

**Standards Committee  
Meeting Agenda  
Wednesday, March 2, 2022 at 9:00am  
Google Meet video conference. E-mail distribution message includes instruction.**

Call to Order

Roll Call of Attendees

Approval of Minutes of 1-12-2022 Meeting

**Unfinished Business** – Standards discussed at last Committee meeting

TITLE	Champion
<p><b>3<sup>rd</sup> time to Committee. Discussed in November and January.</b> <i>Design Directive (DD)-204 Guidance for Use of Project Schedules for Project Under Development.</i></p> <p>The DD is redline copy, showing the propose changes. It has been updated per comments at the last meeting.</p> <p style="color: #FFA500;"><b>Approval is expected in March</b></p>	<p>R.J. Scites</p>
<p><b>2<sup>nd</sup> time to Committee. Discussed in January.</b> Various Design Directive updates based upon AASHTO Green Book 7<sup>th</sup> edition’s moving towards performance based practical design and the definitions of project categories.</p> <p>Four NEW DD’s. are included. See below for list and summary:</p> <ol style="list-style-type: none"> <li>1. <i>DD-600 Geometric Design Project Categories.</i> The DD defines project categories, adopted policies/manuals, and documentation requirements</li> <li>2. <i>DD-601 New Construction Project Category.</i> Defines project type, level of development, policies, and specific WV guidance on geometric and cross-sectional elements for the New Construction Project Category.</li> <li>3. <i>DD-602 Reconstruction Project Category.</i> Defines project type and examples, level of development, policies, and specific WV guidance on geometric and cross-sectional elements for Reconstruction Project Category.</li> <li>4. <i>DD-603 Construction Projects on Existing Roads.</i> Defines project type and examples, level of development, policies, and specific WV guidance on geometric and cross-sectional elements for the construction projects on Existing Road Category</li> </ol> <p>These design directive updates will also affect the following EXISTING DD’s:</p> <ol style="list-style-type: none"> <li>a) <i>Existing DD-601 Geometric Design Criteria for Rural Highways.</i> The DD will be removed and replaced by adopted policy for each project category.</li> </ol>	<p>R.J. Scites</p>

<p>b) <i>Existing DD-602 Interchange Ramps Width.</i> The DD will be shifted to the “Intersections” section and absorbed in DD-624 Ramp Terminals.</p> <p>c) <i>Existing DD-603 Spiral and Superelevation.</i> The DD will be removed. Specific table information will be included in the project categories for superelevation design.</p> <p>d) <i>Existing DD-604, 606 and 609 (3R DDs).</i> These DDs will be removed. These project types fall within the CPER category of projects and guidance has been included on the design of these project. Additionally, RRR projects have problems with proper guidance of their application. Generally, the historical definition of RRR project has not properly identified the scope of the work</p> <p>e) <i>Existing DD-605.</i> Minor rewrite and title change to indicate Controlling criteria direction.</p> <ul style="list-style-type: none"> <li>o Updated <i>DD-605: Controlling Criteria and Design Exception Policy</i> is attached.</li> </ul> <p>f) <i>Existing DD-608 Median and outside slopes, Overlay projects.</i> DD will be removed. Direction will be given for the specific category of project.</p> <p>g) <i>Existing DD-610 Geometric Design Criteria for Urban Highways.</i> the DD will be removed and replaced by adopted policy for each project category.</p> <p>h) <i>Existing DD-624.</i> Updated providing additional direction on cross sectional information.</p> <ul style="list-style-type: none"> <li>o Updated <i>DD-624: Ramp Terminals</i> is attached.</li> </ul> <p>i) <i>Existing DD-817 Minor Preventive Maintenance.</i> The DD will be removed, and direction will be from CPER project types based on Maintenance and Preventive Maintenance project. ADA direction defined in this DD can be found as duplication in DD-811.</p> <p>DD-600, DD-601, DD-602, DD-603, DD-605, and DD-624 have been updated per comments at the last meeting. They are redline copy showing the revisions.</p>	
<p><b>2<sup>nd</sup> time to Committee. Discussed in January.</b></p> <p><i>Structure Directives.</i> The existing Bridge Design Manual (BDM) has been updated and revised title to Structure Directives (SD). The SD are 230 pages total. A few of the key revisions are below:</p> <ul style="list-style-type: none"> <li>• The update revises the number assignments and will make it easier to make additions or updates to the policy.</li> <li>• Only minor changes to direction have been made to clarify policy or replace weak direction (may or should) to (will or shall). Some information has been eliminated due to the direction being out of date or other policies superseding guidance in the BDM (i.e. -information on CADD has been eliminated and guidance on the use of proprietary products are referred to the Design Directive).</li> </ul> <p>In general, users of the SDs should see only minor changes to the flow in comparison to the BDM.</p> <p>No update to the Structure Directives.</p>	<p>R.J. Scites</p>

<p><b>2<sup>nd</sup> time to Committee. Discussed in January.</b></p> <p>Ten proposed drawings and revisions to WVDOH Standard Details Book – Volume 3. The new drawings address steel bridge framing plan layouts which match more closely with the recommendations of the AASHTO/NSBA Guidelines and commentary of AASHTO LRFD Bridge Design Specifications. The following sheets are included:</p> <ul style="list-style-type: none"> <li>a) <i>Sheet # 100FR2A - Framing Steel 3 Girder RF Skew Stagger</i></li> <li>b) <i>Sheet # 100FR3A - Framing Steel 3 Girder LF Skew Stagger</i></li> <li>c) <i>Sheet # 100RF5A - Framing Steel 4 Girder RF Skew Stagger</i></li> <li>d) <i>Sheet # 100FR6A - Framing Steel 4 Girder LF Skew Stagger</i></li> <li>e) <i>Sheet # 100FR8A - Framing Steel 5 Girder RF Skew Stagger</i></li> <li>f) <i>Sheet # 100FR9A - Framing Steel 5 Girder LF Skew Stagger</i></li> <li>g) <i>Sheet # 1100D11A - Abutment Diaphragm C Channel No Skew</i></li> <li>h) <i>Sheet # 1100D12A - Abutment Diaphragm C Channel Skewed</i></li> <li>i) <i>Sheet # 1100D14 - Intermediate Diaphragm C Channel</i></li> <li>j) <i>Sheet # 1100D14A - Intermediate Diaphragm C Channel Skewed</i></li> </ul> <p>No update to the drawings; however, Sheet numbers were added to each sheet.</p> <p><b>Approval is expected in March</b></p>	<p>Barrett Neeley</p>
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**New Business**

TITLE	Champion
<p><b>1<sup>st</sup> time to Committee.</b></p> <p>Six proposed drawings and revisions to WVDOH Standard Details Book – Volume 3. These drawings are for bearings and would be new 1800 section. The following sheets are included:</p> <ul style="list-style-type: none"> <li>a) <i>Sheet # 1800B1 - Plain Elastomeric Bearing Details</i></li> <li>b) <i>Sheet # 1800B2 - Laminated Elastomeric Bearing Details</i></li> <li>c) <i>Sheet # 1800BR1 – Steel Beam Non-Guided Bearing Restraints</i></li> <li>d) <i>Sheet # 1800BR2 – Steel Beam Guided Bearing Restraints</i></li> <li>e) <i>Sheet # 1800BR3 – Steel Beam Fixed Bearing Restraints</i></li> <li>f) <i>Sheet # 1800BR4 – Concrete Beam Non-Guided Bearing Restraints</i></li> </ul>	<p>Barrett Neeley</p>

Next Meeting Date: Wednesday, May 4, 2022.

Deadline for submissions: April 8, 2022.

Adjournment

**Standards Committee  
Meeting Minutes  
January 12, 2022**

**Call to Order:** The meeting was called to order by Acting Chair Steve Boggs a couple minutes after 9:00 AM.

**Attendees:** Committee Meeting attendees are listed below.

**WVDOH:**

Bodnar, Dave - DD  
Boggs, Steve - DS  
Brayack, Dan\* - FM  
Danberry, Sasha - FC

Jack, Shawn - FM  
Long, Travis - DS  
Neeley, Barrett - D1  
Robinson, Chad\* - OM

Scites, RJ\* - DD  
Smith, Shawn\* - FC  
Whitmore, Ted\* - DT  
Zyzka, Mara - DS

\*Voting Delegate

**Others:**

Crane, John - WVAPA  
Parsons, Pat – CAWV  
Smith, Yuvonne - FHWA

Stollings, Kyle – Potesta & Assoc.  
Varney, Billy – Mannik & Smith Group

**Minutes:** Minutes of the 11-3-2021 Meeting were approved without objection.

**Unfinished Business:** Items which were discussed at prior meeting are listed below:

- I. *DD-204 Guidance for Use of Project Schedules for Project Under Development.* A brief review and explanation of updates to Design Directive were given. There was discussion on the major milestones.

**New Business:** Items discussed for first time at committee meeting are listed below:

- II. Various Design Directive (DD) updates based upon AASHTO Green Book 7<sup>th</sup> edition's moving towards performance based practical design and the definitions of project categories were introduced by RJ Scites at the meeting.

Four new DD's:

1. *DD-600 Geometric Design Project Categories.*
2. *DD-601 New Construction Project Category.*
3. *DD-602 Reconstruction Project Category.*
4. *DD-603 Construction Projects on Existing Roads.*

Updates to following existing DD's:

- a) *Existing DD-601 Geometric Design Criteria for Rural Highways.*
- b) *Existing DD-602 Interchange Ramps Width.*
- c) *Existing DD-603 Spiral and Superelevation*
- d) *Existing DD-604, 606 and 609 (3R DDs)*
- e) *Existing DD-605 Controlling Criteria and Design Exception Policy*
- f) *Existing DD-608 Median and outside slopes, Overlay projects.*
- g) *Existing DD-610 Geometric Design Criteria for Urban Highways.*
- h) *Existing DD-624 Ramp Terminals*
- i) *Existing DD-817 Minor Preventive Maintenance.*

**III.** *Structure Directives.* The existing Bridge Design Manual has been updated and revised title to Structure Directives.

**IV.** Ten proposed drawings and revisions to WVDOH Standard Details Book – Volume 3.

- i. *Intermediate Diaphragm C Channel*
- ii. *Intermediate Diaphragm C Channel-Skewed*
- iii. *Abutment Diaphragm C Channel*
- iv. *Abutment Diaphragm C Channel - Skewed*
- v. *Framing Steel 3 Girder LF Skew Stagger*
- vi. *Framing Steel 3 Girder RF Skew Stagger*
- vii. *Framing Steel 4 Girder LF Skew Stagger*
- viii. *Framing Steel 4 Girder RF Skew Stagger*
- ix. *Framing Steel 5 Girder LF Skew Stagger*
- x. *Framing Steel 5 Girder RF Skew Stagger*

**Next Meeting:** The next meeting is on Wednesday March 2, 2022, via Google Meet. Deadline for submissions February 4, 2022.

**Adjournment:** The meeting was adjourned.

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

DRAFT

**DESIGN DIRECTIVE 204**  
**GUIDANCE FOR USE OF PROJECT SCHEDULES**  
**FOR PROJECTS UNDER DEVELOPMENT**

*Draft of January 25, 2022*

*Supersedes June 17, 2021*

This Design Directive provides guidance and instruction, along with sample Project Schedule templates, on how to prepare and submit project schedules for projects under development. These project schedules will be used by project managers, as outlined herein.

## **10. GENERAL**

**10.1 Introduction:** Project schedules are to be submitted by project managers for projects with multidisciplinary involvement, outside resource agency coordination, or have highly technical or complex issues; this includes preliminary study projects. The project schedule shall outline all the required project tasks and the submission dates at project milestones. Design completion dates for projects are established by the schedule and are utilized for tracking project authorization and determining program delivery.

Projects that have few tasks or do not have parallel dependencies that need a critical path identified may not need to use these scheduling procedures. Projects such as contract resurfacing, minor slide repair or emergency projects require few participants and need minor resource documentation. Program milestones will be tracked in the Hub, the replacement software for the legacy Project Tracking System.

