complete removal and mitigation of hazardous toxic metals, if present, and wide latitude in the selection of high-performance repainting materials, which have been proven to provide the best long-term performance characteristics. In light of the fact that an entirely new coating system is being applied, the need for maintenance painting should not be necessary for several years. This is an excellent strategy if the elimination of hazardous toxic metal-containing coatings is required and budgetary restrictions are minimal or if the existing coating were so deteriorated that spot or localized repairs would essentially result in complete removal.
- C. Demolition and replacement of the structure: For older or smaller structures, it has often been found to be more cost effective to replace the structure rather than removing and replacing the coating, particularly if lead paint is present. In the case of smaller bridges, this can be a viable option when the bridge deck also requires replacement. The principal advantage is that costs for the construction of field containment, worker and environmental protection and hazardous waste disposal are instead applied to a tangible asset (the new bridge), which meets updated design technology and requirements. If it is decided to demolish the structure, the demolition plans should note that the existing coating on the structure contains lead or other toxic metals.
- D. Coating assessment: Prior to the decision on which strategy is appropriate for use on a structure, certain information concerning the existing coating must be determined by field inspection and testing.
 - 1. Visual inspection: Rust and coating failures typically appear in a few characteristic places. Carefully examine edges and corners of structural members. Paint is generally thinner at edges and corners. Corrosion often starts at these locations and then undercuts the adjacent intact paint as it spreads away. Inside square corners many times receive an extra thick layer of paint due to multiple coating application passes made over the surface in an effort to coat all angles of the structure. These thicker layers of paint may be prone to cracking, thus exposing the steel to the environment. Carefully examine all areas that may contain deicing salts or retain moisture. Pay particular attention to areas under scuppers, beneath downspouts, inside gusset connections and horizontal surfaces beneath the edges of bridge decks and expansion dams, where deicing salt runoff and debris typically collects. Inspect bolts and rivets, rust detected in these areas may indicate corrosion along the entire length of the bolt or rivet, which may indicate reduced structural integrity. Examine splash zones and surfaces directly exposed to wind, rain, and moisture condensation.

2. Corrosion: The degree of corrosion or coating deterioration (peeling, cracking, lifting, etc.) is important in deciding which painting strategy to use. There are a variety of industry standard procedures, which incorporate photographic standards in order to evaluate and categorize the degree and extent of corrosion on steel surfaces. One method entails a visual evaluation of painted surfaces in accordance with ASTM 610/SSPC-Vis 2, Standard Method for Evaluating Degree of Rusting on Painted Steel Surfaces. This pictorial standard provides an excellent reference for evaluating and quantifying the degree of rusting on painted steel surfaces.
3. Dry film thickness: As the number of coats of paint on a structure and their total thickness increases and the associated stresses and weight of the paint film increase; the risk of future detachment by maintenance painting increases. Measure the paint film thickness in accordance with established industry standards using the equipment manufacturer's instructions for instrument operation. SSPC-PA2, Measurement of Dry Film Thickness with Magnetic Gages, provides a test method for measuring total coating thickness on painted steel surfaces. Measure the total thickness of the paint film at each adhesion test location and at representative locations across the structure. Take measurements on girder surfaces including web, top flange and both top and bottom surfaces of the bottom flange. Measurements should be taken on a representative area of each type of structural member present on the bridge and for each different exposure environment (splash zones, areas directly exposed to weathering, areas protected from weathering). Measurements should also be made on areas where the coating is visually different. Record the number of measurements conducted on each component, the overall range and the average of all readings obtained.
4. Coating adhesion: At some point, the application of additional coating material to an otherwise adherent paint may impart sufficient stress and weight to cause the existing coating to disbond at its weakest point. Adhesion test results help to provide a means for making broad judgments about potential risks of applying additional layers of paint. Locations selected for adhesion tests should represent the thinnest to the thickest areas of the existing coatings. Test area frequency depends upon the size of the structure, the similarity in condition of the coating throughout the structure, coating thickness variations and other factors relating to coating service environments. Coating conditions may vary in spans over water, as opposed to spans over land. The amount of direct sunlight incident to the bridge steel can have varying effects on the condition of the existing coating. Thus, more testing is required if different environments exist. The ultimate factor in reducing test frequency occurs when consistency of results is obtained.

Adhesion testing may be performed in accordance with ASTM D3359, *Standard Test Methods for Measuring Adhesion by Tape Test*, Method A and B. Methods A and B differ in the type of scribe made in the existing coating; the thickness of the existing coating determines which of the methods to use. Both methods assess the amount of coating removal on a scale of 0 to 5, with 5 being the best adhesion. Method A, the X-Cut Method, is more commonly used for the thicker (greater than 5 mils) coating films. Method B is usually reserved for laboratory testing of coating films less than 5 mils in thickness.

5. Substrate condition: Underfilm corrosion or an intact layer of mill scale on a surface, if inactive, may not necessitate the removal of the existing coating. Conversely, if the mill scale, which can be thermodynamically unstable, is beginning to flake and detach from the steel substrate, or if active underfilm corrosion is occurring, such conditions can influence the maintenance painting decision. At each adhesion test site, remove approximately 1 square inch of the coating from the substrate. This can be accomplished by scraping with a sharp knife and wiping with a strong solvent. Care must be taken to avoid damaging the substrate when removing the paint film so as to allow for accurate evaluations of underfilm conditions.

Examine the surface visually for the presence of mill scale and/or corrosion. Viewing under magnification can assist the assessment. A five percent (5%) solution of copper sulfate and tap water can aid in the determination of the presence or absence of mill scale. The solution, when applied to the area, plates out a copper color on the bare steel. Mill scale remains a bluish black color. Document the presence or absence of mill scale and corrosion, describe its condition and identify the locations of tests.

6. Existing coating: The bridge file (available in the District or in Operations Division) should be reviewed to ascertain the type of paint previously applied to the structure. The bridge file should contain a coating history of the structure. If the generic type of the existing coating is unknown, it must be determined prior to specifying repairs to assure that compatible repainting materials are used. The finish and underlying coatings should be sampled separately for laboratory analysis. Samples of coatings may be removed by scraping with a sharp knife or with a chisel. The frequency of the sample sites depends on the conditions found on the structure. If the same number of coats are found throughout and the coating does not appear different, fewer samples are needed. All areas where coatings were removed should be immediately repaired by applying a coat of paint.

Assessing and quantifying the presence of lead and other toxic metals may be determined from coating samples. Samples should comprise of all coats or layers of paint and be frequent enough to represent all coatings on the structure. Atomic Absorption Spectroscopy is a suitable laboratory method to determine the concentration of hazardous metals. Additionally, simple field test methods for lead detection exist. One such method uses LeadCheck® swabs in which a reactive chemical turns a different color, usually reddish or pinkish, in the presence of lead at concentrations higher than 0.5% (5,000 PPM). The limitation of this test is that it is qualitative. Only the presence of lead is detected, not the concentration, and lead levels below 5,000 PPM may not be detected. OSHA 29 CFR 1926.62, *Lead Exposure in Construction* is triggered when airborne concentrations of lead exceed the action level (30 micrograms per cubic meter) as an 8-hour time weighted average (TWA). This can occur at lead levels considerably less than 5,000 PPM. Depending on the removal method and ventilation employed, this method is useful only for a quick field determination of the presence or absence of lead containing coatings. The actual concentration of lead and other hazardous metals must be quantified by appropriate methods prior to beginning contract work.

1074.3.2-Interpretation of Coating Condition Data: Four criteria must be reviewed and interpreted: corrosion, adhesion, film thickness, and substrate condition. The following table gives general guidelines for decision-making.

Criteria	Test Method	Condition 1 Repair Not Required at Present	Condition 2 Repair Needed Soon	Condition 3 Engineering Decision	Condition 4 Total Removal of Coating
Corrosion/Coating Deterioration	ASTM D610	Less than 3%	3 – 10%	10 – 20%	Greater than 20%
Adhesion	ASTM D3359	Greater than 2A or 2B	Greater than 3A or 3B	Less than 3A or 3B	0A or 0B
Film Thickness	SSPC PA2	Less than 20 mils	Less than 20 mils	20 - 30 mils	Greater than 30 mils

These guidelines should not be seen as absolute for making painting decisions because many other factors, such as section loss, structural integrity, available funding and public safety must be included in the decision.

It is apparent that Condition 3 has the most risk for spot repair/overcoating. Coating test patches are a good engineering method to assist in making the correct decision. Placing test patches on a structure is a relatively simple and inexpensive process. ASTM D506401, *Standard Practice for Conducting a Patch Test to Assess Coating Compatibility*, should be used. A few key items in the test patch process are:

- A. Selection of coating material: A good start is to contact a coating manufacturer's representative for their recommendations. At least three different coating materials should be selected.
- B. Selecting test patch locations: Ultraviolet light exposure, airborne contamination (including truck exhaust), roadway splash zones and deicing salt retention areas can all have different microenvironments that may dictate coating conditions and where representative test patches are applied.
- C. Surface preparation: Should be the same as for the application of the repair coating.
- D. Patch application: If possible, the coatings should be applied by the same method that is to be used during the project. Wet film thickness should be monitored during application. In the application of multi-coats, a small portion of each coat should be left exposed so that each individual layer can be evaluated. Six months is the minimum exposure time recommended. Allowing the test patches to weather over the winter provides for several freeze-thaw cycles, which are beneficial to the evaluation.
- E. Patch evaluation: Any sign of peeling, cracking or disbonding indicates the material may not be adequate. Adhesion should be tested around the total patch area. The adhesion should remain in the 3-5A (fair-good) range when evaluated in accordance with ASTM D3359, *Standard Test Methods for Measuring Adhesion by Tape Test*. The new coating must be well adhered to the existing coating and the original layers must have no diminished adhesion qualities as a result of the new coating application. The patch should be periodically evaluated for as long as practical. The results of the patch test should show:

1. If the structure is a candidate for spot repair/overcoating.
2. Which of the test materials is the most appropriate for recoating the structure.
3. The possibility of using a hybrid approach to recoating the structure (i.e., complete removal in some areas and overcoating in others).

1074.3.3-Guidance Document 6 - Guide for Containing Debris Generated During Paint Removal Operations: In order to assist industry in a better understanding of containment, the SSPC published *Guide for Containing Debris Generated during Paint Removal Operations*, (Guide 6). Guide 6 establishes classes of containment for methods of removal in addition to abrasive blast cleaning. These additional classes include hand and power tool cleaning, chemical stripping and wet methods of removal. Guide 6 describes the following: methods of paint removal, containment system classes and methods of assessing the quantity of emissions.

1074.3.3.1-Removal of Coatings Containing Lead and Other Toxic Metals: A designer faces various problems when the existing coating on a structure contains lead or other toxic metals. These problems can be placed in two categories: removal and containment.

The removal of bridge coatings containing lead and other toxic metals can be performed in an environmentally conscious and worker safe manner. The technology of paint removal and containment methods has advanced substantially in the past few years. The Steel Structures Painting Council has two major publications that deal with lead removal; they are *Industrial Lead Paint Removal Handbook*, Volume I, and *Industrial Lead Paint Removal Handbook*, Volume II (Project Design).

For total removal of bridge coatings, abrasive blast cleaning remains the method of choice for both specifiers and painting contractors. It is the most productive of the paint removal methods and provides the highest quality of preparation to enhance the long-term paint performance. The most common methods of abrasive blast cleaning are listed below:

- A. Blast cleaning with expendable abrasives: Disadvantages include, a high level of dust, requirement for elaborate containment and ventilation systems and the volume of waste debris is high. Large amounts of debris can significantly increase the cost of waste handling and disposal.
- B. Blast cleaning with recyclable abrasives: The abrasives are recycled on site using specialized equipment. Disadvantages include the high cost of equipment, equipment breakdown and the resulting lost time. Dust that escapes the containment is typically composed of high levels of steel fines, which can rust and may stain new paint films giving the appearance of premature coating failure on those freshly painted areas.
- C. Wet methods of blast cleaning: In order to reduce the emissions at the point of generation, water alone or water combined with an abrasive may be used. Methods include high pressure water jetting (up to 25,000 PSI) and ultra-high pressure water jetting (up to 40,000 PSI). The water alone is capable of removing loosely adherent coatings or for the selective removal of marginally adherent coatings. The use of water for cleaning leads to efficient removal of chloride from rusted areas. Wet methods are less productive than dry methods, but containment requirements due to no airborne dust, can be reduced. Rust inhibitors are often used to prevent flash rusting prior to coating application. Rust inhibitors can cause problems with paint adhesion; the rust inhibitor must be compatible with the coating system. Disadvantages include the large amount of water that is to be contained, captured and filtered to remove paint chips, etc.

- D. Vacuum blast cleaning: The blasting nozzle is fitted with a source of vacuum to simultaneously clean and collect the abrasive. Disadvantages are a low rate of production, equipment must be relatively close to the work area, difficult to maintain a proper vacuum on irregularly shaped surfaces and special fittings/attachments are often needed.

Whichever method of cleaning is used, the final condition of the steel should provide a surface cleanliness compatible with the prime coating to be used on the project. On existing structures when abrasive blasting is required, the two surface preparation standards usually specified are SSPC-SP6 (NACE #3), Commercial Blast Cleaning, or SSPC-SP10 (NACE #2), Near-White Blast Cleaning. The difference between the two standards is the amount of staining allowed per unit of area. SSPC-SP10 is the more stringent of the two standards. On projects where blast cleaning is not required or not possible, SSPC surface preparation standards such as, SSPC-SP3 Power Tool Cleaning, or SSPC-SP11 Power Tool Cleaning to bare metal can be specified.

In addition to the surface preparation standard, surface profile (roughness of the surface) needs to be specified. The allowable surface profile depth is usually dependent upon the thickness of the coating to be applied. If control of the surface profile is deemed to be significant to coating performance, it should be addressed in the specifications. Surface profile is discussed in SSPC-SP-COM. Designers developing coating specifications should familiarize themselves with SSPC, Steel Structures Painting Manual, *Systems and Specifications*, Volume II.

Containment: When using dry abrasive blast cleaning, sophisticated containment systems are required. It is common for regulators and the public to expect “dust free” operations. While no operation is completely free of any emissions at all times, containment systems being designed and used today are capable of controlling the emissions to the extent that they are not harmful to the environment or the public. The result of the tight containment has been to create an extremely hazardous environment for the workers inside. In order to reduce the toxicity of those exposed inside and to better control the escape of dust, ventilation systems are used. In May 1993, OSHA published *Lead Exposure in Construction* (29 CFR 1926.62). This standard requires that feasible engineering controls be used to reduce worker exposures to lead and other toxic metals. When working in containment, “engineering controls” necessitate the use of substantial dust collection systems to achieve sufficient controlled airflow.

1074.3.3.2-Containment System Classes: Containment classification systems vary in the degree to which the containment design can be effective in controlling emissions into the environment. The degree of emission control is highest in Class 1, high for Class 2, moderate for Class 3 and minimal for Class 4. Four classes of containment are available for abrasive blast cleaning, with three classes for the other methods of removal. Selection of a containment system with greater emission control is recommended when the public is nearby, or the project is close to environmentally sensitive areas. Each Class, 1 through 4, is also designated by a letter which signifies the type of removal method being used. The letters are A for abrasive blast cleaning, W for wet methods, C for chemical stripping and P for power tool or hand cleaning. For example, Class 1A designates Class 1 using abrasive blast cleaning methods. Each Class is defined in Guide 6 with regard to the physical characteristics of the containment, such as type of joint, walls, and the required ventilation.

1074.3.3.3-Methods of Assessing the Quantity of Emissions: There are six methods of assessment; they are the following:

- A. Method A (visible emissions): General surveillance (assessments made without regard to the opacity of emission).
1. Level 0 emissions – no visible emissions.
 2. Level 1 emissions – random emissions of no more than 1% of the workday.
 3. Level 2 emissions – random emissions of no more than 5% of the workday.
 4. Level 3 Emissions – random emissions of no more than 10% of the workday.
 5. Level 4 emissions – emissions are unrestricted and may occur at any time.

Opacity scale (assessment of both the length of time an emission is observed, together with the opacity of the emission on a scale from 0 to 100%).

The WVDOH has, on most projects in the past, specified Level 2 Emissions. The 5% is equivalent to 24 minutes in an 8-hour workday. The opacity scale measurement must be made by trained, certified observers.

- B. Method B (ambient air monitoring for PM-10): High volume air samplers equipped with PM-10 heads are used to assess the total amount of particulate matter, 10 microns or less in size, which escapes the containment. The number of monitors to be used is based upon wind direction and proximity to homes, schools, playgrounds, etc. Monitoring should be conducted for a few days prior to the beginning of work in order to establish background levels.
- C. Method C (occupational monitoring of area emissions for lead): This method uses personal monitors outside the area or equipment that may potentially emit lead.
- D. Method D (EPA ambient air monitors for toxic metals): High volume air samplers are equipped for the collection of total suspended particulate. Same site selection criteria as Method B.
- E. Method E (soil analysis for toxic metals): Pre-job and post-job soil analysis for toxic metals, such as lead, may be useful for determining if adequate ground protection was employed.
- F. Method F (water and sediment analysis): Pre-job and post-job sediment analysis for toxic metals may be useful in determining if adequate water protection was employed. Sampling of water in fast moving streams may or may not be useful. However, for sedentary bodies of water or if drinking water intakes are located nearby, both pre- and post-job water sampling and analysis may be beneficial.

1074.3.4-General Comments:

- A. The Designer should specify the class of containment required and the environment monitoring assessment criteria that will be used to verify the containment suitability. The specified containment must be maintained in accordance with the approved design throughout the project.
- B. The Designer must consider the effect of wind loads on the bridge with the containment in place and provide the minimum wind speed at which the containment must be dropped. The containment structure must be capable of supporting the weight of workers, equipment, and abrasive material.
- C. The specification should identify the methods(s) of assessment, the frequency and duration of the test, the location of the monitor(s) and the acceptance criteria.

- D. Surface preparation material (waste material from cleaning) is not classified as hazardous until tested, although it must be handled as hazardous material until shown to be non-hazardous.
- E. In addition to Guide 6, the following documents are of value to the Design Engineer:
 - 1. Guide 7: SSPC's *Guide for the Disposal of Lead-Contaminated Surface Preparation Debris*.
 - 2. SSPC 95-06: *Project Design, Industrial Lead Paint Removal Handbook*, Volume II.
 - 3. MP 688.03.20: *Guide for Developing the Contractor's Containment/ Disposal Control Plan for Spent Material Prior to Painting Existing Steel Structures*.
 - 4. MP 688.03.21: *Procedure for Sampling "Spent Material" During Removal or Cleaning of Existing Steel Structures Prior to Repainting*.
 - 5. MP 688.03.22: *Procedure for Testing "Spent Material" Recovered During Paint Removal or Cleaning of Existing Steel Structures*.
 - 6. MP 688.03.23: *Evaluation and Approval for Laboratory Testing of Solid Waste*.
 - 7. MP 688.02.20: *Guide for Contractor's and Fabricator's Quality Control Plan for Painting Structural Steel*.
- F. On existing structures, the WVDOH has been requiring two water washing phases.
 - 1. Phase 1, using plain water and low-pressure (3,000 – 4,500 PSI), consists of completely washing the entire bridge deck, expansion joints, sidewalls and drains.
 - 2. Phase 2, wash the structural steel at low pressure with a solution of a commercial brand of soluble salt remover. After the Phase 2 washing, the surface is tested to ascertain chloride concentration. The maximum level of chloride contamination allowed has been 5-micrograms/square centimeter. The WVDOH has been specifying Chlor*Rid® or equal as the soluble salt remover and Chlor*Test™ as the test method.
- G. The wash water from the water cleaning phases is sometimes filtered using a 40 or finer mesh material to catch particles of paint and debris.
- H. In high traffic areas, overpass bridges can be painted at night using rapid mobilization techniques. This technique uses portable truck-mounted containment structures, which are set up and taken down each work night. Using this type of containment with fast curing coatings allows dusk to dawn work with the potential for coating one or two lanes per shift.

1074.4-CONCRETE COATINGS

An important measure used to extend the life of concrete elements, in general, and concrete decks, in particular, is the application of a protective waterproofing sealer. For concrete decks, the protective sealer is typically placed on the deck fascias, an area which is extremely susceptible to deterioration.

In general, protective sealers are available in a variety of forms; those accepted by the WVDOH are Linseed Oil and Epoxy Resin. See Sections 601.13, 707.11, and 707.12 of the Specifications.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1080
COAST GUARD BRIDGE PERMITS**

*May 4, 2022
First Edition*

The Designer shall contact the Engineering Division of the WVDOH at the span arrangement stage to determine the required permits based on anticipated construction methods. The Bridge Designer is responsible for preparing the required permit package for permit checklist. The Project Manager submits the permit to the appropriate agency.

1080.1-COAST GUARD

The USCG requires their prior approval for navigable stream crossings.

The Engineering Director is responsible for obtaining a permit from the Commandant, United States Coast Guard. The permit approves the location and plans for the construction or alteration of any bridge on the State Highway System over navigable streams. Alteration is taken to mean changes to the existing navigation clearances.

Navigation lights and vertical clearance gauges are conditions of the permit and are subject to Coast Guard approval. The Coast Guard, not the State, makes the determination of need for this permit. In general, permits are required throughout West Virginia for streams to the head of navigation and three miles further upstream.

The following is quoted from the *Coast Guard Bridge Permits* publication:

Waterway Requiring Bridge Permits:

Coast Guard jurisdiction is limited to the construction or alteration of bridges and causeways over waterways that have been determined to be navigable waterways of the United States by the U. S. Coast Guard. Navigable waterways of the United States for bridge administrative purposes are: (1) All waters that (i) are used, or are susceptible for use, by themselves or in connection with other waters in highways for substantial interstate or foreign commerce, or (ii) a governmental or nongovernmental body having expertise in waterway improvement determines that the waterway is capable of improvement at a reasonable cost to provide, by its self or in connection with other waters, highways for substantial interstate or foreign commerce; or (2) all waters subject to the ebb and flow or the tide; or (3) tributaries and embayments which are lateral extensions of navigable waterways from the confluence up to the upstream to the limit of ordinary high water (navigable-in-law).

For these reasons, anyone planning to construct a bridge or causeway over a waterway which might be considered navigable under the above criteria, should contact the..[Eighth Coast Guard District, Bridge Branch, 1222 Spruce Street, St. Louis, Missouri 63103-2398, Telephone No. (314) 539-3900, Ext. 2378 for assistance]. However, prospective bridge builders are under no obligation to seek out a determination from the Coast Guard unless the nature and character of the

waterway belies any reasonable conclusion that the waterway is navigable; the burden, if any, rests with the Coast Guard to determine the facts and reach a determination on its own motion or in response to a complaint.

Advance Approval Category Waterways:

The commandant, U. S. Coast Guard, has given his advance approval to the location and plans of bridges to be constructed across certain minor waterways navigable-in-law but not actually navigated other than by logs, log rafts, rowboats, canoes and small motorboats. In such cases, the clearances provided for high water stages will be considered adequate to meet the reasonable needs of navigation. Persons planning to construct a new bridge or causeway or alter an existing bridge or causeway over a navigable waterway are urged to contact the Coast Guard. Any case of reasonable doubt will be resolved by the usual practice of notice or hearing prior to a specific approval of the location and plans for the proposed structure based upon findings made at the time and under the conditions then existing.”

In early project development stages, the designer in consultation with the WVDOH is responsible for assessing the need for a Coast Guard permit. The Designer should initiate contact with the USCG at the earliest possible stage of project development, providing opportunity for Coast Guard involvement throughout the environmental review process in accordance with 23 CFR, Part 771. The environmental section should be consulted for coordination procedures.”

1080.2-US COAST GUARD (TAKEN FROM USCG BRIDGE PERMIT APPLICATION GUIDE, COMDTPUB P16591.3B)

- A. Letter of application for a permit.
 - 1. Applicant information.
 - 2. Consultant information (if employed).
 - 3. Project information.
 - 4. Authority information.
 - 5. Proposed clearances.
 - 6. Existing bridge structure at bridge site.
 - 7. Removal of bridge(s) (if applicable).
 - 8. Construction activity.
 - 9. Environmental effects.
 - 10. Required authorizations.
 - 11. Other federal agencies with jurisdiction over the proposed project.
 - 12. Any applicable attachments.
 - 13. Fill (if applicable).
- B. Adjacent property owners within ½ mile radius.
- C. Underlying studies, reports, and other information.
- D. Drawings.
 - 1. Vicinity map.
 - 2. Plans.
 - 3. Location map.
 - 4. Plan view.
 - 5. Elevation view.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1090
ALTERNATE BRIDGE DESIGN**

*May 4, 2022
First Edition*

On large, complex bridges, it is likely that limiting the design to one material type will unnecessarily limit the number and competitiveness of potential bidders. It is in these cases that alternate designs are economical in terms of the extra design cost and additional time. When alternate designs are required, it is desirable that one superstructure is to be designed with steel and one with concrete. Alternate designs utilizing the same materials, on occasion, can be possible. In certain cases, it may be appropriate to have different superstructure material types for main spans and approach spans.

Alternate designs are required when bridge costs meet the threshold for value engineer analysis per DD 816. Both alternates shall have the same design life, take a similar amount of time to construct, and shall have similar serviceability.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1100
RAILROAD CONSIDERATIONS**

*May 4, 2022
First Edition*

The preliminary plans for bridges to be constructed over railroads shall be submitted to the owner of the railroad by the Project Manager for review and approval. Bridge structures over railroads should generally be prepared following the basic geometric clearances shown in Figure 1100.A. This figure represents the geometric requirements of the Norfolk Southern Corporation and CSX Transportation, Inc. and may vary when applied to other railroads.

1100.1-APPROVAL

The designer shall provide the clearance information for submission to the Railroad for their approval. The data shall identify the railroad division, the railroad valuation station at the centerline of bridge, the distance from the nearest milepost to centerline of bridge, the WVDOH crossing number and the state project number.

1100.2-CRASH WALLS

To limit damage by the redirection and deflection of railroad equipment, piers supporting bridges over railroads and with a clear distance of less than twenty-five (25) feet from the centerline of a railroad track shall be of heavy construction (defined below from CSX *Transportation Criteria for Overhead Bridges*) or shall be protected by a reinforced concrete crash wall, with the exception of rails-to-trails. Crash walls for piers twelve (12) feet to twenty-five (25) feet clear from the centerline of track shall have a minimum height of six (6) feet (10 feet for Norfolk Southern) above top of rail. Piers less than twelve (12) feet clear from the centerline of track shall have a minimum crash wall height of twelve (12) feet above the top of the rail.

The crash wall shall be at least 2.5 feet thick and at least twelve (12) feet long. When two or more columns compose a pier, the crash wall shall connect the columns and extend at least one foot beyond the outermost column parallel to the track. The crash wall shall be anchored to the footings and columns, if applicable, with adequate reinforcing steel extending to at least four (4) feet below the lowest surrounding grade.

“Piers shall be considered of heavy construction if they have a cross-sectional area equal to or greater than that required for the crash wall and the larger of its dimensions is parallel to the track.”

Consideration may be given to providing protection for bridge piers over twenty-five (25) feet from the centerline of track, as conditions warrant. In making this determination, account shall be taken of such factors as horizontal and vertical alignment of the track, embankment height, and an assessment of the consequences of serious damage in the case of a collision.

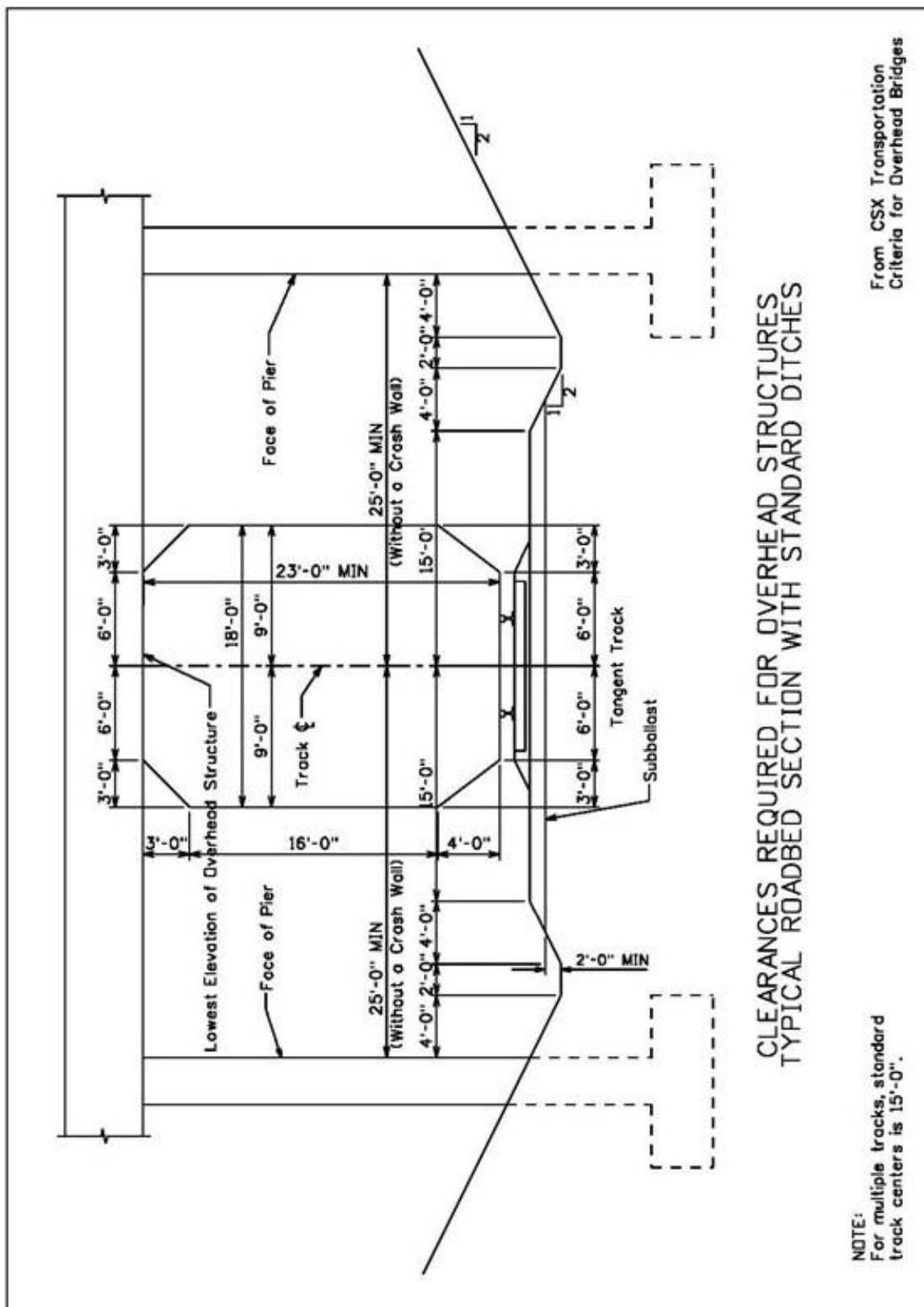


Figure 1100.A

1100.3-CLEARANCES

Minimum vertical clearance above the high rail shall be twenty-three (23) feet. Rehabilitated or widened bridges will generally be allowed to maintain existing vertical clearance, but no less.

The preferred horizontal clearance from centerline of track to the face of the structural element is twenty-five (25) feet. The absolute minimum on all new construction is twelve (12) feet. Rehabilitation bridges will generally be allowed to maintain existing horizontal clearance, but no less.

See Figure 1100.A for an illustration of vertical and horizontal clearances.

The Railway-Highway Provisions of the Specifications state there can be no work within railroad right of way without first obtaining authority from the Railroad's Chief Engineer. The designer should consider these restrictions in the design process to minimize encroachments.

1100.4-DRAINAGE

Proper drainage of railroad right of way must be considered during the layout and design of a railroad overpass.

- A. Substructure units and embankment slopes shall not interfere with railroad ditches.
- B. Deck drains shall be placed to prevent discharge onto the railroad right of way.
- C. The Designer shall verify any additional drainage issues with the Bridge Project Manager.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 1110
BRIDGE DOCUMENT SUBMISSIONS**

*May 4, 2022
First Edition*

A detailed checklist for each submission is included in DD 202. The following sections give a brief description about each submission.

1110.1-DESIGN REPORT

A design report is a preliminary engineering study of proposed alignments for a project. The report will be subject to two scheduled reviews: Design Report Field Review and Design Report Office Review. Refer to DD 202 for more details for each submission requirements.

The Bridge Designer's tasks are to provide the necessary bridge information, cost estimates, and sketches for the report to be developed. This requires the Bridge Designer to look at existing conditions (geometry, major utilities, right of way, existing and/or adjacent structures, etc.), historical data, hydraulic opening, and proposed alignments to assist in the development of the report. The Bridge Designer should develop a cost and a plan and profile drawing for each alternate. These drawings are simple line drawings with minimal details (stationing, grades, vertical and horizontal curve data, etc.). The cost estimates shall be based on historic data for similar structures on a cost per square foot basis.

1110.2-PRE-SPAN ARRANGEMENT MEETING

Prior to the submission of the Span Arrangement Report, the Bridge Designer shall meet with the Bridge Project Manager to discuss the feasible structural systems and span arrangements that should be included in the report. The purpose of this meeting will be to eliminate or add additional alternates for further consideration. This will save considerable time in the Span Arrangement submission stage. The Bridge Project Manager will make this final decision on what alternates should be studied. At this meeting, the Bridge Designer should provide the following:

- A. Preliminary line and grade.
- B. Draft site plan for each recommended alternate showing both plan and profile views. Abutment location should be fairly accurate at this time. Piers should be located for each alternate being studied.
- C. Discussions on why alternates were chosen, and others were not.
- D. Location of utilities, environmental concerns, roads, railroad tracks, etc. Include any obstacles that may influence recommendations.
- E. Superstructure types being considered.

At this time, no cost estimates, or girder analysis should be completed.

1110.3-SPAN ARRANGEMENT STUDY

The Span Arrangement shall be submitted with the Preliminary Field Review Plans for the line and grade studies. The purpose of the Span Arrangement Study shall be to investigate all types of feasible structural systems, span arrangements, and establish all basic design and plan preparation parameters. Following the approval of the Span Arrangement Report and the Preliminary Field Review Plans, the Designer may obtain structural borings for the approved bridge. Approval of the Span Arrangement Report on any project shall not limit or preclude the taking of additional borings required by the WVDOT following review of the TS&L nor does it constitute final acceptance of structure type or span arrangement. Please refer to Section 2.3, Geotechnical Investigations, concerning core borings and foundations.

The following list describes some of the information that should be included in the report. See DD-202 for a detailed list for each submission.

- A. Alignment, grades, typical sections, and superelevations used shall be documented in the Preliminary Field Review Plans.
- B. Consider all viable construction materials. Equal treatment between alternates is essential in ensuring competition and optimum cost-effectiveness. Uniformity of design criteria, material requirements, and appropriate unit costs shall be considered and documented.
- C. Hydraulic study (if crossing a waterway) justifying the proposed span arrangement, scour features, and grade shown in the Preliminary Field Review Plans.
- D. Discuss constructability, any special staged construction, clearance criteria (if crossing a road, railroad, or navigable waterway), freeboard (if crossing a waterway), and maintenance of traffic requirements.
- E. A description of the proposed superstructure depth and preliminary superstructure type utilized in the study, for each alternative span arrangement.
- F. Deck drainage, superstructure joint, and bearing device requirements.
- G. Special environmental, aesthetic, and utility considerations.
- H. A description of the assumed foundation type used for cost estimates and geotechnical data.
- I. A preliminary total structure cost estimate for each span arrangement studied.
- J. Note proposed right of way limits, construction easements, and future maintenance operations.
- K. A detailed discussion documenting the Designer's recommended bridge layouts, to be advanced to the Combined TS&L phase, and the reasons for their selection. In most cases, the Bridge Designer is encouraged to recommend advancement of at least one concrete and one steel superstructure alternate unless only one superstructure material has been deemed feasible as part of the Span Arrangement Study. However, more than two alternates may be advanced.

The decision on what alternates are advanced is based on several factors:

- A. Cost of each alternate (cost differences of 10% or less at this stage are considered to be the same cost).
- B. Future maintenance costs.
- C. Aesthetics (when applicable).
- D. Environmental concerns.
- E. Constructability.

- F. Redundancy.
- G. Additional construction costs inherent of complex structures.

All of these factors shall be taken into consideration, with present cost being the primary concern for most projects, to determine the recommended alternates. The ultimate decision is made by the Bridge Project Manager.

1110.4-TYPE, SIZE AND LOCATION (TS&L)

Preliminary superstructure and substructure plans shall be submitted to the Engineering Division for approval of the recommended alternate prior to proceeding with final bridge design and the Final Detail Plan Submission. Engineering judgement should be used to determine the necessary level of accuracy on a project specific basis.

Based on the approved Span Arrangement Study and approved Preliminary Field Review Plans, various alternates shall be studied to determine the most suitable structure. Consideration shall be given to all superstructures advanced based on economics, serviceability, aesthetics, maintenance, and future use. Each alternate shall be developed equally. Recommendations should not be made until all information is complete for each alternate.

The specified grade(s) of steel and/or concrete strengths used in the design will be as determined through performance requirements, availability, and initial and long-term costs. Comparative studies will be completed during the TS&L Study. Results and recommendations will be provided in a clear and concise format, providing sufficient justification for the recommendations made.

At the time of selecting the preferred material type, life cycle costs may be considered. This consideration may include initial cost as well as expected maintenance cost and service life. For routine structures, this evaluation should be very informal, and it will be the decision of the Bridge Designer to recommend the appropriate superstructure type based on experience and knowledge of the site.

It is recommended that two superstructure types be considered for most bridges in the TS&L stage. However, with the approval of the Bridge Project Manager, some smaller bridge projects may require the consideration of only one alternate.

The following list describes some of the information that should be included in the Combined TS&L Submission. See DD-202 for a detailed list.

- A. Key dimensions for superstructure and substructure. These dimensions should be very close to the final plans.
- B. Final grading.
- C. Deck drainage type.
- D. Design data - live load, future wearing surface, method of design, etc.
- E. Type of expansion joints (if applicable).
- F. Type of bearings and justification for their use.
- G. Detailed hydraulic study and design, including the scour analysis and design form DS-34 shall be submitted as follows:
 - 1. For District Design - Submit copy at TS&L stage to the District Bridge Engineer.
 - 2. For Engineering Division In-House Design - The original copy of DS-34 will be submitted to the District Bridge Engineer.
 - 3. For Consultant Design - The DS-34 form shall be submitted to the Engineering Division, Consultant Review Section.

- H. Discussion and full description of the various superstructure and substructure types that were considered. This shall include a discussion of any structure types that were not considered and the reason for elimination.
- I. Any proposed proprietary components shall be investigated for applicability.
- J. A detailed total structure cost estimate for each appropriate superstructure/substructure combination to be developed in the Final Detailed Plan Submission.
- K. A detailed discussion of the advantages and disadvantages of the various structure types studied.
- L. The recommended superstructure and substructure to be developed in the Final Detailed Plan Submission. The ultimate decision shall be made by the Bridge Project Manager.
- M. A complete list of all project specific Special Provisions.
- N. Discussion of external stability for walls, embankment, and temporary works.