Network diagrams have historically been used by the Division of Highways for tracking project milestones. Network diagrams identify multiple paths of parallel activities. Those paths are reviewed, and a critical path is identified. The critical path is a list of tasks or activities that drive the project's schedule. No tasks on the critical path can be late and allow the project to remain on schedule. This type of schedule management is referred to as Critical Path Method (CPM) and provides a point of emphasis for managing all tasks. The use of CPM is documented in many manuals and reports and should be referred to for detailed information on scheduling.

Example templates of previously used CPMs are included at the end of the DD and provide visual examples of node to node scheduling. These should be referenced and generally followed to ensure disciplines develop at a rate such that design alterations or new data can be managed and incorporated reasonably and without undue undermining of design efforts.

Bar charts, also called Gantt charts, will be used to graphically display a project's schedule. Microsoft Project is available and can be integrated with the Hub to update programmatic milestones. Schedules should document all necessary milestones including review time by Division of Highways staff.

## 20. PROJECT SCHEDULES

**20.1 Initial Schedule:** When it is determined a schedule will be required, an initial review should be developed either prior to programming or shortly after. The schedule should identify all mandatory tasks and define risk based working days. Tasks that involve high probability of risk include major NEPA actions, core drilling operations, right of way acquisition, utility relocations and individual permits.

**20.2 Project Schedule:** The Project Schedule should be updated within a relatively short time before actual project development starts or when previous studies or program rescheduling have identified refinements to project milestones. The integration with the Hub allows for the schedule adjustments to be uploaded and for automated requests for programmatic milestone revisions.

**20.3 Project Schedule Revision:** Revisions to schedules should reflect current expectations and provide realistic delivery schedules. Reasons for major milestone (ENG, ROW, CON) schedule delays shall be documented and reported to development responsibility management. Approved schedule changes shall be forwarded to Programming Division and the Hub updated. The revision documentation will be used for evaluation of both DOH staff and Consultant staff.

## 30. PREPARATION OF PROJECT SCHEDULES

**30.1 General:** Project Schedules will be completed on WVDOH provided software. Schedules will be based on working days, not calendar days. Working time or duration shall be entered in days as the unit. The project manager will be responsible for managing leading activities or predecessors. Immediate supervisors will provide guidance and approve working times and schedules in the original Project Schedule. Templates have been created for standard projects and can be managed out of the Hub.

**30.2 Scheduling Software:** WVDOH will provide software to designated staff who will be responsible for managing schedules. The software will be installed on computers. The software is an industry standard platform and files may be provided to consultants or other firms doing work for the WVDOH. WVDOH will not be responsible for providing software to firms doing work or provide guarantees of compatibility of data files. Compatible templates have been created and can be managed from the Hub.

**30.1.3 Schedule Creation:** Schedule templates can be downloaded from the Hub. The templates provide many standardized tasks that have been identified historically as major milestones of project development.

There are specific project milestone tasks identified in the standard schedule which must not be modified. These tasks tie directly to Hub critical dates for maintaining programmatic schedules. The milestones are in blue with a “post-it note” icon in the indicator column in the software. These tasks for a 300 series project are:

- CP START – “LINE AND GRADE APPROVAL PROCESS”
- RW START – “AUTHORIZE ROW PHASE”
- CP COMPLETE – “UPDATE FINAL PLANS FOR PSE SUBMISSION”
- CP/RW – “PREPARE AND SUBMIT RW CERTIFICATE”
- PSE – “PSE DOCUMENTS ASSEMBLED AND SUBMIT TO CA”
- ADVERTISE – “PROJECT ADVERTISED FOR LETTING”
- LETTING – “LETTING DATE”
- AWARD – “AWARD OF PROJECT”

Design study or 200 series project templates also have key milestones identified in blue that cannot be changed.

**30.4 Project Scheduling:** Scheduling of working time should be based on the amount of time it should take to accomplish the tasks. Resources, or workforce, should be allocated appropriately to meet the scheduled time of the tasks.

Task scheduling is based on many factors. The Division has historically leaned on the institutional knowledge base of existing employees to develop schedules. Staff use the scope of the project, technical needs of the design and risks associated with dealing with outside influences to predict working times.

Some tasks may not be used based on the scope of the project and can be zeroed out. If tasks are deleted, it is the responsibility of the project manager to manage predecessors. If tasks not identified in the standard schedule templates are critical, they may be added but the project manager is responsible for updating any predecessors and assuring that links are correct.

When revising standard template schedules the project managers shall make it a practice to review the following:

- a) Make no changes to programmatic milestones identified in 30.1.3.
- b) Use “auto scheduled” as the task mode. This will update the schedule for future tasks.
- c) Use multiple predecessor tasks when critical activities need to occur prior to task completion. An example of multiple critical predecessors is prior to right of way authorization, environmental documents and acquisition and utility estimates need completed.
- d) Internal review times need scheduled prior to major submittals.

- e) Schedules should be reviewed and concurred by all those responsible for various actions of a project. Different disciplines have more knowledge of resource allocations under their control.
- f) Schedules should follow formal policies when identified. Do not “crash” schedules for working times outside of the project manager’s ability to allocate resources.
- g) Present a realistic schedule based on internal and external resource allocation and risks.

It is important that project managers identify as many risks as possible, hidden time consumers and review times when developing schedules. The timelines should provide a realistic and anticipated flow of project development. Project managers should attempt to control schedules from the earliest stage as possible. Project managers should use design study information to develop the Initial Schedule and once design is initiated update the Project Schedule as resources for design are identified.

Training on project management is available online from AASHTO TC3 program. This service is available free of charge to employees of participating transportation agencies. Staff must set up an account using their state email address. Upon completion of the course, staff will earn continuing education units (CEUs). To access the courses, go to AASHTO’s Store, <https://store.transportation.org>, and search for the following training:

- Critical Path Method (CPM) Scheduling
- Critical Path (CPM) Schedule Management

Training for the use of the scheduling software, MS Project, is available from many online sources. MS Project can be used for many complex project management operations, but the intent of this direction is only to provide simplistic task identification, node to node actions and assign workdays. Project managers will also be able to add tasks, modify predecessors and alter view layouts. It is suggested that the task mode be set to “auto schedule” to enable automatic updates of start and finish dates, and predecessor changes.

**30.5 Schedule Integration with Hub:** Schedules will now be integrated, downloaded, and uploaded to the Hub. It is the responsibility of the project manager to assure that schedules are kept up to date on the Hub. MS Project files may be temporarily stored on user’s computer, but the official Schedule will be the file stored on the Hub.

**30.6 Schedule Storage and Archiving:** For recordkeeping, the project’s final schedule and any revisions shall be saved in the project’s ProjectWise folder. During final evaluation of on time delivery of the project documents, schedule revisions and justifications should be reviewed to determine any issues that need documented for ongoing or future projects.

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

DRAFT

**DESIGN DIRECTIVE 600  
GEOMETRIC DESIGN PROJECT CATEGORIES**

*Draft of January 27, 2022  
ORIGINAL VERSION*

The purpose of the design directive is to provide guidance to designers on the selection of geometric design criteria for the category of projects based on American Association of State Highway and Transportation Officials (AASHTO) and Federal Highway Administration (FHWA) manuals and guidance.

## **1.0 INTRODUCTION**

This directive shall be used by designers to evaluate the project category and apply proper geometric design criteria. Each project category defines the appropriate level of compliance to various levels of geometric criteria. Geometric criteria are defined by AASHTO's Policy on Geometric Design of Highways and Streets ("Green Book") as well as various other approved manuals developed by AASHTO. FHWA also provides guidance and direction for roads on the National Highway System (NHS). Designers will identify the category of the project early in plan development and address any deviations from the criteria with project notes or design exceptions.

Many manuals and documents provide information to designers when adjustments to published values are allowed. Policies are trending to provide more flexibility to rigid standardization allowing for designs to better fit to the environment around them. For many years the Green Book has provided rigid values to be used based on road classification and volumes. The newest version of the Green Book has expanded the definition of the roads from the traditional functional systems to include contextual setting of the roadway. Performance based practical designs are becoming featured to find better solutions for DOTs as funding becomes more critical to the decision making process. Flexibility and context sensitive solutions allow for designers to meet the purpose and need of the project within budgetary limits.

## **2.0 PROJECT CATEGORIES**

Project categories are developed to direct the designer on the proper geometric design criteria that shall be followed for the type of project to be designed. The categories are based on the type of work to be done, whether the route is on the NHS, the functional system characteristics, traffic volumes and design speed. The criteria in the different manuals and guides are based on historical values, studies and engineering judgement. The values provide guidelines to expedite the decision making and documentation of design selections.

The categories used to develop design criteria are new construction, reconstruction and "construction on existing roads". These categories are further defined as project types in the Green Book. Each category uses a performance based practical design approach to the decision making used to develop design values. New construction projects are on new alignments that use strict adherence to values and criteria developed in manuals and guides. Reconstruction projects use the

existing alignment but alter the road type. Construction on existing roads are projects on existing alignments that would not be changing the roadway type and would be developed to address a specific need.

All categories shall be evaluated for their applicability as an alteration project. An alteration project is any project that meets the requirements or project description whereby Americans with Disabilities Act (ADA) facilities must be addressed. Any project defined as alteration must meet the Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right of Way (PROWAG) standards for curb ramp opening, slopes and widths. Refer to DD-811 for a list of projects defined as alteration.

Context and location of the roadway are fundamental items used to develop design values for flexible practical design. New construction can even be evaluated based on context of rural or urban environments. Formal design criteria can be used in rural areas due to fewer constraints than in urban, developed areas. Urban area constraints will influence the geometric design criteria and variations to criteria will need documented using performance factors based on function and context of the roadway.

**2.1. New Construction:** Projects that are developed in the New Construction category will use the design criteria in chapters 2 through 10 of the Green Book or other formal geometric design guidance. Generally, projects in this category shall apply “desirable” or other preferred values to geometric alignments and cross section elements. Functional and context classifications found in chapter 1 of the Green Book should be used to determine values for controlling criteria. There is flexibility allowed for these project categories, but the decisions shall be determined on performance based analysis and thoroughly documented.

**2.2. Reconstruction:** Projects developed under the Reconstruction category utilize existing alignments or minor changes but result in a change in roadway type. Changes in roadway type result in changes to cross sectional elements to address project needs or scope. These projects present problems when trying to adapt documented design criteria to new facilities due to existing context and constraints. These projects may not necessitate forecasting for future performance but should be part of a performance based approach to address facility needs. Green Book chapters 2 through 10 should be reviewed for geometric and cross-sectional guidance but facility context may drive decisions due to constraints within the corridor.

Projects such as intersection improvements, adding lanes and lane or shoulder widening would change the roadway type but would only cause minor changes to the existing alignment. These projects may not use the full criteria found in the Green Book due to nearby or corridor constraints.

Interstate reconstruction is a project type where variable design criteria may be applied. Cross slope information from the “A Policy on Design Standards – Interstate System” may be utilized to correct cross slopes. Criteria may be referenced from other guidance, such as correction of superelevation will use “Green Book” standards. In this case shoulder widths may be substandard to meet new construction criteria, but under the reconstruction category may not be widened since this would alter the roadway type.

**2.3. Construction Projects on Existing Roads:** Projects developed under the Construction Projects on Existing Roads (CPER) category are projects that have no or minor changes to the existing alignment and no changes to the roadway type. [CPER project types are similar to](#)

what has historically been referred to as RRR (restoration, rehabilitation, resurfacing). These projects use flexible criteria based on existing performance to address the facility needs or scope of the project. Projects may use Green Book design criteria, but other manuals may also be used for documenting design criteria of the project. The CPER category will encompass several other subcategories based on the requirements of the project. Projects within the category need to meet different thresholds of documentation and coordination. CPER projects may include ~~3R (restoration, rehabilitation, resurfacing)~~, resurfacing, slide correction, bridge deck replacements, safety upgrade and other preventive maintenance projects.

A subcategory of the CPER is a maintenance project. An example of a maintenance project is the common resurfacing project. These projects use the existing alignments and are not intended to alter the roadway type. These projects are developed to restore rideability and prolong the serviceability of the existing surface.

### 3.0 PROJECT CATEGORY EXAMPLES

The following are definitions and examples of work types for the project categories. The bulleted lists below are not all inclusive, especially for CPER. Any project meeting the criteria of Alteration Project must meet standards defined in PROWAG.

**3.1. Alteration Project:** An Alteration Project is project defined by “DOJ/FHWA Joint Technical Assistance on the ADA Title II Requirements to Provide Curb Ramps When Streets Roads or Highways are Altered Through Resurfacing”. Treatments that are considered alterations of the road surface are:

- Open-graded surface course
- Cape seals
- Mill and fill/Mill and overlay
- Hot in-place recycling
- Microsurfacing/Thin-lift overlay
- Addition of new layer of asphalt
- Asphalt and concrete rehabilitation and reconstruction
- New construction
- Widening of the existing pavement typical section
- Addition of turning lanes
- Pavement rubblization
- Installation of new drainage structures to improve existing drainage characteristic

Treatments that are NOT considered alteration projects are:

- Crack filling and sealing
- Surface sealing
- Chip, Slurry, and Fog seals
- Scrub sealing
- Joint crack seals
- Joint repairs
- Dowel bar retrofit
- Spot high-friction treatment
- Diamond grinding
- Pavement patching
- Shoulder repairs
- Pipe and inlet repairs
- Pulling and restoration of ditches
- Guardrail repair and installation
- Re-stripping

For additional information on alteration project definitions use the following link for guidance.

USDOT FHWA Civil Rights Website

[https://www.fhwa.dot.gov/civilrights/programs/doj\\_fhwa\\_ta\\_glossary.cfm](https://www.fhwa.dot.gov/civilrights/programs/doj_fhwa_ta_glossary.cfm)

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**3.2. New Construction:** Defined as an Alteration Project, on new alignment meeting full compliance with design guidance. Guidance can be found in the latest adopted editions found in section *Policies, Manuals and Guidance*.

- New Interstate
- New Four Lane Divided Highway
- New Two Lane Highway

**3.3. Reconstruction:** Defined as an Alteration Project, on an existing alignment (or make only minor changes to the alignment) that alters the basic roadway type. Designers use standards approved in various design guidance criteria to make performance based practical design decisions that are documented. These concepts generally follow guidance provided in Green Book allowing for more flexibility based on surrounding system characteristics and context.

- Adding Lanes or a Median
- Adding Auxiliary Lanes
- Widening Lanes or Shoulders
- Intersection Improvements
- Bridge Replacement
- Sidewalk Construction
- Interstate Reconstruction

**3.4. Construction Projects on Existing Roads:** Defined as an Alteration Project, on an existing alignment (except for minor changes) that maintains the basic roadway type and uses practical engineering concepts to re-establish some portion of initial serviceability or Level of Service. Projects may not follow the formal DD-202 process, but quality assurance and control may be supplemented by Safety Reviews, ADA Exception Justification Reports and other documentation to support design decisions.

- Resurfacing, Restoration and Rehabilitation (RRR)
- NHS Resurfacing
- Guardrail Installation
- Bridge Deck Replacement

**3.4.1. Construction Projects on Existing Roads (Preventive Maintenance):** Defined as an alteration project, on an existing alignment (except for minor changes) that maintains the basic roadway type and uses practical engineering concepts to re-establish the rideability or corridor functionality.

- NHS Resurfacing
- Non-NHS resurfacing
- Slide Repair
- Bridge Deck Overlays
- Pipe Replacement
- Microsurfacing

**3.4.2. Construction Projects on Existing Roads (Maintenance):** Defined as a non-alteration project, on an existing alignment (except for minor changes) that maintains the basic roadway type and uses practical engineering concepts to extend the service life.

- Joint and Pavement Sealing
- Diamond Grinding
- Joint Repair and Dowel Bar Retrofit
- Pavement Patching
- Guardrail Repairs
- Pulling and Restoration of Ditches
- Restriping

#### 4.0 POLICIES, MANUAL AND GUIDANCE

The following represents a list of approved policies and manuals for designers to document geometric design decisions.

- A Policy on Geometric Design of Highway and Streets, 2018, 7<sup>th</sup> Edition
- A Policy on Design Standards – Interstate System, May 2016
- Highway Safety Manual, 2014, 1<sup>st</sup> Edition
- Roadside Design Guide, 2011, 4<sup>th</sup> Edition
- Manual for Assessing Safety Hardware, 2016, 2<sup>nd</sup> Edition
- LFRD Bridge Design Specifications, 9<sup>th</sup> Edition
- LFRD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Edition
- Manual on Uniform Traffic Control Devices [2009, Revision 1 and 2](#)
- Guidelines for Geometric Design of Low-Volume Roads, 2019, 2<sup>nd</sup> Edition
- WVDOH Design Directives
- WVDOH Bridge Design Manual

Any errata, supplemental or new editions of the listed documents, formally adopted by FHWA on federal aid eligible projects. The use of AASHTO approved documents prior to FHWA adoption on non-NHS projects is acceptable.

#### 5.0 DOCUMENTATION

The following shall provide a minimum of documentation requirements of the project category. The flexibility provided in the geometric design policy may necessitate additional documentation on performance based decisions. The documentation, if not based on published standards, shall thoroughly address design applicability in type, nature and context to function of the corridor or facility. The use of practical engineering concepts for CPER projects should be based on long held performance based decisions that meet system wide goals. When controlling criteria is being addressed by the scope of the project and appropriate limits can not be obtained, variances shall be documented using DD-605 “Design Exception Policy”.

**5.1. New Construction:** The geometric design decisions shall be documented as reference to applicable manuals using tables, charts or section reference. Design exception documents shall be used to identify nonstandard dimensions. The use of less than desirable values for any dimension not defined as a controlling criteria value, shall be submitted for approval by the appropriate Deputy State Highway Engineer.

**5.2. Reconstruction:** The geometric design decisions affecting roadway type shall be documented and referenced to the applicable manual using table, charts or section reference. Design exceptions narrative should reflect performance based practical design decisions that identify the context of the surrounding area and improvements to the facility within funding constraints.

**5.3. Construction Projects on Existing Roads:** Projects designated as CPERs shall use flexible performance based solutions to address facility needs. Depending on facility designation as an NHS route, the documentation will vary. Based on the type of work, the project may be documented using formal documentation, or decisions can be based on

historical performance that meet the yearly fiscal or program monetary constraints. In all cases the designer shall analyze safety concerns, accident data and address ADA concerns when documenting project decisions.

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

DRAFT

**DESIGN DIRECTIVE 601  
NEW CONSTRUCTION PROJECT CATEGORY**

*Draft of January 27, 2022  
ORIGINAL VERSION*

The purpose of the design directive is to provide guidance to designers on the selection of geometric design criteria for the New Construction Project Category.

## **1.0 INTRODUCTION**

New Construction projects are project types in which the desired controlling criteria shall be used from AASHTO, as adopted by FHWA by 23 CFR 625. Geometric and cross sectional guidance, as well as other design information, can be found in the list of published and adopted guides in DD-600. Variance from desired criteria should be documented and detailed in corridor and design studies. These variances should be approved during the Location and Design process, using the design exception policy, and should be made transparent to the public at informational meetings as part of the environmental study.

## **2.0 DEFINITIONS**

- Traveled Way -The portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.
- Roadway - The portion of a highway, including shoulders, for vehicular use. A divided highway has two or more roadways.
- Graded Shoulder - The width of shoulder from the edge of the traveled way to the intersection of the shoulder slope and foreslope.
- Usable Shoulder - The width of shoulder that can be used when a driver parks or makes an emergency stop. If the sideslope is 4:1 or flatter, the usable shoulder width is the same as the graded shoulder width.
- Controlling Criteria – Values used to provide a standardized approach to develop alignments and cross sectional elements. These values provide a consistency to roadway layout and allow a uniform driver expectancy. These controlling criteria are defined as: design speed, design loading structural capacity, lane width, shoulder width, horizontal curve radius, superelevation rate, maximum grade, stopping sight distance, cross slope, and vertical clearance. There are other cross sectional elements that are developed and based on project goals and surrounding context, for instance number of lanes or bike/pedestrian facilities. Controlling criteria shall be used from AASHTO policies, as adopted by FHWA by 23 CFR 625.
- Green Book – Current adopted edition of AASHTO’s “A Policy on Geometric Design of Highways and Streets”.
- Interstate Guide – Current adopted edition of AASHTO’s “A Policy on Design Standards – Interstate System”.

- Urban Area – Areas meeting a population density as defined by the Green Book.
- Rural Area – All areas not meeting the definition of Urban area.

### **3.0 CONTROLLING GUIDANCE**

Controlling guidance for new construction projects will use desired values from Green Book, Interstate Guide, LRFD Bridge Design Specifications, and other formally adopted manuals.

Criteria shall use the most obvious guidance for surrounding context and classification of the road. Due to the abundance of approved policies that provide overlapping direction, the engineer of record shall make the responsible decision on criteria selection.

### **4.0 WVDOH GUIDANCE**

Some geometric and cross sectional element guidance provides variability that must be standardized by the controlling agency. Working with FHWA approved documents and other adopted manuals, the WVDOH directs engineers to use the following geometric criteria parameters when designing New Construction type projects on the following classification of roads.

Guidance in this design directive applies only to geometric and cross sectional elements. Guidance for bridge loading, roadside hardware, signs, pavement markings can be found in other applicable policies.

#### **4.1. Arterial Routes**

##### Freeways:

Freeways represent the highest level of connectivity in which access is controlled and no at grade intersections are allowed. High operating speeds and levels of service are an expectation of the travelling public. Design speeds should be selected early in the development stage. Geometric and cross sectional element criteria will reference the latest policy on design standards for the interstate system. For items not specifically addressed in the latest interstate guide, criteria should be developed from other adopted policies, found in DD-600 section 4.0, using the highest desirable criteria.

Interchanges are another major element of freeway and four lane arterial development. Interchange spacing requirements should be based on the interstate guide. Geometric and cross sectional elements for ramp terminals shall reference guidance on interchange layout from the latest Green Book.

##### Divided and Two Lane Arterial

The designer shall use controlling criteria from the Green Book.

#### **4.2. Collector Routes**

##### Collector Routes

The designer shall use requirements from the Green Book. When traffic counts allow, the use of controlling criteria found in the AASHTO's "Guidelines for Geometric Design of Low-Volume Roads", current adopted edition, can be used in place of those found in the Green Book.

#### **4.3. Local Roads and Streets**

##### Local Roads and Streets

The designer shall use requirements from the Green Book. When traffic counts allow, the use of controlling criteria found in the AASHTO's "Guidelines for Geometric Design of Low-Volume Roads", current adopted edition, can be used in place of those found in the Green Book.

## **5.0 CRITERIA REQUIREMENTS**

### **5.1. Design Speed**

Criteria for design speed is the controlling factor for New Construction and should be established prior to beginning any project documentation. Exceptions to design speed should be rare.

### **5.2. Superelevation**

Maximum superelevation for rural classification shall be 8%. Maximum superelevation for urban classification shall be 8% for freeways and arterials with design speeds equal to or greater than 50 MPH. For other urban routes, maximum superelevation shall be 4%.

### **5.3. Vertical Clearance**

Vertical clearance dimensions from Interstate Guide and Green Book shall be used and increased six inches for future resurfacing.

### **5.4. Terrain Considerations**

The topography of West Virginia contains level, rolling and mountainous types of terrain. New construction projects need to identify the terrain early in project development due to the impact on grade and excavation volumes.

The terrain in West Virginia is generally mountainous for arterial and other high level collectors. The decision on the terrain declaration is based on these alignments either starting near a lower end of a drainage basin and connecting to a top of a mountain range or crossing a drainage basin from ridge line to ridge line. Identification of a rolling terrain may be based on an alignment that parallels a river or stream system and differences in low to high points along the mainline is less than 100' in elevation.