1100.4.1-Geotechnical Report: The Geotechnical Report shall be submitted with the TS&L Submission. It shall contain an evaluation of the recommended foundation type(s) (piling, spread footings, rock socketed drilled shafts, etc.) and pile tips and bearing elevations. It shall also include the background information used, boring logs, core photos, subsurface information, test results, assumptions made, calculations and analysis, and a discussion on the foundations recommended for final design.

External influences (e.g., slope stability, settlement, mining, and scour) must be addressed, and remedial measures must be recommended when required. Potential problems, which might require more information or further consideration before final design or during construction, should also be cited.

Bearing resistance for spread footings and drilled shafts shall be calculated using either the general bearing resistance equation or the lower bound equation presented in the governing specifications. The selection of the general or lower bound equation shall be based on the Rock Mass Rating (RMR) either being less than or greater than 50, respectively. A resistance factor of 0.45 shall be used for both equations. Other bearing resistance equations may be allowed by the Geotechnical Engineer when a demonstrated need is presented to the division.

Lateral earth pressure coefficients shall be included within the report and their application to fixed and movable abutments and wingwalls shall be discussed.

Settlement analyses shall consider preconsolidation pressure for cohesive soils and the report shall discuss whether the soil is considered overly or normally consolidated and why.

Where significant cut or fill slopes are associated with a bridge project, the geotechnical reports shall provide slope and benching recommendations.

1110.5-Final Detail Plans: The Final Detail Plan Submission shall be submitted to the WVDOT for approval prior to submission of the PS&E Plans for comments and approval. At this stage, the plans and design shall be one hundred percent (100%) complete. They should be ready to go to contract if no comments are given.

The following list describes some of the information that should be included with the plan submission. See DD 202 for a detailed list.

- A. Plans in 11 inch x 17 inch format.
- B. Detailed cost estimate.
- C. All applicable special provisions.

- D. Contract completion time chart.
- E. Reference to standard bridge and roadway drawings.
- F. Copy of TS&L comments and actions taken.
- G. Final Hydraulic Report.
- H. Table of loads and factors to be used for bridge rating purposes. See DD 202 for bridge rating checklist

All plans shall be developed in accordance with SD 3000 thru SD 3050 of this manual.

1110.6-PLANS, SPECIFICATIONS AND ESTIMATES (PS&E)

The PS&E Submission is a project level submission, where the bridge plans are incorporated into the roadway plans.

This submission will be used to advertise the project for construction; it may be referred to as the PS&E Package. It shall include all revisions requested at the Final Plan Submission stage.

Plans include the following:

- A. Final plans in 11 inch x 17 inch format.
- B. Standard Details. See SD 3000 thru SD 3050.
- C. Contract Plans. See SD 3000 thru SD 3050.
- D. Right of way plans. See DD 301.
- E. See the following DDs for further information on plans.
 - 1. 701 – Plan Presentation.
 - 2. 702 – Title Sheet Signature Block.
 - 3. 703 – Plan Revision Blocks.
 - 4. 704 – General Notes.
 - 5. 705 – Quantities.
- F. Full size mylar of title sheet, stamped by engineer registered to perform design work in West Virginia.
- G. Electronic plans, if requested.

Specifications include the following:

- A. Standard Specifications. See SD 3000 thru SD 3050.
- B. Supplemental Specifications. See SD 3000 thru SD 3050.
- C. Special Provisions. See 3000 thru SD 3050.

Estimates include the following:

- A. Contract Time Determination. See DD 803.
- B. Engineer's Cost Estimate. See DD 707.

A more detailed description of these components can be found in DD 706.

In order to assure consistent PS&E Packages, the Project Manager is responsible for the inclusion of the PS&E Checklist in the PS&E Package.

1110.6.1-Establishing Contract Duration: The Designer is responsible for the development of a contract duration time bar chart. This chart is to be included in the submittal of a project for PS&E to determine the contract completion date. As a guide in this process, the WVDOH has

published the *Guidelines for Production Rates and Chart for Contract Duration* (DD-803). This document is to be used as a starting point in determining contract duration, and should be augmented with outside sources of information, past experience with work of a similar nature, and good engineering judgment.

1110.6.2-Engineer's Cost Estimate: The Engineer's Cost Estimate shall be an accurate reflection of the anticipated costs of the various items of work that are contained in the construction project. It is the responsibility of the Designer to provide this with the PS&E Package. DD 707 details the requirements for the Engineer's Cost Estimate. In addition, the WVDOH annually compiles a book of Average Unit Bid Prices for the previous year's construction projects. This publication can be used to begin the estimating process for any individual items of work that are to be utilized on a project.

It is advisable to consult past projects of a similar nature and scope near the project that is being estimated to arrive at realistic cost data.

1110.7-TRACINGS

This submission includes the reproducible set of tracings, design calculations and electronic plans, if requested.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2010

DESIGN CRITERIA

May 4, 2022

First Edition

2010.1-LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

All structure designs shall be in accordance with the latest edition (including interim specifications) of the AASHTO Load and Resistance Factor Design (LRFD) Specifications, hereafter referred to as the Governing Specifications or AASHTO.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2012
LOADS AND LOAD FACTORS**

*May 4, 2022
First Edition*

2012.1-LOADS

The Designer must consider all loads that are expected to be applied to the structure. These loads shall include but not be limited to permanent loads, live loads, water loads, construction loads, wind loads, ice loads, earthquake effects, earth pressure, vehicular collision force, force effects due to superimposed deformations, friction forces, and vessel collision forces. These loads shall be in accordance with Section 3 of the Governing Specifications, unless specified otherwise within this document.

The Owner's decisions on various design criteria are listed herein.

2012.1.1-Permanent Loads: Permanent loads shall include dead loads due to the weight of all structural components including future wearing surface, earth surcharge (as applicable) and horizontal earth pressure.

The Designer shall use a load of 15 PSF for permanent deck forms where used. When girder or beam spacing fourteen (14) feet or greater are utilized, the designer shall determine if the 15 PSF for permanent deck forms needs to be increased. All structures shall be designed for a future wearing surface of 25 PSF. Unless a more refined analysis is performed to calculate active earth pressure, the Designer shall use a minimum of 40 PCF for equivalent fluid pressure. See AASHTO 3.11.5.

2012.1.2-Live Loads: All structures shall be designed for the HL-93 live load. Fatigue load frequency, $ADTT_{SL}$ (number of trucks per day in one direction in a single lane over the design life-75 years), shall be provided to the Designer by the Bridge Project Manager. Otherwise, a factor, provided by the Bridge Project Manager, shall be used to reduce the ADTT (number of trucks per day in one direction averaged over the design life-75 years) to a single lane. The design for infinite fatigue life as defined within the Governing Specifications is preferred. Infinite fatigue life shall be used on all critical structures as defined in SD 2012.1.5 or structures meeting the requirements of SD 2012.2C.2 otherwise approved by State Bridge Engineer.

The dynamic load allowance may be reduced for components other than joints, if justified by sufficient evidence, in accordance with the provisions of AASHTO 4.7.2.1, Vehicle- Induced Vibrations). Approval by the WVDOH is required. The dynamic load allowance can be reduced by 50% for timber bridges and wood components of bridges.

2012.1.3-Vehicular Collision Force: Abutments and piers located within a distance of 30.0 FT to the edge of the roadway shall be investigated for collision in accordance with the Governing Specifications. See AASHTO 3.6.5.

2012.1.4-Ice and Snow Loads: Bridge components subject to ice forces shall be designed for these conditions:

- A. Q_{10} elevation
- B. 32 KSF ice load
- C. Six (6) inch ice thickness
- D. If $Q_{50} > 50,000$ CFS, a study to determine the ice forces shall be performed

No special snow loads are required on bridges.

2012.1.5-Earthquake Effects: All bridges in West Virginia are assigned Seismic Performance Zone 1. The Owner shall classify a bridge's importance category for seismic design. These classifications shall be based on the following:

- A. A bridge may be classified as "critical" at the direction of the State Bridge Engineer. A "critical" bridge shall be designed based on a 2,500-year return period event.
- B. All National Highway System bridges are classified as "essential" unless a direct road detour is near the bridge. An "essential" bridge shall be designed based on a 475-year return period event.
- C. All other bridges shall be designed based on a 50-year return period event.

2012.1.6-Force Effects due to Temperature: For force effects due to temperature change, the temperature range for bridges designed in West Virginia, shall be as follows:

- A. Steel or Aluminum: -30° to 120° F
- B. Concrete: 0° to 80° F
- C. Wood: 0° to 75° F

2012.1.7-Vessel Collision Force: Structures subject to vessel collisions in navigable waterways shall be designed based on "regular" criteria.

If barge traffic exists or could exist on a river, as determined by the Bridge Project Manager, a jumbo 3 x 5 barge will be used for collision loads unless otherwise directed.

For further information on vessel collisions, please refer to the AASHTO *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges*.

2012.1.8-Construction Loading: Construction loading shall be in accordance with the Governing Specifications. This loading shall include, but not be limited to, the erection and handling of girders and the effects of deck casting.

2012.2-LIMIT STATES AND LOAD FACTORS

Bridges shall be designed for specified limit states as described in AASHTO 1.3. Each component and connection of the structure shall satisfy the following equation for all limit states:

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n$$

Where:

- η_i = The load modifier factor is a function of ductility factor η_D , redundancy factor η_R and importance factor η_I
- γ_i = load factor from Table 3.4.1.1 of the Governing Specifications
- Q_i = force effect determined from structural analysis
- R_n = nominal resistance determined per the Governing Specifications
- ϕ = resistance factor determined per the Governing Specifications

The ductility factor η_D , redundancy factor η_R and importance factor η_I shall be determined as follows:

- A. Ductility factor $\eta_D = 1.0$ for all structures. Non-ductile components and connections shall not be used.
- B. Redundancy factor $\eta_R = 1.0$, except for components and connections that are found to be fracture critical, use $\eta_R = 1.05$. Where feasible, the Designer is encouraged to design alternate load paths for all members.
- C. Importance factor η_I .
 - 1. $\eta_I = 0.95$ for bridges that meet ALL of the following criteria:
 - a) Length less than fifty (50) feet.
 - b) Design ADT less than five hundred (500).
 - c) Detour capable of carrying all legal traffic less than ten (10) miles or capability of erecting a temporary crossing in less than two (2) days.
 - d) Provides only residential access.
 - 2. $\eta_I = 1.05$ for bridges that meet ANY of the following criteria:
 - a) Total bridge construction cost that exceeds 20 million dollars (\$20,000,000).
 - b) Design ADT greater than 50,000.
 - c) Detour length greater than fifty (50) miles.
 - d) $\eta_I = 1.00$ for all other bridges.

For new bridge design, the Strength II (evaluation permit vehicle) load case is not required.

2012.3-LIVE LOAD DISTRIBUTION FACTORS

For preliminary design, live load distribution factors may be developed using approximate hand calculations based on averages. However, for final design, live load distribution factors shall be calculated per the Governing Specifications, considering variability in span lengths, section properties, etc.

When the line supports are skewed and the difference between skew angles of two adjacent lines of support does not exceed 10° , the bending moments shall be reduced in accordance with AASHTO Table 4.6.2.2.2e-1.

A table of live load distribution factors shall be provided in the plans. The load rating calculations performed in SD 2150 shall also utilize these live load distribution factors.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2013

MATERIALS

May 4, 2022

First Edition

2013.1-CONCRETE

- A. Normal weight concrete shall be used in the design of concrete superstructures and substructures.
- B. The mix design, curing, and placement requirements shall per the Standard Specifications.
- C. Class K or H concrete shall be used in all bridge decks and parapets with a compressive strength of 4,000 PSI.
- D. Concrete compressive strengths for precast beams shall be no more than 8,000 PSI at release (f'_{ci}) with a final compressive strength of 10,000 PSI (f'_c). Precast beams may be designed using high strength concrete with approval of the State Bridge Engineer.
- E. Class B Concrete with a compressive strength of 3,000 PSI should be used for abutments, wingwalls and most piers. Class B Modified concrete with a compressive strength of 4,000 PSI may be used for piers, if required for strength. Concrete strengths greater than 4,000 PSI can be used.

2013.2-REINFORCING STEEL

Reinforcing steel shall be Grade 60 steel conforming to the requirements of AASHTO M 31 with a yield stress, F_y , of 60 KSI and a modulus of elasticity of 29,000 KSI.

2013.3-STRUCTURAL STEEL

Structural steel shall be Grade 50, Grade 50W or High-Performance Steel (HPS) Grade 50W or 70W conforming to the requirements of the AASHTO M270. The use of Grade 36 steel is generally limited to miscellaneous components.

2013.4-PRESTRESSING STEEL

Prestressing steel shall conform to the following specifications:

- A. Strand tendons shall conform to AASHTO M203 Grade 270, low relaxation.
 - 1. Ultimate strength of prestressing steel (f_{pu}) 270 KSI.
 - 2. Yield stress (f_{py}) may be taken as $0.9f_{pu}$ 243 KSI.
 - 3. Apparent modulus of elasticity 28,500 KSI.
- B. Bar tendons (high strength threaded bars) shall conform to AASHTO M275 Grade 150.
 - 1. Ultimate strength of bar (f_{pu}) 150 KSI.
 - 2. Yield stress (f_{py}) may be taken as $0.8f_{pu}$ 120 KSI.
 - 3. Modulus of elasticity 30,000 KSI.

Stress-Relieved strand tendons conforming to AASHTO M203 may be specified with the approval of the Bridge Project Manager.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2014
CURVED SUPERSTRUCTURE DESIGN CRITERIA**

*May 4, 2022
First Edition*

The effects of curvature in determining primary bending moments may be neglected on open cross-sections whose radius is such that the central angle subtended by the span is less than the following:

Number of Beams	Angle for One Span	Angle for 2 or more Spans
2	2°	3°
3 or 4	3°	4°
5 or more	4°	5°

Table 2014.A

When these thresholds are exceeded, specialty computer programs (Grid or Finite Element) shall be used.

Curved bridges shall be evaluated for construction during the design process. The evaluation shall consider erection and handling of the girders, the effects of deck casting and the calculation of camber; properly accounting for deflection due to primary bending and torsion.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2020

DECKS

May 4, 2022

First Edition

2020.1-CONCRETE DECKS

Concrete decks shall be used in all bridge decks. Bridge decks on adjacent box beam structures may be omitted at the Bridge Project Manager's discretion.

2020.2-STEEL GRID DECKS

Permanent open steel grid bridge decks are not permitted. However, open grid may be used, with approval, in such situations as temporary detours, in emergencies where opening the structure to traffic as quickly as possible is of utmost importance, or where the live load capacity of a structure is severely reduced by the weight of the concrete fill. Open grid decks shall be connected to their supporting structures by mechanical fastening.

A steel grid deck may be completely or partially filled with concrete (AASHTO 9.8.2.1). For completely filled decks, provide a 1.5 inch thick structural concrete overfill. Filled and partially filled grid decks shall be connected to their supporting structures through shear connectors or by welding, to transfer shear forces between the adjacent surfaces.

A reinforced concrete slab may be placed on top of an open steel grid deck. Composite action shall be provided between the slab and grid deck, and the grid deck and supporting structure, using shear connectors or other suitable means. AASHTO Section 5, Concrete Structures, shall govern the design of the concrete slab. One layer of reinforcement in each principal direction may be used (AASHTO 9.8.2.4.2).

2020.3-TIMBER DECKS

Glued laminated decks shall normally be used when timber decks are utilized. For panels parallel to traffic, interconnection of panels shall be made with transverse stiffener beams connected to the underside of the glued laminated deck at a spacing not to exceed eight (8) feet. For panels perpendicular to traffic, interconnection of panels may be made with mechanical fasteners, splines, dowels, or stiffener beams.

Nail laminated decks may be utilized on annual plan projects or Category 6 bridges when approved by the Bridge Project Manager.

For decks skewed less than 25°, transverse laminations may be placed parallel to the skew. Otherwise, the transverse laminations shall be placed perpendicular to the supporting members.

Deck panels shall be attached to their supporting component as specified in AASHTO 9.9.4.2. Non-interconnected decks shall only be used on secondary rural roads.

Timber decks shall be provided with a wearing surface (AASHTO 9.9.8).

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2021
DESIGN OF CONCRETE DECK SLABS**

*May 4, 2022
First Edition*

Class K or H concrete shall be used in all bridge decks and barriers conforming to SD 2013.1. The bridge deck width shall not be less than that of the approach roadway section.

The Empirical Design Method for the design of concrete deck slabs (AASHTO 9.7.2) shall be used, provided all required design conditions are met (AASHTO 9.7.2.4). If the Empirical Design Method is not applicable, then the deck shall be designed using the Traditional Method (AASHTO 9.7.3).

The following information is for monolithic concrete bridge decks:

- A. The minimum allowable thickness is eight (8.0) inches.
- B. One (1.0) inch clearance at the bottom of the slab.
- C. 2.5 inches clearance at the top of the slab.
- D. Two (2.0) inches clearance elsewhere.

The following information is for bridge decks utilizing a specialized concrete overlay (SCO):

- A. The minimum allowable thickness is 8.5 inches.
- B. Two (2) inch composite SCO.
- C. One (1.0) inch clearance at the bottom of the slab.
- D. Three (3.0) inches clearance at the top of the slab.
- E. Two (2.0) inches clearance elsewhere.

The top ¼ inch of slab is to be considered a wearing course and shall not be included as part of the structural capacity of slab. The deck haunch shall be vertical from the flange tips.

For bridges with a skew angle of 25° or less, the transverse reinforcement may be placed parallel to the skew where economy warrants; otherwise, it shall be placed perpendicular to the bridge centerline. For bridge decks skewed greater than 25°, additional reinforcement shall be added in the end zones of the deck (AASHTO 9.7.2.5).

Concrete Appurtenances (curbs, barriers, sidewalks, etc.) shall be made structurally continuous.

Typical deck details including end zone reinforcement placement are available from the WVDOH.

2021.1-DECK OVERHANGS

The reinforcing for deck overhangs is not included in the Empirical Design Method. The WVDOH has prepared deck overhang design drawings for our standard barriers referenced in SD 2020. As long as the deck overhang meets the dimensional criteria shown in these drawings, the reinforcement is adequate, and no further design work will be required. If the deck overhang

does not meet this criteria or an alternate crash tested barrier is employed, the overhang shall be designed for all loads including the railing impact loads of AASHTO 9.7.1.5 and the provisions of AASHTO 9.7.2 and detailed to be compatible with the empirical deck system. For bridges with structurally continuous concrete barriers, the minimum total overhang width shall be 3.0 times the depth of the deck, measured from the center of the exterior girder (AASHTO 9.7.2.4). The maximum total overhang width shall be the smaller of 0.625 times the girder spacing and six (6) feet.

Plan notes shall be added requiring the contractor to ensure that precautions have been taken to prevent permanent lateral deformations of the exterior girder, caused by overhang brackets used during the deck pouring operation. See Section 615.2.2 of the WVDOT Specifications.

2021.2-REMOVABLE FORMS

Removable formwork on interior bays is not permitted. When removable forms are to be used, approval of the forming plan will not be required, except when the slab overhang at the fascia girder is greater than two (2) feet from the edge of the flange.

2021.3-PERMANENT METAL DECK FORMS

All bridge decks shall be detailed with permanent metal deck forms, unless the State Bridge Engineer approves another type.

When corrugated stay-in-place forms are used, the design depth is taken as the minimum concrete thickness. Fill corrugations with concrete. The use of foam in the corrugations of stay-in-place formwork is prohibited. Deck forms shall be mechanically tied at common edges and fastened to their support. No welding of steel formwork is permitted. Steel formwork shall not be considered composite with the concrete slab.

2021.4-DECK PROTECTION CRITERIA

All reinforcing in the slab, barriers, medians, curbs and sidewalks shall be epoxy coated, except when alternate protection systems are approved by the State Bridge Engineer.

A dual protection deck system shall be provided for all concrete bridge decks on projects meeting either of the following criteria:

- A. Design ADT greater than 3,500 vehicles per day (VPD).
- B. National Highway System (NHS) bridge.

The dual protection shall be obtained by utilizing a Class H full depth concrete deck on all bridges with a maximum span length less than or equal to 350 feet.

All bridges with spans greater than 350 feet shall utilize a deck system with a specialized concrete overlay in combination with a Class K concrete deck. The overlay is placed after most of the dead load deflections have taken place, thus providing better control over the final elevation of the concrete deck.

To provide necessary information to field personnel in constructing specialized concrete decks and to help prevent rideability problems, Bridge Designers are required to:

- A. Provide cambers and deflections for stringers and floor beams in the contract documents.
- B. Place an overlay on the deck whereby the overlay thickness is part of the three (3.0) inches minimum clearance over the reinforcing steel bars.

2021.5-DECK DRAINAGE

Provide sufficient inlets to drain a 2-year design discharge from the highway or bridge surface. For roadways designed for speeds of 40 MPH or greater, spread of the flow on a bridge deck or curbed section of pavement is generally limited to five (5) feet or the shoulder width, whichever is greater. If the design speed is less than 40 MPH, spread is generally limited to five (5) feet, or the shoulder width plus two (2) feet into the traveled way, whichever is greater. If a parking lane is present, then the spread will be limited to eight (8) feet.

FHWA publication *Design of Bridge Deck Drainage*: Hydraulic Engineering Circular No. 21 shall be utilized for deck drainage design methodology. Deck drains shall be placed, as required, based on the hydraulic computations for the subject bridge (AASHTO 2.6.6.2). The design storm for deck drainage shall be as described in the preceding paragraph. Deck drainage systems shall be designed and sized to remove surface water efficiently and quickly from bridge decks and direct it away from bridge superstructure and substructure elements.

The drainage system shall be designed to be accessible for cleaning.

All components of the deck drainage system shall be of similar color to their adjacent bridge component. Unpainted galvanized components are acceptable adjacent to concrete superstructures and substructures.

Steel components of bridge drain systems shall be galvanized except as noted. Galvanized components will include but not be limited to: scuppers, grates, drain pipes, pipe connectors, clamps, bolts, nuts, and washers.

Galvanizing will be specified to be hot dipped galvanizing in accordance with AASHTO Specification M 111 or M 232 as appropriate.

Fiberglass deck drainage systems may be used with permission from the Bridge Project Manager. Fiberglass components shall also be of similar color to their adjacent bridge component.

On Interstate bridges where pedestrian and bicycle traffic are prohibited, drainage grates shall provide openings parallel to the roadway centerline. Grates shall be designed and detailed to be bicycle and pedestrian safe on all other facilities where drains are required and where pedestrian and bicycle traffic are not prohibited.

2021.6-DECK SURFACE PREPARATION

All concrete bridge decks shall be given a groove finish as described in Section 601.11.4 of the Standard Specifications.

2021.7-SIDEWALKS

The curb height for raised sidewalks on a bridge should be no more than eight (8) inches. If a barrier curb is required, the curb height should not be less than six (6) inches. If the curb height on the bridge differs from the adjacent roadway, it shall be uniformly transitioned over a distance greater than or equal to 20 times the change in height (AASHTO 13.11.2). This transition shall occur outside the bridge limits, normally on the approach slab.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2022

BARRIERS

May 4, 2022

First Edition

All new or replacement bridge barriers shall meet or exceed the following criteria:

- A. Test Level (TL)-3, when any of the following conditions apply:
 - 1. National Highway System (NHS) Bridge.
 - 2. Design speed greater than 45 MPH.
 - 3. Design ADT greater than 3,500 VPD.
 - 4. Deck type is concrete slab on girders.
- B. TL-2, for all other bridges:
 - 1. Design speed must be less than 45 MPH to use a TL-2 barrier.
- C. TL-1, where there is an exceptionally low volume of traffic on a twelve (12) foot wide one lane bridge an exception for use of a TL-1 barrier may be considered if all of the criteria listed in DD-601, *“Conditions for one lane 12’ clear bridge widths on new construction of new roads”* has been met.

The 32 inch Type F barrier is the standard barrier for all new and replacement projects, utilizing a TL-3 barrier. The Designer should note that the 32 inch Type F barrier meets TL-4 requirements. As with all railings, the attachment and supporting elements shall be designed to exceed the strength capacity of the barrier, per AASHTO Section 13. The 42 inch barrier may be specified for special projects based on geometric constraints. If there is a bicycle path adjacent to the barrier, the overall barrier height shall be 54 inches (including railing). Details for these barriers can be obtained from the WVDOH.

Sidewalk barriers shall be designed in accordance with AASHTO Section 13, Railings and Section 2.3.2.2.2. Sidewalk barriers subject to vehicular collision shall meet crash test requirements (AASHTO 13.11.1).

The barrier is constructed without vertical construction joints but is vertically scored for control joints. Longitudinal reinforcement shall be continuous.

The Type F barrier transition and guardrail attachment details can be obtained from the WVDOH. This transition shall occur outside the limits of the bridge.

2022.1-CONTINUOUS BARRIERS FOR DEFLECTION CONTROL OF BRIDGES

Generally, for short to medium span bridges, the AASHTO suggested limits for live load deflection may not govern the design of the main structural members. However, for medium to long span bridges and bridges with HPS 70W steel girders, serviceability criteria, such as live load deflection, become increasingly significant when proportioning the main members.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2023
CONSTRUCTION SEQUENCES**

*May 4, 2022
First Edition*

A deck placement sequence shall be provided on the bridge plans for all multiple span bridges with a cast in place concrete deck. See Figure 2023.A. The purpose of the placement sequence is to reduce deck cracking over the piers. In general, the recommended deck placement sequence shall specify that all positive moment regions of the deck shall be placed first. The negative moment areas shall not be placed until the positive moment regions have reached the minimum strength required in the Standard Specifications. The positive moment region pours shall be limited to a length that can be completed prior to the initial setting of the concrete. If special structural situations warrant, consideration may be given to an alternate deck placement sequence. The bridge superstructure shall be analyzed for the proposed deck placement sequence.

Under some circumstances, the deck may be placed in one continuous pour. These circumstances include, but are not limited to, single span bridges or multi-span bridges of a length that can be placed in one casting operation. On multi-span bridges, the initially placed concrete must remain plastic during the entire casting operation.

The Designer shall be aware of, and take into consideration, the proximity of concrete facilities, pour volumes, and quantity of concrete a contractor can place when determining the deck placement sequence.

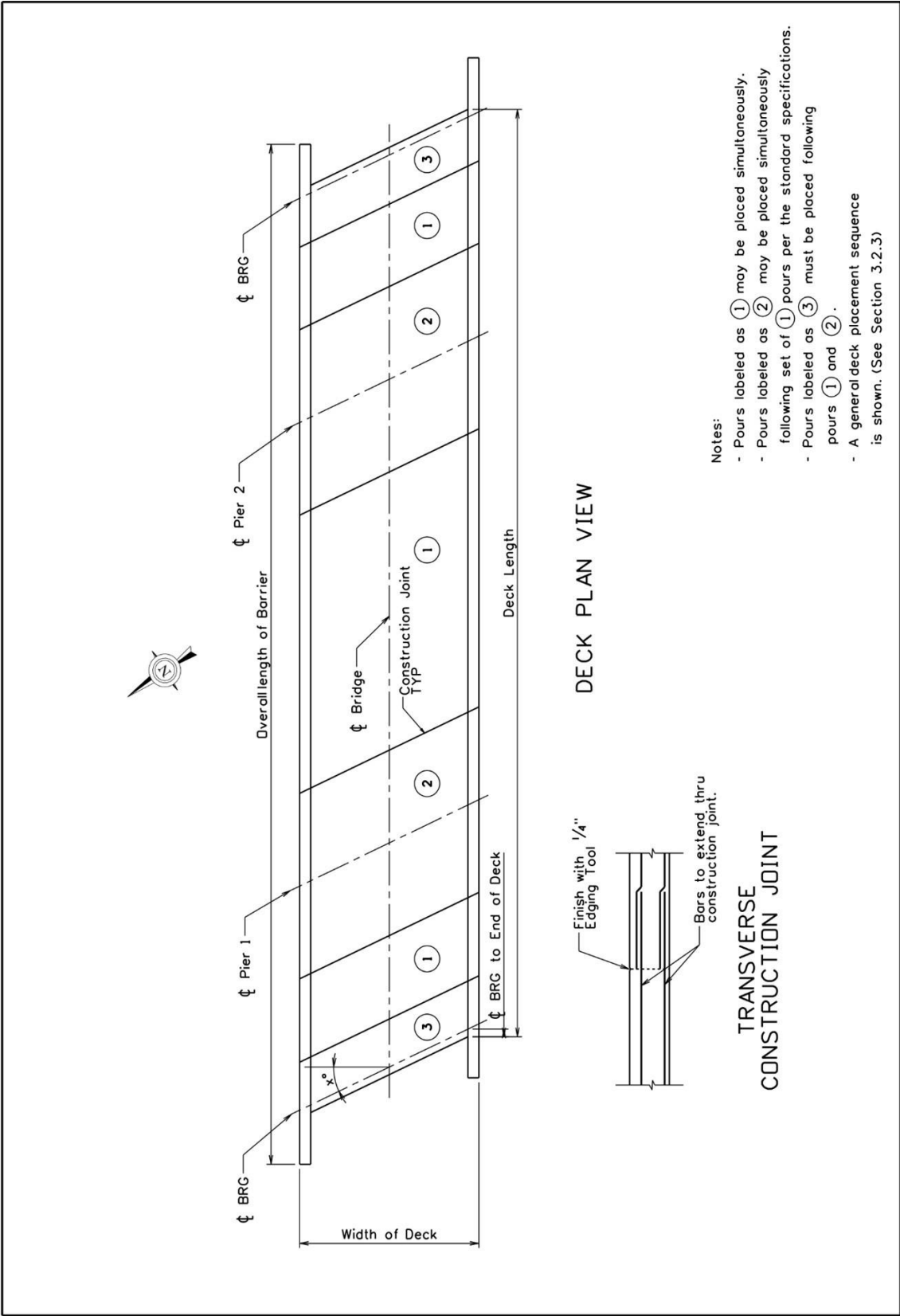


Figure 2023.A

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2030
STEEL SUPERSTRUCTURE**

*May 4, 2022
First Edition*

The Designer is not limited to the span-to-depth ratios provided by the Governing Specifications.

The load carrying capacity of exterior beams shall not be less than the load carrying capacity of interior beams.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2031

MATERIAL PROPERTIES

May 4, 2022

First Edition

2031.1-SPECIFYING GRADE, TOUGHNESS LEVEL AND SPECIAL PROVISIONS

Typical steel grades considered for fabrication of bridges are as specified by the Governing Specifications. Grade 100 and HPS 100W are not permitted for bridge structures unless approved by the State Bridge Engineer.

2031.2-GRADE

The specified grade(s) of steel used in the design will be as determined through performance requirements, availability, and initial and long-term cost. Comparative studies will be completed during the TS&L study. Results and recommendations will be provided in a clear and concise format, providing sufficient justification for the recommendations made.

2031.3-TOUGHNESS

Fracture toughness requirements for all main components and connections sustaining tensile forces shall be identified on the contract drawings. Toughness requirements shall conform to Zone 2 Charpy V-Notch impact test requirements of AASHTO M 270.

2031.4-SPECIAL PROVISIONS

Special provisions shall be prepared for materials, fabrication or construction not specifically addressed in the Governing Specifications.

2031.5-LIMITATIONS OF GRADE SELECTION

The use of high-strength steel is desirable when strength is the major design criterion. Historically, when deflection or other service criteria governed, the use of Grade 36 steel was preferred. However, the cost differential between high-strength Grade 50 steel and Grade 36 steel has narrowed or may even be negligible. Therefore, the added strength of Grade 50 steel is worth the minimal cost premium.

The use of HPS 70W steel shall be considered in the design of all structures. Hybrid girders (configured as described below) are generally most economical for tangent structures. This type of girder utilizes different grades of steel in the beam cross section. Generally, the use of HPS 70W steel in the top and bottom flanges of the negative flexure regions and the bottom flange of the positive moment regions with Grade 50 steel in the top flange of the positive moment regions and in the web throughout is considered to be the most economical design. Other configurations may be studied at the request of the Bridge Project Manager.

The use of uncoated high-strength weathering steel provides significant initial and long-term cost savings and will be considered at TS&L for all projects meeting the requirements of SD 2039, Weathering Steel.

The use of weathering steel requires unique specification requirements for fabrication, painting, and construction techniques. These unique requirements are contained in a set of standard plan notes included in SD 3000, Miscellaneous Plan Notes. The notes were developed to apply to all structure types, including integral bridges, and will require slight modification to suit individual project conditions and designs. Designers must verify that these notes are applicable to their project and do not conflict with other plan notes.

When the use of coated or painted steel structures are specified, see SD 1074 for coating specifications.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2032

DESIGN GUIDELINES

May 4, 2022

First Edition

The following addresses design guidelines that shall be considered during the analysis and detailing of steel bridge structures, as appropriate.

2032.1-WEB PLATES

The minimum web thickness for plate girders is $\frac{7}{16}$ inch. Increment the web thickness by $\frac{1}{16}$ inches minimum. It is generally more economical to maintain a constant web thickness throughout a project. However, web thickness may be varied at field splices, or less desirable, at shop splices. Design partially stiffened webs, defined as webs stiffened near support locations with positive flexure regions largely unstiffened. The Designer shall consult with steel fabricators to determine the most economical location of a splice, and whether or not changing the web thickness will offset the added cost of the additional web thickness.

For long-span bridges, evaluate haunched webs versus constant depth webs. Typically, haunched webs should be fabricated using a straight taper. Parabolic haunches will be considered for aesthetic reasons with approval of the WVDOH. The Designer shall consult with a steel fabricator to determine the additional cost of fabrication.

2032.2-TRANSVERSE INTERMEDIATE STIFFENERS

Transverse intermediate stiffeners are used to increase the shear-buckling strength of girders with slender webs as defined by the Governing Specifications. “Transverse stiffeners shall consist of plates or angles welded or bolted to either one or both sides of the web” (AASHTO 6.10.8.1.1).

For fascia girders, design intermediate transverse stiffeners on the interior side of the web only. For interior girders, intermediate transverse stiffeners may be paired at the same location on opposite sides of the web. See Figure 2032.A for details of transverse stiffeners.

2032.3-DIAPHRAGM CONNECTION PLATES

Connection plates serve to attach diaphragms and cross frames to the girders. Connection plates will be designed to meet the criteria of transverse stiffeners as prescribed in the Governing Specifications and as follows:

- A. Connection plate width will be sized to suit the connection details.
- B. Connection plates will be connected to both the top and bottom flanges to prevent distortion-induced fatigue cracking.
- C. In the absence of more detailed loading information, for tangent non-skewed structures, the attachment of the connection plate to the girder will be designed for a transverse force of 20 KIPS.

See Figures 2032.B and 2032.C for details of connection plates.

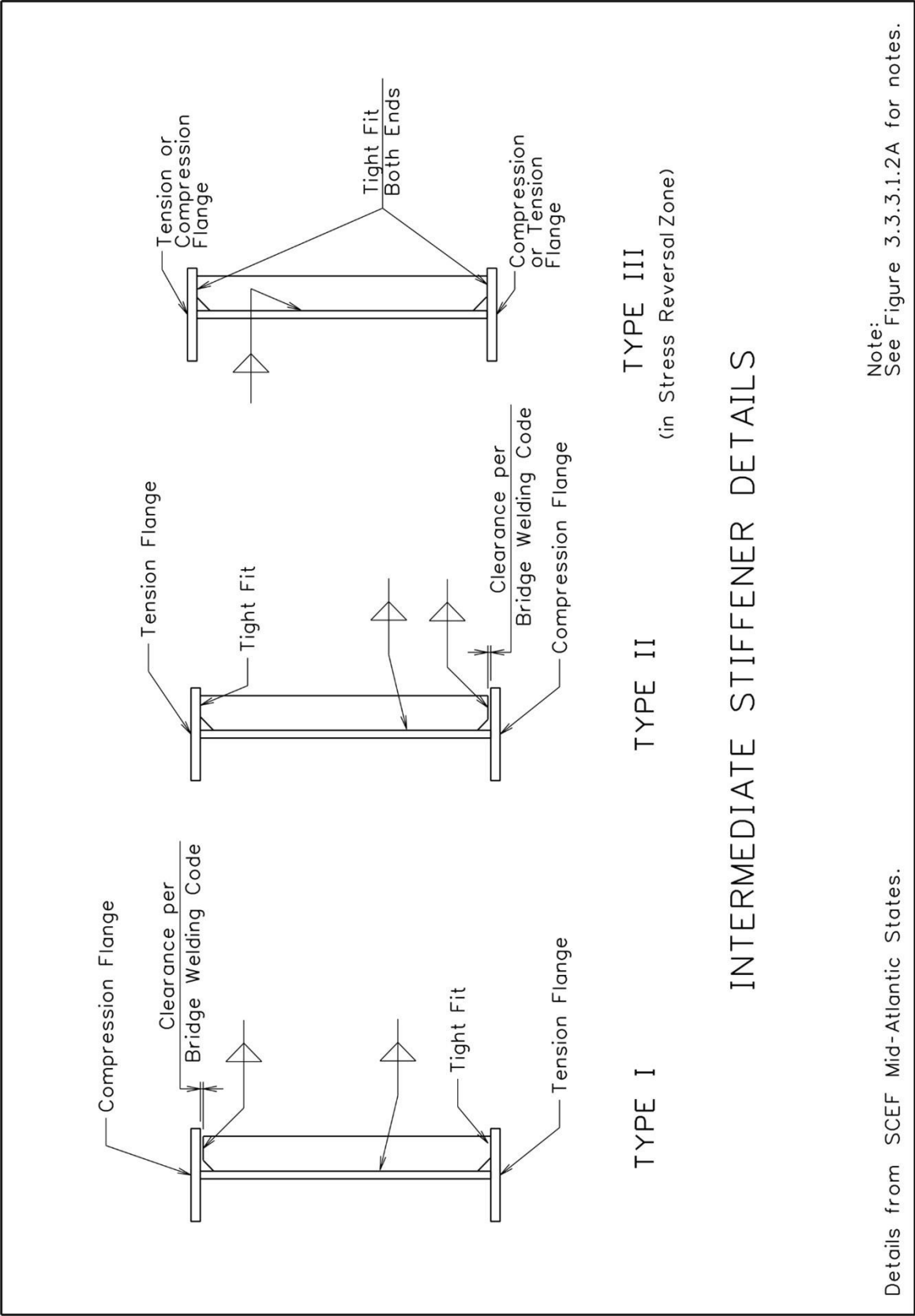
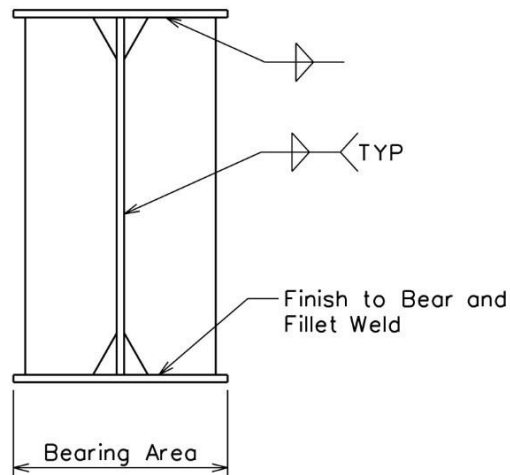
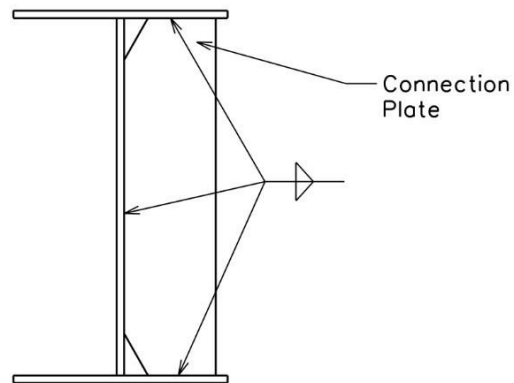


Figure 2032.A

STIFFENER DETAILS



BEARING STIFFENER



CONNECTION PLATE

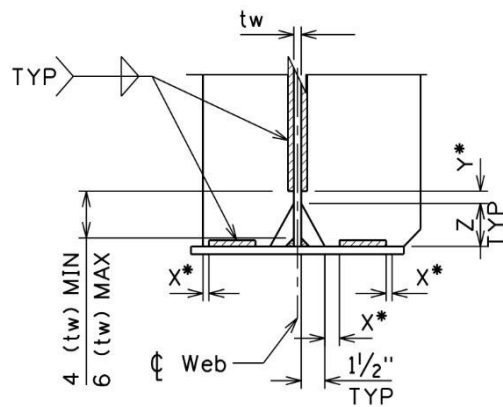
Notes:

1. Stiffener size must be shown on plans.
2. Fillet weld size shall be shown on plans unless minimum weld size as per AASHTO/AWS D1.5 is to be used.
3. If a bearing stiffener is used as a connection plate for cross frames, fillet welds are required.
4. When longitudinal stiffeners are required, place all transverse stiffeners on one side of the web and place the longitudinal stiffener on the opposite side.
5. Details similar for paired stiffeners.