### **5.5. Other Geometric and Cross Sectional Element Criteria**

The designer should reference approved or adopted policies for other geometric and cross sectional design criteria. The most appropriate document should be identified and referenced for use on New Construction projects.

## **6.0 DOCUMENTATION**

Geometric criteria for new projects should be documented in initial design documentation. Most new projects are vetted in corridor studies, design studies or traffic analysis. These documents often establish design criteria and cross sectional elements are defined to provide a decision document based on the purpose and need. The study documents should reference the policy used or exception to the approved policy. Design exceptions on new projects should be evaluated and only rarely used. The Design Exception policy should be used to document all variances to controlling criteria.

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

DRAFT

**DESIGN DIRECTIVE 602  
RECONSTRUCTION PROJECT CATEGORY**

*Draft of January 27, 2022  
ORIGINAL VERSION*

The purpose of the design directive is to provide guidance to designers on the selection of geometric and cross sectional element design criteria for the Reconstruction Project Category.

## **1.0 INTRODUCTION**

Reconstruction category of projects are those in which no or minor changes to the existing roadway alignment are made. These projects identify a specific need to the project corridor that improves connectivity, level of service, upgrade roadside hardware features or regenerate the service life of the facility. Generally, a reconstruction type project has specific goals to meet the needs of the facility. The designer will need to narrow their scope and use only the criteria necessary to address purpose of the project.

It is important to correctly identify projects that meet the reconstruction type projects category. Reconstruction projects are projects that address the needs of the corridor and require a higher order of accuracy and development. These projects meet specific needs and are meant to provide a higher level of capacity, improve safety or restart the service life of the facility.

The designer shall make every effort to identify all safety and ADA requirements. ADA accommodation shall be made on all alteration projects. Refer to DD-811 for a list of projects defined as alteration. If curb ramps can not be constructed to standard, an ADA Design Exception shall be submitted following the steps outlined in DD- 811.

Example Project Types:

- Interstate Reconstruction
- Roadway Reconstruction
- Horizontal or Vertical Curve Correction
- Adding Auxiliary Lanes
- Widening Lanes or Shoulders
- Upgrading Curb Ramps
- Adding Pedestrian or Bicycle Facilities

## **2.0 DEFINITIONS**

- Traveled Way -The portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.
- Roadway - The portion of a highway, including shoulders, for vehicular use. A divided highway has two or more roadways.
- Graded Shoulder - The width of shoulder from the edge of the traveled way to the intersection of the shoulder slope and foreslope.
- Usable Shoulder - The width of shoulder that can be used when a driver parks or makes

- an emergency stop. If the sideslope is 4:1 or flatter, the usable shoulder width is the same as the graded shoulder width.
- Controlling Criteria – Values used to provide a standardized approach to develop alignments and cross sectional elements. These values provide a consistency to roadway layout and allow a uniform driver expectancy. These controlling criteria are defined as: design speed, design loading structural capacity, lane width, shoulder width, horizontal curve radius, superelevation rate, maximum grade, stopping sight distance, cross slope, and vertical clearance. There are other cross sectional elements that are developed and based on project goals and surrounding context, for instance number of lanes or bike/pedestrian facilities. Controlling criteria shall be used from AASHTO policies, as adopted by FHWA by 23 CFR 625.
  - Green Book – Current adopted edition of AASHTO’s “A Policy on Geometric Design of Highways and Streets”
  - Interstate Guide – Current adopted edition of AASHTO’s “A Policy on Design Standards – Interstate System”.
  - Urban Area – Areas meeting a population density as defined by the Green Book.
  - Rural Area – All areas not meeting the definition of Urban area.

### 3.0 CONTROLLING GUIDANCE

Controlling guidance for reconstruction projects will use the variability of values from Green Book, Interstate Guide, LRFD Bridge Design Specifications, and other formally adopted manuals.

Criteria shall use the most obvious guidance for surrounding context and classification of the road. Due to the abundance of approved policies that provide overlapping direction, the engineer of record shall make the responsible decision on criteria selection.

### 4.0 WVDOH GUIDANCE

Some geometric and cross sectional element guidance provides variability that must be standardized by the controlling agency. Working with FHWA approved documents and other adopted manuals, the WVDOH directs engineers to use the following geometric criteria parameters when designing Reconstruction type projects on the following classification of roads.

Guidance in this design directive applies only to geometric and cross sectional elements. Guidance for bridge loading, roadside hardware, signs, pavement markings can be found in other applicable policies.

#### 4.1. Arterial Routes

##### Freeways

Design speeds should be developed based upon the use of posted or operating speeds. The project purpose or scope should identify the specific criteria to be followed. The designer will document the project’s geometric and cross sectional criteria, referencing the latest policy on design standards for the interstate system. For items not specifically addressed in the latest interstate guide, guidance should be developed from other adopted policies, found in DD-600 section 4.0, using the highest desirable criteria.

### Divided and Two Lane Arterial

Design speeds should be developed based upon the use of posted or operating speeds. The designer shall reference and document controlling criteria from the Green Book that addresses the purpose of the project.

### **4.2. Collector Routes**

#### Collector Routes

Design speeds should be developed based upon the use of posted or operating speeds. The designer shall reference and document controlling criteria from the Green Book that addresses the purpose of the project. When traffic counts allow, the use of controlling criteria found in the AASHTO's "Guidelines for Geometric Design of Low-Volume Roads", current adopted edition, can be used in place of those found in the Green Book.

### **4.3. Local Roads and Streets**

#### Local Roads and Streets

Design speeds should be developed based upon the use of posted or operating speeds. The designer shall reference and document controlling criteria from the Green Book that addresses the purpose of the project. When traffic counts allow, the use of controlling criteria found in the AASHTO'S "Guidelines for Geometric Design of Low-Volume Roads", current adopted edition, can be used in place of those found in the Green Book.

## **5.0 CRITERIA REQUIREMENTS**

### **5.1. Design Speed**

Design speeds should be developed based upon the use of posted or operating speeds. Exceptions to design speed should be rare.

### **5.2. Superelevation**

Maximum superelevation for rural classification shall be 8%. Maximum superelevation for urban classification shall be 8% for freeways and arterials with design speeds equal to or greater than 50 MPH. For other urban routes, maximum superelevation shall be 4%.

### **5.3. Vertical Clearance**

Vertical clearance dimensions from Interstate Guide and Green Book shall be used and increased six inches for future resurfacing.

### **5.4. Terrain Considerations**

The topography of West Virginia contains level, rolling and mountainous types of terrain. New construction projects need to identify the terrain early in project development due to the impact on grade and excavation volumes.

The terrain in West Virginia is generally mountainous for arterial and other high level collectors. The decision on the terrain declaration is based on these alignments either starting near a lower end of a drainage basin and connecting to a top of a mountain range or crossing a drainage basin from ridge line to ridge line. Identification of a rolling terrain may be based on an alignment that parallels a river or stream system and differences in low to high points

along the mainline is less than 100' in elevation.

### **5.5. Other Geometric and Cross Sectional Element Criteria**

The designer should reference approved or adopted policies for other geometric and cross sectional design criteria. The most appropriate document should be identified and referenced for use on Reconstruction projects.

Projects on divided four lane routes shall have 4:1 median and outside side slopes. In areas that have above statewide average roadway departure accident rates, median and outside side slopes shall be graded to 6:1 and drainage adjusted.

## **6.0 DOCUMENTATION**

Criteria that will be addressed in the reconstruction project should be identified early in development either by scope or by location and design approval document. Design exceptions should be provided for any identified criteria that does not meet approved policies. The Design Exception policy should be used to document all variances to controlling criteria for all geometric and cross sectional elements being addressed with the reconstruction project.

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

DRAFT

**DESIGN DIRECTIVE-603**  
**CONSTRUCTION PROJECTS ON EXISTING ROADS CATEGORY**  
*Draft of January 27, 2022*  
*ORIGINAL VERSION*

The purpose of the design directive is to provide guidance to designers on the selection of geometric and cross sectional element design criteria for the Construction Projects on Existing Roads Category.

## **1.0 INTRODUCTION**

Construction Projects on Existing Roads (CPER) category of projects are those in which no or minor changes to the existing roadway alignment are made and no changes to the roadway type. These projects use flexible criteria based on existing performance to address the facility needs or scope of the project. Generally, a CPER type project has specific goals to meet the needs of the facility. The designer will need to narrow their scope and use only the criteria necessary to address purpose of the project.

It is important to correctly identify projects that meet the CPER type category. CPER projects are projects that address the needs of the corridor and require a lower order of accuracy and development. These projects meet specific needs and are meant to re-establish rideability or corridor functionality.

Example Project Types:

- Pavement resurfacing (one lift of material, not including scratch or patching and leveling)
- Slide repair
- Emergency repairs (declared emergency)
- Roadside hardware repair/replacement
- Concrete pavement repair
- Structure replacement/rehabilitation
- Drainage structure

## **2.0 CONSTRUCTION PROJECTS ON EXISTING ROADS CLASSIFICATION**

This classification of projects encompasses a vast majority of the projects developed for construction by district and division design staff. Projects of this type have specific goals and requirements that may use engineering judgement for practical solutions. There may be lower levels of development and accuracy for plan development and design. Project development, dimensions and limits shall use the surrounding context of the corridor for preparing the controlling criteria. Within the project limits there may exist the need to provide different levels of development for critical elements.

CPER project types are similar to what has historically been referred to as RRR (Resurfacing, Restoration and Rehabilitation). The policy and guidance included in this design directive will

establish WVDOH's RRR policy. The designer should use practical solutions to determine design criteria based on the surrounding corridor context. When practical, or if identified by accident data, the designer shall incorporate safety enhancements into the project scope.

It is of utmost importance that the scope of work and purpose of the project are identified through all levels of management. The engineer of record shall identify the level of development and accuracy of critical elements prior to beginning design.

Controlling criteria documents will be used as a guide for reference. Design exceptions will be unnecessary when there is no intent for construction activities to allow for the complete buildout of the geometric and cross sectional elements found in controlling guidance. When construction activities allow for geometric and cross sectional elements to be evaluated, variance to controlling criteria will be formally documented using the design exception policy.

The designer shall make all attempts to identify all safety and ADA requirements. The designer shall document all CPER resurfacing projects using the attached Federal-Aid Resurfacing Operational & Safety Review Checklist 2016 (Safety Checklist). ADA accommodation shall be made on all alteration projects. Refer to DD-811 for a list of projects defined as alteration. If curb ramps can not be constructed to standard, an ADA Design Exception shall be submitted following the steps outlined in DD- 811.

### 3.0 DEFINITIONS

- Traveled Way -The portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.
- Roadway - The portion of a highway, including shoulders, for vehicular use. A divided highway has two or more roadways.
- Graded Shoulder - The width of shoulder from the edge of the traveled way to the intersection of the shoulder slope and foreslope.
- Usable Shoulder - The width of shoulder that can be used when a driver parks or makes an emergency stop. If the sideslope is 4:1 or flatter, the usable shoulder width is the same as the graded shoulder width.
- Controlling Criteria – Values used to provide a standardized approach to develop alignments and cross sectional elements. These values provide a consistency to roadway layout and allow a uniform driver expectancy. These controlling criteria are defined as: design speed, design loading structural capacity, lane width, shoulder width, horizontal curve radius, superelevation rate, maximum grade, stopping sight distance, cross slope, and vertical clearance. There are other cross sectional elements that are developed and based on project goals and surrounding context, for instance number of lanes or bike/pedestrian facilities. Controlling criteria shall be used from AASHTO policies, as adopted by FHWA by 23 CFR 625.
- Green Book – Current adopted edition of AASHTO's "A Policy on Geometric Design of Highways and Streets"
- Interstate Guide – Current adopted edition of AASHTO's "A Policy on Design Standards – Interstate System".
- Urban Area – Areas meeting a population density as defined by the Green Book.
- Rural Area – All areas not meeting the definition of Urban area.

#### 4.0 CONTROLLING GUIDANCE

Controlling guidance for geometric and cross sectional elements on CPER projects will use values from Green Book, Interstate Guide and other formally adopted manuals as a guide when making decisions based on the context of the corridor. Reference criteria shall use the most obvious guidance for surrounding context and classification of the road. Due to the abundance of approved policies that provide overlapping direction, the engineer of record shall make the responsible decision on criteria selection.

Controlling guidance on CPER projects for structure design shall follow current LRFD Bridge Design Specifications.

Controlling guidance on CPER projects for roadside hardware, traffic signs and signal and maintenance of traffic shall follow the current approved edition of Manual for Assessing Safety Hardware, LRFD Standard Specification for Structural Supports for Highway Signs, Luminaires, and Traffic Signals and Manual on Uniform Traffic Control Devices.

#### 5.0 WVDOH GUIDANCE

Some geometric and cross sectional element guidance provides variability that must be standardized by the controlling agency. Working with FHWA approved documents and other adopted manuals, the WVDOH directs engineers to use the following geometric criteria parameters when designing CPER type projects on the following classification of roads.

##### 5.1. Arterial Routes

###### Freeways

Design speeds should be developed based upon the use of posted or operating speeds. The project purpose or scope should identify the specific criteria to be followed. The designer will document the project's geometric and cross sectional criteria, referencing the latest policy on design standards for the interstate system. For items not specifically addressed in the latest interstate guide, guidance should be developed from other adopted guidance using the highest desirable criteria.

###### Divided and Two Lane Arterial

Design speeds should be developed based upon the use of posted or operating speeds. The designer shall reference and document controlling criteria from the Green Book that addresses the purpose of the project.

##### 5.2. Collector Routes

###### Collector Routes

Design speeds should be developed based upon the use of posted or operating speeds. The designer shall reference and document controlling criteria from the Green Book that addresses the purpose of the project. When traffic counts allow, the use of controlling criteria found in the AASHTO's "Guidelines for Geometric Design of Low-Volume Roads", current adopted edition, can be used in place of those found in the Green Book.

##### 5.3. Local Roads and Streets

###### Local Roads and Streets

Design speeds should be developed based upon the use of posted or operating speeds.

The designer shall reference and document controlling criteria from the Green Book that addresses the purpose of the project. When traffic counts allow, the use of controlling criteria found in the AASHTO's "Guidelines for Geometric Design of Low-Volume Roads", current adopted edition, can be used in place of those found in the Green Book.

## **6.0 CRITERIA REQUIREMENTS**

### **6.1. Design Speed**

Design speeds should be developed based upon the use of posted or operating speeds. Exceptions to design speed should be rare.

### **6.2. Superelevation**

Maximum superelevation for rural classification shall be 8%. For rural routes with design speeds less than 45 mph and ADTs 500 or less, superelevation can be 6% maximum if issues with over supply of superelevation causes issues with maximum relative gradient or excessively long project termini. For rural routes with design speeds equal to or less than 35 mph and ADTs 500 or less, superelevation can be 4% maximum if issues with over supply of superelevation causes issues with maximum relative gradient or excessively long project termini. The use of superelevation other than 8% maximum on rural routes is only allowed on projects when accident rates are lower than the average statewide roadway departure accident rate within the corridor.

Maximum superelevation for urban classification shall be 8% for freeways and arterials with design speeds equal to or greater than 50 MPH. For other urban routes, maximum superelevation shall be 4%.

### **6.3. Vertical Clearance**

Vertical clearance dimensions from Interstate Guide and Green Book shall be used. The designer shall verify that any increase in the pavement surface will provide a minimum vertical clearance.

### **6.4. Terrain Considerations**

The topography of West Virginia contains level, rolling and mountainous types of terrain. New construction projects need to identify the terrain early in project development due to the impact on grade and excavation volumes.

The terrain in West Virginia is generally mountainous for arterial and other high level collectors. The decision on the terrain declaration is based on these alignments either starting near a lower end of a drainage basin and connecting to a top of a mountain range or crossing a drainage basin from ridge line to ridge line. Identification of a rolling terrain may be based on an alignment that parallels a river or stream system and differences in low to high points along the mainline is less than 100' in elevation.

### **6.5. Other Geometric and Cross Sectional Element Criteria**

The designer should reference approved or adopted policies for other geometric and cross sectional design criteria. The most appropriate document should be identified and referenced for use on CPER projects.

The designer shall determine the existing roadside departure accident rate for all CPER

projects. For projects that have above statewide average roadway departure rates, the designer shall investigate corrective measures for the corridor or spot corrections and document and implement actions. Projects on divided four lane routes that have above statewide average roadway departure accident rates within the project limits shall have median and outside side slopes graded to 6:1 and drainage adjusted. Types of corrective measures could include but not limited to are:

- Cross slope correction
- Superelevation correction
- Skid friction improvement
- Curve widening
- Shoulder paving
- Shoulder widening
- Lane departure warning devices
- Pavement marking and signing

## 7.0 DOCUMENTATION

Criteria that will be addressed in the CPER project should be identified early in development either by scope or by location and design approval document. Documentation of criteria should be based on addressing specific corridor needs and above statewide average roadway departure rates. For projects classified as an alteration project, all necessary improvements are to be included with the project documents. Any ADA design exception to an alteration project shall be documented per DD-811 “Curb Ramps and Sidewalks”.

The designer needs to address the criteria used based on the component under design. Engineers are cautioned the practical designs based on historical solutions that provide acceptable to exceeding in-service performance still require documentation of design decisions. Plan documents shall address repair and replacement of any roadside hardware devices, signing and markings. Information on accident data shall be required on all projects and a careful review shall be made to determine if there are critical location or corridor issues that need addressed.

For Interstate and four-lane highways, the designer will contact FHWA to conduct a field review after receiving preliminary information required for completing the Federal-Aid Resurfacing Operational & Safety Review Checklist 2016. Recommendations from the field review will be incorporated into the plans or noted as future projects or maintenance activities on the Federal-Aid Resurfacing Operational & Safety Review Checklist. The checklist, and any supporting documents, will be submitted with the PS&E package.

## Federal Aid Resurfacing Operational & Safety Review Checklist 2016

You will receive the following information from the Traffic Engineering Division. This information must be included in the review package before submitting for PS&E:

- Include copy of crash data (generally excel spreadsheet-see attached example)
- Include crash rate analysis (overall, wet, night as compared to statewide average-see attached example)
- NOTE:* For non-expressway facilities, if no crash rates are above the statewide average (overall, wet, night) check here, attach the data and no further review is required.
- Include the preliminary recommendation(s) for proposed improvements to address any identified rates above statewide average-see attached example)

The Operational Review & Safety Report prepared by District Design/District Traffic Engineering should include the following:

### 1. *Project Overview*

- Project Limits
- AADT
- Roadway classification (NHFC)
- ROW Width
- Location Map (WVDOH)/Google Map

### 2. *Operational Experience*

- Using the crash data (excel spreadsheet) received from the Traffic Engineering Division match the mile points and the project stations included in the maintenance project construction documents. As an alternative location of crashes can be shown on Google map, Planning Straight Line or Cultural Straight Line
- Summary of any fatal crashes (narrative only)
- List any identified/recurring maintenance or operational concerns

### 3. *Pavement Conditions*

	Existing	Proposed
Lane Width		
Shoulder Width		
Shoulder Material		
Drop Off		
Safety Edge Presence		
Pavement Condition (IRI if available)		

### 4. *Conclusions/Recommendations*

- Summary of any operational or safety enhancements included within project
- Summary of how the initial recommendations for improvements were addressed
- Any recommendations for improvements to be included in a separate project (send summary for all projects to Traffic Engineering Division for consideration in HSIP)

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

DRAFT

**DESIGN DIRECTIVE 605**  
**CONTROLLING CRITERIA AND DESIGN EXCEPTION POLICY**  
*Draft of January 27, 2022*  
*Supersedes February 9, 2017*

Attached for your use is the Division of Highways (DOH) Controlling Criteria and Design Exception Policy. It shall be used on all applicable projects.

**1.0 CONTROLLING CRITERIA**

All new construction or reconstruction projects on the National Highway System (NHS) shall be designed in accordance with the AASHTO criteria, such as criteria found in the most current approved editions of AASHTO’s “A Policy on Geometric Design of Highway and Streets” (Green Book) and the Roadside Design Guide. All RRR projects shall be designed in accordance with the current ~~FHWA-approved State RRR criteria~~ Construction Project on Existing Roads (CPER) guidance. Those criteria related to design speed, lane and shoulder widths, design loading structural capacity, horizontal curve radius, maximum grade, stopping sight distance, cross slope, superelevation rate, and vertical clearance are the controlling criteria that require formal design exceptions when not met. Stopping sight distance (SSD) applies to horizontal alignments and vertical alignments except for sag vertical curves.

Of the 10 controlling criteria, only design loading structural capacity and design speed apply to all NHS facility types. The remaining eight criteria are applicable only to "high-speed" NHS roadways, defined as Interstate highways, other freeways, and roadways with a design speed greater than or equal to 50 mph (80 km/h). However, any of the remaining eight criteria that are not met on projects that do not require a formal design exception for these criteria shall be documented to the project file, with any mitigation strategies indicated in the report. The following chart is included for clarity.

**Application of Controlling Criteria**

<b>All NHS Roadways:</b>	<b>Interstate highways, other freeways, and roadways with design speed ≥ 50 mph:</b>
Design Speed	Design Speed
Design Loading Structural Capacity	Design Loading Structural Capacity
	Lane Width
	Shoulder Width
	Horizontal Curve Radius
	Superelevation Rate
	Maximum Grade
	Stopping Sight Distance
	Cross Slope
	Vertical Clearance

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According to the Stewardship & Oversight Agreement, the WVDOH is responsible for preparation and approval of all design exceptions, including those on the NHS. For Full Oversight and other non-Delegated projects, a copy of the design exception should be forwarded to the FHWA for acceptance.

~~The criteria are included in the attached flowcharts and “Design Exception Justification Report”. These criteria will be referred to throughout this document as the “10 Controlling Criteria”. A formal design exception is not required and should not be submitted unless the existing/proposed value for any of the “10 Controlling Criteria” fails when compared to the design values for any of these criteria. The designer should follow “Step 1” of the attached flowchart when determining which design directive to use in establishing design values.~~

## 2.0 DESIGN EXCEPTIONS

All efforts should be made to adhere to the specified criteria. However, under unusual conditions, it may be necessary to use values that are less than the minimum values that have been established. If lesser values are proposed for use, a Design Exception Justification Report shall be developed and approved. ~~The approved Design Exception Report shall be submitted to Planning Division at PS&E for filing and inclusion on the straight line diagrams.~~ A copy of the approved design exception shall be included in the project file.

As stated previously, design exceptions shall only be submitted when one or more of the “10 Controlling Criteria” design values is violated by the existing/proposed values. Design exceptions are not required and shall not be submitted on projects defined as ~~Construction Projects on Existing Roads (CPER) as shown in the attached flowcharts~~ project types. The designer should refer to DD-600 “Project Categories” for guidance in determining which project types are classified as CPER maintenance projects and not defined as an alteration project.

Design criteria that are not satisfied and are not one of the “10 Controlling Criteria” shall not be documented and submitted as a formal design exception. These criteria exceptions shall be documented in the project files only.

## 3.0 DESIGN EXCEPTION JUSTIFICATION REPORT

The following information, which affects the design values selected, must be considered and documented as a part of the design exception request: (1) the existing roadway characteristics, the minimum design criteria values, specific design criteria that will not be met, the proposed design values, and the criteria source (controlling design directive or AASHTO section) must be identified; and (2) a narrative documenting that the acceptance of the design exceptions is prudent, cost-effective and will not compromise the safety of the traveling public. This narrative will discuss the following items:

1. The effect of the variance from the design criteria on the safety and operation of the facility and other impacts such as right-of-way, community, environmental, cost, and usability by all modes of transportation; and safety mitigating measures considered and provided;
2. The compatibility of the design and operation with adjacent sections of roadway;
3. Amount and character of traffic using the facility;
4. Accident history (type, location, severity, etc.);
5. Alternatives considered;

6. Comparative cost of full design criteria versus lower design criteria being proposed, or other practical alternatives;
7. The long-term effect of the reduced design criteria versus full design criteria (effect of capacity reduction);
8. Difficulty in obtaining full design criteria (cost, right of way involvement, delay, environmental impacts, etc.);
9. Level of Service for full design criteria versus reduced design criteria; and,
10. Any other design criteria that is not being met, i.e., cumulative effect of more than one standard that is being reduced.