Details from SCEF Mid-Atlantic States.

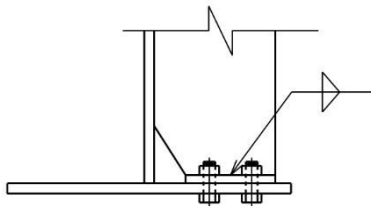
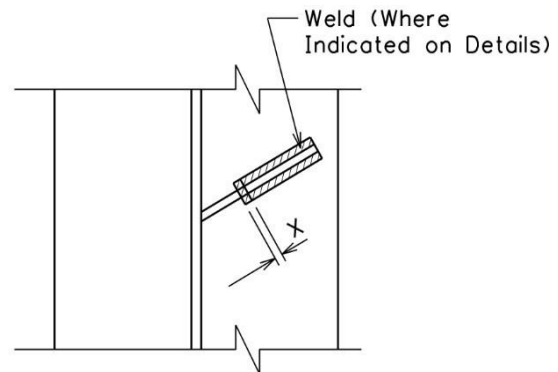
Figure 2032.B

STIFFENER DETAILS

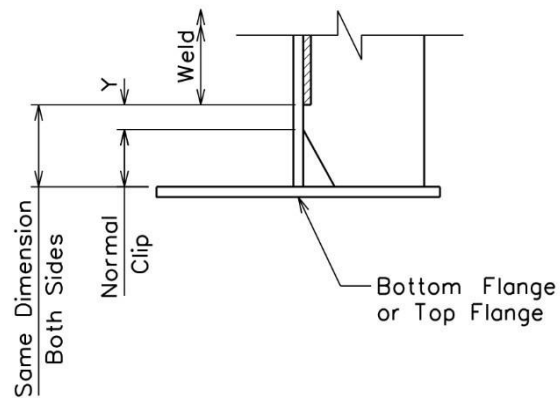


$$\begin{aligned}
 X &= \frac{1}{4}'' \pm \frac{1}{8}'' \\
 Y &= \frac{1}{2}'' \pm \frac{1}{4}'' \\
 Z &= \begin{cases} 2\frac{1}{2}'' & \text{for } \frac{1}{2}'' \text{ web} \\ 3'' & \text{for } \frac{9}{16}'' \text{ web} \\ 4'' & \text{for } \frac{3}{4}'' \text{ web} \end{cases} \\
 * &= 0'' \text{ for groove weld}
 \end{aligned}$$

DETAIL AT END
OF STIFFENER
OR CONNECTION
PLATE



ALTERNATE DETAIL
AT TENSION FLANGE
WHERE STRESS RANGE
EXCEEDS CATEGORY C



STIFFENER
WELDING
DETAIL

For Skewed Stiffeners

Details from SCEF Mid-Atlantic States.

Figure 2032.C

2032.4-BEARING STIFFENERS

Bearing stiffeners are used to resist bearing reactions and other concentrated loads, either in the final state or during construction. Bearing stiffeners are to be placed on the web of girders at all bearing locations and other points of concentrated loads. “Bearing stiffeners shall consist of one or more plates or angles welded or bolted to both sides of the web. The connection to the web shall be designed to transmit the full bearing force due to the factored loads” (AASHTO 6.10.8.2.1). The plates, or outstanding legs of the angles, should extend as close as practical to the outer edges of the bottom flange. The stiffeners shall be finished to be tight fit against the bottom flange and fillet welded. See Figures 2032.B and 2032.C for details of bearing stiffeners.

2032.5-AUXILIARY STIFFENERS

Auxiliary stiffeners are partial depth bearing stiffeners used at bearing locations to reinforce the girder web and ensure full bearing of the girder over the bearing device. Auxiliary stiffeners are positioned and sized to resist a portion of the total reaction based on the projected area of the bearing device. The resistance of the auxiliary stiffeners is disregarded when designing bearing stiffeners, defined in 3.3.2.1.3, for the full bearing reaction in accordance with the Governing Specifications. The length and connection of auxiliary stiffeners to the girder web will be sufficient to transmit the design force into the girder web. See Figure 2032.D for details of auxiliary stiffeners.

2032.6-LONGITUDINAL STIFFENERS

Longitudinal stiffeners increase the resistance to flexural buckling of the girder web. Longitudinal stiffeners should consist of a welded plate or bolted angle positioned longitudinally on one side of the web. Longitudinal stiffeners are not permitted without the permission of the State Bridge Engineer.

2032.7-JACKING STIFFENERS

Jacking stiffeners are installed at future jacking locations to reinforce the girder web.

The design of jacking stiffeners for new structures shall be in accordance with the AASHTO provisions for bearing stiffeners. For rehabilitation projects, the detail length and connection of jacking stiffeners to the girder web will be at a minimum sufficient to transmit the design jacking force into the web.

2032.8-FLANGE PLATES

The minimum flange plate size is $\frac{3}{4}$ inch thick by ten (10) inches wide. Refer to the Governing Specifications for minimum plate sizes with respect to handling, transportation and erection considerations.

Typically, no more than one flange plate thickness transition is economical between interior support locations and field splices. For some long-span girders, bottom flange transitions may prove economical in positive flexure regions. All transitions must be evaluated for economy during the design process. The Designer shall consult with fabricators to gain up-to-date cost factors for design. Items to consider for the design of flange plates:

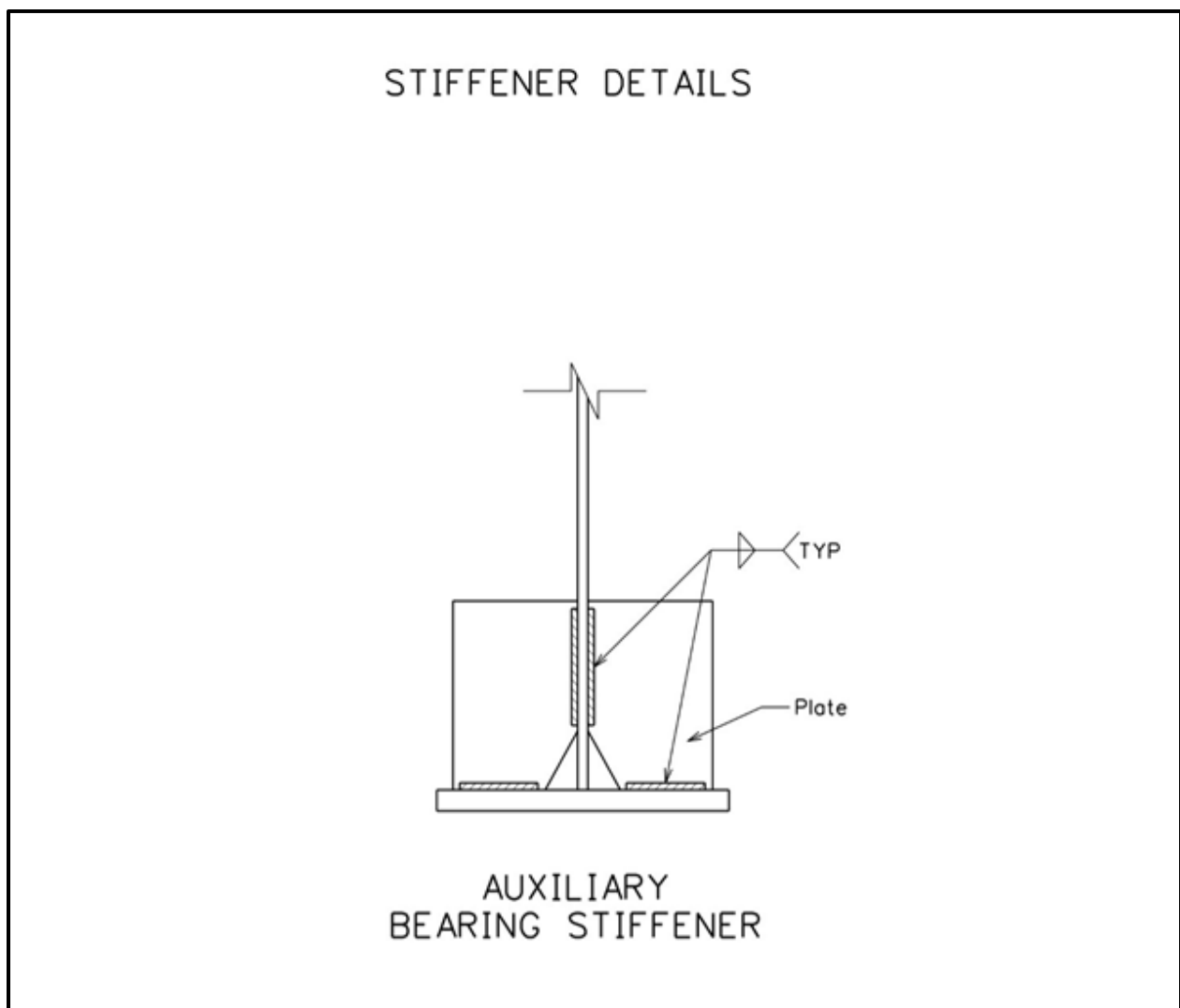
- A. Maintain constant-width flanges within field sections.
- B. Group flange sizes during design and locate flange transitions for adjacent girders at equal distance from support locations to facilitate fabrication of multiple girders.

- C. Minimize specifying plates of different thickness in girder details. Whenever possible, use common plate sizes throughout the girder.
- D. Do not reduce the cross-sectional area at flange transitions by more than one half the area of the larger flange plate.
- E. Avoid bottom flange lateral bracing for wind whenever possible.
- F. Consider making the top flange width the same for entire length of structure to reduce deck framing costs.

2032.9-ERECTION REQUIREMENTS

The Designer shall provide the anticipated erection scheme, which is to be consistent with the project's permitting.

SEE SD 3000 for Erection Plan Notes.



NOTE: SEE Figure 2032.B for notes.
SEE Figure 2032.c for dimensions.

Figure 2032.D

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2033

FATIGUE

May 4, 2022

First Edition

Fatigue shall be considered in the design of a new structure, reconstruction, or major rehabilitation of an existing structure. See SD 2012.1.2, Live Loads for fatigue load frequency.

Structural details identified as Detail Category D, E, or E' by AASHTO are not acceptable for the design of fatigue susceptible regions of new structures.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2034
NONREDUNDANT STEEL TENSION MEMBERS**

*January 17, 2023
Supersedes May 4, 2022*

As recommended by the Governing Specifications, the design of new structures will employ continuity or redundancy to provide one or more alternate load paths. Where the use of nonredundant steel tension members (NSTM) is unavoidable and approved by the WVDOH, the NSTM should be clearly designated on the contract drawings with the appropriate tension zones indicated and shall be fabricated according to Section 12 of AASHTO/AWS D1.5M/D1.5: current version, *Bridge Welding Code*.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2035
DIAPHRAGMS, CROSS FRAMES AND LATERAL BRACING**

*May 4, 2022
First Edition*

Cost-effective design of steel bridges involves particular attention to the design of the details. Cross frames and diaphragms are a significant example. Cross frames and diaphragms typically account for a small percentage of the total structure weight of steel. Nevertheless, due to labor-intensive fabrication, they usually are a significant percentage of the total erected steel cost. The need for, and location of cross frames and diaphragms shall be carefully evaluated. Cross frame spacing is to be established by rational analysis for all stages of construction and the final condition. Diaphragms, in lieu of cross frames, shall be used on beams less than 36 inches.

Designer shall consult AASHTO/NSBA for cross frames and diaphragms layout. For regions near skewed supports, cross frame forces can become large due to significant differential deflection of adjacent girders. The Designer shall investigate the elimination of specific cross frames near skewed supports to relieve “stiffness” of the framing as needed to reduce cross frame forces.

In order to mitigate distortion and fatigue-prone details on heavily skewed bridges (30° or more), slot the diaphragm or cross frame connection holes. These connections shall not be completely tightened until all of the superstructure concrete is placed.

For analysis using refined methods, the cross frame members will be included in the structural model and subsequently be designed for the forces generated by the analysis. The force effects considered in design shall at a minimum include dead load, live load, wind load and, as required, thermal forces. When approximate methods of analysis are used, dead load, live load, and thermal forces cannot be easily obtained. The cross frame members will be designed, at a minimum, for wind loading and slenderness requirements as specified by AASHTO. When cross frames are designed members, they then become subject to the same fracture toughness testing as the main members. As discussed previously, cross frame forces near the supports of skewed bridges can be significant in magnitude. Careful evaluation is needed regardless of the analysis method used.

Cross frames may be K-type or X-type. K-type cross frames will be used when the ratio of the girder spacing to the girder depth is greater than 1.5:1. K-type cross frames will be oriented with diagonals connecting to the mid-point of the bottom strut. At support locations, K-type diaphragms will be used, except for the purposes of jacking, where solid plate diaphragms are needed. K-type diaphragms at end supports will be oriented with the diagonals attached to the mid-point of the top strut. Generally, for aspect ratios less than 1.5:1, X-type cross frames should be considered. Top struts and/or bottom struts in either type may be eliminated only when accompanied by an analysis and plan showing that adequate flange support will be provided during construction.

Member selection is based upon the design forces. For highly loaded cross frame members in compression, WT sections connected through the flange usually provide an efficient section, otherwise use single angles. The designer should consult with fabricators prior to the selection of

member types to avoid specifying shapes not readily available. Since significant premiums are levied on “non-stocked” shapes, the least weight shapes are not necessarily the least costly.

The AASHTO Standard Specifications (LFD) do not accurately address the behavior of single angles subjected to combined flexure and compression. When designing using the LRFD Method, the Governing Specifications (commentary to Article 6.12.2.2.4b) refers designers to the AISC *Specification for Load and Resistance Factor Design of Single Angle Members*, contained in the AISC *Load and Resistance Factor Design Manual of Steel Construction*, (AISC LRFD). Therefore, design of single angle members will be in accordance with the AISC LRFD Specifications.

Permanent steel end diaphragms or cross frames shall not be used in conjunction with integral or semi-integral abutments that utilize permanent concrete end diaphragms. Temporary bracing may be required to ensure a stable system while the deck slab is placed.

The need for lateral bracing shall be investigated for all stages of the assumed construction sequence. Girder flanges attached to the decks with sufficient rigidity to brace the flange will not require lateral bracing. The wind load stresses in steel girders can be reduced by either changing the flange size, reducing the diaphragm or cross-frame spacing or by adding lateral bracing in the exterior bays. See Section 6.7.4 of the Governing Specifications for additional information.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2036

CONNECTION DESIGN

May 4, 2022

First Edition

Design of connections shall be in accordance with the Governing Specifications. The connection shall be designed for moment, shear, and tension as applicable. Refer to the Governing Specifications for loads and load combinations. Connections should be detailed as simple as practical, using a minimum number of fasteners and/or weld length.

2036.1-WELDED CONNECTIONS

When designing welded connections, the following shall be considered:

- A. No field welding is permitted unless approved by the WVDOH.
- B. Use fillet welds wherever possible. The minimum fillet weld size is $\frac{1}{4}$ inch for materials up to and including $\frac{3}{4}$ inches thick, and $\frac{5}{16}$ inch for materials over $\frac{3}{4}$ inches. Fillet weld sizes should generally be limited to a maximum of $\frac{5}{16}$ inch in order to allow for single pass welds in fabrication.
- C. Minimize the fit up requirements for the connection using pre-qualified joints per Bridge Welding Code.
- D. Avoid the use of partial and full-penetration welds. If required, specify them as partial joint penetration (PJP) or complete joint penetration (CJP) respectively on the contract drawings to permit the Fabricator to select a least cost pre-qualified joint for approval. PJP joints require appropriate indication of minimum effective weld throat as per design requirements.
- E. Unless specifically required for strength, full penetration welds should not be specified except for splicing flanges, webs, longitudinal stiffeners, and any other member subjected to live load and wind load tension and flexural stresses.

2036.2-BOLTED CONNECTIONS

When designing bolted connections, consider the following:

- A. Use high-strength AASHTO M 164 bolts, minimum $\frac{7}{8}$ inch diameter. Other sizes or strengths may be used with the approval of the Bridge Project Manager.
- B. Bearing-type connections shall be permitted only for joints subjected to axial compression and joints on secondary/bracing members (AASHTO 6.13.2.1.2). All other bolted connections shall be designed as slip-critical connections (AASHTO 6.13.2.1.1).
- C. Bolt size, spacing, gage, and edge distance will be in accordance with AASHTO. The Designer should refer to AISC LRFD for general detailing and the requirements for assembly of bolted connections.

- D. Type B faying surfaces shall be specified for slip-critical connections on new structures. Appropriate faying surfaces shall be established for rehabilitation of existing structures.
- E. If the design permits, eliminate gusset plates by bolting cross frame members directly to the stiffeners. If gusset plates are required, shop-weld cross frame members to the gusset plates. Field connections for cross frames shall be bolted.
- F. Stems of WT sections and outstanding legs of angles shall be turned in common orientation to facilitate fabrication of cross frames.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2037
NON-DESTRUCTIVE TESTING (NDT) REQUIREMENTS**

*May 4, 2022
First Edition*

3037.1-NON-DESTRUCTIVE TESTING (NDT) REQUIREMENTS

The Bridge Welding Code (BWC) adequately describes the types of welds (groove, fillet, etc.) that require NDT and specifies the type of NDT (radiographic, ultrasonic, magnetic particle, etc.) appropriate for the type of welds. In general, the BWC only requires NDT for welds on “main members” in the “tension and/or stress reversal” area of the structure. The Designer’s responsibility is limited to assuring that the contract documents adequately describe which members are main members, and adequately identifies the tension and stress reversal areas. When curved, box beam, truss, tied arch, or other complex designs are utilized, the Designer must consider the minimum weld quality requirements and develop a Special Provision which will define NDT requirements for the entire structure.

2037.2-STRAIGHT BEAM AND GIRDER BRIDGES

In general, for these structures, the main members are the longitudinal beams and girders. Diaphragms, cross frames, bearings, expansions dams, drainage, etc. would be considered secondary members and not subject to NDT. Additional plan notes or Special Provisions are not required to identify main members. However, the Designer must assure that plan detail sheets for girder/beam elevation views adequately define the tension and stress reversal areas of the structure.

2037.3-CURVED OR OTHER COMPLEX BRIDGES

Complex bridges include, but are not limited to, box beam, stringer/sub-stringer with floor beams, trusses, tied arches, and heavily skewed bridges. For these types of structures, the members to be considered main members are not readily obvious to all parties. Designers must add appropriate notes to define and identify the main members. All members so designated then become subject to NDT requirements. In addition, the Designer must assure that plan detail sheets adequately define the tension and stress reversal areas of the structure. Additional NDT can be specified for highly stressed areas or members, welds, or members subject to high stress ranges and fatigue, or as otherwise deemed appropriate by the designer, by preparation of extensive plan notes or a Special Provision.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2038
COMPOSITE CONSTRUCTION**

*May 4, 2022
First Edition*

Generally, composite construction shall be used for all single-span and continuous bridges. Non-composite design may be used if it results in a more economical design than composite construction, with approval of the Bridge Project Manager.

For continuous spans, the beams shall be designed for composite action in both positive and negative moment regions. One percent (1%) slab reinforcement shall be used to control flexure cracking throughout the dead load negative moment region. The Designer may elect to design for composite action only in the positive moment region with the approval of the Bridge Project Manager.

Moment redistribution shall be used where appropriate following an elastic analysis.

2038.1-LIMITS OF COMPOSITE REGIONS

Proportion composite sections in single spans and positive flexure regions of continuous spans such that the neutral axis of the composite section lies within the steel member. Concrete in tension will not be considered effective when calculating section properties of the composite section for strength.

In the negative flexure regions of continuous spans, only the deck slab reinforcement is considered to act composite with the girder when calculating section properties for strength.

In positive flexure regions of continuous spans, and in single spans, the deck slab is considered to act composite with the girder when calculating section properties for strength. The deck reinforcing steel may be ignored in calculating the composite section properties.

2038.2-STEEL STUD SHEAR CONNECTORS

Use welded stud type shear connectors, minimum $\frac{7}{8}$ inch diameter, that are sufficient to resist both horizontal and vertical movement between the deck slab and girders.

Shear connectors shall be positioned to satisfy the provisions of the Governing Specifications and are grouped such that a minimum of two studs are placed in a single transverse row across the top flange.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2039

WEATHERING STEEL

May 4, 2022

First Edition

2039.1-GENERAL GUIDELINES

Weathering steel for bridge components is available as AASHTO M 270 Grade 50W and HPS 70W in plates up to four (4) inches, depending on the grade of steel.

Standard shapes in Grade 50W weathering steel are generally available. Availability should always be verified from fabricators/suppliers prior to being specified in design.

The use of uncoated weathering steel shall be governed by the guidelines as contained in the FHWA Technical Advisory; T 5140.22, *Uncoated Weathering Steel in Structures* dated October 3, 1989 (FHWA Technical Advisory).

2039.2-SITE SELECTION AND DESIGN DETAILS

Uncoated weathering steel shall not be used in the following situations:

- A. An acidic or corrosive environment.
- B. Where the structure is subjected to saltwater spray or frequent fog conditions.
- C. Over depressed roadways (less than 20 feet clearance) that create tunnel-like conditions allowing salt-laden water from the roadway surface below to be carried up to the bridge structure.
- D. Low under-clearance conditions where the steel is less than ten (10) feet over standing water and eight (8) feet over moving water.
- E. Regions where the steel is continuously wet, buried in soil or covered with vegetation.
- F. In structure types where accumulation of debris may be a problem.
- G. Truss bridges of all types.

When weathering steel is specified, consideration shall be given to the following:

- A. The number of expansion joints shall be minimized or eliminated completely.
- B. Use of details that minimize the retention of water and debris.
- C. The number of bridge deck scuppers shall be minimized.
- D. Drip bars shall be used at support locations.
- E. Substructures shall be protected from staining by use of special drainage details and/or protective coatings SD 1074.

The Designer is responsible to assure the bridge site and design details are in accordance with the requirements of the FHWA Technical Advisory for the use of weathering steel.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2040
STEEL CURVED GIRDER BRIDGES**

*May 4, 2022
First Edition*

The method of analysis will be evaluated during the span arrangement study and submitted for approval in the Span Arrangement Report.

Wider flanges generally provide better performance in curved girders. Therefore, curved girders shall be proportioned using wider flange plates that also satisfy the slenderness requirements of AASHTO.

Whenever possible, cross frames and diaphragms shall be oriented radial to the centerline of the bridge. Typically, closer cross frame spacings are required for curved bridges than that required for tangent bridges. The commentary to the Curved Girder Specifications provides suggested maximum cross frame spacing for curved bridges.

Avoid the use of irregular framing arrangements whenever possible. When irregular framing arrangements are unavoidable, the following shall be considered:

- A. A refined analysis shall be employed.
- B. Complete evaluation of force effects in all girders, bracing, cross frame members, connections and details shall be performed.
- C. Check for and resolve any uplift condition at bearings.

If uplift conditions at the bearings are present, employ the following to mitigate the effects:

- A. Diaphragm and cross frame members may be left un-tightened in slotted holes until the concrete deck has been cast.
- B. Use tie downs or counterweights at affected support locations.

The effects of temperature change on curved bridges must be considered. Thermal movements must be allowed in directions radiating from the fixed supports. Whenever multiple fixed supports are utilized, or when guided expansion bearings are not oriented to satisfy the provisions of the Curved Girder Specifications, a thermal analysis of the superstructure is required. A thermal analysis will also be required for complex and irregular framing arrangements.

The Designer should consider the possibility of requiring full-width shop assembly for curved bridges with complex geometry. This will require special plan notes and the approval of the Bridge Project Manager.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2045
CONCRETE SUPERSTRUCTURES**

January 17, 2023

Supersedes May 4, 2022

Concrete compressive strengths for precast beams shall be no more than 8,000 PSI at release (f'_{ci}) with a final compressive strength of 10,000 PSI (f'_c). Precast beams may be designed using higher strength concrete with approval of the State Bridge Engineer.

AASHTO beams shall be designed utilizing straight or straight and draped prestressing strands. These strands shall be AASHTO M 203, Grade 270, seven-wire, low-relaxation strands. Strand properties are shown in Table 2045.A.

Diameter	Area	Ultimate Strength	Applied Prestressing
0.5 Inch	0.153 IN ²	41.3 KIPS/strand	31.0 KIPS/strand
0.5 Inch (Special)	0.167 IN ²	45.1 KIPS/strand	33.8 KIPS/strand
0.6 Inch	0.217 IN ²	58.6 KIPS/strand	44.0 KIPS/strand

Table 2045.A

The FHWA currently requires a strand development length of 1.6 times the AASHTO development length requirement. This development length requirement shall be used for all strand sizes and spacing. The Designer should be aware that this might affect the use of beams in the 20 feet to 30 feet range.

All reinforcing bars are to be tied at all intersections except where spacing is less than twelve (12) inches in each direction; in which case, every other intersection shall be tied. Tack welding of steel reinforcing cages is not allowed. Designers shall assure that all submissions, such as shop drawings, fabrication details, erection plans, etc., do not reflect alternate fastening methods.

Prestressed beam spans shall be designed for the dead and live loads carried by the composite action of the slab and girders. Multi-span beams shall be designed as continuous for live load purposes.

In a situation where two or more beams of the same size require a slightly different number of strands, resulting from differences in design loadings (i.e., interior and exterior beams), use the greater number of strands if possible. This makes fabrication easier and reduces confusion during construction.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2047
PRESTRESSED CONCRETE BEAMS**

*May 4, 2022
First Edition*

2047.1-REQUIREMENTS FOR PRESTRESSED BEAMS

Stress computations resulting from the prestressing of bonded or debonded strands shall be in accordance with the Governing Specifications. In analyzing stresses or determining the required length of debonding, stresses shall be limited to the values specified in the Governing Specifications.

The use of AASHTO M 203, Grade 270, low-relaxation prestressing strands, either straight or draped strand profiles, are preferred for the design of prestressed/pretensioned concrete beams. Straight strands are preferred where practical.

Bridges with varying span lengths, skew angles, beam spacings, beam loads or other design criteria may result in very similar individual designs. The designer should consider combining similar designs into groups of common materials and strand patterns based on the following priorities:

- A. 28-Day compressive concrete strength (f'_c).
- B. Strand pattern (size, number and location).
- C. Compressive concrete Strength at release (f'_{ci}).
- D. Debonding.

In order to achieve uniformity and consistency in designing strand patterns, the following criteria may be used:

- A. The maximum strand pattern for precast beams shall utilize an odd number of strands per row (placing a strand on the centerline of the beam) and a minimum side cover of three (3) inches, except for AASHTO Type III, V, and VI beams for which strand patterns with an even number of strands per row shall be utilized.
- B. The actual strand pattern used for design shall be symmetrical about the vertical axis of the beam.
- C. In the end regions of the beam, a maximum of twenty-five percent (25%) of the strands may be debonded to satisfy the allowable stress limits, but debonded strands shall not exceed forty percent (40%) of the total strands in any one row (AASHTO 5.11.4.3). In applying these percentage limitations, the following may be used:
 - 1. Strands shall be debonded in a pattern that is symmetrical about the vertical axis of the beam.
 - 2. The theoretical number of debonded strands shall be rounded to the closest even number of strands, except that debonded strands cannot be permitted in rows containing three strands or less.
 - 3. Exterior strands in all rows shall be fully bonded.

4. Debonded strands shall be distributed laterally throughout the strand patterns as uniformly as practical. Debonded strands shall be separated by at least one fully bonded strand, whenever possible.

Reinforcement shall be provided in the top flange of all beams to control tensile cracks prior to, during and after release of the prestressing force. The reinforcement may consist of a combination of AASHTO M 31, Grade 60, deformed reinforcing bars and AASHTO M 203, Grade 270 prestressing strands.

The strands in the bottom flange shall be confined for a distance from the beam end equal to 1.5 times the depth of the beam. The confinement reinforcement shall be provided by deformed ties, AASHTO M 31, Grade 60, spaced at 6 inches maximum (AASHTO 5.10.10.2).

The vertical stirrup reinforcement for fully pretensioned beams shall be symmetrically spaced about the center of the beam.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2048

ADJACENT BOX BEAMS

August 8, 2022

Supersedes May 4, 2022

Prestressed box beam designs for spans ranging from twenty (20) feet to one hundred (100) feet are included in the Standard Details Book Volume 3 – Standard Bridge Plans by the WVDOH. Design and Assembly Details and Design Tables are included for box beams ranging in size from seventeen (17) inches to forty-two (42) inches in height. Also included in the Standard Bridge Plans are Design and Assembly Details (3000 Series) that show the various details and notes that are to be included in the construction plans. The Standard Bridge Plans also include design tables for twelve (12) inch plank beams for spans ranging from ten (10) feet to twenty-two (22) feet. As of this printing, the box beams and plank beams in the Standard Bridge Plans have been designed using the AASHTO LRFD 1998 Edition and are to be used only with the permission of the Bridge Project Manager.

Adjacent box beams are to be placed so that a $\frac{3}{4}$ inch gap is obtained between the beams to allow for the placement of a full depth shear key. The procedures for detailing and placement of the shear key are located on Standard Sheet 3000MB2 of the Standard Bridge Plans. If these standard plans are not used, these details shall be included in the contract plans.

In addition to the full depth shear key, adjacent box beams are to be transversely post tensioned using 1 inch diameter, 150 KSI thread bars conforming to AASHTO M 275, Type II. For skewed and non-skewed bridges, the full-length (transverse direction) thread bars shall be post-tensioned to a force of 80 KIPS. Thread bars that do not penetrate the full width of the structure shall be stressed to 40 KIPS.

When adjacent box beams are used that do not meet the standard drawings, a full design of the superstructure must be completed. If this design is performed, and the standard sheets are altered, the Designer must ensure that the words “Standard” are removed from the sheets.

2048.1-ALTERATION OF STANDARD SHEETS

The WVDOH has coordinated the development of these standards with the various suppliers of precast box beams.

When standard plan sheets are incorporated into a set of plans for a specific bridge, no alterations of these sheets are to be made. If changes are necessary to adapt the plans to some unusual situation, then approval to alter the plan sheets must be obtained from the Bridge Project Manager. Exception to this is permitted where the design must be modified in accordance with SD 2048.2, Odd Length Beams, to accommodate odd span lengths not listed on the standard plan sheets.

2048.2-ODD LENGTH BEAMS

The standard plans contain design and manufacturing details for specific span lengths. If the span lengths are between those listed on the standard plans, it cannot be assumed that the design and manufacturing data for the closest shorter or longer listed span length will be acceptable,

particularly if a concrete barrier is being utilized. In this situation, the data for the next highest listed beam length shall be chosen. The initial tension at the beam end may be greater than the allowable, which may require additional debonding of strands to alleviate the overstressing.

It will be necessary to perform a design for span lengths other than those shown on the standard plan design data chart. It is requested that Engineering Division review designs prepared by consulting engineering firms or by district offices prior to extensive plan development.

2048.3-ADDITIONAL DESIGN INFORMATION

Omission of a wearing surface or an asphalt concrete wearing surface is appropriate on bridges with a low ADT (average daily traffic) that are not on part of the Coal Resource Transportation System (CRTS). On higher ADT bridges, a concrete deck, reinforced with a single mat of reinforcing steel, should be considered. Horizontal shear bars shall be provided between the interface of the box beam and the concrete deck. The top surface of the beams shall have a roughened surface when a composite deck is used.

The use of backwalls is encouraged on all new designs of adjacent precast/prestressed box beam bridges. When backwalls are used, expansion joint details shall be used as shown in Standard Sheet 3000MB3. If a composite deck with backwall is used, see Figure 2048.A.

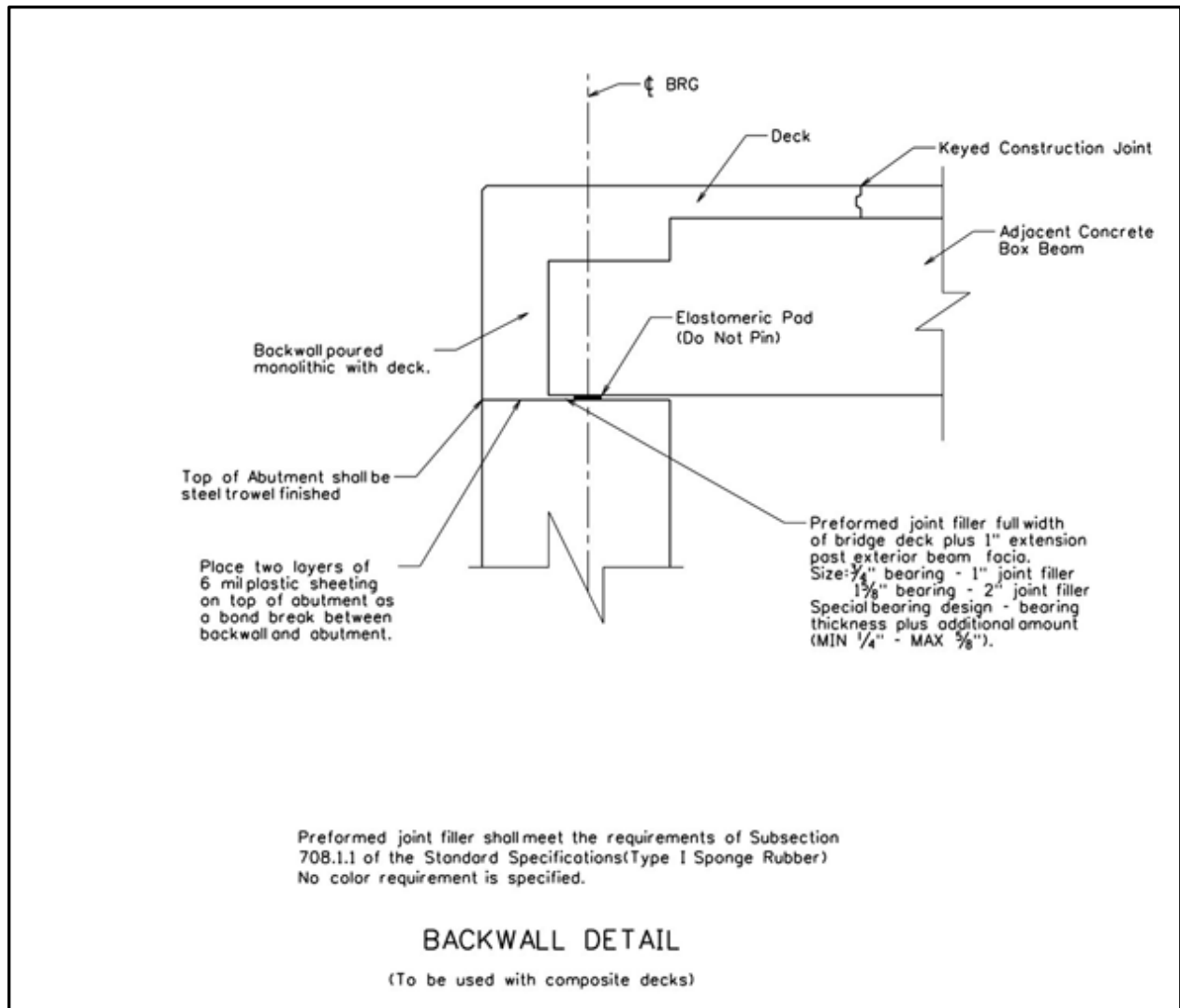


Figure 2048.A

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2049
SEGMENTAL CONCRETE STRUCTURES**

*May 4, 2022
First Edition*

2049.1-INTRODUCTION

The following design criteria apply to the design of precast or cast-in-place segmental concrete bridges. The design shall satisfy the requirements of Section 5.14.2 of the Governing Specifications.

2049.2-METHODS OF CONSTRUCTION

Different construction methods can be used for segmental structures. The most commonly used methods are:

- A. Span-by-Span Method: In this method a temporary erection truss or a fabricated girder, spans from one pier to the next to support the precast segments (including I-girders) for an entire span while they are placed, assembled and post-tensioned together to become a self-supporting structure. The span range for this method is between 100 feet and 180 feet.
- B. Balanced Cantilever Method: In this method, the segments are placed symmetrically about a pier so that upon application of the permanent cantilever tendons, the superstructure is balanced. Successive cantilevers are connected together through mid-span closure pours using longitudinal continuity tendons. The recommended clear span between piers may range from 150 feet to 450 feet for precast construction. For a span range of 450 feet to 700 feet, cast-in-place construction with a form traveler can be used. For spans greater than 600 feet, the weight of the haunched segments near the piers, diminish the feasibility of using such method and cable-stay construction becomes more competitive.