This documentation is essential for each design exception requested. The level of analysis should be commensurate with the complexity of the project.

Design speed and design loading structural capacity are fundamental criteria in the design of a project. Exceptions to these criteria should be extremely rare and the documentation will provide the following additional information.

Design Speed exceptions:

- Length of section with reduced design speed compared to overall length of project,
- Measures used in transitions to adjacent sections with higher or lower design or operating speeds.

Design Loading Structural Capacity exceptions:

- Verification of safe load-carrying capacity (load rating) for all State unrestricted legal loads or routine permit loads, and in the case of bridges and tunnels on the Interstate, all Federal legal loads.

Design values chosen or being considered, which are exceptions to the appropriate minimum design criteria values, shall be approved as early as possible in the design process prior to considerable detailed design work being accomplished. The ~~Deputy State Highway Engineer—District Operations~~ appropriate ~~Depute State Highway Engineer overseeing the plan development,~~ will approve those exceptions necessary on projects ~~that are District "Design Responsibility". The Deputy State Highway Engineer—Development will approve those exceptions necessary on projects that are a Central Office Division "Design Responsibility".~~ The DOH is responsible for the preparation and approval of all design exceptions.

During the design process, a continuing review of exceptions shall be made, and any additions or modifications documented and placed in the project files. Approved design exceptions will be shown on the title sheet and a copy saved to the “project file”. Design exceptions on all projects must be approved prior to submission of the PS&E package to Contract Administration Division.

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**DESIGN EXCEPTION JUSTIFICATION REPORT**

**PROJECT DATA**

State Project No. \_\_\_\_\_ Date: \_\_\_\_\_

Federal Project No: \_\_\_\_\_ County: \_\_\_\_\_

Project Name: \_\_\_\_\_

Project Description: \_\_\_\_\_

WVDOH Representative: \_\_\_\_\_

FHWA Representative: \_\_\_\_\_

**HIGHWAY ROUTE DATA**

- AASHTO Functional Classification
- 1.  Urban  Rural
  - 2.  Arterial  Collector  Local Road
  - 3.  Freeway  Divided/Arterial  Two-Lane Arterial
  - 4.  Interstate

**TERRAIN TYPE**  Level  Rolling  Mountainous

**TRAFFIC DATA** Current Year: \_\_\_\_\_ Design Year: \_\_\_\_\_

ADT: \_\_\_\_\_ ADT: \_\_\_\_\_

DHV: \_\_\_\_\_ DHV: \_\_\_\_\_

**SPEED LIMIT:** \_\_\_\_\_  POSTED/  REGULATORY

**ACCIDENT DATA**

Accident Rate: \_\_\_\_\_

Base Accident Rate (Statewide Average): \_\_\_\_\_

Nature of Area: \_\_\_\_\_

**DISTRICT DESIGN PROJECTS**

**DESIGN CRITERIA DATA (Document Only Exceptions)**

<b><u>10 Controlling Criteria</u></b>	<b><u>Existing Condition</u></b>	<b><u>Minimum Design Criteria</u></b>	<b><u>Proposed Value</u></b>	<b><u>Criteria Source</u></b>
1. Design Speed	_____	_____	_____	_____
2. Lane Width	_____	_____	_____	_____
3. Shoulder Width	_____	_____	_____	_____
4. Stopping Sight Distance	_____	_____	_____	_____
5. Horizontal Curve Radius	_____	_____	_____	_____
6. Maximum Grade	_____	_____	_____	_____
7. Cross-Slope	_____	_____	_____	_____
8. Superelevation	_____	_____	_____	_____
9. Vertical Clearance	_____	_____	_____	_____
10. Design Loading Structural Capacity	_____	_____	_____	_____

**APPROVAL SIGNATURES**

**RECOMMENDED:**

1. \_\_\_\_\_  
Responsible Charge Engineer

2. \_\_\_\_\_  
District Engineer/**Director** or  
Engineer of Record

**APPROVED:**

\_\_\_\_\_ **Deputy State Highway Engineer –**  
**(Project Development Oversight)**

\_\_\_\_\_ **Federal Highway Administration**  
***(Full Oversight and non-Delegated Projects Only)***

CENTRAL OFFICE DESIGN PROJECTS

DESIGN CRITERIA DATA (Document Only Exceptions)

<u>10 Controlling Criteria</u>	<u>Existing Condition</u>	<u>Minimum Design Criteria</u>	<u>Proposed Value</u>	<u>Criteria Source</u>
1. Design Speed	_____	_____	_____	_____
2. Lane Width	_____	_____	_____	_____
3. Shoulder Width	_____	_____	_____	_____
4. Stopping Sight Distance	_____	_____	_____	_____
5. Horizontal Curve Radius	_____	_____	_____	_____
6. Maximum Grade	_____	_____	_____	_____
7. Cross-Slope	_____	_____	_____	_____
8. Superelevation	_____	_____	_____	_____
9. Vertical Clearance	_____	_____	_____	_____
10. Design Loading Structural Capacity	_____	_____	_____	_____

APPROVAL SIGNATURES

RECOMMENDED: \_\_\_\_\_ APPROVED: \_\_\_\_\_

1. \_\_\_\_\_ Deputy State Highway Engineer – Development  
Responsible Charge Engineer

2. \_\_\_\_\_ Federal Highway Administration  
Division Director (Full Oversight and non-Delegated Projects Only)



WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

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**DESIGN DIRECTIVE 624**

**RAMP TERMINALS**

*Draft of January 27, 2022*

*Supersedes July 11, 2012*

**1.0 GOVERNING CRITERIA**

Ramp terminals shall be designed based on criteria found in the latest approved edition of the AASHTO's "A Policy on Geometric Design of Highways and Streets" (Green Book).

**2.0 GEOMETRIC GUIDANCE**

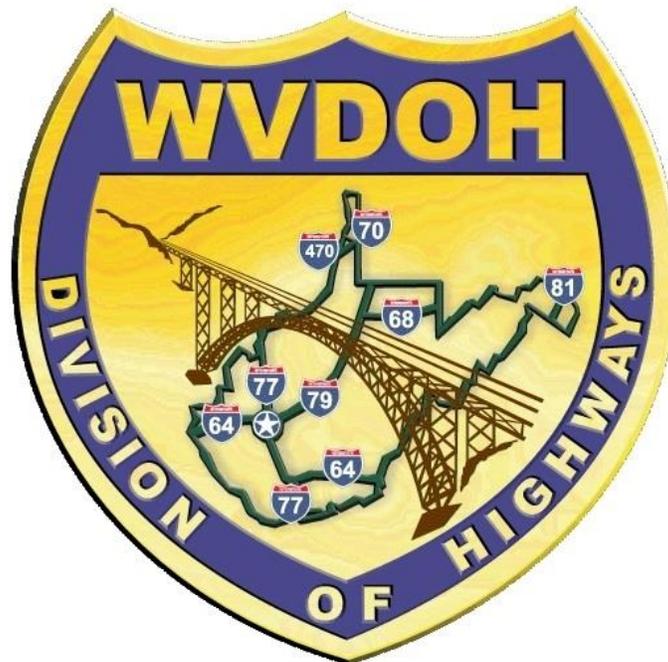
Ramp geometric layout shall use the taper configuration. Parallel configuration may be used if excessive acceleration and deceleration lengths are needed or when mainline geometry issues cause poor angles for motorists to use side and rear view mirrors. The use of parallel configuration will be determined and documented during the initial design study phase and approved prior to preliminary engineering activities.

**3.0 CROSS SECTIONAL ELEMENT GUIDANCE**

Ramp lane and shoulder widths shall be determined from the Green Book using the table referring to "Design Widths of the Traveled Way for Turning Roadways". For one lane operation, Case II "Provision for passing stalled vehicle" and condition C shall govern for roadway widths. For two lane operation condition C shall govern for roadway widths. Full shoulders will be carried on to ramp structures up to 200 feet or greater. Ramp bridges greater than 200 feet require special consideration and reduction in width should be approved and documented prior to preliminary design.

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# STRUCTURE DIRECTIVES



WEST VIRGINIA DEPARTMENT OF  
TRANSPORTATION  
DIVISION OF HIGHWAYS

January 2022

Electronic copies of this Manual can be obtained via the Internet at  
<https://transportation.wv.gov/>

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WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

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**STRUCTURE DIRECTIVE 1010**  
**PROJECT DESIGN CRITERIA**  
**Date of Implementation**  
**First Edition**

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, Road and Bridges* (Standard Specifications) including the latest supplemental specifications.

See DD 600 information that is applicable to the 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 WVDOH will determine the line and grade on a project. If a Consultant is designing the project, then the line and grade will be determined by the Consultant.

**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, among other items.

**1010.4-HIGHWAY DRAINAGE, HYDROLOGY AND HYDRAULICS, 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**

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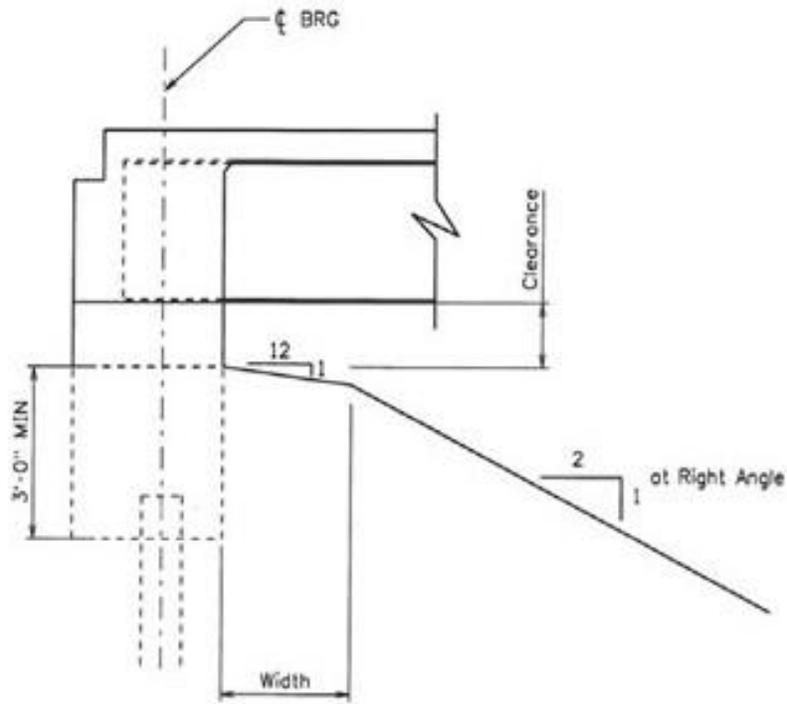
**STRUCTURE DIRECTIVE 1020  
BRIDGE LAYOUT CRITERIA  
Date of Implementation  
First Edition**

**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: See Figure 1020.A:
  - 1. A minimum berm width of 3 FT shall be used under dry conditions.
  - 2. For wet conditions, a berm width of 5 FT is preferred.
  - 3. When very steep terrain is encountered, a berm width of 10 FT 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.B.
  - 1. A minimum 1.5 FT clearance between the berm and superstructure is required except that a minimum of 3.0 FT 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 10 FT a minimum 3 FT 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 3 FT minimum clearance is preferred.
- C. The maximum desirable skew is 30°; 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.