2049.3-SPECIFICATIONS

For items not specifically addressed in the Governing Specifications, the following may be consulted:

- A. *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges.*
- B. *AASHTO Standard Specification for Highway Bridges.*
- C. *AASHTO Guide Specifications of Distribution of Loads for Highway Bridges.*

2049.4-NOTATION

Unless otherwise indicated, the notations used in these criteria correspond to the notations contained within the codes upon which these criteria are based. The codes are specified where necessary.

2049.5-LOADINGS

All applicable loadings specified in the Governing Specifications, Section 3, shall be considered in the design. The load modifier, which is a factor accounting for ductility, redundancy and the operational importance of the bridge shall follow SD 2012.

In addition to the loading combinations specified in Table 3.4.1-1 of the Governing Specifications, the following combination shall be investigated at the service limit state: DC + DW + EH + EV + ES + WA + CR + SH + TG + EL, where all loads are as defined in AASHTO 3.3.2.

Construction loads and construction loading combinations for the service limit state and the strength limit state shall follow Section 5.14.2 of the Governing Specifications.

2049.5.1-Thermal Effect:

- A. For positive temperature gradient, Section 3.12.3 of the Governing Specifications shall be implemented considering Zone (3) in Table 3.12.3-1. Negative temperature values shall be obtained by multiplying the specified positive values by (–0.3) for plain concrete decks and (–0.20) for asphalt overlay.
- B. In lieu of project specific information, the load factor for temperature gradient may be taken as:
 - 1. 0.0 for strength and extreme event limit states.
 - 2. 1.0 for service limit state when live load is not present.
 - 3. 0.5 for service limit state when live load is present.

2049.5.2-Creep and Shrinkage: The Governing Specifications shall be used to determine strains due to shrinkage and to estimate creep coefficient. For major and long span bridges, a creep test is recommended. Moment redistribution in a continuous structure, which occurs after the construction of the closure pours, for the effect of creep and shrinkage shall be carefully considered. Resulting stresses shall be calculated assuming a realistic construction sequence submitted to the Design Engineer.

2049.5.3-Prestressing: Sections 5.9 and 5.10 of the Governing Specifications shall be referenced. The structure shall be designed for both initial and final prestressing forces. The design shall satisfy the following criteria:

- A. All prestressed box girder deck slabs shall be transversely post-tensioned, unless the box wings are relatively short so that mild reinforcement would be sufficient to resist the applied loads. However, the use of transverse posttensioning in short wings will increase deck durability and thus justifying any extra expense.
- B. If draped tendons are used in the deck slab, consideration shall be given to the final location of the center of gravity of the strands within the duct. Critical eccentricities over the web and at the centerline of the box shall be reduced by ¼ inch from the theoretical value to account for construction tolerances.
- C. The effect of structural indeterminacy, which reveals itself in the form of secondary moments due to prestressing, shall be carefully examined; otherwise, substantial secondary effects may arise causing unexpected internal stresses.
- D. In the design of horizontally curved bridges, special consideration shall be given to the effect of the lateral force component of the curved tendons. This force may cause additional transverse bending stresses in the box crosssection. Also, the spacing

- between the prestressing ducts shall be sufficient to allow enough concrete presence, to prevent any tendon overlap resulting from the straightening of the tendons. When a tendon curves in two planes, the inplane and out-of-plane forces shall be added vectorially.
- E. Unless a vibration analysis indicates otherwise, the unsupported length of external tendons shall not exceed 25 feet.

2049.6-MATERIALS

2049.6.1-Concrete: Minimum 28 days cylinder compressive strength (f'_c) for concrete shall be as follows:

- A. Cast-in-place (CIP) segmental superstructure 6,500 PSI
- B. Precast segmental superstructure (including CIP joints) 6,500 PSI

Clear concrete cover for the superstructure shall be as follows:

- A. Top riding surface 2.0 inches
- B. Outside face of webs 2.0 inches
- C. Other locations 1.5 inch

The Governing Specifications shall be referenced for other cover requirements.

2049.6.2-Prestressing Steel: Prestressing steel shall conform to the following specifications:

- A. Strand tendons shall conform to AASHTO M203 Grade 270, low relaxation:
 - 1. Ultimate strength of prestressing steel (f_{pu}) 270 KSI.
 - 2. Yield stress (f_{py}) may be taken as $0.9f_{pu}$ 243 KSI.
 - 3. Apparent modulus of elasticity 28,500 KSI.
- B. Parallel wires shall conform to AASHTO M204.

Other parameters:

- A. Apparent modulus of elasticity 28,500 KSI.
- B. Friction coefficient 0.25 per RAD.
- C. Wobble coefficient 0.0002 per foot
- D. Anchor set 0.375 inches

Bar tendons (high strength threaded bars) shall conform to the following specifications:

- A. Bar tendons shall conform to AASHTO M275 Grade 150.
 - 1. Ultimate strength of bar (f_{pu}) 150 KSI.
 - 2. Yield stress (f_{py}) may be taken as $0.8f_{pu}$ 120 KSI.
 - 3. Modulus of elasticity 30,000 KSI.

Other parameters:

- A. Modulus of elasticity 30,000 KSI.
- B. Friction coefficient 0.30 per RAD.
- C. Wobble coefficient 0.0002 per foot.
- D. Anchor set 0.063 inches.

2049.7-ALLOWABLE STRESSES

2049.7.1-Concrete Allowable Stresses: Concrete allowable stresses, in the transverse and longitudinal directions, under temporary construction loads at service limit state shall follow Section 5.14.2.3.3 of the Governing Specifications.

Concrete allowable stresses for permanent loading conditions shall satisfy section 5.9.4 of the Governing Specifications.

2049.7.2-Prestressing Allowable Stresses:

For prestressing tendons:

- | | |
|--|--|
| A. Maximum jacking stress | $0.8 f_{pu}$ but not to exceed $0.90 f_{py}$. |
| B. At anchorage after anchoring | $0.70 f_{pu}$. |
| C. At other locations after anchoring | $0.74 f_{pu}$. |
| D. At service limit state after losses | $0.80 f_{py}$. |

For prestressing bars:

- | | |
|--|-----------------|
| A. Maximum jacking stress | $0.70 f_{pu}$. |
| B. At anchorage after anchoring | $0.70 f_{pu}$. |
| C. At service limit state after losses | $0.80 f_{py}$. |

2049.8-BOX GIRDER PROPORTIONS

The box girder cross-section dimensions shall satisfy the requirements of Section 5.14.2.3 of the Governing Specifications. The overall dimension of the box girder cross-section shall preferably not be less than that required to limit the live load plus impact deflection, calculated using the gross section moment of inertia and the secant modulus of elasticity, to 1/1000 of the span.

See the commentary of Section 5.14.2.3.10d of the Governing Specifications for determining girder depth and web spacing.

The proposed precast AASHTO-PCI-ASBI standard box girder for span-by-span and balanced cantilever constructions can also be used for the bridge cross section.

Segment weight shall be suitable for construction and erection equipment.

2049.9-ANALYSIS METHOD AND MATHEMATICAL COMPUTER MODELING

Analysis of segmental bridges shall follow the guidelines outlined in Section 4.6.2.9 of the Governing Specifications. Elastic analysis and beam theory can be used to determine design moments, shears, and deflections.

The analysis shall be conducted in both the transverse and longitudinal directions. The effect of secondary moment due to prestressing shall be included in stress calculations at the service limit state. At the strength limit state, the secondary force effects induced by prestressing, with a load factor of 1.0, shall be added algebraically to the force effects due to all other factored loads.

The effective flange width for box girders shall be based on Section 4.6.2.6 of the Governing Specifications. The capacity of the cross section at the strength limit state may be determined by considering the full compression flange width effect. Shear lag shall be considered

for service load stress calculations. If the width (tip to tip of wings) to depth ratio exceeds 6, the designer must analyze for the effect of shear lag.

Segments of horizontally curved bridges with torsionally stiff closed sections whose central angle subtended by a curved span or portion thereof is less than 12° may be analyzed as if the segments were straight.

Mathematical models shall include loads, geometry, and material behavior, and where appropriate, response characteristics of the structure foundation and soil-structure interaction. Any available segmental computer software approved by the WVDOH may be used. The computer model shall verify the anticipated structural behavior and must include all the construction stages suggested for the project. It must also account for the final status of the bridge under service loads as well as the effect of time dependent factors such as creep, shrinkage, and prestressing losses.

2049.10-DESIGN METHOD

All applicable limit states (strength, extreme events, service, and Fatigue) shall be satisfied in accordance with the Governing Specifications.

The service limit state covers cracking, deformations, deflections, and concrete stresses. For the strength limit state, the resistance factors shall follow Section 5.5.4.2 of the Governing Specifications. The structure as a whole shall be proportioned to resist collapse due to extreme events specified in Table 3.4.1-1 of the Governing Specifications as may be appropriate to its site and use.

The deck shall be designed per AASHTO 9.7.6, Deck Slabs in Segmental Construction. The deck overhang shall be designed considering the three load cases specified in the Governing Specifications Section A13.4.

2049.11-SHEAR AND TORSION

Section 5.8 of the Governing Specifications shall be applied for shear and torsion design. The analysis is based on the Modified Compression Field Theory (MCFT) instead of the traditional ACI empirical equations.

It is conservative to design based on maximum shear and maximum torsion. However, it is sufficient to design using the maximum shear and the corresponding torsion and the maximum torsion with its associated shear. The vertical component of the longitudinally draped tendons shall be algebraically added to the acting shearing force.

In a box girder, the stresses due to shear and torsion will be additive on one side of the web, while counteracting each other on the other side. Therefore, the transverse web reinforcement shall be based on the summation of reinforcement due to shear and torsion.

Checks for principal tensile stresses shall be performed as a method of preventing cracking under service load conditions.

2049.12-TOTAL WEB REINFORCEMENT

The web reinforcement consists of flexural reinforcement required by transverse analysis and that required from the shear and torsion design. The total reinforcement may be taken as the largest of:

- A. 50% of flexural reinforcement + 100% of shear and torsion reinforcement.
- B. 100% of flexural reinforcement + 50% of shear and torsion reinforcement.
- C. 70% of flexural reinforcement + 70% of shear and torsion reinforcement.

2049.13-SEGMENTAL CONSTRUCTION JOINTS

Joints in precast segmental bridges shall be either cast-in-place closures or match cast. Cast-in-place concrete joints and epoxy joints between precast units shall be considered as Type A joints. Dry joints shall be considered as Type B joints.

The resistance factors of the joints in flexure and shear shall follow that of Table 5.5.4.2.2-1 of the Governing Specifications.

For Type A joints, auxiliary bonded reinforcement through the joint may be provided at a stress of $0.5 f_y$ to carry the allowable tensile force in accordance with Tables 5.9.4.1.2-1 and 5.9.4.2.2-1 of the Governing Specifications.

The concrete strength of the cast-in-place closures shall not be less than that of the precast concrete. The width of the closure shall permit the development of the reinforcement in the joint or coupling of the tendons ducts if used.

Type B joints (dry joints) are not permitted.

2049.14-SHEAR KEYS

There are two types of shear keys in the match-cast joint between precast segments:

- A. Web shear keys: Located on the faces of the webs of precast box girders. The total depth of shear keys shall be approximately 75% of the section depth and its width shall be at least 75% of the web thickness.
- B. Alignment keys: Located in the top and bottom slabs. Alignment keys are not expected to transfer the shear forces at the joints, but rather to correct alignment of the two match-cast segments being erected. For a single-cell box, normally a minimum of three alignment keys is required on the top slab and one on the bottom slab.

Both shear and alignment keys shall not be located in the tendon duct zones.

2049.15-ERECTION SCHEDULE AND CONSTRUCTION

The method of construction assumed for the design shall be shown in the contract documents. Temporary supports required prior to the time the structure, or its components, can support itself and subsequently applied loads, shall be clearly shown in the contract documents.

A typical erection schedule and anticipated construction system shall be incorporated into the design documents in an outlined schematic form. The assumed erection loads along with time of application and removal of erection loads shall be clearly stated in the plans.

2049.16-CONSTRUCTION DATA ELEVATIONS AND CAMBER CURVE

Construction data elevations shall be based on the vertical and horizontal highway geometry. Camber curves shall be calculated and based on the assumed erection loads and schedule used in the design as well as the assumed construction sequence. Camber curve data shall be provided at the centerline of the box. Camber is the amount by which the concrete profile, at the time of casting, must differ from the geometric profile grade in order to compensate for all structural dead load, post tensioning, long and short-term time dependent deformations and the effects of construction loads and sequence of erection.

2049.17-INTEGRATED DRAWINGS

Contract drawings shall be prepared in accordance with Method A as explained in Section 5.14.2.3.9 of the Governing Specifications. Congested areas of post-tensioned concrete shall be

shown on integrated drawings with an assumed post-tensioning system. Such areas include, but are not necessarily limited to, anchorage zones, areas containing embedded items for the assumed post-tensioning system, areas where post-tensioning ducts deviate both in the vertical and transverse directions and other highly congested areas as determined by the Designer and/or the WVDOH.

The assumed post-tensioning system, embedded items, etc. shall be selected in a manner that will accommodate competitive systems (e.g., using standard available anchorage sizes). Integrated drawings utilizing the assumed system shall be detailed to show reinforcing and post-tensioning steel in two-dimensions (2-D) and, when necessary, in complete three-dimensional (3-D) drawings.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2050
MISCELLANEOUS SHAPES/TYPES**

*May 4, 2022
First Edition*

Other shapes such as spread boxes, T-beams, double-T's and slab structures may be considered for use in bridge projects provided the beams satisfy all required design criteria.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2055
TIMBER SUPERSTRUCTURES**

*May 4, 2022
First Edition*

Timber structures are not an economical alternative for most bridge sites. However, the aesthetic benefits of a structure made from timber may outweigh the costs associated with construction and maintenance.

2055.1-LIMITATIONS

Typically, single span structures on low ADT roads are good candidates for timber bridges. Timber structures may be considered for pedestrian bridges. While timber structures are encouraged for smaller bridges within West Virginia, they are not practical for most sites.

A partial list of limitations to the use of timber structures:

- A. The inability of timber to be made continuous over interior supports makes it an unfavorable alternate on multiple spans.
- B. Spans over one hundred (100) feet pose deflection and camber limitations.
- C. Members are considerably deeper than competing materials, thereby potentially restricting waterway openings.

2055.2-TYPES

2055.2.1-Glulam System: A glulam system consists of a series of transverse glulam deck panels supported on straight or slightly curved longitudinal glulam beams. Glulam components are manufactured from 1 ½ inch to 2 inch thick lumber laminations that are bonded together on their wide faces with a waterproof structural adhesive. This system is the most practical for clear spans of twenty (20) feet to one hundred (100) feet (although they can span even larger distances) and are widely used on single-lane and multiple-lane roads and highways. Beams are the principal load-carrying components of the bridge superstructure. For additional information, see the Timber Bridge Manual, Chapter 7, Part 1.

2055.2.2-Longitudinal Glulam Deck: Longitudinal glulam deck bridges are composed of a series of glulam panels placed edge to edge across the width of the bridge. These systems are suitable for spans up to thirty five (35) feet. The panels are not post-tensioned or interconnected with dowels. Continuity is achieved through transverse stiffener beams bolted directly to the panels through brackets below. For further details of longitudinal glulam deck bridges, see the Timber Bridge Manual, Chapter 8.

2055.3-PROCEDURE

All timber bridges shall be designed according to Section 8 of the Governing Specifications.

The United States Department of Agriculture – Forest Service has a publication on Timber Bridges: *Timber Bridges – Design, Construction, Inspection, and Maintenance*, 1992 (Timber Bridge Manual). This manual may serve as a useful tool during the design process. Note that this manual was written as a design aid using the LFD Design Method. Special considerations shall be given when applying this manual to the current LRFD Design Method.

The dynamic load allowance shall be reduced by fifty percent (50%) for timber bridges and wood components of bridges.

Lumber size shall be selected based upon available sizes. Structural calculations shall be based upon available sizes. Structural calculations shall be based on actual dimensions rather than nominal dimensions.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2060

BEARINGS

May 4, 2022

First Edition

Bearing devices are mechanical systems that transmit loads from the superstructure to the substructure. Also, bearing devices provide for movement due to thermal expansion and contraction as well as rotational movement associated with the deflection of primary members. There are two principal types of bearings: fixed and expansion. Fixed bearings only allow rotation while expansion bearings permit both rotation and translation. All bridge bearing designs shall be in accordance with the Governing Specifications.

The applicability of certain types of bearings will vary depending on the loads and movement the bearing is required to sustain. Elastomeric bearings are preferred for most span arrangements. Polytetrafluorethylene (PTFE or Teflon) expansion bearing assemblies or pot bearings may be used when span lengths, curvature, or load limits for the standard elastomeric pads are exceeded. Elastomeric bearing pads shall be designed in accordance with the Governing Specifications using an elastomer with a hardness of 50-60 Durometers: "Method A" will be used for the design of unreinforced pads and "Method B" will be used for the design of steel reinforced pads.

Concrete surfaces in contact with the bearings shall be adequately reinforced to prevent bursting, splitting and/or spalling. This also applies to any jacking pockets or jacking locations provided for the future replacement of bearings.

Use epoxy grit to increase the coefficient of friction between the contact surfaces to reduce the potential for slip.

Any components to be welded to the superstructure steel, such as sole plates, shall be painted as superstructure steel. Plates, fasteners, or other components fabricated as part of, and permanently attached to elastomeric bearings may be painted.

Steel components of bearing devices shall be galvanized/metalized except as noted. These components shall include, but not limited to, masonry plates, rockers, sliding bearing plates, pins, bolts, nuts, washers, anchor bolts, nuts, and washers. Galvanizing shall be hot dipped galvanizing in accordance with AASHTO.

Proper consideration shall be given to those components that have finished surfaces such as sliding bearing surfaces, finished surfaces of pins, and pin holes where galvanizing may not be permissible. Plates receiving Teflon pads or stainless-steel sheets shall not be galvanized. The plates shall be SSPC-SP-6, commercial blast cleaned, and except for areas with special facings, shall be painted in accordance with the Specifications.

Weathering steel may be used for bearing devices. There are no known issues of having galvanized parts in contact with weathering steel.

Slotted holes are not permitted on fixed bearings.

All bridge bearings shall be accessible for inspection and maintenance with the exception of integral abutment bearings. The bearings shall also be replaceable without damage to the structure and without removing anchorages permanently attached to the substructure.

2060.1-BEARING DESIGN CRITERIA

Combinations of load, rotation and translation anticipated during construction shall be incorporated into the design of the bearings with allowance for construction tolerances and variation of temperature at installation. It is possible that the rotation and translation of the bearings may be significant during construction and may not be fully relieved, resulting in “locked-in stresses” in the bearing. If not sufficiently accounted for in design, these effects from construction could potentially cause an overstress under normal service conditions and/or adversely affect service life of the bearing. The Designer shall evaluate construction loading and movement in the design of the bearings and incorporate the most cost-effective of the following alternatives to control or relieve stress in the bearings from construction:

- A. Prior to allowing traffic on the newly constructed bridge, jack the bearing assemblies to relieve possible stresses that may have occurred during construction. This excludes bearings at integral supports. For concrete beam structures made continuous for live load, jack bearing assemblies prior to casting the continuity joint.
- B. Design the bearings for additional movement/rotation during construction that includes sufficient tolerance for; temperature variation at installation (from assumed ambient temperature), anticipated rotations and out of level support surfaces at the bearing seats.
- C. Prescribe the installation temperature for the bearings and require beam seats to be level or within defined dimensions. These requirements shall be incorporated into the design of the bearings, specified in the construction documents, and verified during construction.

Elastomeric bearings for integral abutments must be designed to support non-composite dead load reactions and beam rotations. Thermal forces are not considered since the time between beam placement and final closure pour is assumed to be small. Reactions and rotations shall be based on actual span configuration. No further design cases are required. All superimposed dead loads and live loads at the final configuration are supported by the closure pour. The minimum pad thickness is ½ inch.

Elastomeric bearings for semi-integral and expansion abutments must support dead load plus live load reactions and must resist dead and live load rotations. Before the abutment closure pour, if applicable, bearings must be designed to support non-composite dead load reactions and beam rotations. Reactions and rotations shall be based on actual span configuration. For concrete beams, rotations due to dead loads at the final stage shall be calculated to include long-term deflections considering creep and shrinkage. In addition, the bearings must resist lateral forces due to thermal expansion and contraction.

2060.2-BEARING REPLACEMENT

Provisions for future jacking of the structure, for the future replacement or realignment of bridge bearings, shall be included in the design of new bridge projects. Jacking locations shall be shown in the TS&L Study and on Final Detailed Bridge Plans. The Designer shall develop a suitable jacking scheme with consideration given to available equipment such as jacks, controls and false work. Appropriate modeling techniques shall be employed to determine jacking forces in the structure and to evaluate the bridge to determine the effects due to jacking. The Designer will consider jacking forces in bridge components, identifying and evaluating potential adverse effects to components such as cross frames, diaphragms, deck slab, substructures, etc. The

Designer shall consider jacking forces in the design of bridge components, providing appropriate sizing and/or details sufficient to resist the anticipated jacking forces. The jacking scheme shall be provided on the contract plans in sufficient detail and clarity to ensure successful jacking of the structure. Jack locations, jacking forces, maximum jacking height and all pertinent information and restrictions, including any special traffic restrictions, shall be included.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2070

EXPANSION DEVICES

May 4, 2022

First Edition

All bridges shall be designed to eliminate deck expansion joints where practical. If expansion devices are required, they shall be detailed to prevent damage to the structure from water, deicing chemicals and roadway debris. AASHTO Section 14, Joints and Bearings, shall govern the design for all expansion devices. The Designer shall provide joint opening information on the plans with provisions for temperature variations. The following features shall be considered for all designs:

- A. Multi-span bridges shall be continuous over interior supports when applicable.
- B. All proposed deck expansion joints shall be clearly shown in the TS&L.

2070.1-TRANSVERSE JOINTS

20170.1.1-Thermal Movement Less Than Two (2) Inches: When appropriate, structures with a total thermal movement of two (2) inches or less shall be designed in accordance with SD 2090.1, Integral Abutments.

2070.1.2-Thermal Movement from Two (2) to Four (4) Inches: When semi-integral abutments cannot be used, strip seal expansion devices shall be used for a thermal movement range of two (2) inches to four (4) inches. The scope of this specification is limited to preformed non-reinforced, polychloroprene (neoprene) strip seal glands that mechanically lock into steel retainers. The steel retainers shall be anchored into the structure in accordance with the contract requirements.

See SD 3000, Miscellaneous Plan Notes, for notes to be included when strip seals are specified.

2070.1.3-Thermal Movement Greater Than Four (4) Inches: Finger plate expansion devices with a drainage trough may be utilized for thermal movements greater than four (4) inches. The minimum thickness of a finger plate shall be two (2) inches. Proprietary modular expansion devices may be used for these movements through the propriety approval process. See DD 106.

Two types of drainage troughs may be used: either rubberized or preformed fabric material. Rubberized trough material shall consist of neoprene. Preformed fabric material shall consist of multi-ply polyester fabric and rubberized trough material vulcanized to form a laminate. The designer shall detail the trough with a minimum slope of 1:12, to allow for proper drainage.

All bridge expansion devices must be designed bicycle/pedestrian safe by attaching a flat plate on top of the expansion device. The plate shall be welded to the approaching traffic side of the device as shown in the Figures 2070.A and 2070.B.

See SD 3000, Miscellaneous Plan Notes, for notes to be included when finger plates are specified.

2070.2-LONGITUDINAL JOINTS

Longitudinal joints may be used on bridge decks only when justification is provided, showing that standard construction sequencing is unfeasible.

When longitudinal joints are used, they shall be placed where drainage is not a consideration.

Two types of longitudinal joints that may be considered are two-part silicone sealant or a strip seal and must be designed for the movements of the structure.

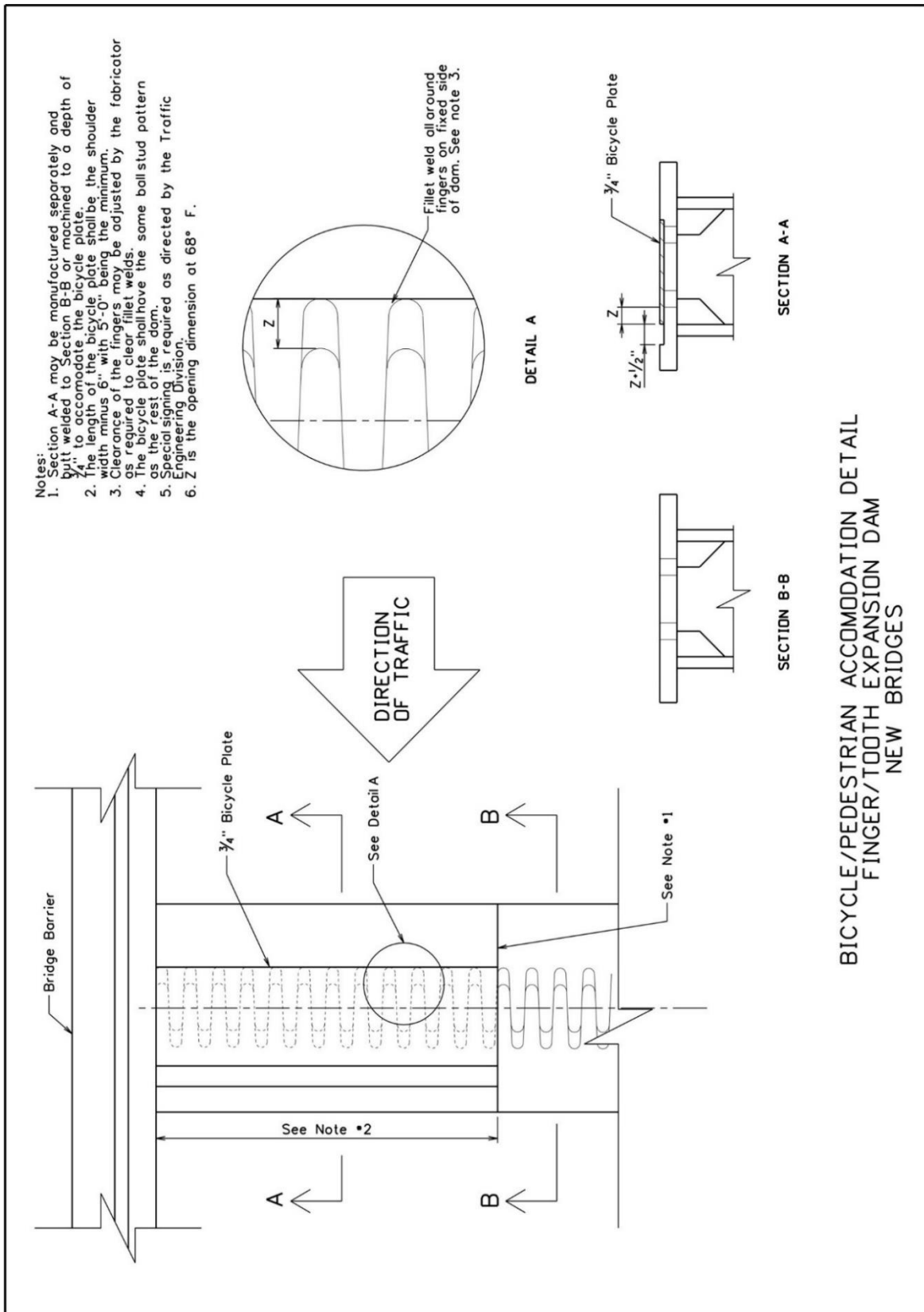


Figure 2070.A

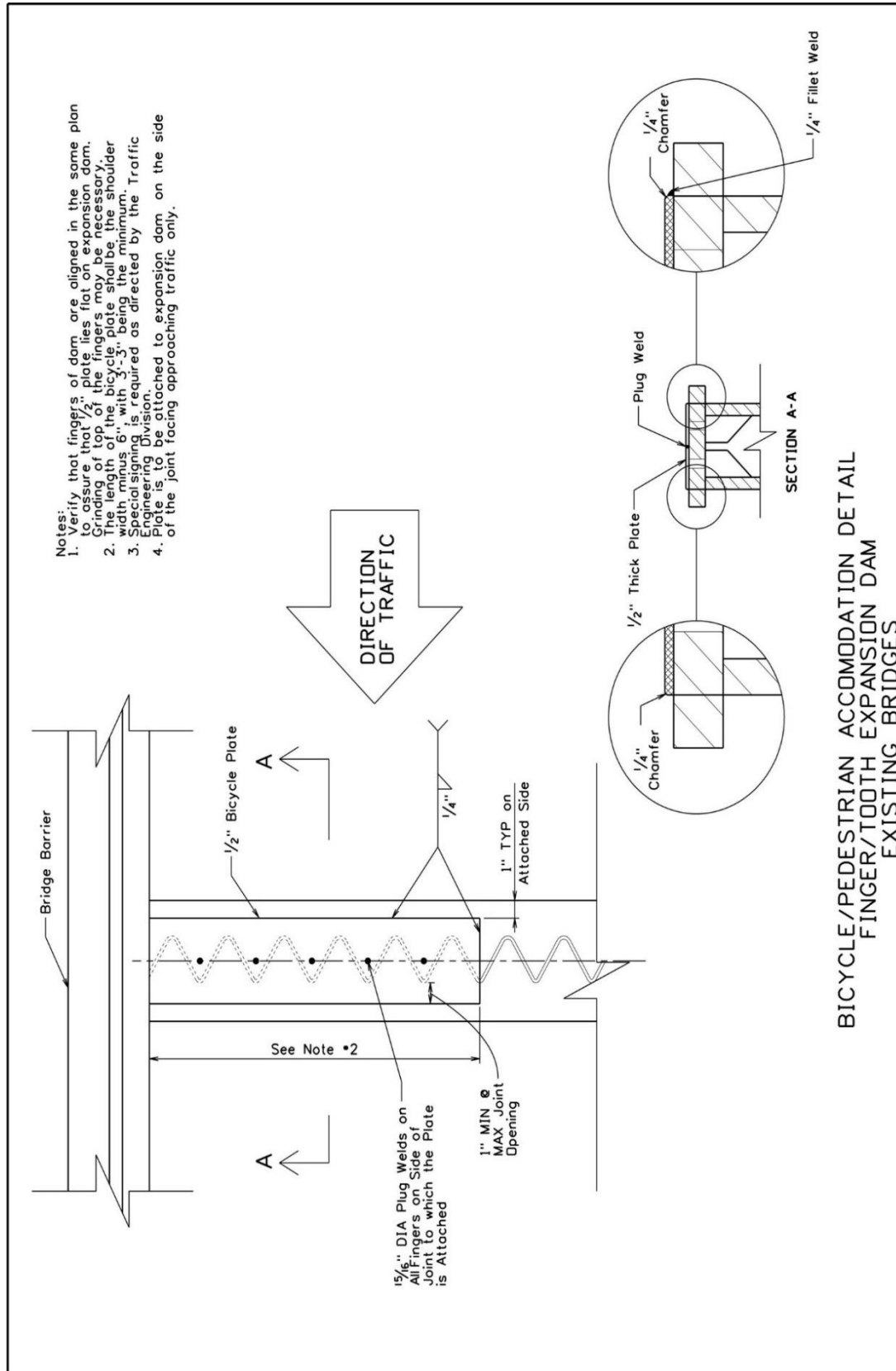


Figure 2070.B

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2080

ABUTMENTS

May 4, 2022

First Edition

2080.1-ABUTMENT DESIGN CRITERIA

The main parts of an abutment are the bridge seat, backwall, stem, wingwalls and foundation. See Figures 2080.A thru 2080.D. Several types of abutments can be utilized for a particular bridge site. See SD 1043, Abutment Types. Aesthetics can sometimes be a factor when selecting an abutment type. Abutment designs shall be in accordance with Section 11 of the Governing Specifications.

Class B Concrete with a compressive strength of 3,000 PSI should be used in most abutments. The minimum reinforcing throughout the abutment and wingwalls shall be No. 5 bars spaced at one (1) foot. All reinforcing steel above the beam seat shall be epoxy coated.

2080.2-BRIDGE SEAT

The width of the bridge seat must be sufficient to accommodate the beams or girders, jacking operations and meet the seismic requirements of the Governing Specifications. Adequate room between the beams or girders and the backwall shall be provided to allow for expansion joint inspection, where applicable.

Due to roadway cross-section slopes or skewed abutments, it may be necessary to provide beam seats of different elevations. When detailing abutments, the difference in elevations between adjacent beam seats may be achieved by stepping the top of the stem, or by utilizing a sloped stem to eliminate the effects of cracking in large bridge seat steps. See Figure 2080.E. The sloped stem option is preferred when the slope of the berm allows.

When the beam longitudinal grades are between 0.5% and 2%, the beam seats shall be sloped parallel to the beam grades. Otherwise, the beam seats shall be level to true elevation. The bridge seat elevations at the centerline of bearing shall be shown on the plans.

Minimum reinforcement shall be according to the Governing Specifications. The bearing area under each beam may be subjected to very large localized compressive and shearing stresses. Additional reinforcement directly under the bearing may be necessary to prevent the formation of visible cracks or possible spalling. This additional reinforcing may be required for beam seats that are stepped four (4) inches or more or when the standard reinforcing is not sufficient to prevent cracking or spalling.

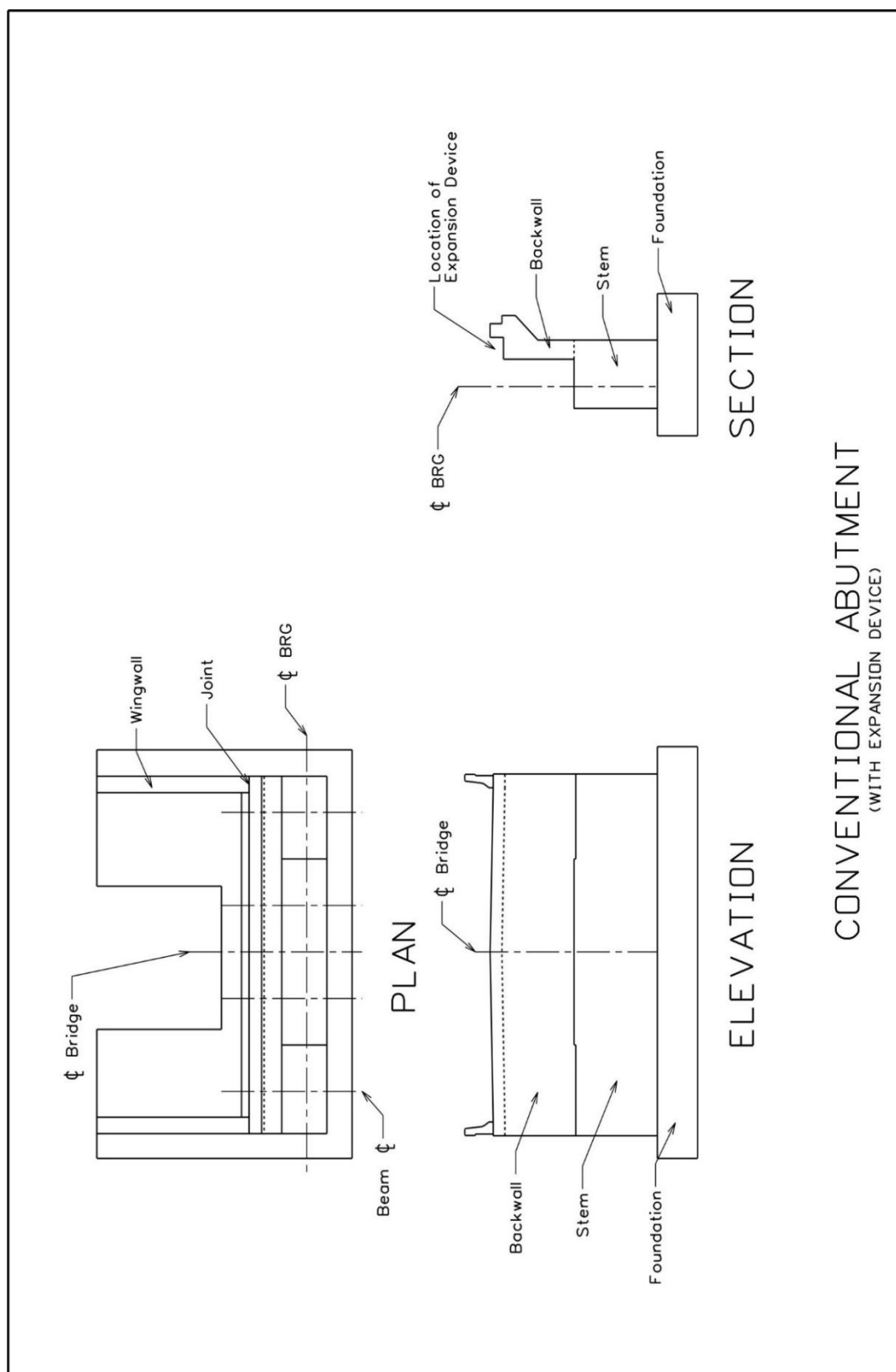


Figure 2080.A

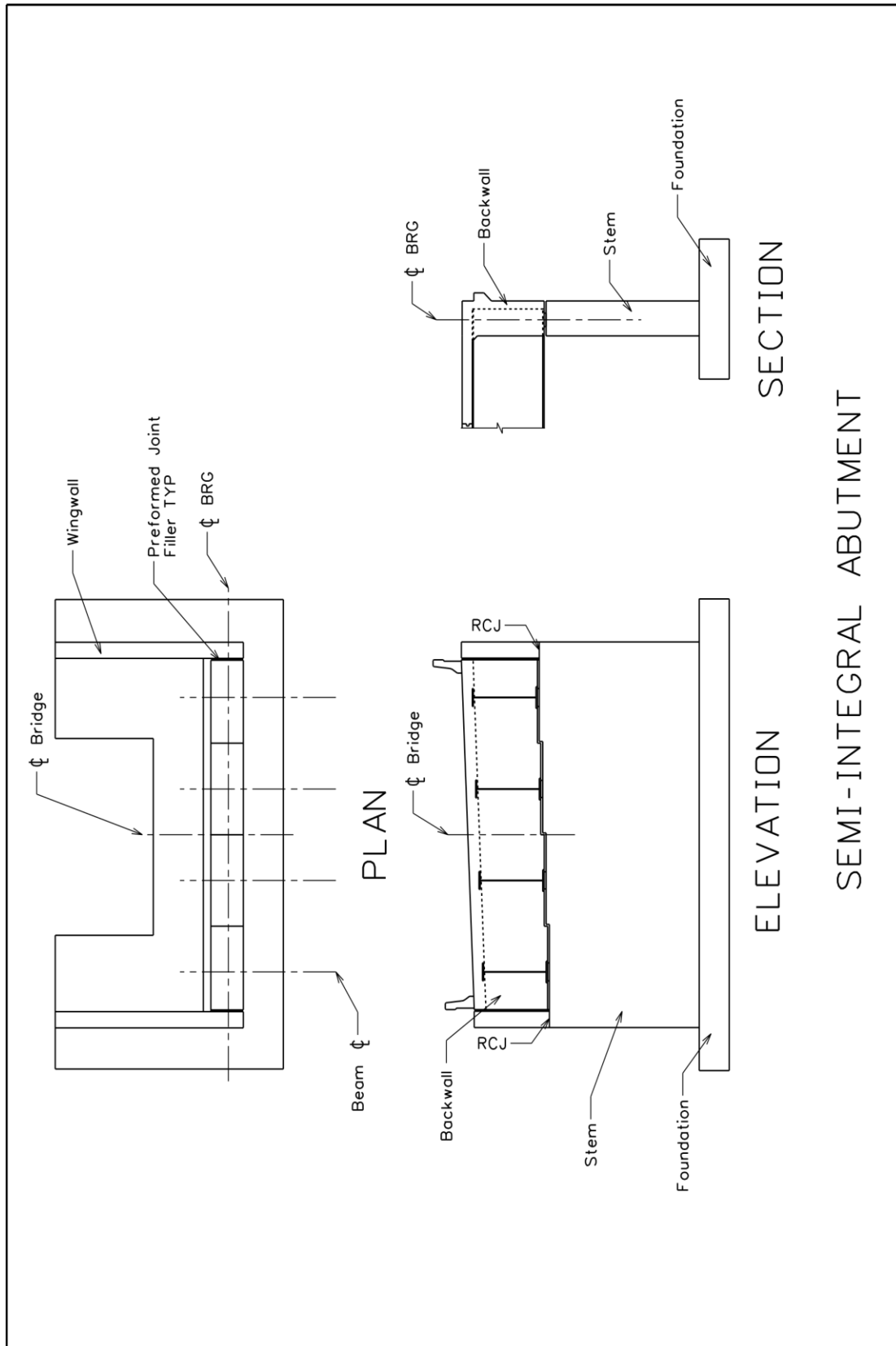


Figure 2080.B

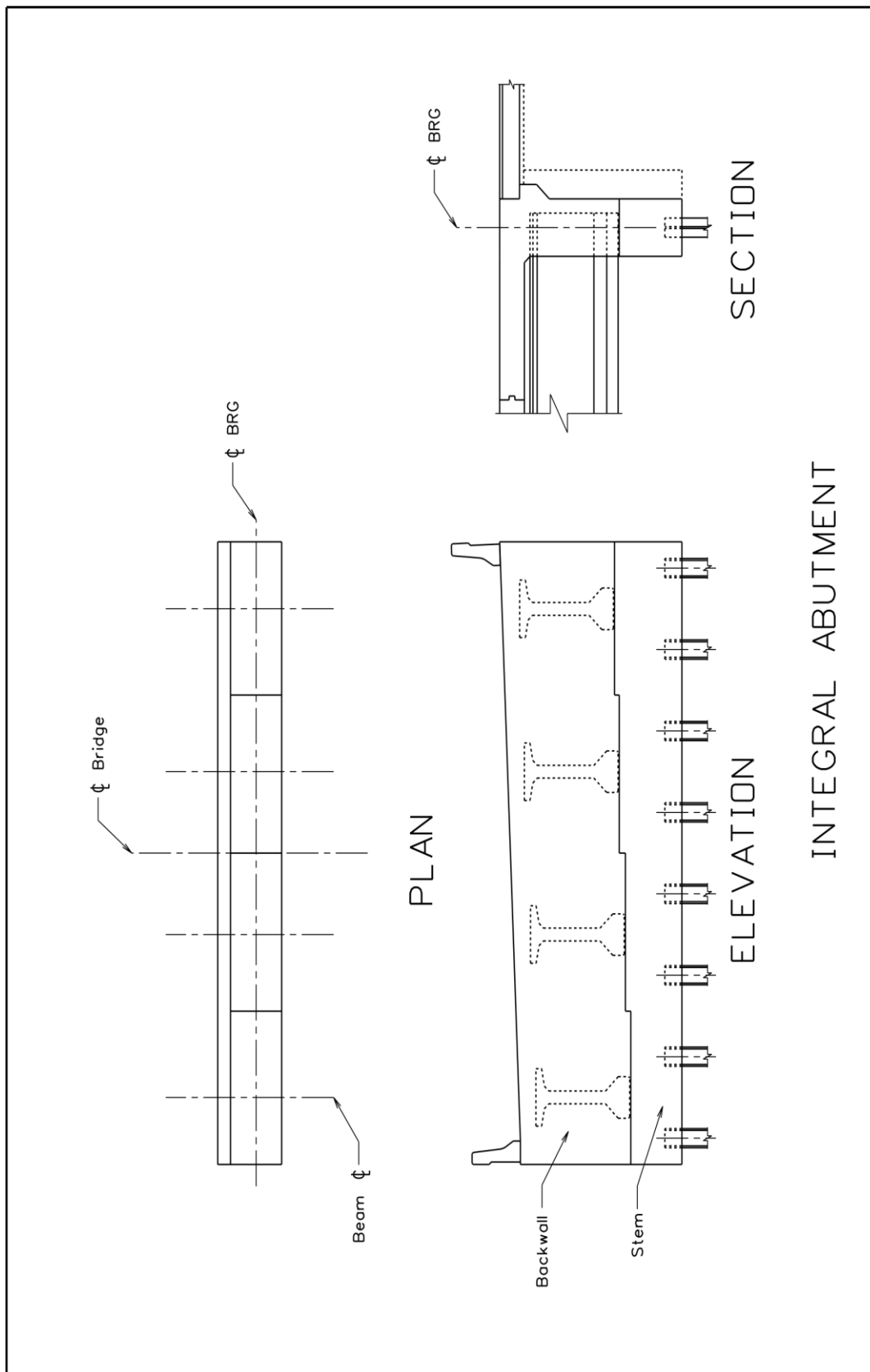


Figure 2080.C

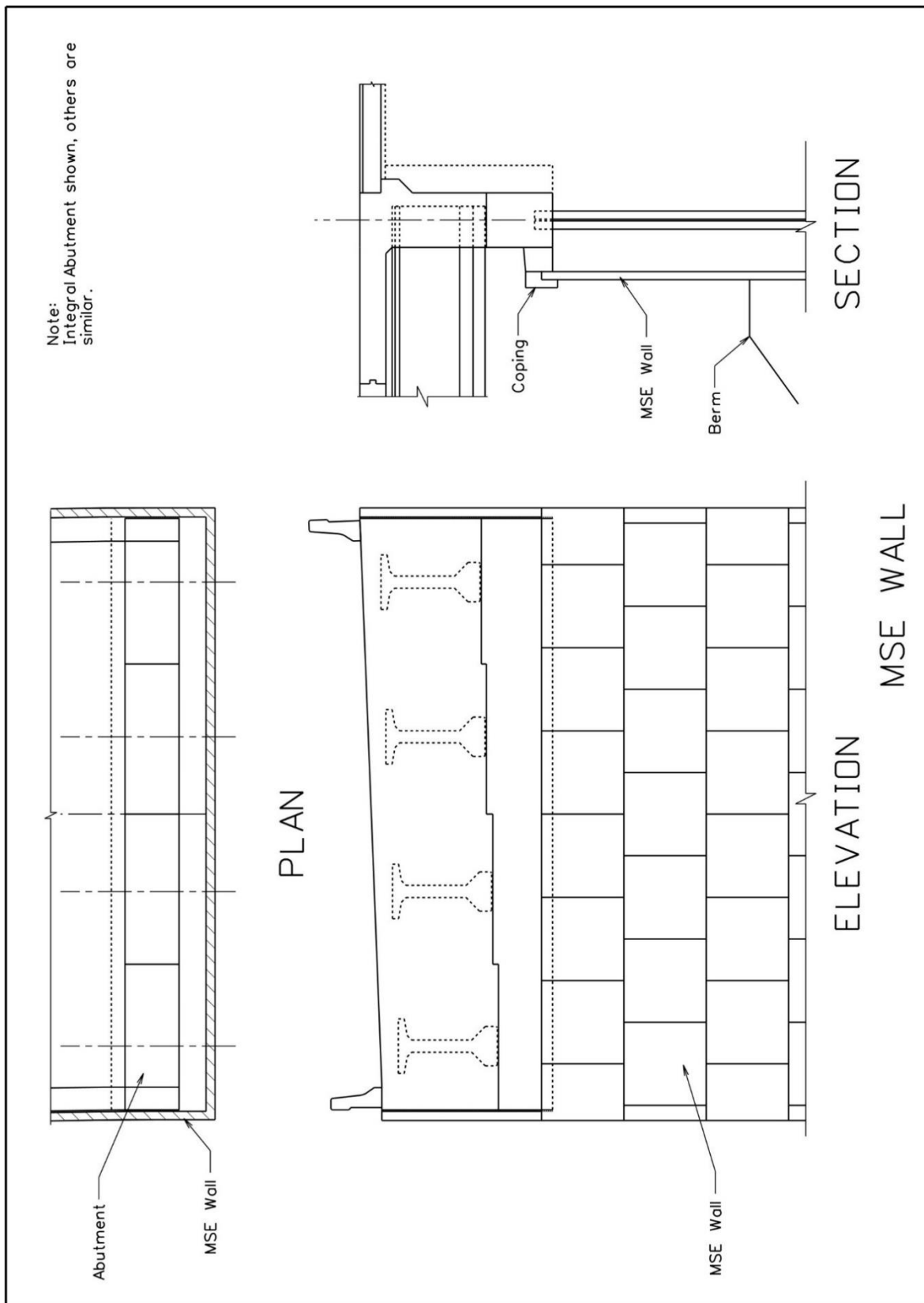
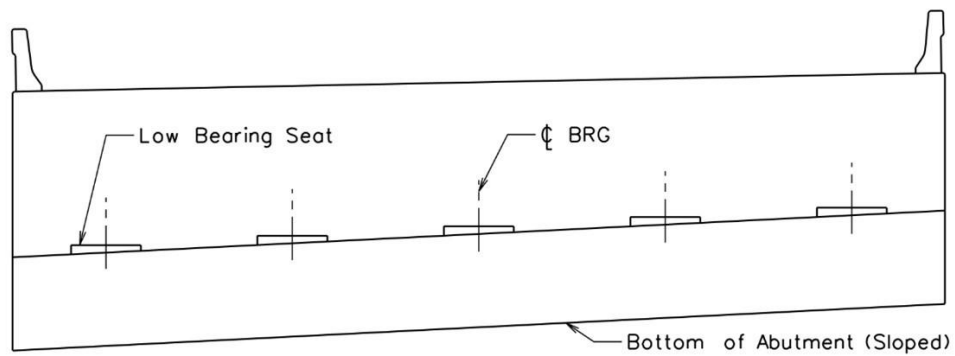
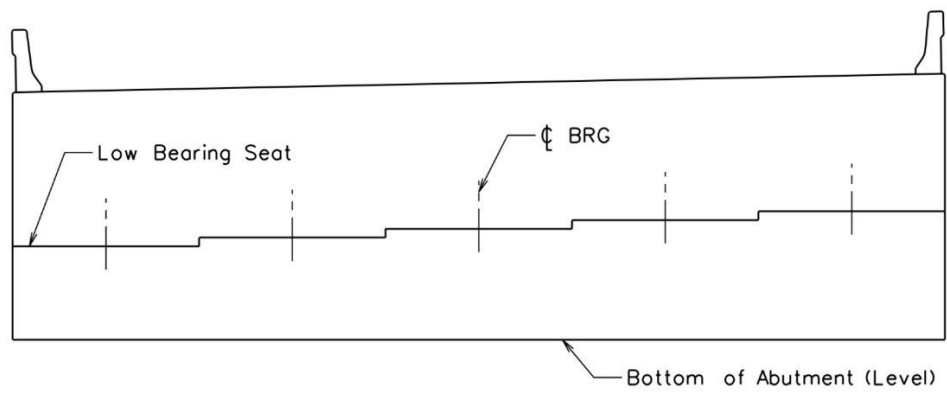
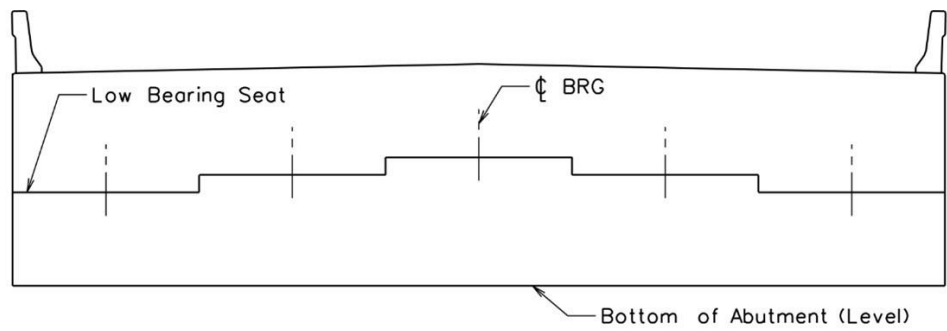


Figure 2080.D



BRIDGE SEATS

Figure 2080.E

2080.3-BACKWALLS

A backwall shall be provided on all bridges to retain the embankment behind the bridge, support the approach slab (if applicable) and to protect the bridge seat from water intrusion. The minimum thickness for a backwall shall be one (1) foot. For backwall requirements on jointless bridges. See SD 2090.

A roughened horizontal construction joint shall be provided between the bridge seat and the backwall extending the entire width of the abutment. This construction joint may be optional for small abutments. Another method is to provide an optional construction joint one (1) feet below the paving notch. Concrete above this joint shall not be placed until the deck slab is in place.

The roadway elevations at the face of the backwall shall be given on the plans.

2080.4-STEM

The stem is a structure that retains the embankment behind the abutment and transmits the loads from the superstructure to the foundation. While there is no minimum thickness requirement, the stem thickness shall be designed to support the required loading and provide sufficient space for bearing devices.

Stems may be level, stepped or sloped based on bridge geometry. SEE SD 2080.2.

2080.5-WINGWALLS

Wingwalls must be long enough to retain the roadway embankment based on the embankment slopes. The positioning of the wingwalls depends on the site itself (e.g., cut vs. fill, etc.). The Designer must study the existing and proposed contours and determine which type of wingwall best fits the site. Approach roadway slopes must be considered.

Generally, an embankment slope of 2:1 is used. When this is not possible, 1½:1 may be acceptable, with approval of the Bridge Project Manager. Wingwalls may be placed parallel to the roadway, at some angle or on the same alignment as the centerline of abutment bearings.

The following calculations and Figure 2080.F are representative examples of how to determine wingwall lengths. The Designer must consider the requirements stated above when laying out wingwalls and determining their length.

Calculation of wingwall lengths:

- D = Distance from shoulder to berm
- S = Embankment slope (S = 2 for a 2:1 slope)
- q = Angle from backwall to wingwall
- d = Width of bridge seat
- WL = Wingwall length

Wingwalls where $q > 90^\circ$ (right wingwall in Figure 2080.F):

$$WL = \frac{D(S)}{\cos(q - 90)} - d + 1 \text{ FT}$$

Wingwalls where $q \leq 90^\circ$ (left wingwall in Figure 2080.F):

$$WL = D(S) - d + 1 \text{ FT}$$

Wingwalls shall be designed as horizontal cantilevers when supported by the abutment or as vertical cantilevers if they are supported by a foundation.

A vertical construction joint may be required between the wingwall and abutment. The need for a construction joint is determined based on size and configuration. There are situations where the wingwall is a separate structure from the abutment. This requires an expansion seal or other expansion material between the two components.

2080.6-FOUNDATION

Abutment and wingwall foundations shall be designed and located as specified in SD 2120, Foundations.

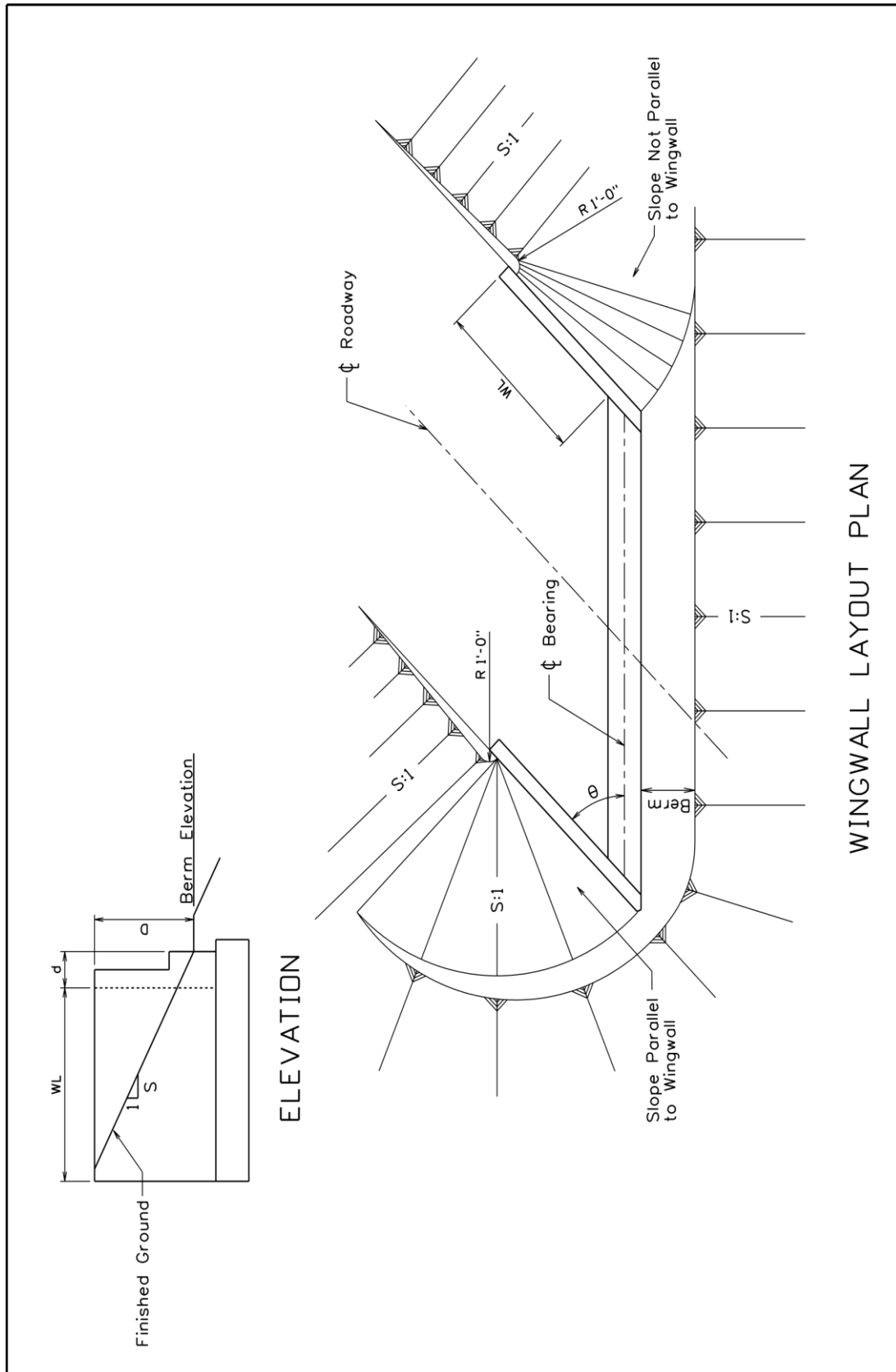


Figure 2080.F

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2090
JOINTLESS BRIDGE ABUTMENTS**

August 6, 2025

Supersedes May 4, 2022

Fully integral and semi-integral abutments shall be used whenever possible to eliminate deck expansion joints. Jointless bridge abutments serve to reduce maintenance and construction costs, improve aesthetics and long-term serviceability of a structure. Integral abutments are the preferred jointless bridge abutments and shall be used when the anticipated thermal movement is two (2) inches or less. Semi-integral abutments may be used for instances that are not appropriate for integral abutments. Steel diaphragms or cross frames at bearings are not required for superstructures with integral or semi-integral abutments. If required for construction stability, temporary bracing may be placed adjacent to the abutment and removed after the concrete has cured.

2090.1-CRITERIA FOR INTEGRAL ABUTMENTS

Integral abutments refer to short stub-type abutments connected rigidly to the bridge deck without joints. This rigid connection allows the abutment and the superstructure to act as a single structural unit. As a result, the bridge superstructure, abutment, and foundation piles are all subjected to cyclic loading.

2090.1.1-Approach Slabs: Approach slabs are encouraged for all integral abutments and are required for integral abutments having a total anticipated thermal movement exceeding ½ inch. Approach slabs shall be anchored to the abutment by reinforcing steel bars and detailed to accommodate appropriate thermal movements. Approach slabs shall be twenty (20) feet unless geometrically constrained. In such cases, the length of the approach slab should be maximized. Approach slabs shall be in accordance with SD 2140 and the most recent Standard Details Book, Volume III.

2090.1.2-Expanded Polystyrene (EPS): Integral abutments shall utilize a minimum six (6) inch thick expanded polystyrene material behind the abutment to reduce earth pressure forces generated by thermal movement. Refer to the most recent Standard Details Book, Volume III for expanded polystyrene detailing behind approach slabs. Omission of expanded polystyrene material shall require Project Manager's approval and supplemental structural and geotechnical design outside of the scope of this directive.

2090.1.3-Bridge Length: The maximum bridge length for which integral abutments are recommended shall be 400 feet and 600 feet for steel and concrete superstructures respectively. These bridge lengths are based on past experience with jointless bridges which did not show serious construction or maintenance problems.

When economically feasible, the use of integral abutments on bridges with lengths exceeding the limits specified herein should be investigated. Special design considerations and details may be necessary for integral abutments on bridge lengths exceeding those specified.

The ratio between the span lengths in the bridge shall be chosen such that no net negative beam reaction is produced at any limit state unless approved by the Project Manager. No net negative force shall be allowed on any pile at any limit state.

2090.1.4-Skew Angle: Earth pressure acts in a direction perpendicular to the abutments. For skewed bridges, the earth pressure forces on the abutments produce a torque that causes the bridge to twist in plan. This twisting can cause cracking at the ends of the deck. Limiting the skew angle reduces this effect. The maximum allowable skew angle is:

- A. 20° for integral abutments without approach slabs.
- B. 20° for multiple span structures with adjacent end spans greater than 140 feet.
- C. 30° for multiple span structures with adjacent end spans less than 140 feet.
- D. 30° for single span steel structures less than 140 feet or single span concrete structures between 140 feet and 90 feet.
- E. 45° for single span concrete structures less than 90 feet.

Integral abutments for simple span structures should be skewed at the same angle. Different skew angles are allowed for multiple span structures but shall not differ by more than 10°. Superstructures may be skewed greater than the above limits with the State Bridge Engineer's approval and supplementary calculations and detailing, as required, to limit transverse movement.

2090.1.5-Horizontal Alignment: Straight beams should be used with integral abutments. Curved superstructures utilizing straight beams may be used when centrifugal forces are considered in the integral abutment's design. Horizontally curved beams are discouraged but are allowed if they meet all the following criteria:

- A. Beams are concentric.
- B. Bearing lines are not skewed more than 10° from radial.
- C. The stiffnesses of all beams is similar in both planes.
- D. The span eccentricity does not exceed 2.5 percent of the span length.
- E. Crossframes or diaphragms are designed as primary members and installed at spacing not to exceed 25 feet for steel I-Beam superstructures.
- F. Appropriate centrifugal forces are considered in the integral abutment's design.

Integral abutments outside of the above limits may be used with the State Bridge Engineer's approval and supplementary calculations and detailing as required.

2090.1.6-Vertical Alignment: Integral abutments may be used on bridges with vertical curvature. The grade between the abutment and nearest fixed support should not exceed five percent (5%). Integral abutments on steeper slopes may be used with additional design considerations.

2090.1.7-Geometric Constraints: Integral abutments measured from the top of deck to bottom of pile cap shall not exceed ten (10) feet. A taller integral abutment may be allowed with supplementary calculations and thicker expanded polystyrene layer with the Project Manager's approval. The berm elevation should be approximately constant along the front face of the integral abutment to avoid uneven earth pressure during contraction of the bridge.

The height of the pile cap section shall be minimized to avoid developing high passive earth pressures or premature failure of the expanded polystyrene. The pile cap portion of the integral abutments shall be no less than three (3) feet by three (3) feet.

Wingwalls should be cantilevered and shall not extend more than six (6) feet from the edge of the abutment transversely nor extend more than twelve (12) feet longitudinally. Wingwalls shall have a minimum thickness of twelve (12) inches. The wingwalls may be tapered as needed for aesthetics and constructability. Cantilevered wingwalls shall be connected to the abutment using a one (1) foot chamfer. Refer to Figure 2090.A.

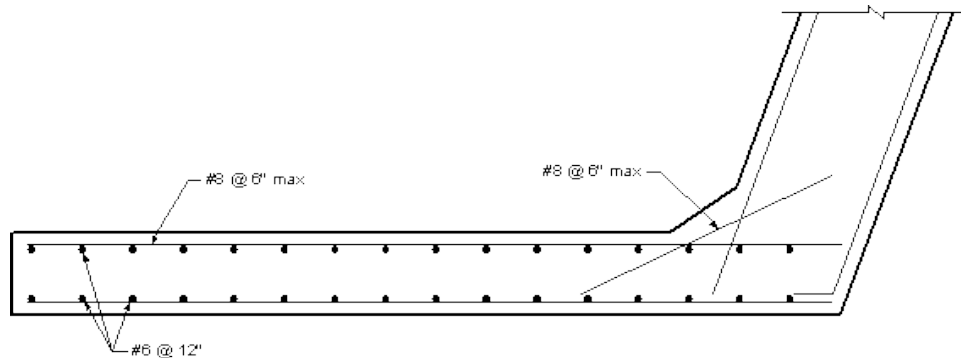


Figure 2090.A

Structurally isolated wingwalls may be used for instances that are not appropriate for cantilevered wingwalls. Appropriate preformed joint material shall be used between the integral abutment and structurally isolated wingwalls to allow unhindered thermal movement. The use of structurally isolated wingwalls with integral abutments requires approval from the Project Manager.

2090.1.8-Piling Constraints: Piles shall be a single row of steel H-Piles aligned so that the flanges are parallel to the direction of thermal movement. Piles shall be HP12 or HP14 sizes unless otherwise approved by the Project Manager. All piles shall be embedded into the pile cap a minimum of two (2) feet. Additional embedment may be necessary for integral abutments with large thermal movements. The distance from the side of any pile to the nearest edge of the abutment shall not be less than nine (9) inches. Pile spacing shall not exceed ten (10) feet unless approved by the Project Manager. Piling lengths of ten (10) feet minimum to fifteen (15) feet shall be predrilled to the top of rock. Piling lengths greater than fifteen (15) feet shall be predrilled a minimum of fifteen (15) feet. Pre-drilling shall be in accordance with Section 616 of the Specifications. Pile points are permitted but are not considered a substitute for pre-drilling integral abutment piling. Piling shall not be allowed under wingwalls.

It is good practice to position piles directly under bridge beams for load transfer. Where this is not practical, the distance between the centerline of any given beam to the centerline of the nearest pile should not exceed the effective shear depth (d_v) of the pile cap. If this requirement cannot be satisfied the pile cap shall be designed using the Strut-and-Tie Methodology. For adjacent box beam bridges the pile cap may be designed assuming superstructure forces are distributed uniformly across the abutment's width. A minimum of four (4) piles shall be used

unless otherwise approved by the State Bridge Engineer. Piles shall be evenly spaced using the same pile size.

2090.1.9-Closure Pour: To reduce cracking in integral abutments, a closure pour consisting of the backwall, and an adjacent “X” feet minimum of deck shall not be placed until all other deck pours have been placed and cured. The distance “X” is equal to 0.5 feet + the effective slab length measured from the front face of the abutment.

2090.1.10-Scour and Drainage: Protective countermeasures are recommended around all integral abutments to avoid erosion. Protective countermeasures are required around all integral abutments for which any of the following criteria are satisfied:

- A. Water surface elevations from the design storm reaches the abutment.
- B. Water surface elevations from the check storm reaches the bottom of the beams.
- C. The toe of the abutment fill falls within ordinary high water.
- D. The abutment fill slope exceeds 4:1.
- E. Scuppers are present near the abutment or along the abutment’s fill slope.

Design and check storms are defined within the West Virginia Department of Transportation, Division of Highways Drainage Manual. Integral abutments supporting bridges in scour prone areas shall be evaluated for structural stability after scour has occurred. The evaluation shall show the bridge does not incur damage that hinders its ability to be operational after the roadway fill has been re-established.

2090.2-DESIGN OF INTEGRAL ABUTMENTS

Integral abutments and piles shall be designed to resist all applicable force effects. Integral abutments are assumed to act as pinned connections. Flexural forces between the pile cap and closure pour shall be evaluated by modeling the frame action for integral abutments that do not meet the requirements of SD 2090.1.4 – SD 2090.1.8.

2090.2.1-Loads and Load Combinations

2090.2.1.1-Permanent Loads: All permanent loads on the abutment including beam dead load, wearing surface, approach slabs, abutment self-weight, attached wingwalls, down drag, etc. shall be considered. Permanent loads that are not transferred through the bridge beams, excluding those from cantilevered wingwalls, may be assumed to act uniformly across the abutment width. Self-weight of cantilevered wingwalls may be assumed to act as point loads at the ends of the abutment pile cap. Permanent loads carried by the approach slab should be applied to the abutment assuming the approach slab acts as a simple span beam between the abutment and sleeper slab.

2090.2.1.2- Live Loads: All live loads on the abutment including truck loads, lane loads, pedestrian loads and impact forces shall be considered. Impact forces may be omitted or reduced for the design of the piles in accordance with AASHTO LRFD Bridge Design Specifications.

For the design of the integral abutment and the piles, live loads are assumed equally distributed to all beams in the cross section due to the high rigidity of the abutment. Multiple presence factors shall be omitted for abutments carrying more than two (2) design lanes to avoid underestimation of pile forces on wide bridges where the length-to-depth ratio of the abutment beam is relatively high. The total live load on the abutment shall be determined assuming the largest number of traffic lanes that may be allowed as defined within AASHTO LRFD Bridge Design Specifications. For the design of integral abutments, the live load reaction of any beam may be computed as follows:

$$R_V = \frac{R_{BRG} N_L}{g_{INT} N_B}$$

Where:

R_V	=	Beam reaction for integral abutment design
R_{BRG}	=	Interior beam reaction from the beam design program
g_{INT}	=	Interior beam shear distribution factor from the beam design program
N_B	=	Number of beams in the cross section
N_L	=	Maximum number of traffic lanes allowed by the bridge clear width

Live load from the approach slab shall be considered and applied assuming the approach slab acts as a simple span beam between the abutment and sleeper slab. It is unlikely the position of the design vehicle that produces maximum beam reactions will concurrently produce significant live load on the approach slab therefore, in absence of more thorough analysis, only lane load needs be applied to the approach slab for integral abutment design.

Centrifugal forces shall be applied to integral abutments that lie within horizontal curves. The centrifugal forces may be applied as an increase in vertical live load from rotation of the superstructure as a rigid body.

Braking force shall be applied to the integral abutment for the design of the closure pour, backwall, and pile cap. Thermal movement controls the design of the piles therefore braking forces should not be considered in their design.

2090.2.1.3-Wind Loads: Wind uplift on the superstructure and transverse wing loads on the superstructure and on live load should be considered in the design of integral abutments for large multiple span bridges. The magnitude of wind forces on small bridges does not often control the design of integral abutments and may be omitted based on engineering judgement.

2090.2.1.4-Other Transverse Loads: The load combination for which water and ice loads are investigated does not often control the design of integral abutments and may be omitted based on engineering judgement.

2090.2.1.5-Thermal Movements: The integral abutment thermal movement due to uniform temperature change shall be computed as:

$$\Delta = \alpha \Delta t_{max} L$$

Where:

Δ	=	Thermal movement due to uniform temperature change
α	=	Coefficient of thermal expansion of the bridge material
Δ_{tmax}	=	Uniform temperature difference ($\Delta_{tmax} = 115^{\circ}\text{F}$ for steel superstructure) ($\Delta_{tmax} = 85^{\circ}\text{F}$ for concrete superstructure)
L	=	Distance from abutment centerline and point of fixity

The design thermal movement at the abutment shall be based on the difference in temperatures specified in SD 2012.1.6 and temperature at which the pile cap is made integral with the superstructure. The structural deck concrete shall be poured and allowed to cure between 45°F and 85°F therefore the design thermal deflection shall be calculated from a uniform temperature difference (Δ_{tmax}) of 115°F and 85°F for steel and concrete superstructures respectively.

The length between centerline of abutment bearing and nearest point of fixity (L) shall be used to compute the thermal deflection. The point of fixity shall be defined as the location on the bridge which does not move longitudinally during changes in temperature. Appropriate stiffnesses shall be considered in determining the point of fixity.

The distance to fixity (L) should be equal to half of the span length for simple spans with approximately constant width and height with integral abutments at both ends. For simple spans with differing widths or heights with integral abutments at both ends, the distance to fixity (L) for integral abutments may be estimated as:

$$L_{A1} = L_{BRIDGE} \left(\frac{W_{A2} H_{A2}}{W_{A1} H_{A1} + W_{A2} H_{A2}} \right) \geq 0.4 L_{BRIDGE}$$

$$L_{A2} = L_{BRIDGE} - L_{A1} \geq 0.4 L_{BRIDGE}$$

Where:

L_{A1}	=	Distance from Abutment 1 to the point of fixity
L_{A2}	=	Distance from Abutment 2 to the point of fixity
L_{BRIDGE}	=	Bridge length
W_{A1}	=	Abutment 1 width
H_{A1}	=	Abutment 1 average height
W_{A2}	=	Abutment 2 width
H_{A2}	=	Abutment 2 average height

The distance to fixity (L) should be equal to the span length for simple spans with integral abutment at only one end. The distance to fixity (L) is defined as the distance from the abutment to the nearest fixed bearing for continuous structures where the fixed bearing is founded on a stiff

substructure. For all other configurations the thermal movement for the subject integral abutment shall be calculated as:

$$\Delta_i = \frac{F}{k_i}$$

$$F = k_{eq} \Delta_{tmax}$$

$$\frac{1}{k_{eq}} = \sum_{i=1}^n \frac{1}{k_i}$$

Where:

Δ_i	=	Design deflection of the i-th abutment
F	=	Force required to deflect bridge the design movement (Δ_{tmax})
k_i	=	Spring stiffness of the i-th substructure unit
k_{eq}	=	Equivalent spring stiffness of springs in series
n	=	Number of substructure units
Δ_{tmax}	=	Uniform temperature difference

The spring stiffness of each substructure unit shall be determined with consideration of bearing movement, pier deflections, pile deflection, etc. as appropriate. For horizontally curved bridges the consideration of out of plane thermal deflection should be considered as needed. The pile flanges shall be oriented parallel to the assumed direction of thermal movement for horizontally curved bridges. See Figure 2090.B.

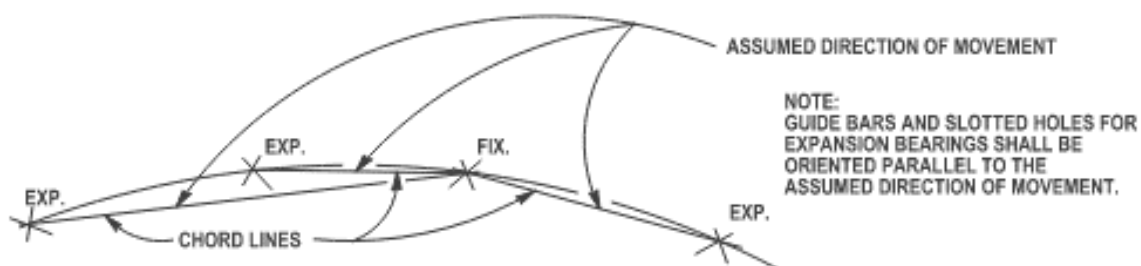


Figure 2090.B

2090.2.1.7-Other Loads: Secondary load effects from creep and shrinkage shall be considered. Other secondary loads may be evaluated at the engineer's discretion.

2090.2.2-Abutment Design: The components of the abutment are made up of the combined closure pour and backwall, pile cap, piles and wingwalls. Longitudinal forces shall be considered for design of the abutment for out of plan flexure and bi-axial bending. Passive earth pressure shall be computed in accordance with code requirements. Earth pressure forces may be reduced for

integral abutments using expanded polystyrene but shall not be less than 40 PCF equivalent fluid pressure.

2090.2.2.1-Closure Pour: Reinforcing steel within the closure pour shall match standard deck details. Refer to Figure 2090.C.

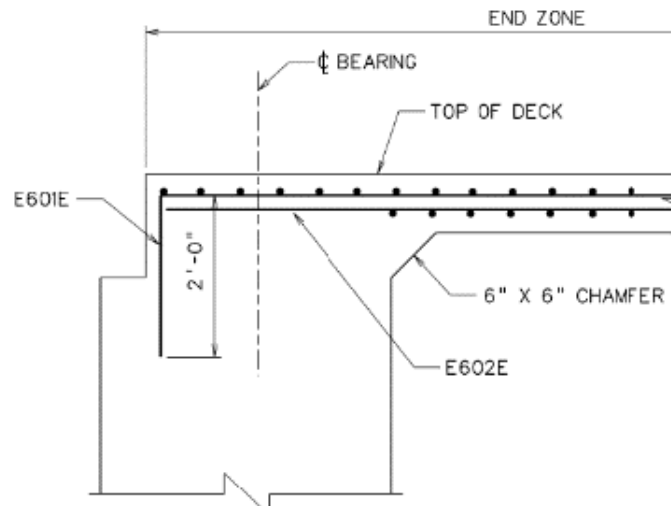


Figure 2090.C

2090.2.2.2-Backwall: The backwall shall be designed without consideration of strength from reinforcing steel in the closure pour. The backwall shall be designed as a horizontal beam resisting earth pressure and all other applicable horizontal forces. Refer to Figure 2090.D.

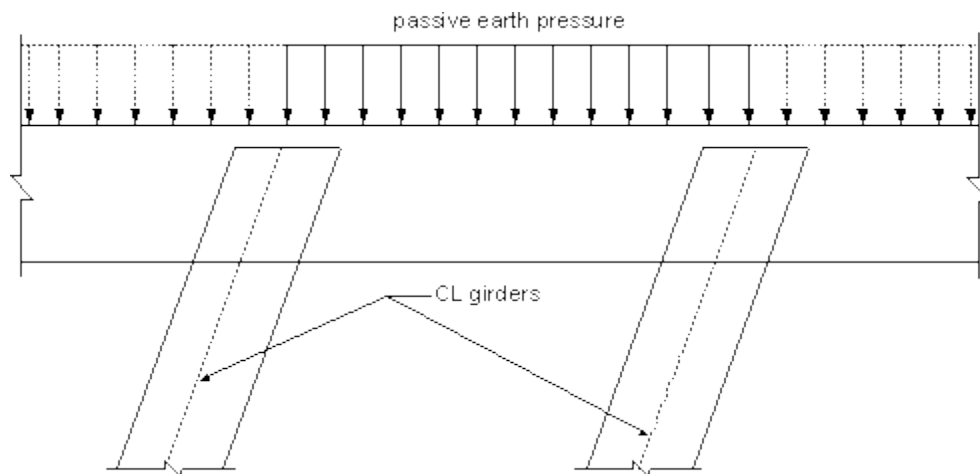


Figure 2090.D

It is best practice to use the same longitudinal reinforcing steel bar size in the front and back faces to avoid unintentional construction errors. The longitudinal reinforcing should be detailed for maximum flexure force effects unless significant cost saving can be realized. Longitudinal bars shall be contained within closed stirrups designed for appropriate shear effects.