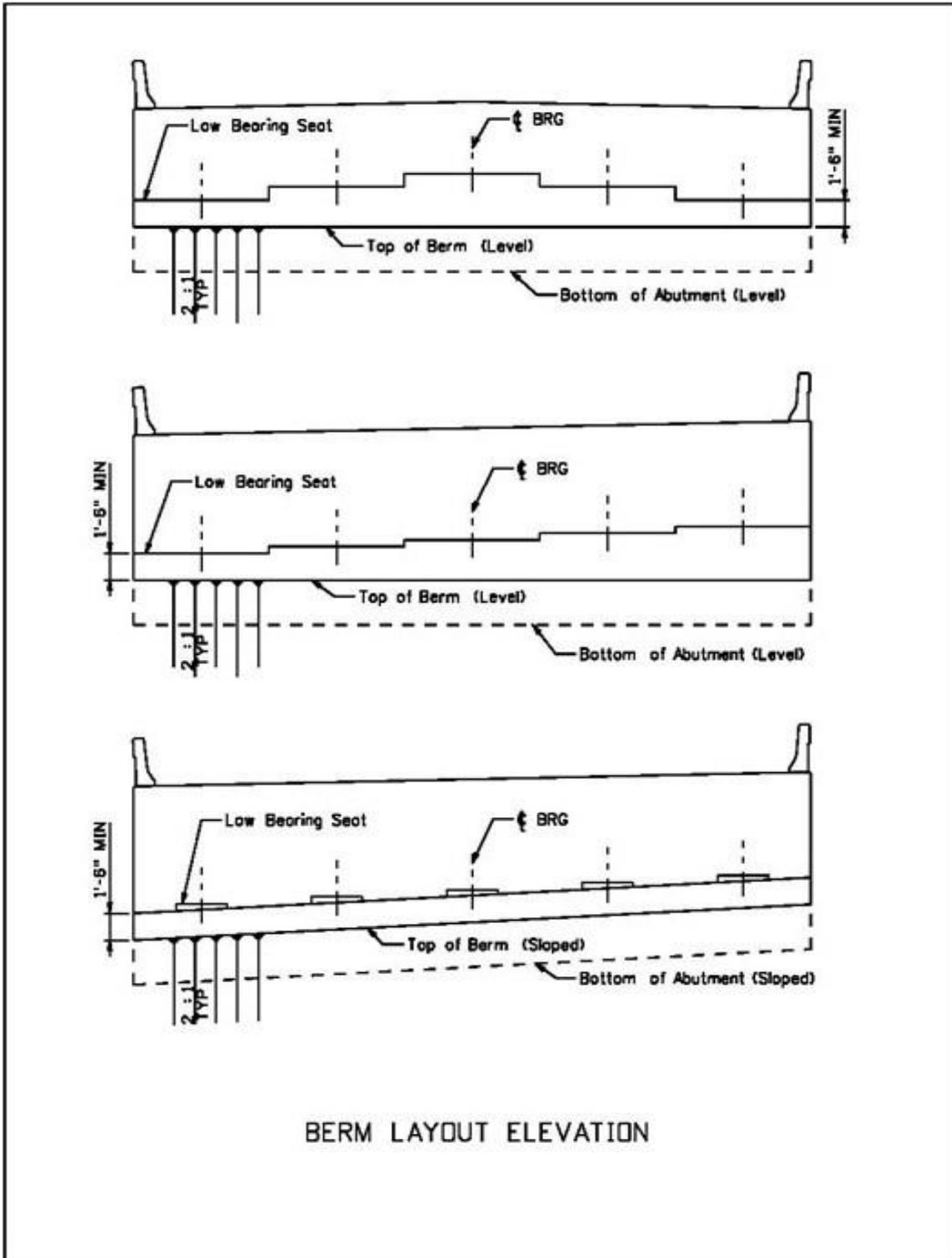
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BERM LAYOUT SECTION

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 WVDOT Drainage Manual for further guidance.

Freeboard, the clear distance above the design discharge elevation and the lowest portion of the superstructure, shall be 2 FT with assurance that the bridge bearings are above the design discharge elevation unless otherwise approved by the Director of Engineering.

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 toe of the embankment shall not encroach into the stream channel.

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 below. 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 and the final grade shall provide 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.

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

**Table 1020.C**

\* 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 FT when passing under an Interstate Route adjacent to the Interstate Interchange.

**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.

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

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**STRUCTURE DIRECTIVE 1030  
GEOTECHNICAL INVESTIGATIONS  
Date of Implementation  
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

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**STRUCTURE DIRECTIVE 1031**  
**STRUCTURE BORING REQUIREMENTS**  
**Date of Implementation**  
**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 5.0 FT 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 structures, 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. 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 100 FT 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 10.0 FT of rock. For spread foundations, drill two times the estimated breadth of the foundation or a minimum of 10.0 FT into rock at the discretion of the Geotechnical Engineer. For drilled caissons, core a minimum of three times the estimated shaft diameter into rock. If shaft locations are known, drill one boring at each shaft location. 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 50/6 IN but no deeper than 50/3 IN.

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 100 FT, as close to the structure's foundation as possible. Borings shall be extended below the bottom of the foundation at least 10 FT into competent material and SPT sampling on 2.5 FT 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 10 FT 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:

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- 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**

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**STRUCTURE DIRECTIVE 1033  
SCOUR ANALYSIS  
Date of Implementation  
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.

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- (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 Yes \_\_\_\_ No \_\_\_\_ (13) Screening Category \_\_\_\_\_
- (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.

Yes \_\_\_\_\_ No \_\_\_\_\_ Low Risk 1 D 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: \_\_\_\_\_

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.

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

DRAFT

**STRUCTURE DIRECTIVE 1040  
STRUCTURAL SYSTEM SELECTION  
Date of Implementation  
First Edition**

**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. The following sections discuss some of the steel and concrete superstructure types that are used by the WVDOH. 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 and piers founded on various types of foundations. Common abutment and pier types along with foundation types are also described later in this section.

**1040.1-STEEL SUPERSTRUCTURE TYPES**

Steel superstructures should be considered for any span length ranging from 20 FT to 650 FT or more. Generally, the following table, Table 1040.A, can be used as a guideline for selecting steel superstructure types.

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

**Table 1040.A**

The superstructure should be designed such that the structure has redundant load paths and is not considered fracture critical. Some designs, especially truss and tied arch designs, are generally, by their very nature, fracture critical. As defined in the Governing Specifications, a Fracture-Critical Member (FCM) is a “*Component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function*”. The Designer is to declare at Span Arrangement or TS&L if the structure is fracture critical. 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 WVDOT's policy on high performance steel, SD 2031.

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.

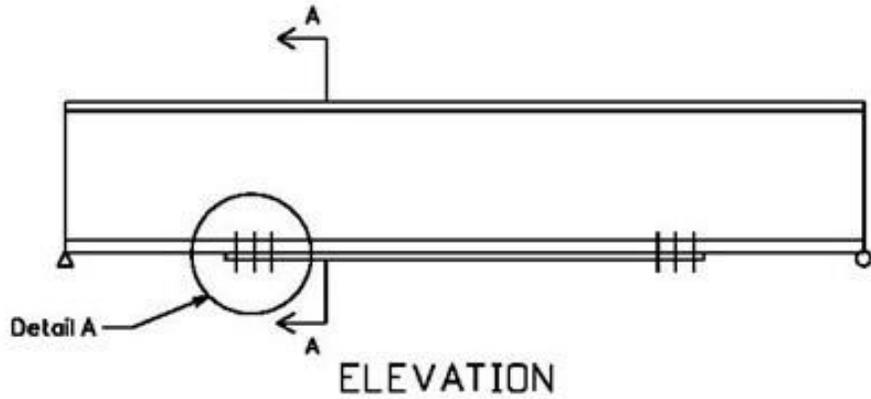
The following section discusses the various types of steel superstructure types and guidelines for when to consider them.

**1040.1.1-Rolled Beams:** Rolled beams should be considered for any span length ranging from 20 FT to 100 FT. With cover plates, the span range of rolled beams can be extended to 130 FT. However, only end bolted cover plates shall be used. See Figure 1040.B. The Designer shall determine the availability of any rolled section considered, including lengths and grade of steel.

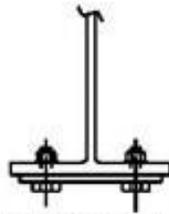
The Designer should minimize the number of beam lines. Rolled beam bridges should have a minimum of three stringer lines.

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.

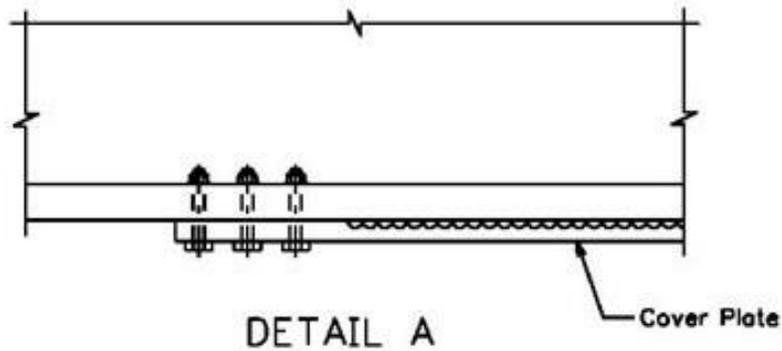
### END BOLT COVER PLATE DETAIL



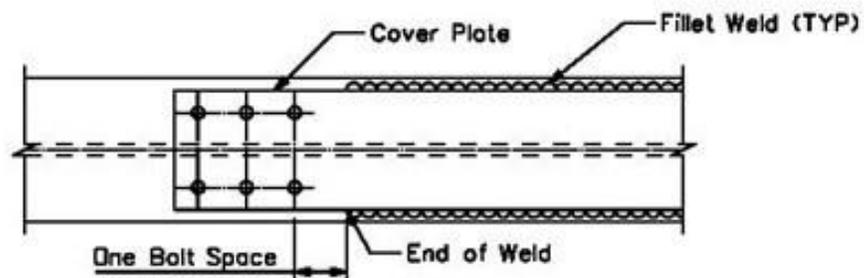
ELEVATION



SECTION A-A



DETAIL A



PLAN

Note:  
Connection to be  
designed by the  
Engineer.

Figure 1040.B  
SD 1040  
Page 3 of 13

**1040.1.2-Plate Girders:** Plate girders should be considered for any span length ranging from 80 FT to 400 FT. 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 one girder line, in some cases, may increase the total weight of the steel. However, the savings realized through fabrication of one less girder, 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 unless the system is structurally redundant and not fracture critical.
- 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 considered to eliminate any uplift problems at the abutments. 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).

Detailing interior and exterior girders the same is often desirable for curved bridges with three to five girder lines and for most tangent bridges. Therefore, when designing tangent bridges, consider “balancing” the total factored design moment 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. For curved structures with five or more girders, consider grouping the girders into two similar designs, one for girders interior of the centerline of the bridge and one for girders exterior to the centerline of the bridge. Consult with fabricators to ascertain the least cost approach.

Limit girder spacing to 15 FT for typical girder structures. For girder/sub-stringer framing arrangements, the main girders may be efficiently spaced at 20 FT to 22 FT. 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  $\frac{7}{16}$  IN. Increment the web thickness by a minimum of  $\frac{1}{16}$  IN. 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.

**1040.1.3-Box Girders:** Steel box girders can be considered as an alternate for steel plate girders for span length ranging from 200 FT to 400 FT.

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.

**1040.1.4-Trusses:** Trusses can be used for bridges over navigable river crossings with spans from 400 FT to 900 FT. 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.

**1040.1.5 -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.

**1040.1.6-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.

**1040.2-Concrete Superstructure Types:** Concrete superstructure types should be considered for any span length ranging from 20 FT to 650 FT or more. Generally, the following, Table 1040.C, can be used as a guide for selecting concrete superstructure types.

SPAN LENGTH (FT)	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 1040.C**

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 6000 PSI (5500 PSI for WVDOH Standard Box Beams) at release ( $f'_{ci}$ ) with a minimum final compressive strength of 8000 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 10000 PSI and a release strength of up to 9000 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 following discusses the various types of concrete superstructure types and guidelines for when to consider them.