Additional longitudinal reinforcing bars shall be placed through holes drilled or cast in the beam ends. The beam anchorage reinforcing steel shall be #8 bars at twelve (12) inches maximum spacing, placed 5½" from the front face of the abutment. A minimum of three (3) longitudinal bars are required to pass through the beam web unless otherwise approved by the Project Manager. Longitudinal bars should be detailed to lap or mechanically couple between beams for ease of placement.

Vertical reinforcing steel shall be approximately equally spaced across the width of the abutment except at beams where modifications may be necessary. The closure pour – pile cap interface shall be connected by fully developed vertical reinforcing steel. Flexure forces at the closure pour – pile cap interface shall be computed assuming frame action for integral abutments not meeting the requirements of SD 2090.1.7.

2090.2.2.3-Pile Cap: The pile cap shall be designed in accordance with AASHTO LRFD Bridge Design Specifications with consideration of both vertical and horizontal forces. Primary flexural reinforcing bars shall be of equal number and size in the top and bottom mat, shall be continuous along the cap's length if possible, and be hooked at the pile caps ends. Secondary flexural reinforcing bars shall be of equal number and size in the front and back mats and shall be continuous along the cap's length if possible. If required, splices in mats of reinforcing steel should be made near points of contraflexure. Shear stirrup size and spacing shall remain constant along the cap's length unless significant savings can be realized. Shear stirrups shall enclose flexural reinforcement except where they conflict with the embedded piles in which case U-Shape stirrups shall be used. Punching shear of the pile through the cap without consideration of the backwall shall be investigated when the positioning of beam and piles requires Strut-and-Tie modeling as specified in SD 2090.1.

2090.2.2.4-Cantilevered Wingwalls: Cantilevered wingwalls shall be designed in accordance with AASHTO LRFD Bridge Design Specifications with consideration of both vertical and horizontal forces. Horizontal forces from earth pressure introduce torsion in rectangular wingwalls that shall be considered as part of their design.

Constant thickness rectangular cantilevered wingwalls are recommended for ease of construction. It is best practice to use the same bar size for horizontal tension reinforcing steel unless significant savings can be realized. Wingwalls should use the same reinforcing steel bar size throughout to avoid unintended construction errors if practical.

2090.2.2.5-Isolated Wingwalls: Isolated wingwalls may be used with integral abutments when cantilevered wingwalls are not practical. Preformed joints shall be used to allow free expansion of the integral abutment without contacting the isolated wingwalls. Foundations for isolated wingwalls shall also be isolated from the abutment foundation. Use of isolated wingwalls with integral abutments requires the Project Manager's approval.

2090.2.3-Pile Design: Steel piles shall be designed in accordance with AASHTO LRFD Bridge Design Specifications with the P-Δ methodology using LPILE or similar software package. Steel pile design shall use resistance factors defined within AASHTO LRFD Bridge Design

Specifications for driven piles. Steel piles subject to thermal movements less than ½” may be designed without consideration of the flexural forces introduced as part of the lateral analysis. Design of piles using lateral analysis shall follow the following steps:

STEP 1: Perform borings, field reconnaissance and computations to establish geotechnical parameters needed to complete lateral pile analysis. Modify geotechnical parameters as specified within AASHTO LRFD Bridge Design Specifications for closely spaced piles.

STEP 2: Select pile size based on available information. Use axial loads to estimate pile size if flexural forces are not yet available.

STEP 3: Create LPILE model assuming a fixed head condition. Apply shear and moments at the pile head to establish the deflected fixed head condition. Limit moment to the plastic moment of the pile. Run modeling.

STEP 4: Summarize flexural forces predicted from modeling. Document the first point of zero moment (L_{i1}), second points of zero moment (L_{i2}) and maximum moment between the two points (M_u) as shown within Figure 2090.E.

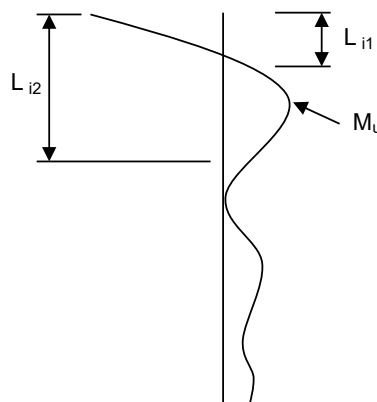


Figure 2090.E

STEP 5: Determine axial pile capacity with an unbraced length equal to the difference between the first and second points of zero deflection.

STEP 6: Design pile in accordance with AASHTO LRFD Bridge Design Specifications for axial and flexural force M_u . Return to Step 3 if pile does not have sufficient capacity.

2090.3-CRITERIA FOR SEMI-INTEGRAL ABUTMENTS

Semi-integral abutments may be used for instances that are not appropriate for integral abutments and where foundation sites rule out the use of an abutment on a single row of piles.

2090.3.1-Approach Slabs: Approach slabs are required for all semi-integral abutments unless approved by the State Bridge Engineer. Approach slabs shall be anchored to the semi-integral abutment diaphragm by reinforcing steel bars and detailed to accommodate appropriate thermal movements. Approach slab geometry shall meet the requirements of SD 2090.1.1.

2090.3.2-Expanded Polystyrene (EPS): Expanded polystyrene for semi-integral abutments shall be in accordance with the requirements of SD 2090.1.2. Expanded polystyrene shall be placed along the full height of the diaphragm that thermally deflects. The thickness of the expanded polystyrene material behind the abutment shall be derived from engineering computations where thermal movements exceed 2 inches but shall not be less than twelve (12) inches.

2090.3.3-Skew Angle: Semi-integral abutments experience similar earth pressure and torsional effects as described in SD 2090.1.4. Semi-integral abutments may be used for skew angles up to 30° without supplementary analysis. Semi-integral abutments for structures should be skewed at the same angle. Different skew angles are allowed for multiple span structures but shall not differ by more than 10°. Superstructures may be skewed outside these limits with the Project Manager's approval and supplementary calculations and detailing, as required, to limit transverse movement.

2090.3.4-Horizontal Alignment: Semi-integral abutments are subject to the horizontal alignment requirements of SD 2090.1.5. Semi-integral abutments that do not meet those requirements may be used with the Project Manager's approval.

2090.3.5-Geometric Constraints: The pile cap portion of a semi-integral abutment shall be no less than three (3) feet by three (3) feet. Wingwalls shall meet the same geometric constraints as described in SD 2090.1.7.

2090.3.6-Foundations: Semi-integral abutments shall be supported by direct bearing on rock, steel piling, drilled caissons, or micropiles. Geotechnical information and design shall be used to establish the preferred foundation type.

2090.3.6.1-Piling Constraints: A minimum of two rows of steel H-Piles aligned so that the flanges are perpendicular to the direction of thermal movement shall be used. All piles shall be embedded into the pile cap a minimum of two (2) feet. The distance from the side of any pile to the nearest edge of the abutment shall not be less than nine (9) inches. Pile size, drilling, and driving criteria shall be at the discretion of the Geotechnical Engineer. The front row of piles may be tapered to resist longitudinal loads if necessary.

2090.3.6.2-Drilled Caisson Constraints: A single row of caissons shall be used and designed to resist applicable longitudinal loads. All caissons shall be embedded into the pile cap a minimum of twelve (12) inches. Caisson reinforcing steel shall be fully developed in accordance with AASHTO LRFD Bridge Design Specifications. The distance from the side of any caisson to the nearest edge of the abutment shall not be less than nine (9) inches. Drilled caissons shall be a minimum of three (3) feet in diameter unless otherwise approved by the Project Manager.

2090.3.6.3-Micropile Constraints: Micropiles should be investigated where access is limited or when longitudinal loads would require taper of conventional piling. Micropile foundations shall

be designed and detailed in accordance with FHWA Micropile Design and Construction Guidelines.

2090.3.7-Abutment Seat: A full-length curb on the top of the semi-integral abutment stem shall be used to help retain backfill when the bearing height exceeds 1 ½ inches. Neoprene seals shall be used for waterproofing. Refer to Standard Details – Volume III.

2090.4-DESIGN OF SEMI-INTEGRAL ABUTMENTS

Semi-integral abutments and piles shall be designed to resist all applicable force effects in accordance with AASHTO LRFD Bridge Design Specifications. Semi-integral abutments shall be limited to 4 inches of thermal movement computed in accordance with SD 2090.2.1.5.

2090.5-SEMI-INTEGRAL ABUTMENT CONVERSION

The conversion of an existing non-integral abutment to a semi-integral abutment should be carefully considered in major rehabilitation projects. These considerations should include at a minimum the cost premium for the semi-integral abutment conversion, anticipated rehabilitated bridge service life and differences in maintenance costs. Project Manager approval is required for any semi-integral abutment conversion.

Applicable requirements of SD 2090.4 shall apply for all semi-integral abutment conversions unless otherwise approved. The engineer shall carefully consider the life cycle costs and risks of using semi-integral abutment conversions which fall outside the recommendations of SD 2090.4. All new works associated with the semi-integral abutment conversions shall be designed to resist all applicable force effects in accordance with AASHTO LRFD Bridge Design Specifications.

The preferred method of conversion involves full height removal of the backwall to bearing seat elevation and reconstruction in accordance with Standard Details – Volume III. When this method of construction poses challenges, partial height backwall removal may be considered. The condition of remaining segments of backwall shall be in good condition or rehabilitated as needed.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2100

RETAINING WALLS

May 4, 2022

First Edition

Retaining walls are used to control fills and may be used in cut situations. They may be necessary due to the geometric constraints of a roadway or as a measure against erosion. Typically, retaining walls are utilized to transition the roadway grade around bridge abutments to the embankment or side slopes of the obstacle being spanned by the bridge. The height of the fill being retained mandates the type of retaining wall to be designed.

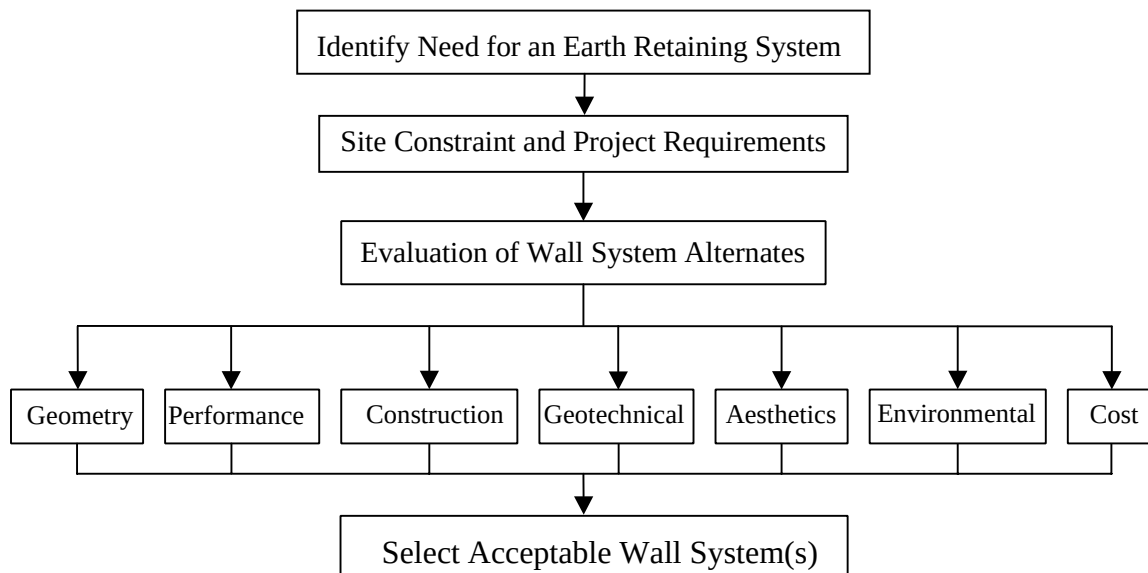
The selection and design of the appropriate retaining wall system is an integral part of the structural design of highway projects. Proper application of retaining system type and the use of proprietary retaining systems requires guidance and advance approval of the wall types and their manufacturers.

The decision to select a particular retaining wall system for a specific project requires an evaluation based on cost-effectiveness, practicality of construction, slope and structural stability, aesthetics, and environmental consistency with its surroundings.

The most common types of retaining wall systems are gravity, cantilever, mechanically stabilized earth (MSE) walls, pile and tie-back (anchored) walls. Gravity, cantilever, pile and tie-back walls are considered conventional retaining walls.

MSE walls are proprietary walls. This wall type requires the designer to complete certain design elements (e.g., external and global stability) and the wall supplier to complete other design elements (e.g., internal stability). MSE walls will typically be chosen from a list of pre-approved suppliers or manufacturers.

The following shall serve as a guide to illustrate the selection process.



**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2101
RETAINING WALL SYSTEM SELECTION PROCESS**

*May 4, 2022
First Edition*

2101.1-IDENTIFY NEED FOR AN EARTH RETAINING SYSTEM

A decision on the permanency of the wall system is to be made. If sufficient right-of-way (ROW) is available, a stable soil slope may be constructed thus eliminating the need for a wall system.

2101.2-SITE CONSTRAINTS AND PROJECT REQUIREMENTS

Items affecting the wall selection can be gathered during a preliminary site review and can include the following:

- A. Site accessibility and space restrictions that may include limited ROW and headroom, availability of on-site storage for wall materials, access for specialized construction equipment, and restrictions on traffic disruption.
- B. Maximum estimated wall height.
- C. Location of underground utilities and nearby structures.
- D. Aesthetic requirements imposed by project surroundings.
- E. Environmental concerns including permitting requirements and encroachment on existing waterways (scour).

Based on these items, several wall systems may be eliminated from consideration while others may be recommended for further consideration. Insight on subsurface conditions may also be gained based on site investigations.

2101.3-EVALUATION OF WALL SYSTEM ALTERNATES

The remaining wall systems shall then be evaluated. This involves a more detailed study based on cost, design, performance, and construction. In addition to cost, the following specific factors shall be evaluated:

- A. Wall system geometry
- B. Performance requirements
- C. Constructability issues
- D. Aesthetic requirements
- E. Environmental concerns and requirements

Based on the above study, one or more of the remaining wall systems may be eliminated from further consideration. Finally, based on geotechnical information, cost estimates and project requirements, the preferred wall systems are then selected for further consideration.

2101.4-WALL SYSTEM SELECTION

Fill walls are generally constructed in conjunction with fills such as bridge approach embankments. Cut walls are generally constructed in situations where the finished grade will be substantially below the existing grade or for bridge approaches in a cut situation. Tables 2101.A, 2101.B and 2101.C are selection charts for cut and fill wall systems that are commonly used. These tables summarize the key selection factors and are modified from the FHWA publication, *Earth Retaining Systems* – FHWA-SA-96-038. Fill wall systems can be constructed in a cut situation; however, temporary shoring as well as space for excavation to construct the wall will be required.

Height ranges are given in the chart for various wall types but should only be used as a guide for economy. The WVDOT has certain height limitations on individual wall systems as discussed in SD 1043.1

Wall Type	Perm.	Temp.	Cost Effective Height Range	Lateral Movements	Water Tightness	Advantages	Disadvantages
Sheet-Pile Wall ⁽²⁾	X	X	Up to 16 Feet	Large	Fair	• Rapid construction • Readily available	• Difficult to construct in hard ground or through obstructions
Soldier Pile/Lagging wall	X	X	Up to 16 Feet	Medium	Poor	• Rapid construction • Soldier Piles can be drilled or driven	• Difficult to maintain vertical tolerance in hard ground • Potential for ground loss at excavated face
Tie-Back or Anchored Wall	X	X	15-60 Feet ⁽¹⁾	Small – Medium	N/A	• Can resist large horizontal pressure • Adaptable to varying site conditions	• Requires skilled labor and specialized equipment • Anchors may require permanent easements

(1) For soldier pile and lagging wall only.

(2) Sheet-Pile wall can also be used in FILL situation as a temporary wall when feasible.

Table 2101.A - System selection chart for CUT walls

Wall Type ⁽²⁾		Perm.	Temp.	Cost Effective Height Range	WVDOH Height Range	Differential Settlement Tolerance ⁽¹⁾	Advantages	Disadvantages
Concrete Gravity Wall				3-10 Feet		1/500	• Durable • Requires smaller quantity of select backfill as compared to MSE walls • Concrete can meet aesthetic requirements	• Deep foundation support may be necessary • Relatively long construction time
Concrete Crib Wall		X		6-35 Feet		1/300	• Does not require skilled labor or specialized equipment • Rapid construction	• Difficult to make height adjustments in field
Concrete Cantilever Wall		X		6-30 Feet		1/500	• Durable • Requires smaller quantity of select backfill as compared to MSE walls • Concrete can meet aesthetic requirements	• Deep foundation support may be necessary • Relatively long construction time
Concrete Counterforted Wall		X		30-60 Feet		1/500	• Durable • Requires smaller quantity of select backfill as compared to MSE walls • Concrete can meet aesthetic requirements	• Deep foundation support may be necessary • Relatively long construction time
MSE Wall (Precast Facing)		X		10-65 Feet	0 - 60 Feet	1/100	• Does not require skilled labor or specialized equipment • Flexibility in choice of facing	• Requires use of select backfill Subject to corrosion in aggressive environment (metallic reinforcement)
MSE Wall (Wire Face)		X	X	6-50 Feet	0 - 60 Feet	1/60	• Does not require skilled labor or specialized equipment • Flexibility in choice of facing	• Facing may not be aesthetically pleasing • Geosynthetic reinforcement is subject to degradation in some environments
MSE Wall (Mod. Block)	Class I	X		6-23 Feet	0 - 40 Feet	1/200	• Does not require skilled labor or specialized equipment • Flexibility in choice of facing • Blocks are easily handled	• Requires use of select backfill • Subject to corrosion in aggressive environment (metallic reinforcement) • Positive reinforcement connection to blocks is difficult to achieve
	Class II				0 - 20 Feet			
	Class III				0 - 10 Feet			
	Class IV				0 - 5 Feet			

(1) Ratio of the difference in vertical settlement between two points along the wall to the horizontal distance between points.

(2) Sheet-Pile wall can be used in FILL situations as a temporary wall if feasible.

Modular Block Walls have the following Classifications	
Class	Wall Type
I	HITEC approved systems
II	Soil Reinforcement has a positive connection to the Wall Face.
III	Soil Reinforcement has a friction connection to the Wall Face.
IV	Has no Soil Reinforcement

Table 2101.B – System selection chart for FILL walls

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2102
PERMANENT RETAINING WALLS**

*May 4, 2022
First Edition*

2102.1-GRAVITY WALLS

Gravity walls come in many forms and are designed to resist overturning moments and horizontal sliding forces by their own weight. They shall be designed so that the resultant force acts within the kern of the section (within the middle third of the footing base). Two acceptable forms include un-reinforced concrete or cribbing walls (either timber or concrete). The backfill should be a free-draining granular soil to prevent water pressure build-up.

2102.2-CANTILEVER WALLS

The cantilever retaining wall is the most economical type of conventional retaining wall system and is widely used. They are comprised of a concrete stem and a base footing, which form an inverted “T”. The wall is fully reinforced to resist applied loads. The design is based on the assumption that the base and stem are fixed at their junction and the stem acts as a cantilevered beam. The footing should be founded below the frost line. Cantilevered walls can be supported on spread footings, piles or rock socketed drilled shafts depending on geotechnical recommendations. The stem may be tapered or straight, as the design requires. Drains should be located at regular intervals to prevent water pressure from acting on the back of the wall. Counterforts may be employed where needed to accommodate taller fills.

2102.3-PILE WALLS

There are two accepted types of pile walls: sheet pile walls and soldier pile walls. Sheetpile walls consist of driven, vibrated, or pushed interlocking steel or concrete pile sections. The design theory assumes that the passive resistance of the soil in front of the wall plus the flexural strength of the sheet-pile effectively resists the lateral forces from the soil behind the wall.

Solder-pile walls are constructed of vertical steel H-piles with concrete or timber lagging between them. Its design theory is based on internal arching of the soil acting on the piles. The function of the lagging is to support the soil so that the arching effect is not interrupted. Soldier-pile walls can be constructed with tiebacks.

2102.4-TIE-BACK WALLS (ANCHORED WALLS)

Tie-back walls or anchored walls are non-gravity cantilever walls that rely on anchors for additional lateral support. An anchor or tie-back is a structural system engineered to transfer tensile loads from the earth to the wall. The principal disadvantage of tie-back walls is the anchoring devices. They must be carefully evaluated for corrosion resistance. The non-gravity wall may be either reinforced concrete or soldier-pile with lagging.

2102.5-PROPRIETARY WALLS

Mechanically stabilized earth walls are considered proprietary walls. They consist of concrete facing units that are anchored to either steel reinforcing strips or steel bar mats to allow the loads to be transferred through shear to the concrete face. Generally, these walls are suitable for most applications under thirty (30) feet in height. Situations that require taller walls may take advantage of a stepped MSE configuration. Various finish techniques can be called for in construction plans to meet aesthetic requirements. Contact a manufacturer for additional details. Refer to the WVDOT Approved Product List (APL) website for a list of approved MSE wall vendors.

The Manufacturer shall design the tiebacks and connections to resist the loads and accommodate the wall layout provided to them. Supply the following data to the manufacturer:

- A. Vertical geometry: a general cross-sectional sketch of the wall in relation to the traffic lanes.
- B. Existing grade elevations at constant intervals or stations.
- C. Finished grade elevations or top of wall elevations at constant intervals or stations.
- D. Horizontal geometry: a plan view of wall and its relation to traffic lanes.
- E. Core borings.
- F. Soil parameters.
 - 1. Unit weight (wet).
 - 2. Angle of internal friction.
 - 3. Unit cohesion.
 - 4. Ultimate bearing capacity.
 - 5. Ph.
- G. External loads and special conditions.
 - 1. Truck loads.
 - 2. Seismic condition.
 - 3. Traffic condition.
- H. Spread footings or abutments located in reinforced volume.
- I. Miscellaneous appurtenance loads such as coping requirements.
- J. Location of potential obstructions to wall system components (e.g. piling, drainage structures, light poles, sign supports, etc.).

The contract plans for a bridge containing an MSE Wall shall refer to the system as the generic term "MSE" and allow for a competitive bid between the approved Manufacturers.

2102.5.1-Concrete Panel: Facing panels are typically square, rectangular, hexagonal or cruciform in shape and employ either metallic or geosynthetic reinforcement that is positively connected to the panel to create a reinforced soil mass.

3.10.2.5.2-Modular Block: Modular block walls consist of vertically stacked, dry cast concrete blocks that employ geogrid, metallic grid or geotextile reinforcement. The reinforcement may be connected to the wall face through friction developed between vertically adjacent blocks or through the use of special connectors.

2102.5.3-Wire Faced: Wire faced walls consist of continuous or semi-continuous layers of geotextile, geogrid or welded wire mesh laid down alternately with horizontal layers of compacted

soil backfill. The wall facing can be constructed by wrapping each layer of reinforcement around the overlying layer of backfill, then re-embedding the free end into the backfill. Other systems have a panel facing that can be attached to the wire mesh.

2102.5.4-Advantages and Disadvantages: The advantages of a MSE wall system are:

- A. Wall system construction is relatively rapid and does not require specialized labor or equipment.
- B. Limited foundation preparation is required.
- C. Concrete panel and wire faced wall systems are flexible and can accommodate relatively large total and differential settlements without distress. Modular block systems can accommodate relatively large settlements without distress.
- D. Reinforcement is light and easy to handle. Modular blocks are also light and easy to handle.
- E. Concrete panels permit greater flexibility in the choice of facing and architectural finishes.
- F. Modular blocks permit flexibility in the choice of sizes, shapes, weights, textures and colors, and can also adapt to sharp curves and significant front batter.

The disadvantages of a MSE wall system are:

- A. The wall system may not be economical for cut applications due to the additional cost associated with constructing temporary shoring to provide sufficient base width for construction.
- B. Wall systems require relatively large base widths.
- C. The use of metallic reinforcement requires that the backfill meet minimum electrochemical requirements for corrosion protection.
- D. The allowable load for geosynthetic reinforcement must be reduced to account for creep, durability and construction damage.
- E. Wire faced walls may not meet aesthetic requirements and the geotextile or geogrid life may be reduced due to exposure to ultraviolet light.
- F. Wall systems may not be appropriate for the following applications:
 - 1. Where it may be necessary to gain future access to underground utilities.
 - 2. At locations subject to scour - involving significant horizontal curvature (panel walls).
- G. Typically, wall systems are not cost-effective for temporary applications.

2102.5.5-Designer Notes:

- A. The Designer must analyze each wall system for external stability (e.g., sliding, overturning, bearing, global stability) according to the Governing Specifications. The wall manufacturer shall determine internal stability for MSE walls.
- B. The minimum embedment length of reinforcing for MSE walls shall not be less than seventy percent (70%) of the wall height. Shorter reinforcement lengths, but no less than eight (8) feet, can be considered if smaller compaction equipment is used, facing panel alignment can be maintained, and minimum requirements for wall external stability are met.
- C. The bottom of all wall system footings or leveling pads shall be located below frost depth (minimum 36 inches for West Virginia). As an alternative, the soil within the

- depth and lateral extent of frost penetration below the wall may be removed and replaced with non-frost susceptible clean granular soil.
- D. MSE walls, under most circumstances, are the most economical choice of wall systems when compared to traditional gravity or cantilever type cast-in-place concrete walls. Since the list of approved MSE wall systems has increased, the need to have a CIP wall alternative on a project may not be necessary.
 - E. A special design is required if the wall must support structure foundations, other retaining walls, noise walls, or other types of surcharge loads. The wall is considered to be supporting the surcharge load if the surcharge is located within a 1H:1V slope projected from the bottom of the back of the wall. For MSE walls, the back of the wall is considered the back of the soil reinforcement layers.
 - F. When aesthetics govern, MSE walls should be held to a maximum height of thirty (30) feet when possible. Limitations imposed on MSE wall systems by the WVDOH are based on a combination of technical evaluation report data, FHWA demonstration projects, NHI training course literature and taking into account human factors when dealing with tall structures and the traveling public.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2103
TEMPORARY RETAINING WALLS**

*May 4, 2022
First Edition*

Temporary walls may be required for some projects utilizing staged construction. They may also be necessary for extreme events where failures of slopes or existing walls occur, and temporary measures are needed to maintain safety and service. Under these circumstances, temporary walls are subject to the same design requirements, including shop drawing review and approval as for permanent wall systems.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2110

PIERS

August 6, 2025

Supersedes May 4, 2022

Piers are intermediate supports in a multi-span bridge system. All feasible pier types must be considered in the preliminary phases of a project. Refer to SD 1044 for a general description of various types of piers. The use of integral pier caps, steel bents, and prestressed pier components fall outside the scope of this directive. In such cases the Engineer shall coordinate with the Project Manager to establish geometric and design constraints.

2110.1-PIER CAP GEOMETRY

All pier caps must be wide enough to accommodate the bridge bearings and jacking points. The edge of the bearing masonry plates must be a minimum of three (3) inches from the face of the pier cap. Instances of pier caps being wider than the column, necessitated by skewed bearings and dual bearings are acceptable.

Pier cap depths shall be determined by strength and clearance requirements. The minimum size is three (3) feet vertically by three (3) feet horizontally and must extend one (1) foot beyond the fascia beam bearings. Pier caps are usually haunched in the region beyond the face of the exterior column or stem. Any pier cap longer than five (5) feet beyond the face shall be haunched unless approved by the Project Manager. A haunch ratio between 2:1 and 4:1 shall be used unless approved by the Project Manager. The use of parabolic haunches or similar complex geometry should be avoided to reduce forming costs. The ends of the pier caps may be either plane vertically or shaped (i.e., cylindrical to mirror the columns).

Beam seats shall be stepped and finished level. The sloped stem option, per SD 2080, is preferred when steps exceed four (4) inches.

2110.2-PIER CAP DESIGN

Pier caps shall be designed to resist all applicable force effects in accordance with AASHTO LRFD Bridge Design Specifications. The Engineer shall use the strut and tie modeling for thick concrete elements as defined within AASHTO LRFD Bridge Design Specifications. Use this method to determine internal force effects near supports and points of concentrated loads. Class B Concrete with a compressive strength of 3,000 PSI should be used for piers. If required for strength, Class B Modified concrete with a compressive strength of 4,000 PSI may be used.

Minimum horizontal reinforcement shall be #5 bars spaced at twelve (12) inches. Reinforcing steel shall be placed to avoid anchor bolts. Tension steel bars shall not be larger than #11 nor spaced closer than six (6) inches to limit concrete consolidation problems unless approved by the Project Manager. Tension steel bars may be bundled, or a second row of reinforcing steel placed six (6) inches below the top mat may be used to attain additional flexural capacity. The pier

cap geometry should be re-evaluated if sufficient flexural capacity cannot be attained within these limits.

2110.3-PIER COLUMNS

The minimum longitudinal column diameter or stem thickness shall be three (3) feet unless otherwise approved by the Project Manager. Column tapers or section changes shall not be used unless cost savings can be demonstrated. Multi-column piers should be evaluated if the diameter of the stem thickness of a single-column pier exceeds eight (8) feet. Hollow shafts piers should be evaluated for columns in excess of one hundred (100) feet in height or ten (10) feet in diameter. Multi-column piers should generally not be used in a flood plain. Single circular column, T-type or wall type piers may be used in the flood plain with rounded ends and shall be oriented parallel to the stream flow. Crash walls shall be considered in accordance with AASHTO LRFD Bridge Design Specifications and AREMA requirements.

2110.4-PIER COLUMN DESIGN

Pier columns shall be designed to resist all applicable force effects in accordance with AASHTO LRFD Bridge Design Specifications. Torsional forces from non-symmetric longitudinal loads shall be considered on single-column pier shafts. A second-order nonlinear analysis ($P-\Delta$ analysis) is preferred over approximate methods of moment magnification. A second-order analysis shall be used for all columns with slenderness (KL/r) exceeding 50 unless otherwise approved by the Project Manager.

2110.5-PIER FOUNDATIONS

Pier foundations shall be located and designed as specified in SD 2120, Foundations.

2110.6-EVALUATION OF EXISTING PIERS

Major rehabilitation projects may involve repurposing or reusing existing piers. Where superstructure replacements, deck replacement or similar work are done the Engineer shall evaluate the increase in deadload on the pier and, if information is available, estimate the load carrying capacity of the existing foundations. Where insignificant increases in deadload are found and a field assessment shows no signs of distress, settlement and similar global issues the pier may be reused without the need for retrofitting. Where these increases are not insignificant, or the superstructure configuration allows for more traffic lanes than the existing condition, a more thorough evaluation may be necessary. All aspects of evaluation, repurposing and reuse of existing piers shall require approval from the Project Manager.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2120

FOUNDATIONS

May 4, 2022

First Edition

2120.1-GENERAL

All substructures are to be founded upon bedrock; whether by spread footings, piles or rock socketed drilled shafts. Only end bearing piles, either driven or predrilled and driven, are acceptable. Friction or combination friction and end bearing piles shall not be used.

The Geotechnical Report shall list design assumptions and recommend appropriate foundations. The Designer shall determine if the structure can accommodate the design assumptions (i.e., settlement, lateral movement, etc.).

2120.2-SPREAD FOOTINGS

Spread footings shall be proportioned to distribute the total vertical and horizontal forces in such a manner that the required structural stability is obtained and that the allowable design bearing pressures are not exceeded.

Minimum thickness for a spread footing is three (3) feet and shall be keyed into rock a minimum of one (1) foot. Allowable bearing pressures shall be used to size the footing.

For grade separation structures, the top of the footing shall be a minimum of one (1) foot and preferably two (2) feet below the finished ground line. The top of the footing should be at least one (1) foot below the bottom of any adjacent drainage ditch. Deeper footings may be necessary to find the spread footing on competent material. The Designer shall consider constructability with regard to adjacent structures such as railroad tracks, existing foundations, etc. Cofferdams may be necessary to facilitate construction. The designer shall review railroad requirements to assure compliance.

For water crossing structures, the bottom of footings should be founded on competent bedrock. Unless directed otherwise, the top of the footing shall be located below the anticipated scour event. An exception to placing the top of the footing below the scour event may be founding the footing in non-scourable bedrock. Non-scourable bedrock is defined as rock having a minimum RQD of fifty percent (50%) and is resistant to slaking, as determined by the Engineer. Cofferdams may be required for construction and shall be anticipated in the plans.

Due to the possibility of stream meander, footings placed in the overflow section of the water crossing shall be investigated to assure the footings will remain stable should the stream migrate.

2120.3-PILE FOUNDATIONS

In order to facilitate the driving of non-predrilled steel bearing piles the following minimum size of piles shall be specified:

- A. HP 10x42 for lengths up to forty (40) feet.
- B. HP 12x53 for lengths between forty (40) and eighty (80) feet.
- C. HP 14x73 for lengths over eighty (80) feet.

The design may require larger piles, for the above lengths, based on vertical and/or lateral loads. The minimum piling length shall be ten (10) feet. The pile embedment is a minimum of one (1) foot into the cap. The Designer shall verify the availability of any section and pile accessories necessary for construction. See SD 1050, Piling, for additional information.

For water crossing structures, the top of the pile cap shall be placed below the anticipated scour event. See AASHTO 2.6.4.4.2.

Steel H piles, as a minimum, shall conform to the requirements of AASHTO M160, and M270, Grade 36. High strength steel piling such as Grade 50 may be used.

The Geotechnical Engineer shall determine the required bearing capacity and perform a drivability analysis to determine if the pile can be installed to the required elevation without damage. The Geotechnical Report should also recommend if pile tips are required.

Pile footings shall be proportioned to distribute the total vertical and lateral forces in such a manner that the required structural stability is obtained and that the allowable design bearing values are not exceeded. Minimum thickness for a pile footing is three (3) feet. Generally, the pile footing should be designed so that no pile experiences uplift. If uplift is anticipated, the pile-cap connection shall be checked along with the pullout resistance of the pile.

The Designer shall check steel H piles as a structural column when determining the necessary pile size. This is especially critical for trestle type bents after the anticipated scour event. If bracing is required, it shall be designed and detailed on the plans along with connection details.

The location of battered piles should be checked to ascertain that the piles remain within the right-of-way and do not interfere with piles from adjacent structures, existing substructure units nor conflict with portions of staged construction.

The Designer shall provide any special provisions that will be necessary for any load test required for the design.

In congested areas and/or areas adjacent to existing structures, vibration and noise during pile driving and predrilling shall be considered. The Designer shall provide appropriate mitigation measures to the Bridge Project Manager. The Designer shall provide any special provisions regarding vibration and noise remediation.

For grade separation structures, the footing shall be designed and located as specified in SD 2120.2.

2120.4-ROCK SOCKETED DRILLED SHAFT FOUNDATIONS

Rock socketed drilled shafts shall be designed in accordance with the Governing Specifications and shall accommodate both vertical and lateral loads. The rock socket shall extend below the anticipated scour depth and remain adequate to support the bridge, ignoring passive earth pressures.

If site conditions dictate that temporary casings may be uneconomical, the use of permanent steel casings should be considered. The Designer shall proportion the permanent steel casings, including wall thickness and material properties.

The effect of drilled shaft construction on existing structures shall be considered, with appropriate mitigation measures.

In congested areas and/or areas adjacent to existing structures, vibration and noise during construction shall be considered, especially during installation of steel casings, both temporary and permanent. The Designer shall provide appropriate mitigation measures to the Bridge Project

Manager. If necessary, the Designer shall provide any special provisions regarding vibration and noise remediation.

The Designer shall provide any special provisions that will be necessary for any load test required for the design.

2120.5-SLOPE AND FOUNDATION PROTECTION

Foundation protection is the armoring of a bridge foundation, for protection from scour caused by moving water, as in the case of a stream channel crossing. Slope protection is the armoring of an embankment, for protection from erosion caused by surface runoff, in a grade separation crossing. If erosion is to be prevented, embankment and foundation protection must be anticipated, and the proper type and amount of protection must be provided. Slope protection may also be used in urban locations and on Interstates where aesthetics is a factor or where maintenance is impractical.

Section 218 of the Specifications describes the types of slope and foundation protection and the construction methods for their installation.

2120.5.1-Slope Protection: Slope protection is required at all bridge embankments. Slope protection shall be eight (8) inches crushed rock with confinement curbing. Confinement curbing is reinforced concrete curbing a minimum of six (6) inches wide by two (2) feet high used to retain the crushed rock slope protection on bridge embankments (see Standard Specifications).

Due to failures encountered with wire mesh baskets; gabions will no longer be used for slope protection.

If the Designer is confident that an abundance of adequate material exists, select embankment, meeting the requirements of the rock borrow specifications, may be specified for slope protection. Otherwise, rock borrow shall be specified on the plans.

The slope protection shall generally extend three (3) feet beyond the outside limits of the superstructure fascias and from the toe of the slope to the abutment face. Slope protection is not required between dual structures when the distance between adjacent slope protection edges exceeds ten (10) feet, unless deemed necessary for aesthetic or maintenance purposes.

2120.5.2-Foundation Protection: Foundation protection shall be used to protect bridge embankments, abutments and pier foundations from erosion and streambed scour at stream channel crossings. If the Designer is confident that an abundance of adequate material exists, select embankment, meeting the requirements of the rock borrow specifications, may be specified for foundation protection. Otherwise, rock borrow shall be specified on the plans.

Foundation protection design procedure shall follow the following Federal Highway Administration Publications:

- A. Design of Riprap Revetment, Hydraulic Engineering Circular No.11, (HEC 11).
- B. Evaluating Scour at Bridges, Fourth Edition, Hydraulic Engineering Circular No. 18, (HEC 18).

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2130

SCOUR

May 4, 2022

First Edition

Scour potential shall be considered in all designs. Wherever necessary, adequate details shall be incorporated into the design and plans to minimize the effects of scour. All designs will be performed in accordance with the Federal Highway Administration Technical Advisory T5140.23, "Evaluating Scour at Bridges". A DS-34 form will be completed during the design phase of the project. Refer to WVDOH Bridge Maintenance Directive (BMD) S-102-2 for additional information regarding the DS-34 form.

For structures designed by a consulting engineering firm, the Consultant will submit Form DS-34 to the WVDOH at the time of submittal of the preliminary drawings. The Consultant Review Section will transmit the DS-34 form to the appropriate District Bridge Engineer who will submit a copy to the Highway Operations Division Evaluation Section at the time of the initial inventory of the bridge, when the new bridge is opened to traffic.

For structures designed by the Engineering Division In-House Design Section, the DS-34 form will be submitted to the appropriate District Bridge Engineer at the time of completion of the PS&E. The District Bridge Engineer will submit a copy to the Highway Operations Division Evaluation Section at the time of the initial inventory of the bridge, when the new bridge is opened to traffic.

For structures designed in the district office, a copy of the DS-34 form will be submitted with the TS&L or with the preliminary plans for review and approval by Engineering Division. A copy of the DS-34 form will be submitted to the Highway Operations Division Evaluation Section at the time of the initial inventory of the bridge when the new bridge is opened to traffic.

It will be WVDOH policy to design new structures to be classified as having "no scour potential" or if it is classified as having scour potential, it will be classified as being "low risk". In this regard, Form 21, Item 23, should be coded as 5, 7, 8, 9 or N.

The section of Form DS-34 form related to screening (Item 12 through Item 18) and the section on corrective action (Item 26 through Item 29) will not be required to be completed for a new structure.

Scour calculations are based upon the discharge created by the flood of one percent (1%) annual incidence of return (Q_{100}) and the "super flood" defined as 0.2% annual incidence of return (Q_{500}). Scour depth, average stone size (D_{50}) and any necessary designs shall be based upon the provisions of the following FHWA publications:

- A. *Design of Riprap Revetment*, Hydraulic Engineering Circular No.11, (HEC 11).
- B. *Evaluating Scour at Bridges*, Fourth Edition, Hydraulic Engineering Circular No. 18, (HEC 18).

Other publications considered relevant concerning the topic of bridge scour are:

- A. *Stream Stability at Highway Structures*, Third Edition, Hydraulic Engineering Circular No. 20, (HEC 20).
- B. *Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance*, Second Edition, Hydraulic Engineering Circular No. 23, (HEC 23).

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2140

APPROACH SLABS

May 4, 2022

First Edition

2140.1-GENERAL

Approach slabs shall be required on all bridges except those on State Local Service Roads with an ADT less than 500 and an ADTT less than 100. Jointless bridges may require approach slabs per SD 2090.1.

2140.2-DESIGN

The approach slab length shall be a minimum of twenty (20) feet measured along the centerline of the roadway or edge of pavement. The minimum width of the approach shall be from gutter line to gutter line of the bridge. The thickness of the approach slab shall consist of twelve (12) inch of concrete topped with two (2) inches of asphalt skid resistant overlay (14 inch total thickness).

2140.2.1-Reinforcement: Longitudinal reinforcement shall consist of No. 5 bars spaced at twelve (12) inches (top) and No. 8 bars spaced at six (6) inches (bottom).

Transverse reinforcement shall consist of No. 5 bars spaced at twelve (12) inches (top and bottom).

2140.2.2-Joint Type Between Approach Slab and Approach Pavement or Bridge Transition Pavement: For integral bridges, a Type H joint (Standard Detail Sheet PVT2) is required to accommodate the thermal movement when using flexible approach pavement. Rigid bridge transition pavement requires a Type B joint (Standard Sheet PVT1) between the approach slab and the bridge transition pavement for movements up to 0.25 inch and a Type J joint (Standard Sheet PVT5) for movements greater than 0.25 inches.

Bridges with expansion joints require a Type H joint (Standard Detail Sheet PVT2) when the approach pavement is flexible and a Type A joint (Standard Sheet PVT1) when the bridge transition pavement is rigid.

2140.2.3-Detailing: The approach slab detail sheet(s) included in the plans shall be an all-inclusive sheet(s) with pay items, quantities, and bar schedule. The items on this sheet are considered roadway pay items and are included in the roadway summary and estimate of quantities.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2150
LOAD RATING OF NEW BRIDGE DESIGN**

January 4, 2023

Supersedes May 4, 2022

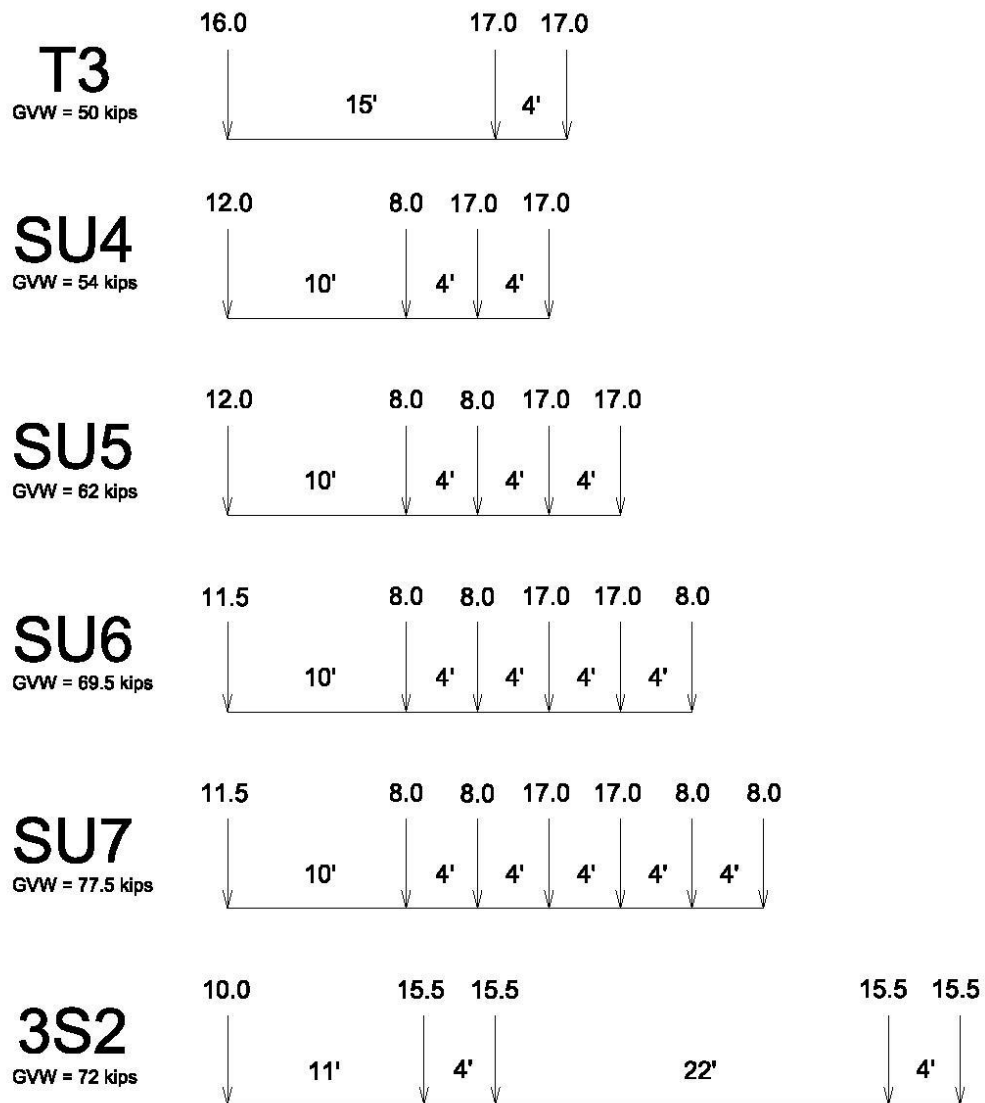
Load and Resistance Factor Rating (LRFR) is consistent with the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD BDS) in using a reliability-based limit states philosophy and extends the provisions of these specifications to the areas of inspection, load rating, posting and permit rules, fatigue evaluation, and load testing of existing bridges. The LRFR methodology has been developed to provide uniform reliability in bridge load ratings.

2150.1-LOAD RATING OF NEW AND REPLACEMENT BRIDGES

Load rating analysis shall be performed for all new or replacement bridges, including value engineering or value engineering change proposals submitted by the contractor, using the LRFR method found in the current edition of the AASHTO Manual for Bridge Evaluation (MBE). All applicable limit states per MBE Table 6A.4.2.2-1 will be satisfied, including those listed as optional checks.

Each bridge shall be load rated at inventory and operating levels for AASHTO's HL93 loading as presented in the MBE on all routes. In addition, a legal load evaluation shall be completed for each West Virginia legal load on all routes. Bridges on a Coal Resource Transportation System (CRTS) route shall be load rated for four additional trucks (WV-SU40, WV-SU45, WV-3S55, and WV-3S60) during the legal load evaluation. The axle configurations and loads for the WV Legal Trucks are shown in Figure 2150.A, CRTS Trucks and Emergency Vehicle Live Loads are shown in Figure 2150.B, and Wood "Pup" Truck Live Loads are shown in Figure 2150.C.

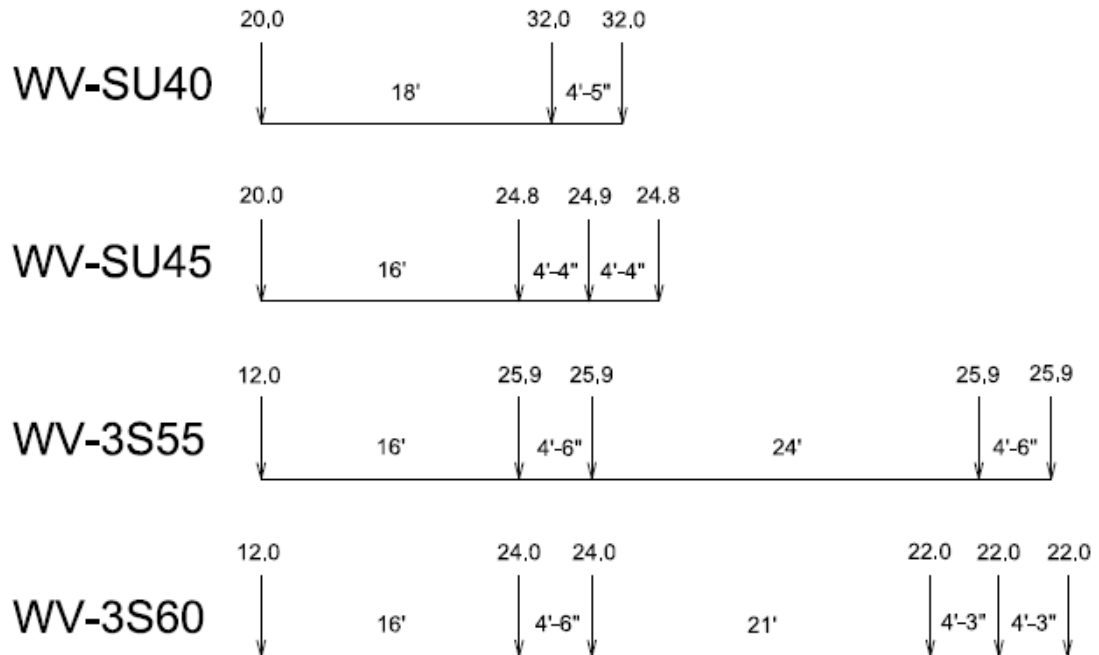
LEGAL VEHICLE LIVE LOADS



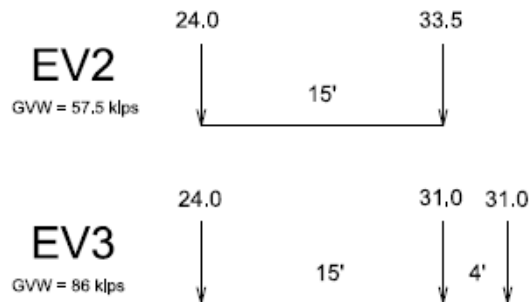
Note: All axle weights are in Kips

Figure 2150.A

CRTS VEHICLE LIVE LOADS



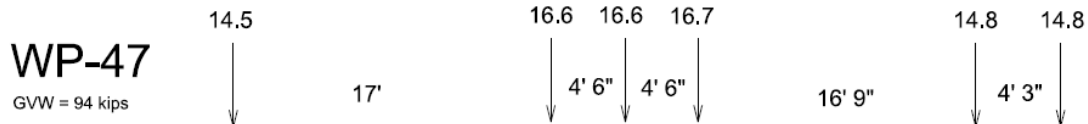
EMERGENCY VEHICLE LIVE LOADS



Note: All axle weights are in kips

Figure 2150.B

WOOD "PUP" TRUCK PERMIT LOAD



Note: All axle weights are in kips

Figure 2150.C

The bridge load rating analysis using the LRFR method shall be performed concurrent with the beam/girder final design to assure proper design and adequate rating. The target ratings for new or replacement bridge designs are shown in Table 2150.D.

Table 2150.D -Target Ratings

Design- Inventory (Factor)	Legal (Tons)													
	Standard							CRTS*				Emergency		Permit**
HL93	Type 3	SU4	SU5	SU6	SU7	Type 3S2	Lane***	SU- 40	SU- 45	3S- 55	3S- 60	EV2	EV3	WP47
1.00	41	49	51	51	51	55	50	53	60	73	79	36	54	59

* Required for CRTS (Coal Resource Transportation System) routes only, considered as legal load evaluation

** Although WP47 is a permitted truck, it shall be load rated as a legal load configuration

*** Not required for single spans less than and equal to 200 feet

The designer will perform the load rating and submit all required information, as detailed in this article, to the bridge project manager. A request for an independent load rating check shall be submitted to the Evaluation Section of Operations Division (OM) by the bridge project manager during the load rating submission. The request shall contain the following information:

- A. Load rating sheets containing tabulated section properties, live load distribution factors (and conversion factors, if needed), dead load moments and shears, and live load moments and shears at critical locations in each span and at all supports.
- B. Controlling rating factors (design) and tonnages (legal loads) for all required configurations, as shown in Table 2150.D.
- C. A full set of current bridge design plans.
- D. The CP/RW date of the project.

If requested, the designer shall also be required to submit to OM, through the bridge project manager, original rating computations included with the design calculations and shall clearly identify or include the following information:

- A. Design specifications.
- B. Design live load.
- C. Member capacities.
- D. Method of analysis – line girder, grid, or finite element.
- E. Method used for calculation of live load distribution factors.
- F. Live load distribution factors.
- G. Table of applicable load factors.
- H. Controlling limit states.
- I. Design, legal, and permit ratings for all required loadings for consultant designed bridges if required by project scope.
- J. Relevant computer input and output information for consultant designed bridges if required by project scope.

Upon submission, OM or the District Office will perform an independent load rating of the bridge. If the independent load rating check performed by the WVDOT indicates a rating less than any of the target values (shown above in Table 2150.D), the bridge project manager in coordination with the Evaluation Section of Operations Division shall be contacted immediately to determine what actions are to be taken before proceeding further with the final design and detailing.

The designer shall state in the plans when redistribution of negative moments is utilized for use in the permit rating of the bridge. See AASHTO LRFD BDS 4.6.4.

2150.2-LOAD RATING OF NEW OR REPLACEMENT FRAMES, ARCHES, THREE SIDED STRUCTURES AND CULVERTS

The load rating analysis shall be performed by the designer in accordance with the governing specifications and the MBE using the live load models presented in this document. If it is determined the depth of fill is such that live load effects can be neglected (per AASHTO MBE), the structure would have an infinite safe load capacity for live loads as long as the structure has residual capacity remaining after dead load effects have been considered.

A 3D Finite Element Analysis shall be performed for any structure constructed on a longitudinal slope to determine the out of plane load effects on the structure in the final condition. This requirement shall apply to structures with their entire longitudinal length or any portion of the structure on a longitudinal slope.

Calculations shall be submitted to the bridge project manager for approval prior to fabrication of any primary structural elements.

2150.3-LOAD RATING OF REHABILITATED OR WIDENED STRUCTURES

Load rating of structures using combination specifications within the superstructure (e.g. a superstructure designed by LRFD for the new widened superstructure elements and the original superstructure elements designed by Load Factor Design) shall not be permitted.

Load rating of structures partially reconstructed resulting in the use of combination of specifications between substructure and superstructure elements (e.g. a reconstructed

superstructure designed by LRFD supported by the original substructure designed by Allowable Stress Design, Load Factor Design, or unknown specifications) is permitted. The method of analysis for a reconstructed superstructure shall be LRFR.

2150.4-CONVERSION FACTORS FOR REFINED ANALYSIS

When structures are designed using refined analyses, conversion factors shall be developed. The refined analyses methods include line girder analyses based on refined live load conversion factors, grid analyses and finite element analyses. The conversion factors indicate the relationship of live load design moments and shears obtained from the refined analysis to the live load moments and shears obtained from a standard line girder analysis with a live load distribution factor of 1.0 for a single lane (a single lane equals two wheels). The conversion factors for both the maximum and minimum moments and shears are developed separately for each live load under consideration. For example, the Type-3 legal load configuration for a continuous span steel I-girder bridge would have conversion factors provided for maximum and minimum moment effects, as well as maximum and minimum shear effects.

Do not use AASHTO distribution factors for the line girder analysis.

The conversion factors for refined analyses shall be computed using the following equation:

$$CF = \frac{\text{Moment or Shear (refined analysis)}}{\text{Moment or shear (line girder analysis)}}$$

Subsequent analyses of the structure may be completed using a standard line girder analysis with a live load distribution factor 1.0 for a single lane (a single lane equals two wheels). Do not use AASHTO distribution factors for the line girder analysis. For additional loadings, or re-evaluation of the design vehicle, the live load moments and shears obtained from the standard line girder analysis shall be multiplied by the conversion factors obtained from refined analysis at appropriate girder location under investigation. For example, for Girder 3 at mid-span of span 2, the equivalent refined moment for a particular live load can be calculated as follows:

Girder 3, Location: Span 2.5

CF = 1.026 (listed in the table on the original plans)

M(line girder) = 3175.8 K-FT (live load moment from line girder analysis for the live load)

M(refined) = 3175.8 K-FT (1.026)
 = 3258.4 K-FT (equivalent refined live load moment for the live load)

2150.5-LOAD RATING PLAN SHEETS

The required information for the plan sheet submittal is located in SD 3042.18. Example plan sheets are also available for reference on the WVDOT website.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2160

GUARDRAIL

May 4, 2022

First Edition

In all cases, it is necessary to provide approach guardrail to effectively protect the motorist from the spearing effect at the end of the guardrail/barriers. See DD-662 for approach guardrail requirements. Whenever possible, the approach guardrail should extend twenty-five (25) feet, be flared and as strong as the bridge guardrail. The flares should be four (4) feet wherever possible.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2170
BRIDGE IDENTIFICATION PLATE**

*May 4, 2022
First Edition*

Section 104.8 of the Specifications requires that a bridge plate be placed on each bridge. Although the specification provides some general requirements for location of the bridge plate, the specific location of the plate must be designated on the construction plans or in the contract documents.

The following will be addressed in the development of each bridge project:

- A. A bridge plate will be required unless the construction drawings or contract documents specifically indicate that a bridge plate is not required.
 - 1. If a bridge plate is not required, the construction drawings or contract documents will indicate that the plate is not required. Some repair and renovation projects may not require a bridge plate.
- B. The specific location for the bridge plate shall be specified on the construction plans or in the contract documents:
 - 1. On steel through truss bridges, the bridge plate shall be placed on the end post approximately eight feet above the deck surface.
 - 2. On deck type structures or other bridges with concrete barriers, the bridge plate will be placed on the roadway face of the barrier.
 - 3. On bridges with steel guardrail and no suitable location on the superstructure, the plate will be mounted on the front face of the wingwall.
 - 4. On other unusual bridges, the Designer will specify the most suitable location where the bridge plate can be observed and read from the roadway or sidewalk surface.
- C. The bridge plate will always be mounted at the beginning station end of the structure and on the right-hand side of the structure, when approached facing ahead station.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2180
CONDUIT ON BRIDGES**

*May 4, 2022
First Edition*

Conduit shall be placed in the parapet for all bridge replacement and rehabilitation projects. For standard thirty-two (32) inch tall parapet two (2) - 1½ inch diameter polyvinylchloride (PVC), schedule 40, conduits will be used. For all other parapet configurations, larger diameters should be investigated and used. Conduit location and type shall be identified in the plans and paid for as Lump Sum Pay Item 662003-001, Polyvinylchloride Conduit.

Numerous situations have been observed where clamps have been used to attach conduit or supports to the structural steel of bridges. The clamps cut into the structural steel and damage the paint coating. This causes accelerated deterioration of the structural steel due to the damage to the coating, as well as providing a location for dirt, moisture, and chlorides to collect.

Any device that damages paint, provides areas that trap moisture, dirt, chlorides, and debris or prevents adequate clearance to clean and paint should not be approved to use to attach the conduit.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 2190

FUTURE INSPECTIONS

May 4, 2022

First Edition

Bridges shall be detailed and constructed to allow for proper inspection. Methods for inspection access shall be described in the TS&L Report. The Designer may need to consider manholes, etc. to facilitate access. The anticipated means of access from a walkway to all hand-holds shall be documented. Consideration of the following features should be made:

- A. Plate girders over fifty-four (54) inches deep shall have rail hand-holds in their inside web faces. Webs deeper than seven (7) feet shall have two parallel rails.
- B. A walkway in conjunction with hand-holds shall be provided when both of the following conditions exist:
 - 1. Underdeck depth exceeds six (6) feet.
 - 2. Bridge length exceeds 1,500 feet.
- C. Access to hollow piers and large box girders shall be provided. Electrical conduit and outlets may be required inside hollow members. The use of these items shall be with approval of the Bridge Project Manager.

Inspection walk access ladders and other bridge access ladders shall be designed and detailed in a manner that precludes easy access by unofficial persons. The bottom rung of ladders provided for initial access shall be placed a minimum of eight (8) feet above ground such that another ladder is necessary or the ladder shall have a locked gate. Ladders providing access downward over the edge of bridge decks shall have locked gates. Other details shall be used, as necessary, to discourage access.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2200
BRIDGE DEMOLITION/DISMANTLING PLAN**

*May 4, 2022
First Edition*

All bridge projects requiring the demolition/dismantling of structures shall be in accordance with Section 203 of the Specifications.

Designers and/or Project Managers are responsible for determining the presence or absence of lead-based coatings by requesting a field survey be conducted by the Division of Highways, Materials Control, Soils and Testing Division.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2210
TEMPORARY STRUCTURES**

*May 4, 2022
First Edition*

All temporary structures shall be designed in accordance with the Governing Specifications and shall be so noted on the plans.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 2220
SALVAGEABLE MATERIALS**

*May 4, 2022
First Edition*

Bridge projects may involve salvaging certain materials from an existing bridge, temporary bridge or other facility. A plan note is required in the contract documents to clearly inform the Contractor of their responsibility for the dismantling, care, delivery, delivery location, etc. of the salvaged material. In addition to specifying the Contractor to deliver the material to a specified site, it is vitally important that the notes specify the Contractor to unload and store the material at the designated site. In addition, any particular instructions for unloading and/or storage must be specified.

If specific instructions requiring the Contractor to unload the materials are not contained in the contract documents, it will be assumed and interpreted by the Contractor and the Construction Administrators that the Department is to unload the material. This can cause extensive disruption of schedules for the maintenance forces that will be required to unload the materials.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 3000
MISCELLANEOUS PLAN NOTES**

*May 4, 2022
First Edition*

3000.1-MANDATORY PLAN NOTES

3000.1.1-Structural Steel: All structural steel, except as noted, shall meet AASHTO M270 Grade 50W, except girder flanges, webs, and splice plates shall meet Grade 50W-T2.

3000.1.2-Blast Cleaning and Painting: Upon completion of all fabrication operations in the shop, and before shipment to the project site, all weathering steel bridge components shall be blast cleaned to a Near White surface condition according to SSPC-SP 10. Prior to the start of any blast cleaning, all oil, grease, cutting fluids, or other foreign matter shall be removed from the surfaces of the steel by solvent cleaning according to SSPC-SP 1.

The members or portions of members listed below shall be blast cleaned and shop painted according to Section 688 of the Specifications, Field Painting of Metal Structures, using the Zinc Rich, Low VOC System, Section 711.22. Apply the full paint system in the fabrication shop, except faying surfaces of high strength bolted connections, which shall be shop painted with primer only. The color of the final topcoat shall be 30045 according to Federal Standard 595 and the Gloss at angle of 60 degrees shall not exceed 25.

- A. For integral and semi-integral abutment structures, paint the ends of the girders and all other structural components encased in the concrete abutment plus one additional foot in length.
- B. Where expansion joints are specified, paint all steel components under the joint and in both directions from the centerline of the joint for a distance of 1.5 times the girder depth, or 10 FT, whichever is larger. Components specified to be hot dip galvanized do not require painting.

Include cleaning and painting costs in Item 615001-*, Steel Superstructure.

3000.1.3-Identification Marking Steel Members: All steel mill and fabricator identification markings for steel plates, shapes, or fabricated members shall be by metal tags, soapstone, or some other readily removable material; or shall be marked in an area of the completed member which will be encased or covered with concrete. Marking methods and locations are subject to approval of the Engineer.

Do not use paint or wax-based crayons for marking.

3000.2-DECK REMOVAL:

3000.2.1-Deck Removal-Grinding Note: After removal of the deck, the tension and stress reversal areas of the beam top flanges shall be inspected for the presence of unauthorized welds which may have been placed during the construction of the original deck, or during subsequent maintenance operations. Any such welds discovered shall be removed by thermal cutting the welds to within ¼ IN of the flange surface followed by grinding the remaining weld flush with the beam flange, or as may be otherwise directed by the Engineer. After grinding, the ground area of the beam flange shall be inspected by the contractor using magnetic particle (MT) testing to assure the absence of any cracks. Magnetic particle testing shall be performed in accordance with the currently adopted ANSI/AASHTO/AWS Bridge Welding Code D1.5 (BWC). Personnel performing the MT shall also be qualified in accordance with the BWC.

All grinding and MT shall be witnessed by individuals qualified as a Certified Welding Inspector (CWI) in accordance with the American Welding Society Standard for Qualification and Certification of Welding Inspectors QC-1. The Contractor shall notify the Engineer at least 48 hours prior to the start of any grinding or nondestructive testing. All work and costs associated with removal of the unauthorized welds, including MT, and witnessing the work by the CWI, shall be paid to the contractor as Force Account Work in accordance with Section 109.4 of the Standard Specifications. Appropriate time extensions will be given due consideration by the Engineer.

3000.3-TYPICAL PLAN NOTES:

3000.3.1-Finger Joints: The fabrication and erection of the fingerplate shall be in accordance with the approved shop drawings and bridge deck grade and crown (profile). The openings shall be preset prior to shipment and assembled with temporary shipping angles. The fingerplate shall be installed under the supervision of the supplier.

The drainage trough shall not be spliced unless indicated on the approved shop drawings. When splices are indicated, the splices shall be shop vulcanized by the Manufacturer. Longitudinal splices are not permitted.

The Manufacturer shall be required to submit a detailed report substantiating the testing performed on its joint design and showing the corresponding fatigue resistance line generated from the actual fatigue testing data.

After the expansion joint is installed, it shall be tested for water tightness by flooding the expansion joint with water and inspecting from below.

3000.3.2-Lead Based Paint Coating: Project plans for repair, renovation, rehabilitation, replacement, or demolition of existing highway bridges that contain lead-based coatings shall contain a note as follows:

The Contractor's attention is directed to the fact that the existing structure contains lead-based paint coatings.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 3010

STANDARD DRAWINGS

May 4, 2022

First Edition

3010.1-STANDARD DETAILS AND DRAWING FILES

A number of standard details and drawings are located on the WVDOH webpage. These are updated as necessary. For a listing of the standard bridge details, refer to Standard Details, Volume III, which can be viewed, downloaded, or printing from this website: <https://transportation.wv.gov/highways/engineering/Pages/Details.aspx>.

All other details and drawings not listed on the website can be obtained from the WVDOH.

3010.1.1-Standard Prestressed Concrete Box Beam Files: Prestressed concrete box beams and AASHTO plan sheets formatted as used by the WVDOH are available electronically. See Standard Details, Volume III.

3010.2-DIMENSIONS

All dimensions and MicroStation working units are to be in Customary U. S. Units. Plan detailing and precision are to be as shown in Table 3010.A.

DETAIL	UNIT	PRECISION
Stationing	Hundred feet (e.g., 150.25 Feet = 1+50.25)	Two decimal places (.01)
Length	Feet-Inches	Two decimal places ($\frac{1}{8}$ Inch)
Finished Ground Elevation	Feet	Two decimal places (.01)
Finished Concrete*	Feet	Two decimal places (.01)
Beam Seat Elevation	Feet	Three decimal places (.001)
Slope/Cross-Slope	%	Two decimal places (.01)
Skew Angle	DEG (°), MIN (′), SEC (″)	Two decimal places (.01″)
Ultimate Foundation Pressure	TSF	Two significant figures
Temperature	°F	Whole number (1)
Drawing Dimensions	Feet-Inches	$\frac{1}{8}$ Inch
Clear Cover	Inches	$\frac{1}{2}$ Inch
Structural Steel	Feet-Inches	$\frac{1}{16}$ Inch
Reinforcing Steel	Feet-Inches	1 Inch straight, $\frac{1}{4}$ Inch bent
Deep Foundation Tip Elevation	Feet	1 Foot
Pile Batter/Cut-Fill Slope	Ratio (e.g., 1:1)	--
Core Boring Elevation	Feet	One decimal place (.1)

* Except Beam Seat

Table 3010.A

3010.3-PHYSICAL SIZE OF DRAWINGS, SCALES

The following guidelines are intended to establish a standard format for plan sheets. These standards shall be followed. In rare instances, it may be necessary to deviate slightly from recommended formats. These exceptions should be held to a minimum and used only when sound judgment indicates that a deviation is necessary.

3010.4-SHEET SIZE AND FORMAT

All contract plans are published electronically as .pdfs, where B is 11 x 17 Inch and D is 22 x 34 Inch. The border for the D size sheet is 21 x 32 Inch providing a 0.5 Inch border on the top, bottom, and right sides with a 1.5 Inch left border for binding.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 3020
ESTIMATE OF QUANTITIES**

*May 4, 2022
First Edition*

3020.1-ACCURACY OF QUANTITY ESTIMATES

The Designer shall use diligence in determining the actual quantity for each item needed to construct the project.

Established bid items shall be used whenever possible. When a needed unit of work has not been assigned an item number, the Designer has two options: combine work or obtain a new bid item number.

3020.1.1-Combine Work: A unit of work can be combined with or made incidental to an established bid item. In either case, a plan note must indicate to the Contractor where to include their cost for the combined or incidental work that is required. This should only be done with the permission of the Bridge Project Manager.

3020.1.2-New Bid Item Number: A list of established pay items has been developed and is maintained by the Technical Support Division. Designers must use pay items that are on this list. When a needed unit of work to construct the project does not fit into any of the pay items already on the list, then a new bid item may be added.

Plan notes or Special Provisions (see SD 3030) shall be written to explain to the Contractor the scope of work involved with the new bid item and what the units of payment will be.

3020.2-ROUND OFF

The Designer shall provide the estimated quantities in whole units. When calculations produce fractional numbers, the value shall be rounded to the next higher whole number.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 3030
STANDARD SPECIFICATIONS AND SPECIAL PROVISIONS**

*May 4, 2022
First Edition*

3030.1-STANDARD SPECIFICATIONS

All highway and bridge projects must be in accordance with the latest edition of the *West Virginia Division of Highways Standard Specifications, Road and Bridges* (Standard Specifications), and shall be specified on the General Notes sheet in the contract plans.

3030.2-SUPPLEMENTAL SPECIFICATIONS

The WVDOH periodically publishes Supplemental Specifications; a compilation of all specification revisions which have been approved for use by the WVDOH, industry, and the FHWA, since the last publication. Each Supplemental Specification is noted as accompanying a particular Standard Specification. Supplemental Specifications are cumulative, in that the current Supplemental Specifications also contain the approved specification changes from previous issues of the Supplemental Specifications. When new Standard Specifications are published, the appropriate specification revisions from the then current Supplemental Specifications are incorporated into the new Standard Specifications. Therefore, a reference to the Standard Specifications must also contain a reference to the appropriate Supplemental Specification.

3030.3-SPECIAL PROVISIONS

Special provisions are governing specifications for a specific item of work on a project for which applicable specifications are not included in the Standard or Supplemental Specifications. Special provisions can also be developed to modify, add to, or delete all or part of the current Standard or Supplemental Specifications. When the Designer determines a portion or all of the work (materials or construction) is not adequately specified by the Standard or Supplemental Specifications, the Designer is responsible for assuring that an appropriate special provision is included in the project PS&E package. If an appropriate special provision does not currently exist, the Designer is responsible for preparation of the special provision. Special provision format and presentation is strictly controlled to assure compatibility with the governing specifications. Designers should consult with the Specification Engineer for guidance in preparation of special provisions.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

STRUCTURE DIRECTIVE 3041

DRAWING SEQUENCE

May 4, 2022

First Edition

Bridge plans are composed of drawings covering each of the following aspects of a structure. Where applicable, all drawings shall be drawn to an appropriate scale. The order of bridge plan sheets is as follows:

- A. General Plan and Elevation.
- B. Index of Drawings.
- C. General Notes.
- D. Estimate of Bridge Quantities.
- E. Construction Sequence.
- F. Temporary Construction Access.
- G. Foundation Layout.
- H. Substructure Drawings.
- I. Superstructure Drawings.
- J. Deck Drawings.
- K. Roadway Elevations and Dead Load Deflections.
- L. Miscellaneous Details.
- M. Erection Details.
- N. Approach Slabs.
- O. Situation Plan.
- P. Core Borings.
- Q. Existing Bridge Plans.
- R. Load Rating.

The above sheets may be combined, when practical, for simple structures and details.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 3042
GENERAL PLAN AND EVALUATION**

*May 4, 2022
First Edition*

The following is a general format for detailing and may be modified depending on the complexity of the project. See the “Final Detail Bridge Plan Submission Certification” checklist in DD-202 for a complete listing

3042.1-GENERAL PLAN AND ELEVATION

The General Plan and Elevation sheet(s) shall contain a drawing of the entire bridge in plan and elevation view. The sheet(s) shall contain, but not be limited to the following applicable information:

A. Plan View.

1. Plan view of bridge, including outline of all substructure units.
2. North arrow.
3. Centerline of roadway.
4. Work points and station of each substructure unit.
5. Skew angle of bridge.
6. Bridge layout dimensions (bridge, lane, and shoulder widths).
7. Span lengths.
8. Centerline of bearing of each substructure unit.
9. Route number and name (if applicable).
10. Horizontal alignment information, including all complex geometry data.
11. Highway signage, lighting, and utility features.

B. Elevation View.

1. Elevation view of bridge, including all substructure units.
2. Beginning and ending stations of bridge along profile grade line.
3. Length of bridge: paving notch to paving notch along baseline.
4. Length of bridge: centerline of bearing to centerline of bearing along baseline.
5. Existing ground line.
6. Proposed profile grade line.
7. Profile grade data, including vertical curve data.
8. Location of fixed and expansion bearings.
9. Elevations of top of footings for shallow foundations.
10. Bottom of footing elevations for deep foundations and the estimated pile (or rock socket drilled shaft) tip elevation.
11. Elevation of low bridge seat for each substructure unit.
12. Berm elevation and width.
13. Slope protection type, thickness, and slope.
14. Highway signage, lighting, and utility features.

3042.1.1-Bridges Over Waterways: This sheet(s) shall contain the following additional information:

A. Plan View.

1. Stream name.
2. Edge of stream at normal water elevation.
3. Stream flow direction.
4. Ordinary high water.

B. Elevation View.

1. Normal water elevation.
2. Ordinary high water.
3. The appropriate design frequency discharge and elevation.
4. Backwater elevation due to larger stream.
5. Clearances, horizontal and vertical.
6. +2% flowline (if navigable stream).

3042.1.2-Bridges Over Railroads: This sheet(s) shall contain the following additional information:

A. Plan View.

1. Name of railroad.
2. Valuation station of railroad and bridge at the respective centerlines.
3. Direction information (e.g., to Charleston).
4. Skew angle of the centerline of railroad track(s) with respect to the centerline of roadway.
5. Track horizontal alignment data.
6. Crash wall locations.
7. Location of critical vertical clearance point with station and offset.
8. Actual and required horizontal clearance dimensions.

B. Elevation View.

1. Actual and required vertical and horizontal clearance dimensions.

3042.1.3-Bridges Over Roadways: This sheet(s) shall contain the following additional information:

A. Plan View.

1. Route number and name (if applicable).
2. Skew angle of roadway with respect to bridge centerline.
3. Intersection station.
4. Horizontal alignment data for both roads.
5. Minimum horizontal clearance lines.
6. Actual and required horizontal dimensions.

B. Elevation View.

1. Location of critical vertical clearance point with station and offset.
2. Minimum horizontal clearance lines.
3. Other features of the roadway underneath the proposed bridge (e.g., ditches, guardrail).

3042.2-INDEX OF DRAWINGS

The Index of Drawing sheet(s) shall contain a table of all the drawings in the set of plans. The table shall contain the sheet name and number.

3042.3-GENERAL NOTES

The Designer shall use the General Notes sheet to address project specific items, revise or add to the Standard Specifications. The General Notes sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Governing Specifications.
- B. Design Criteria.
 - 1. Design specifications.
 - 2. Welding codes.
 - 3. FWS allowance.
 - 4. Special loads (e.g., ship impact).
- C. Design Data.
 - 1. Concrete (for each class used).
 - 2. Prestressed concrete beams.
 - 3. Structural steel (for each grade used).
 - 4. Reinforcing steel.
 - 5. Prestressing strands.
- D. Material and Construction Notes.
 - 1. Concrete.
 - 2. Structural steel.
 - 3. Reinforcing steel.
 - 4. Fasteners.
 - 5. Excavation.
 - 6. Backfill.
 - 7. Joint filler.
 - 8. Piling / rock socketed drilled shafts.
 - 9. Bearing and bearing areas.
 - 10. Handling and storing steel members.
 - 11. Construction joints.
 - 12. Blast cleaning and painting.
 - 13. Protection of concrete substructure.
 - 14. Environmental.
 - 15. Scour protection.
- E. Maintenance of Traffic.
 - 1. Temporary structure requirements.
 - 2. Phased construction.
- F. Dismantling Structure Requirements.
 - 1. Disposal of material.
 - 2. Limits of removal.
 - 3. Bridge demolition/dismantling plan.
- G. Deck Overhang.
- H. Bridge Specific Agreements and Permits.

1. Special railroad requirements.
 2. Special Coast Guard requirements.
 3. Special environmental requirements.
- I. Erection Notes and Procedures.
 - J. Shop Drawings.
 - K. Section Cut Symbology.
 - L. Highlighted Special Provisions: The Designer shall list in this section all known special provisions for the bridge project. This note shall be placed on the General Note sheet - *“The following is not a complete list of special provisions. See the proposal for a listing of all special provisions”*.
 - M. Abbreviations.
 - N. Special Notes: Place the notes that do not fit any of the above headings in this section.

3042.4-ESTIMATE OF BRIDGE QUANTITIES

The Estimate of Bridge Quantities sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Pay item numbers and descriptions.
- B. Reinforcing steel bar quantities with breakdown for each substructure and superstructure unit.
- C. Concrete quantities with breakdown for each substructure and superstructure unit.
- D. Type and weight of structural steel including total weight and breakdown by different grade.
- E. Quantities split for items to be divided between governmental jurisdictions.

3042.5-CONSTRUCTION SEQUENCE

The Construction Sequence sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Traffic control plan notes.
- B. Transverse section showing traffic lanes maintained for various phases of construction.
- C. Maintenance of traffic.
- D. Quantities.
- E. Cut-line location and appropriate removal details for deck and parts of substructure.
- F. Construction phasing and closure pour details.
- G. Temporary support details.
- H. Temporary shoring details.

3042.6-TEMPORARY CONSTRUCTION ACCESS

The Temporary Construction Access sheet(s) shall contain, but not be limited to, the following pertinent information:

- A. Notes.
- B. Quantities.
- C. Detours.
- D. Construction method and sequence.
- E. Plan view of causeways with north arrow.
- F. Section view with details of temporary access.

- G. Hydraulic information (if applicable).
- H. North arrow.

3042.7-FOUNDATION LAYOUT

A Foundation Layout sheet shall be included in the plans for all bridges designed. The foundation layout sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Centerline of roadway and centerline of substructure bearings.
- B. Skew angle(s) of substructure units (if applicable).
- C. Work point numbers with stations, northings, eastings and bearing between the work points.
- D. All dimensions necessary for the layout of work points.
- E. Work point number and stations.
- F. All dimensions necessary for construction of the substructure units.
- G. North arrow.

3042.8-SUBSTRUCTURE DRAWINGS

3042.8.1-Abutments: The Abutment sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Plan View.
 - 1. Plan view of the abutment with all dimensions necessary for construction of the abutment.
 - 2. Centerline of roadway and centerline of bearing.
 - 3. Station direction.
 - 4. Skew angle of abutment.
 - 5. Work point number.
 - 6. Center line of beams or girders with spacing.
 - 7. Beam or girder seat dimension.
 - 8. Wingwall lengths and skew.
 - 9. North arrow.
 - 10. Weep drain spacing.
 - 11. Location of construction and expansion joints.
 - 12. Anchor bolt layout.
 - 13. Pile size, batter, and spacing.
 - 14. Rock socketed drilled shaft size and spacing.
 - 15. A spread footing abutment requires a footing plan with reinforcing layout and bar marks.
- B. Abutment Elevation View.
 - 1. Elevation view of the abutment including piles or rock socketed drilled shafts if any.
 - 2. Beam seat elevations.
 - 3. Backwall elevations at centerline of roadway and gutter lines.
 - 4. Bottom of footing elevation, if on piles or rock socketed drilled shafts.
 - 5. Depth of rock socketed drilled shafts.
 - 6. Top of footing elevation for spread footing with minimum footing thickness.

7. Weep drain holes and elevations.
 8. Construction joints.
 9. Section arrows.
 10. Reinforcing steel layout with bar marks.
 11. All dimensions necessary for construction of the abutment.
 12. Berm line with elevation.
- C. Wingwalls – Elevation View.
1. Reinforcing steel layout with bar marks.
 2. Dimensions necessary for construction.
 3. Elevations at the top of the wingwalls are given at the gutter line, at the intersection of the wingwall and backwall, as well as at the end of the wingwall.
 4. If the wingwall cantilevers from the abutment, show the elevation at the bottom of the wingwall.
 5. If the wingwall is on a spread footing, show the elevation at the top of footing.
 6. If the wingwall is on a pile supported footing, show the elevation at the bottom of footing.
 7. Construction joint/locations and type.
 8. Existing ground line.
- D. Abutment and Wingwall Sections.
1. Reinforcing steel with bar marks.
 2. Centerline of anchor bolts.
 3. Dimensions necessary for construction.
 4. Limits of structure excavation.
 5. Limits of select material for backfill.
 6. Weep hole or other drainage items.
 7. Pile spacing and size or rock socked drilled shafts size.
 8. Embedment of piles or rock socked drilled shafts.
 9. Berm line and berm elevation.
 10. Construction joints.
 11. Existing ground line.
 12. Utilities passing through the wall.
- E. Reinforcing Schedule. See SD 3042.
- F. Barrier End Details.
1. Plan view showing all dimensions necessary for construction.
 2. Elevation view showing reinforcing steel bars with bar marks and any construction joints.
 3. Guardrail end anchorage details.
- G. Notes.
1. Specify the allowable bearing capacity and the maximum design bearing capacity for spread footings, the allowable drilled shaft or pile load capacity and the maximum design load per rock socketed drilled shaft or pile for deep foundations.
 2. Any abutment specific notes not covered in the General Notes.

3042.8.2-Piers: The Pier sheet(s) shall contain, but not be limited to, the following applicable information:

A. Plan View.

1. Plan view of the pier with all dimensions necessary for construction of the pier.
2. Centerline of roadway and centerline of bearing.
3. Station direction.
4. Skew angle of pier.
5. Work point number.
6. Center line of beams or girders with spacing.
7. Beam or girder seat and pedestal dimensions.
8. North arrow.
9. Anchor bolt layout.
 - a. Piling layout for pile supported footings.
 - b. Rock socketed drilled shaft layout for drilled shaft supported footings.

B. Elevation View (Looking ahead station).

1. Bridge seat elevations.
2. Bottom of footing elevation, if on piles or rock socketed drilled shafts.
3. Depth of rock socketed drilled shafts.
4. Top of footing elevation for spread footing with minimum footing thickness.
5. Construction joints.
6. Section arrows.
7. Reinforcing steel bar layout with bar marks in footing(s), column(s) and pier cap.
8. Dimensions necessary for construction of the pier.
9. Limits of structure excavation and/or wet excavation.
10. Limits of select embankment and/or scour protection.
11. Existing ground line.

C. Side Elevation View.

1. Reinforcing steel with bar marks in footing and column.
2. Dimensions necessary for construction.
3. Construction joints.
4. Limits of structure excavation and/or wet excavation.
5. Limits of select embankment and/or scour protection.
6. Existing ground line.

D. Section Views.

1. A section through the column looking down on the pier showing reinforcing steel. bar layout with bar marks.
2. A section through the pier cap showing reinforcing steel with bar marks.
3. Anchor bolt layout.

E. Reinforcing Schedule. See DD 3043.

3042.9-SUPERSTRUCTURE DRAWINGS

3042.9.1-Framing Plan: The Framing Plan shall contain, but not be limited to, the following applicable information:

- A. Centerline of roadway.
- B. Centerline of bearing at each substructure unit.
- C. Skew angle.
- D. Line drawing of structural framing.

- E. Beam or girder spacing.
- F. Work point number.
- G. Beam or girder identification mark (use letters for beam marks, e.g., Beam A, Girder A, etc.).
- H. Intermediate diaphragms or cross frames and spacings.
- I. Distance from centerline of bearing to centerline of splices.
- J. Length of each span from centerline of bearing to centerline of bearing (along center line of roadway).
- K. Total length of structure along center line of roadway.
- L. North arrow.
- M. Use numbers to identify splices (e.g., Splice 1).
- N. Type of bearing (e.g., fixed, expansion).
- O. For all structures other than conventional straight multi-beam and multi-girder types, notes shall be added to clarify which members are to be considered “main members” (including cross frames) for purposes of steel toughness (CVN) requirements and for nondestructive testing (NDT) of welds.
- P. Location of all stiffeners other than stiffeners used for cross frame connections.
- Q. Partial segment of inspection handrail.
- R. Future jacking points.
- S. Notes: Any framing plan notes not covered in the General Notes.

3042.9.2-Beam or Girder Elevation: The Beam or Girder Elevation shall contain, but not be limited to, the following applicable information:

- A. Steel Superstructure.
 - 1 Distance from centerline bearing to centerline of splices and dimensions for flange transition points for each beam or girder.
 - 2 Total length of each beam or girder.
 - 3 Number, spacing and location of studs.
 - 4 Size and weight of rolled beams (e.g., W24 x 162).
 - 5 For plate girders, show sizes of the web plate, top and bottom flange plates, along with grade of steel and welds.
 - 6 Location and extent of tension zones and stress-reversal areas in girder flanges.
 - 7 Intermediate, bearing, and auxiliary stiffeners with size.
 - 8 Jacking loads in Tons.
- B. Concrete Superstructure.
 - 1 For prestressed AASHTO I-beams, use the standard detail sheet for the type specified and include required design and detailing information.
 - 2 For prestressed box beams, use the standard detail sheet for the appropriate size of box and include required design and detailing information.
 - 3 For post-tensioned beams include the required design and detailing information.
 - 4 Jacking loads in tons.

3042.9.3-Superstructure Details: The Superstructure Details shall contain, but not be limited to, the following applicable information:

- A. Field Splice Details.
 - 1 Plan view(s) showing flange splice plates and bolt layout and spacing.

- 2 Elevation view showing flange and web splice plate sizes and web bolt spacing.
- B. Cross Frames or Diaphragms.
 1. Steel cross frames or diaphragms.
 - a) Elevation views of abutment, pier and intermediate cross frames or diaphragms with actual cross-slope(s).
 - b) Bearing stiffener plate size.
 - c) Member sizes of cross frames or diaphragms.
 - d) Plan view of cross frames or diaphragms as needed.
 - e) Weld and clip details.
 - f) Elevation view of bolt layout and intermediate stiffener plate thickness.
 2. Concrete diaphragm.
 - a) Elevation views of abutment, pier and intermediate diaphragms with actual cross-slope(s).
 - b) Reinforcing steel layout with bar marks.
 - c) All dimensions necessary for construction.
- C. Girder Post-tensioning Details.
- D. Bearing Stiffener, Intermediate Stiffener and Auxiliary Stiffener Details.
- E. Shear Connector Details.
- F. Bearings.
 1. Design and data table including the following applicable information:
 - a) Location.
 - b) Bearing type – fixed, expansion, guided or non-guided.
 - c) Service loads – DL (minimum and maximum), LL and horizontal seismic loads.
 - d) Design movement (longitudinal and transverse).
 - e) Design rotation.
 - f) Roadway slope.
 - g) Bearing orientation angle.
 - h) Bearing dimensions (including beveled sole plate, sole plate and masonry plates).
 - i) Total bearing depth.
 - j) Anchor bolt information (number, diameter, and embedment length).
 - k) Miscellaneous items (e.g., PTFE data, guides, stiffeners etc.).
 2. Bearing layout diagram showing the bearing type and orientation for each beam at all substructure locations.
 3. Plan view of the bearing showing all items necessary for their fabrication, including anchor bolt locations.
 4. Elevation view labeling all items included in the bearing device along with any welding or bonding.
 5. Specific details necessary to fabricate uncommon bearing items (e.g., embedded plates with studs for prestressed concrete beams and elastomeric bearing with PTFE sliding surfaces).
 6. Steel reinforced elastomeric bearing pads.
 - a) Plan view of bearing pad showing overall dimensions.
 - b) Include sole or masonry plates if necessary.
 - c) Elevation view showing overall height, internal and external layer thickness, steel shim thickness and edge distances.

- d) Specific details necessary for the fabrication of the bearing pad.
- 7. Any bearing detail notes not covered in the General Notes sheet.

3042.10-DECK DRAWINGS

3042.10.1-Deck Layout: The Deck Layout sheet(s) shall contain, but not be limited to, the following applicable information. See SD 2022.2, Figure 2022.A.

- A. Plan view of deck showing reinforcing steel layout and bar marks both transversely and longitudinally, top and bottom.
- B. Additional reinforcing steel in the top of slab over piers, and their placement.
- C. Overall length of deck slab.
- D. Overall barrier length.
- E. Skew angle.
- F. Centerlines of bridge, bearings, and piers.
- G. North arrow.
- H. Radius of deck fascia for curved bridges.
- I. Deck placement sequence. See SD 2022.3.
- J. Distance from the centerline of bearing to the end of the deck.
- K. Deck slab construction joints must be at the same location as the curb, sidewalk, and parapet joints.
- L. Construction joints.
- M. Notes regarding deck concrete, deck slab overhang and deck placement sequence (state allowable options).
- N. Bar marks, number of bars and bar location (top and/or bottom).
- O. The minimum lap lengths for the bar sizes being used in tabular form.
- P. Special reinforcement details for sharply skewed bridges.
- Q. Deck drain locations.
- R. Location of appurtenances.

Sample Deck Reinforcing Plan sheets are available from the WVDOT.

3042.10.2-Typical Deck Section: The Typical Deck Section sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Overall width of deck slab.
- B. Type of barrier and/or any sidewalk information.
- C. Gutter to gutter dimension between parapets.
- D. Dimension to centerline of deck.
- E. Width of parapets.
- F. Thickness of deck.
- G. Overlay thickness.
- H. Deck overhang dimensions.
- I. Girder spacing.
- J. Profile grade line location.
- K. Location of lanes and shoulders.
- L. Cross-slope of bridge deck.
- M. Reinforcing steel with bar marks.

- N. Reinforcing clearances.
- O. Lap lengths.
- P. Construction joints.

For General Typical Deck Section, see Figure 3042.A.

3042.10.3-Barrier and/or Sidewalk Details:

- A. Show a section view with all dimensions and construction details
- B. Note - barrier dimensions vary according to the deck system being used
- C. Show a section view of barrier and/or sidewalk with reinforcing steel and bar marks

Typical Barrier Section drawings are available from the WVDOH.

3042.10.4-Deck Construction Joints: Place a transverse, vertical construction joint between adjacent deck pours. Extend the longitudinal bars thru the construction joint a minimum of the lap length. Note that the joint is finished with a ¼ inch edging tool. See SD 2022.2, Figure 2022.A.

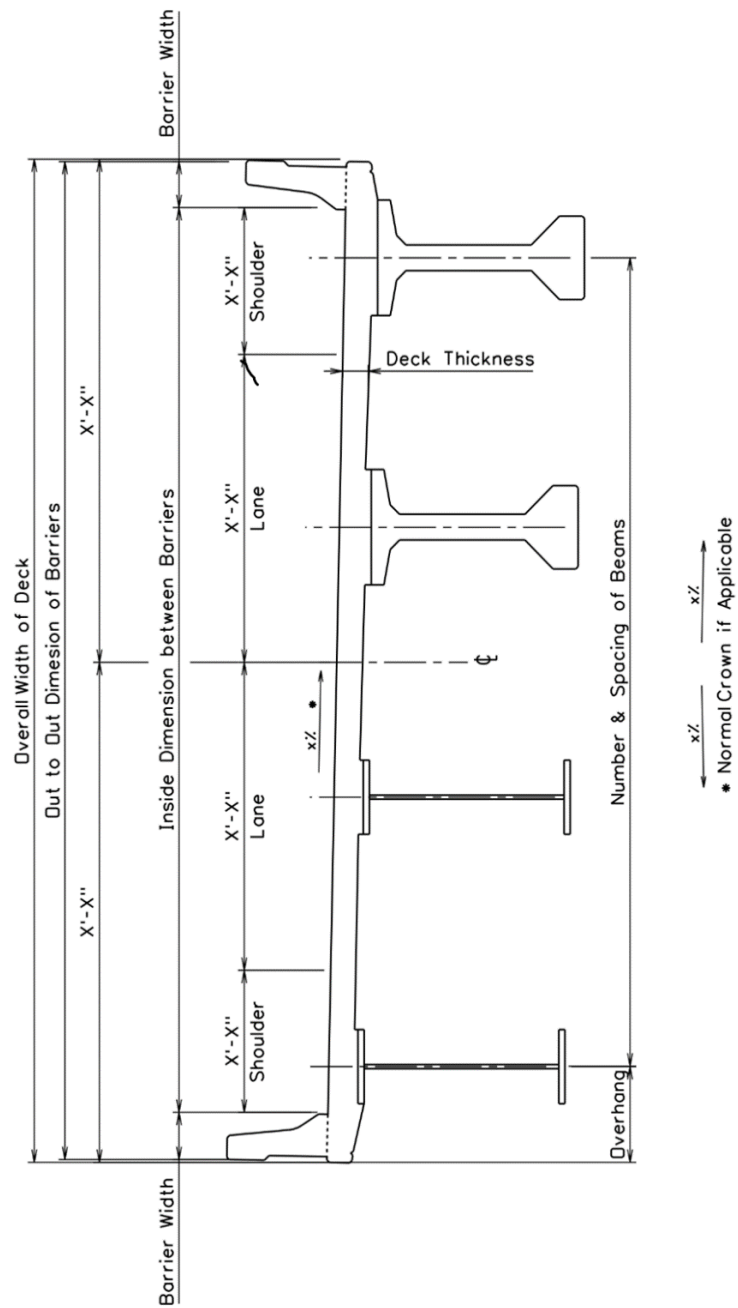
3042.11-ROADWAY ELEVATIONS AND DEAD LOAD DEFLECTIONS: This sheet(s) shall contain the roadway elevations at the centerline of each beam or girder, the gutters, fascia's and profile grade line. These elevations shall be given at equal spaces per span (typically 10 to 15 Feet). The length of the structure also needs to be shown.

Dead load deflections due to the deck placement sequence and appurtenance placement shall be shown. If necessary, on long-span bridges, deflections shall be shown for individual pour sequences.

For steel beam/girder superstructures, a table of various dead load deflections (including deflections due to steel weight), vertical curve corrections and required shop camber shall be given for all beams/girders. A camber diagram shall be provided showing the location of points along the length of the beam, for which camber and deflections are given, along with the required blocking dimensions.

For concrete beam superstructure, a table of dead load deflections shall be given for each beam. A camber diagram including appropriate tables shall be provided showing the estimated initial camber and net final camber.

A section showing the placement of the deck over the beams or girders, and the control depth from the roadway surface to the top of the beams or girders.



DECK SECTION VIEW

Figure 3042.A

3042.12-MISCELLANEOUS DETAILS: These sheets shall contain any details necessary for the structure that has not been presented on any other sheet in the plans such as:

- A. Expansion joints.
- B. Inspection railing.
- C. Walkways.
- D. Railings.
- E. Deck drainage.
- F. Overhead signs.
- G. Navigational lighting.
- H. Lighting.
- I. Utility supports.
- J. Pedestrian screening.

These details shall contain all information necessary for construction.

3042.13-ERECTION DETAILS: A proposed girder erection sequence shall be given for large or complicated structures if required by the Bridge Project Manager or the scope of work.

The erection sequence diagrams shall show the profile view of the structure with the following information given for the individual phases of erection where applicable:

- A. Proposed substructure units, including excavation limits.
- B. Placement of superstructure field sections/units over the piers, including pertinent erection criteria and bracing information including applicable notes.
- C. Sequences for placing additional field sections/units and closure sections.
- D. Show proposed method to stabilize the structure.
- E. Temporary bracing.
- F. Crane locations.
- G. Ballast for cantilevered construction.
- H. Post-tensioning details (for drop-in spans).
- I. Indicate whether a girder is designed for a particular construction method (e.g., specific pick point locations, balanced cantilever construction, etc.).
- J. Necessary jacking information including stiffness parameters for the pier columns.
- K. Diaphragm or cross frame connection notes.
- L. Deck casting notes.
- M. Other project specific notes not covered in the General Notes.

Add the following note to the Erection Details sheet: "Following the proposed erection sequence does not relieve the Contractor from the requirements of Section 615 of the Specifications".

3042.14-APPROACH SLABS: This sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Plan View.
 - 1. Dimensions necessary for construction.
 - 2. Reinforcing steel layout with bar marks.
 - 3. Section arrows both transverse and longitudinal.

4. For integral/semi-integral abutments, detail the gap between the approach slab and rigidly supported items, to allow for anticipated movements.
- B. Section Views
 1. Dimensions of each section.
 2. Reinforcing steel layout with bar marks.
- C. For integral/semi-integral abutments the following joint details at the end of the approach slab shall be used unless directed otherwise by the Bridge Project Manager.
 1. Use Type H joint for asphalt approach.
 2. Use Type J joint for concrete approach (this joint has its own pay item) .
- D. Notes.
 1. Quantity table of the area of each approach slab.
 2. Reinforcing bar list for each approach slab.
 3. All notes regarding the approach slab.
 4. Note - approach slabs and their quantities are roadway items; not bridge items.

3042.15-SITUATION PLAN: This sheet(s) shall include all items from the General Plan and Elevation sheet plus the following applicable information:

- A. Plan View.
 1. Right-of-way limits, including all easements and utilities.
 2. Existing structures within project limits (e.g., existing bridge, dwellings, walls, etc.).
 3. Core boring hole locations.
 4. Contours including finished grading contours.
 5. Utilities and their disposition.
 6. Highway signage and lighting features.
 7. Dimension limits of scour protection.
 8. Dimension limits of slope protection.
 9. Fill slopes.
 10. Temporary bridge.
 11. Temporary detour.
 12. Limits of select embankment or foundation protection.
 13. Berms, ditches, etc.
 14. Deck drains.
- B. Elevation View.
 1. Actual footing elevation for pile footings and the estimated and minimum pile tip elevation.
 2. Elevation of low bridge seat for each substructure unit.
 3. Limits of foundation protection at substructure units.
 4. All applicable hydraulic information.
- C. Additional information.
 1. Hydraulic data.
 2. Traffic data.
 3. Core boring locations table.

3042.16-CORE BORINGS: This sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Complete boring logs including technical descriptions and general descriptions of strata.
- B. All necessary geotechnical parameters including RQD, percent recovery, blow count, sample type, etc.
- C. Elevations at the top of borings.
- D. Foundation and pile tip elevation.
- E. Station and offset of each boring.

3042.17-EXISTING BRIDGE PLANS: Sheets from the existing bridge plans, if available, shall be attached.

3042.18-LOAD RATING SHEETS: The Load Rating sheets shall contain the following information. Sample rating sheets are available from the WVDOH.

- A. Girder elevation showing critical rating locations.
- B. Section properties for steel beam/girder bridges, including:
 - 1. Span number.
 - 2. Location in span.
 - 3. Yield strength for top flange, bottom flange, and web.
 - 4. Top and bottom flange thickness or size of rolled beam.
 - 5. Web depth and thickness.
 - 6. Section modulus with respect to the top and bottom flanges for the Non-composite section.
 - 7. Composite section for positive moment (steel girder plus deck concrete for both n and $3n$ modular ratios).
 - 8. Negative flexure section (steel girder plus longitudinal deck reinforcing).
- C. Section properties for prestressed concrete beam bridges, including:
 - 1. Span number.
 - 2. Location in span.
 - 3. Concrete compressive strength f'_c , and allowable tension and compression.
 - 4. Girder cross-section area.
 - 5. Section modulus with respect to the top and bottom flanges for the Non-composite section.
 - 6. Composite section.
 - 7. Section modulus with respect to the top and bottom rows of prestressing strands.
 - a) Non-composite section.
 - b) Composite section.
 - 8. Girder flexural strength and shear strength at all critical section locations.
- D. Design service moment and shear for steel beam/girder bridges. Service load moment and shear due to non-composite dead load (DL_1), composite dead load (DL_2) and live load including impact for the rating vehicle (HS20 for tangent structures and HS25 for curved structures) at all critical section locations and flange transitions along the length of the girder.
- E. Design service moment and shear for prestressed concrete beam bridges.

1. Service load moment and shear due to non-composite dead load (DL₁), composite dead load (DL₂) and live load including impact for the rating vehicle (HS20) at all critical section locations.
 2. Axial load and moment due to the effective prestress force after all losses at all critical section locations.
 3. Secondary moments due to prestressing forces at all critical locations for post-tensioned concrete beams only.
- F. For tangent structures designed using line girder analysis, provide live load distribution factors for moment and shear calculated based on the Governing Specifications.
- G. For curved structures and tangent structures designed using refined analysis, provide conversion factors for live load moment and shear.
1. Provide sample calculations on the plan sheet that demonstrate the derivation and use of the conversion factors provided. The example calculations shall be based on actual values provided for the design girder on the plans, such that the reader can follow the computations by comparing them to the original plans.
 2. Identify the number of traffic lanes loaded in the refined analysis.
- H. Sample rating calculations should be performed at the location where the loading rating is critical and included with the load rating sheets.
- I. Load rating table.
1. Provide a load table as shown below.
 2. Rating values provided shall be given in tons.

LOAD RATING TABLE (TONS)				
RATING	H	HS	3S2	TYPE 3
INVENTORY (IR)				
OPERATING (OR)				

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 3043
REINFORCING SCHEDULES**

*May 4, 2022
First Edition*

Reinforcing schedules shall contain a table(s) of each component listed (e.g., Abutment 1 Pier 1, etc.), and all bars pertinent to that component. Details of all bent bars used shall be provided on the sheet(s). Refer to CRSI for bend type and nomenclature. The minimum lap splices for each bar size shall be listed. Any specialized notes pertaining to reinforcing steel not included in the General Notes shall be placed on this sheet(s).

Use common bar marks for the same bars in different substructure units. Change prefixes as needed (e.g., PA501 or PB501, for Pier A and Pier B respectively).

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 3050
BASIC LAYOUT – RETAINING WALLS**

*May 4, 2022
First Edition*

The following order of sheets should be followed for the preparation of contract plans for Retaining Wall projects:

- A. General Notes.
- B. Index of Drawings.
- C. Estimate of Quantities.
- D. Plan and Elevation.
- E. Typical Wall Sections.
- F. Reinforcing Schedule.
- G. Miscellaneous Details.
- H. Situation Plan.
- I. Core Borings.

3050.1-GENERAL NOTES

This sheet shall contain the following information:

- A. Governing Specifications.
- B. Design strengths for concrete and reinforcing steel.
- C. Material notes for concrete, reinforcing steel, steel or concrete piling, sheet piling and lagging and rock socketed drilled shafts.
- D. Design loads.
- E. Excavation.
- F. Safety factors utilized in the wall design.
- G. Wall design parameters and notes.
- H. Allowable bearing pressure pile / rock socketed drilled shaft design loading.
- I. Requirements for wall surface texture or other aesthetic treatment.
- J. Special provisions.
- K. Construction sequence.
- L. Maintenance of traffic and phase construction.
- M. Special railroad requirements.
- N. Special environmental requirements.
- O. Special Coast Guard requirements.
- P. All abbreviations used in the set of plans.
- A. Special notes that are project or site specific.

3050.2-INDEX OF DRAWINGS

This sheet shall consist of an index of drawings table with the sheet number and title.

3050.3-ESTIMATE OF RETAINING WALL QUANTITIES

The Estimate of Retaining Wall Quantities table shall contain the following applicable items along with pay item numbers and descriptions:

- A. Concrete quantities coordinated with detail sheets.
- B. Area of MSE retaining wall.
- C. Reinforcing steel bar quantities coordinated with detail sheets.
- D. Excavation and backfill quantities.
- E. Sheet piling and concrete lagging quantities.
- F. Piling or rock socketed drilled shaft quantities.
- G. Miscellaneous quantities associated with wall (sealing, riprap, etc.).

3050.4-PLAN AND ELEVATION

The Plan and Elevation sheet(s) shall contain a plan and elevation drawing for each of the retaining walls of the project. The sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Plan View.
 - 1. North arrow
 - 2. Plan view of the wall.
 - 3. Plan view of the footings, for cast-in-place (CIP) walls only.
 - 4. Horizontal alignment information including all complex geometric information.
 - 5. Beginning and ending wall work points by station and offset with respect to centerline of construction.
 - 6. Additional points as necessary to tie wall alignment to adjoining roadway alignment(s).
 - 7. Limits of MSE wall embankment.
 - 8. Locations of typical sections.
 - 9. Locations of construction and expansion joints.
 - 10. Indicate the exterior face of wall and the type of barrier if any.
 - 11. Beginning and ending of barrier by station, offset and roadway alignment Retaining wall layout dimensions.
 - 12. Substructure footing dimensions referenced to front face of the retaining wall for CIP walls only.
 - 13. Rock socketed drilled shaft size and spacing for CIP walls only.
 - 14. Pile size, batter, and spacing.
 - 15. CIP walls require a footing layout plan showing reinforcing layout and bar marks.
 - 16. Wall mounted signing and lighting and any utilities.
- B. Elevation View.
 - 1. Station and elevation for each critical point on the wall.
 - 2. Length of wall segments and total wall length.
 - 3. Existing ground line along wall alignment.
 - 4. Finished ground surface in front of the wall.
 - 5. Top of retaining wall, top of coping and gutter line.
 - 6. Vertical alignment data along with top of wall elevations at twenty-five (25) feet intervals.
 - 7. Top of barrier and top of CIP pads.
 - 8. Elevations of top of footings for CIP walls only.

9. Footing elevation for pile or rock socketed drilled shaft footings and the estimated pile or drilled shaft tip elevation for CIP walls only.
10. Elevation of top of leveling pad and depth of undercut for MSE walls.
11. Elevation of top and bottom of lagging and steel piling for a retaining wall with steel soldier pile and lagging.
12. Approximate rock surface profile.
13. Location of section views.
14. Reinforcing steel layout with bar marks for CIP walls only.
15. Locations of construction and expansion joints.
16. All dimensions necessary for construction of the retaining walls.

3050.5-TYPICAL WALL SECTIONS

A. CIP retaining walls.

1. Reinforcing steel with bar marks.
2. Dimensions necessary for construction.
3. Limits of structure excavation.
4. Limits of select material for backfill.
5. Weep hole or any drainage items.
6. Pile spacing and size or rock socketed drilled shaft size.
7. Embedment of piles or rock socketed drilled shafts.
8. Construction joints.
9. Existing ground line.
10. Finished ground line in front of the wall.
11. Finished grade behind the wall including any roadway drainage.
12. Wall mounted signing and lighting.
13. Utilities passing through the wall.
14. Parapet details.
15. Specific Notes.

B. MSE retaining walls.

1. Coping details.
2. Barrier and moment slab details.
3. Reinforcing steel for coping, barrier, and moment slab with bar marks.
4. Dimensions necessary for construction.
5. Limits of structure excavation.
6. Limits of select material for backfill.
7. Limits of random material for backfill.
8. Drainage items.
9. CIP leveling pad.
10. Bottom of undercut.
11. Existing ground line.
12. Finished ground line in front of the wall.
13. Finished grade behind the wall including any roadway drainage.
14. Wall mounted signing and lighting.
15. Utilities passing through the wall.
16. Barrier for roadway in front of the wall.
17. Specific MSE wall design notes.

18. Soil reinforcement data.
19. Pipe sleeve for any adjacent abutment piling.
- C. Soldier pile with lagging retaining walls.
- D. Coping details.
 1. Barrier and moment slab details.
 2. Reinforcing steel for coping, barrier, and moment slab with bar marks.
 3. Dimensions necessary for construction.
 4. Limits of structure excavation.
 5. Limits of select material for backfill.
 6. Drainage items.
 7. Lagging
 8. Pile size.
 9. Bottom of pile tip elevation.
10. Existing ground line.
11. Finished ground line in front of the wall.
12. Finished grade behind the wall including any roadway drainage.
13. Wall mounted signing and lighting.
14. Utilities passing through the wall.
15. Barrier for roadway in front of the wall.
16. Specific soldier pile wall design notes.
17. Specific notes.

3050.6-REINFORCING SCHEDULES

See SD 3043.

3050.7-MISCELLANEOUS DETAILS

This sheet shall contain any of the following details and any other details necessary for the construction of the retaining walls:

- A. Architectural surface treatment detail.
- B. CIP gutter details.
- C. Wall expansion and contraction joint details.
- D. Wall corner detail(s).
- E. Barrier reinforcement details.
- F. Temporary shoring details.

3050.8-SITUATION PLAN

This sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Right-of-way limits and utilities.
- B. Core boring hole locations.
- C. Contours including finished grading contours.
- D. Designation of all walls.
- E. Wall location with station and offset with respect to centerline of project for each critical point.
- F. Roadway alignment information.
- G. Barrier locations.
- H. Limits of MSE wall embankment.

- I. North arrow.
- J. Temporary detour.

3050.9-CORE BORINGS

This sheet(s) shall contain, but not be limited to, the following applicable information:

- A. Complete boring logs including technical descriptions and general descriptions of strata.
- B. All necessary geotechnical parameters including RQD, percent recovery, blow count, sample type, etc.
- C. Elevations at the top of borings.
- D. Foundation and pile or rock socketed drilled shaft tip elevation.
- E. Station and offset of each boring.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 3060
SHOP DRAWINGS – REQUIRED INFORMATION**

*May 4, 2022
First Edition*

3060-SHOP DRAWINGS

All shop drawings submitted shall be in accordance with DD-102 and the Standard Specifications, including the latest Supplemental Specifications. Additional information regarding shop drawings may be found in *Shop Detail Review/Approval Guidelines* developed by the AASHTO/NSBA Steel Bridge Collaboration.

A list of Material Procedures (MP's) is also available at <https://transportation.wv.gov/highways/mcst/Pages/default.aspx>, which contain information on how inspection procedures are done and what procedures are used.

If the proposed shop drawings alter the General Contractor's operations, the Designer shall obtain approval from the General Contractor prior to approving the shop drawings.

The following items are required information for shop drawing submittal:

3060.1-GENERAL

- A. Title Block.
 - 1. State project number.
 - 2. Federal project number.
 - 3. Bridge name and (design) number.
 - 4. Contractor's name.
 - 5. Fabricator's name and mailing address.
 - 6. Sheet number.
- B. Specifications.
 - 1. Correct version identified.
 - 2. Correct materials.
 - 3. Current AASHTO designations.

3060.2-STRUCTURAL STEEL

- A. Framing plan with girder layout.
- B. Principal controlling dimensions.
 - 1. Span lengths.
 - 2. Girder lengths.
 - 3. Lateral beam spacing.
 - 4. Splice locations and bolt pattern layout.
 - 5. Diaphragm/cross-frame location.
 - 6. Section sizes.
 - a) Web and flange plates of welded members.
 - b) Rolled beams.

- C. Bearing locations.
- D. Material properties.
 - 1. Material and material testing.
 - 2. Designation of material, tension zones and welds for fracture critical members (FCM's), including applicable nondestructive testing.
- E. Connections.
 - 1. Stiffener and connection plates.
 - 2. Bolted splices.
 - 3. Number and spacing of bolts in floor beam and cross girder connections as well as special attachments.
- F. Cross-frames and diaphragms.
- G. Camber and/or mid-ordinate for cambered rolled beams or girder sections.
- H. Elevation at center of span or segment, field splice, abutment, and pier ordinates on shop assembly diagrams.
- I. Erection framing plan details.
- J. Incorporation of all necessary revisions into the shop detail drawings.
- K. General notes and detail sheets relative to cleaning and painting.

3060.3-PRESTRESSED CONCRETE

- A. Framing plan with girder layout.
- B. Cross section.
 - 1. Dimensions.
 - 2. Strand locations.
- C. Principal controlling dimensions.
 - 1. Span lengths.
 - 2. Girder lengths.
- D. Material strengths.
- E. Stresses.
 - 1. Initial.
 - 2. Final.
- F. Prestressing strand.
 - 1. Type (e.g., low relaxation).
 - 2. Size.
 - 3. Position.
 - 4. Draping.
 - 5. Jacking force.
 - 6. Strand cutting procedure and order.
- G. Reinforcing steel bars.
 - 1. Grade.
 - 2. Size.
 - 3. Position-**no tack welding**.
 - 3. Reinforcement detail.
- H. Debonding.
 - 1. Correct location/length.
 - 2. Shielding.

- I. Special details.
 - 4. Post-tensioning details.
 - 5. Drain location and type.
 - 6. Inserts for attachments (e.g., guardrail attachments, downspout supports, cross-frames or diaphragms).

3060.4-TIMBER

- A. Species and grade.
- B. Member dimensions.
- C. Preservative treatment.
- D. Bolt types.
- E. Holes.
 - 1. Position.
 - 2. Size.
 - 3. Method.
- F. Camber.

3060.5-RETAINING WALLS

Retaining walls and their components that require shop fabrication are subject to shop drawing review and approval procedures. See SD 6000 thru SD 6040.

3060.6-OTHER ITEMS SUBJECT TO APPROVAL

- A. Dimensions, laminations for reinforced elastomeric bearings and PTFE bearings.
- B. Material specifications.
- C. Bolts (anchor bolts, etc.).
- D. Special cases: Vendor designed items such as pot bearings, modular expansion devices, etc.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 3061
SHOP DRAWINGS – APPROVAL PROCEDURES**

*May 4, 2022
First Edition*

3061.1-PRIOR TO APPROVAL

The Fabricator shall submit three (3) copies of each set directly to the WVDOH or Consultant with copies of all transmittal letters to the Contractor, WVDOH Central Office and the District Construction Engineer. Copies of all subsequent letters shall also be sent to these offices.

Depending on the complexity and number of corrections required, the reviewer may convey comments to Fabricator by:

- A. Telephone.
- B. Marked drawings.
- C. Letter.
- D. Fax.
- E. E-mail.

The Designer/Consultant returns one (1) set to the Fabricator.

3061.2-APPROVAL

When all corrections are made, the Fabricator/Detailer shall submit nine (9) sets to the WVDOH or Consultant for final approval and distribution. The shop drawing shall then be stamped “APPROVED”.

Shop drawings shall **not** be marked as follows:

- A. “Approved except for...”.
- B. “Approved as corrected”.
- C. “Approved as marked”.
- D. etc.

Stamp all drawings for each submission with the following:

- A. Signature/initials of Reviewer.
- B. Date.

3061.3-AFTER APPROVAL

Submit completely approved sets in accordance with DD 102.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 3070
CONTRACT PLAN REVISIONS**

*May 4, 2022
First Edition*

Consultants shall obtain approval from the Bridge Project Manager before any approval can be made on shop drawing details or specifications that are different from that in the Contract Plans or Documents. Bridge Project Manager approval may be requested verbally or in writing. If verbal approval is given, the Consultant shall send a letter to the Bridge Project Manager describing the change and confirming the verbal approval.

Given this information, the Bridge Project Manager will distribute the revisions to the Shop Inspectors and any other necessary sections to facilitate approval of the structural fabrication.

Consultants shall make the required changes or revisions to the plans and denote on them that revisions have been made. Replacement plan sheets shall be submitted to the Bridge Project Manager for approval of all changes.

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS**

**STRUCTURE DIRECTIVE 4000
COMPUTER DESIGN SOFTWARE**

*May 4, 2022
First Edition*

The Designer may use any program to produce the required design or other output. When computer files are transferred as part of the design or review process, the files shall be compatible with the WVDOH's computer programs as follows:

- A. Word processing documents shall be compatible with current operating software.
- B. Spreadsheets should be compatible with current operating software.
- C. Databases should be compatible with current operating software.
- D. CADD drawings shall be compatible with the current version of WVDOH CADD operating software.

4000.1-HYDRAULIC PROGRAMS

See current WVDOH Drainage Manual for acceptable hydraulic analysis software.

4000.2-ACCOUNTABILITY

The Designer is ultimately responsible for the use and interpretation of all computer-generated results, whether the program is at fault or not.