**1040.2.1-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 30 FT in length.

**1040.2.2-Box Beams:** For short span bridges of 100 FT 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.

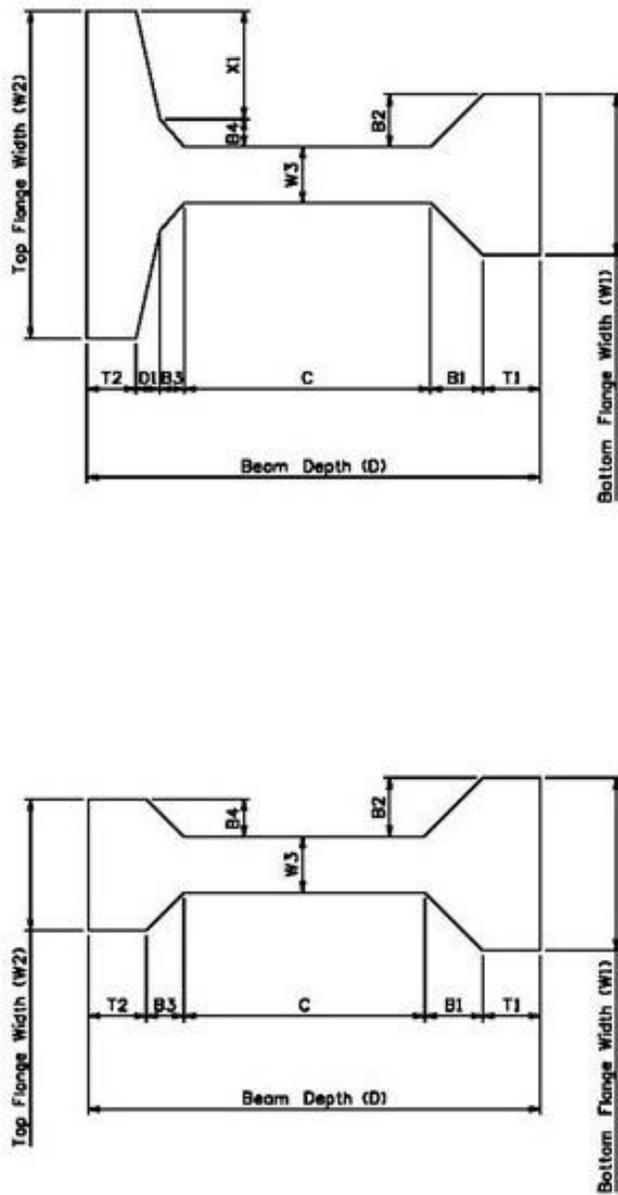
Factors involved in the choice of box beam configuration design should include but are not limited to economics, traffic type and volume, time constraints, and method of construction (whether by contract or state construction crews which generally have limited construction capabilities). The Bridge Designer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

**1040.2.3-Prestressed Concrete Beams:** AASHTO Type I, II, III, IV or Type IV Modified prestressed concrete beams should be considered for bridges with spans from 25 FT to 145 FT. The maximum span length is based on the haul capacity 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 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 one of the beam sections from Figure 2.4.2.3. AASHTO Type V and Type VI sections shall not be used.

Alternate beam sections may be used for special design situations. 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 1040.E

WVDDOT-DOH STANDARD PRESTRESSED CONCRETE I-BEAM SECTIONS



AASHTO I-BEAM  
Typical - Type II, III & IV

AASHTO I-BEAM \* & VI\*  
Typical-Type IV MDD, V & VI\*

Beam Designation	Top Flange Width (IN) W2	Bottom Flange Width (IN) W1	Depth (IN) D	Depth (IN)						Flange (IN)			Web Thickness (IN) W3	Basic Beam Properties		
				T2	D1	B3	C	B1	T1	X1	B4	B2		Area (IN <sup>2</sup> )	I <sub>x</sub> (IN <sup>4</sup> )	
II	12	18	36	6	-	3	15	6	6	-	3	6	6	369	15.8	50,980
III	16	22	45	7	-	4 1/4	19	7 1/2	7	-	4 1/2	7 1/4	7	560	20.3	125,390
IV	20	26	54	8	-	6	23	9	8	-	6	9	8	789	24.7	260,730
IV MDD	36	26	60	4	2	3	34	9	8	11	3	9	8	880	28.8	384,248
IV MDD	36	26	66	4	2	3	40	9	8	11	3	9	8	908	31.6	491,660
IV MDD	36	26	72	4	2	3	46	9	8	11	3	9	8	956	34.4	615,361
IV MDD	36	26	78	4	2	3	52	9	8	11	3	9	8	1,004	37.3	756,222
IV MDD	36	26	84	4	2	3	58	9	8	11	3	9	8	1,052	40.2	915,113
V	42	28	63	5	3	4	33	10	8	13	4	10	8	1,013	31.9	521,160
VI	42	28	72	5	3	4	42	10	8	13	4	10	8	1,085	36.4	733,320

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Figure 1040.D

**Approximate Maximum Span Lengths (FT)**

		Beam Spacing (FT)				
		14	12	10	8	6
		AASHTO Type	I	25	30	35
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

**Table 1040.E**

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 IN Type F barriers, no sidewalks and utilizing concrete with a release strength ( $f'_{ci}$ ) of 6000 PSI and a final strength ( $f'_c$ ) of 8000 PSI.

**1040.2.4-Post-Tensioned I-Beams (Drop-In):** Using post-tensioned drop-in spans can increase span lengths for prestressed concrete beams. The drop-in segments will be field spliced with pier segments and then the entire girder will be post-tensioned. At the field splice locations, temporary shoring towers or strongbacks may be required.

**1040.2.5-Segmental Concrete Boxes:** Segmental concrete boxes are an economical solution for bridges with span lengths over 100 FT 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.

**1040.2.6-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 prestressed concrete beams, 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.

**1040.3-Abutment Types:** 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:

- A. Wall Type Abutment.
- B. Pedestals.
- C. Stub Abutment.
- D. Integral Abutment.
- E. Semi-Integral Abutment.
- F. Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS).

**1040.3.1-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 drilled caissons.

The maximum exposed face should generally be 30 FT, 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 or aesthetics are not a governing factor. Otherwise, where walls greater than 30 FT are required, a stepped (terraced) wall configuration shall be used.

**1040.3.2-Pedestals:** The beam seat is supported on columns/drilled caissons or pedestals resting on individual footings. This configuration is useful for meeting unique construction problems, e.g., widely varying elevations of competent rock.

**1040.3.3-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 drilled caissons.

**1040.3.4-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 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.

**1040.3.5-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.

**1040.3.6-Geosynthetic Reinforced Soil –Integrated Bridge System Abutment (GRSIBS):** 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 consist 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 8 IN 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.

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.

**1040.3.7-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 U-shaped wingwalls may be tapered parallel to the roadway slope to match the finished grade.

#### **1040.4-PIER TYPES**

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

**1040.4.1-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 100 FT to 150 FT, the use of a compression strut at mid-height, similar to the pier cap, shall be investigated. The individual columns will be supported on an appropriate foundation.

**1040.4.2-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 has to be investigated during the preliminary design phase of the project.

**1040.4.3-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.

**1040.4.4-Steel Pier Caps:** Steel pier caps are fracture critical. If used, the design shall allow for reasonable access to the interior for future maintenance, inspection, and repair.

**1040.4.5-Foundation Types:** All feasible foundation types must be considered in the preliminary phases of the project. The WVDOH's policy is to found all new bridge foundations on rock. However, bridges may be allowed to be supported on Intermediate Geomaterial (IGM) at the discretion of the Geotechnical Engineer.

**1040.5.1-Spread Footing:** Spread footings have been found to be economical for depths to 20 FT. 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.

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.

**1040.5.2-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 Bridge Project Manager, 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 10 FT. See SD 2120.

For integral abutments, single-line piling systems shall be used, predrilled 15.0 FT deep using 1.0 FT diameter for soil or 2.0 FT 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.

**1040.5.3-Drilled Caissons:** Drilled caissons 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, drilled caissons may eliminate the need of caisson caps, for certain configurations such as single or multiple column piers.

Drilled caissons shall be designed using soil-structure intersection 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, but are not limited to, pre-installation core holes, providing a test hole for shafts 10.0 FT and larger in diameter and cross hole sonic logging (CSL) testing.

Results from the CSL testing may show inadequate structural integrity and continuity. Further investigations such as core drilling for each of the unacceptable caissons would then be required, causing construction down time and added expenses.

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

DRAFT

**STRUCTURE DIRECTIVE 1042  
CONCRETE SUPERSTRUCTURE TYPES  
Date of Implementation  
First Edition**

Concrete superstructure types should be considered for any span length ranging from 20 to 650 FT or more. Generally, the following table, Table 1042.A can be used as a guide for selecting concrete superstructure types.

SPAN LENGTH (FT)	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 6000 PSI (5500 PSI for WVDOH Standard Box Beams) at release ( $f'_{ci}$ ) with a minimum final compressive strength of 8000 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 10000 PSI and a release strength of up to 9000 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.

**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 30 FT in length.

### **1042.2-BOX BEAMS**

For short span bridges of 100 FT 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.

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.

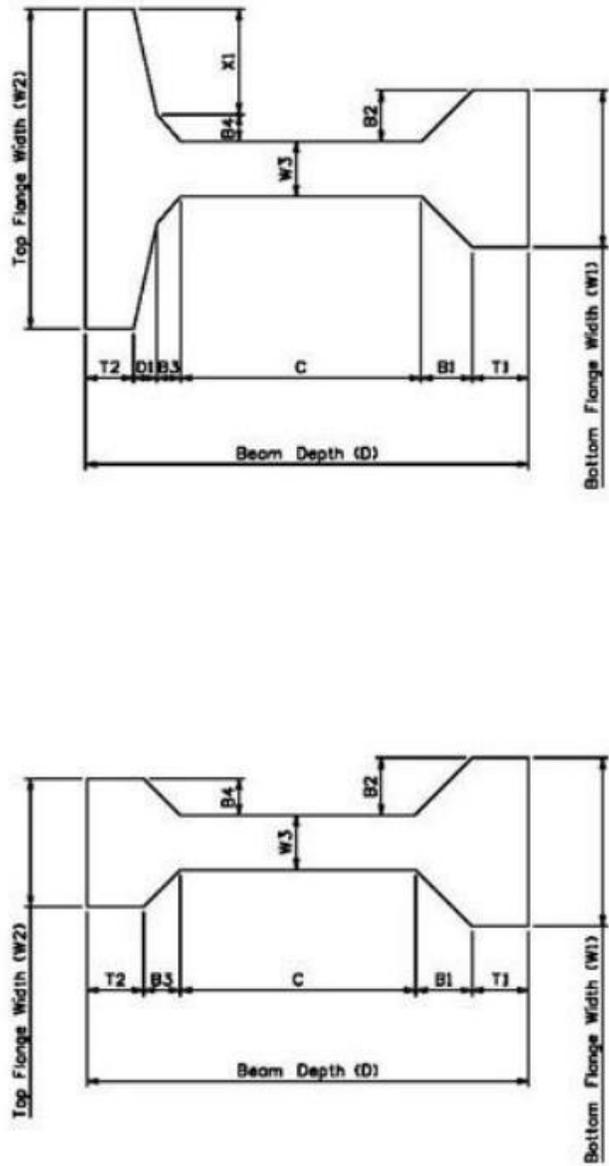
### **1042.3-PRESTRESSED CONCRETE BEAMS**

Prestressed concrete beams should be considered for bridges with spans from 25 FT to 145 FT. 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 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 locally available. Beam sections shown in Figure 1042.B represent a historical list of available shapes of prestressed concrete beams.

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WVDDT-DDH STANDARD PRESTRESSED CONCRETE I-BEAM SECTIONS



AASHTO I-BEAM  
Typical-Type IV, V & VI

AASHTO I-BEAM  
Typical-Type II, III & IV

Beam Designation	Top Flange Width (IN)		Bottom Flange Width (IN)		Depth (IN) D	Depth (IN)						Flange (IN)			Web Thickness (IN) W3	Basic Beam Properties	
	W2	W1	T2	B3		C	B1	T1	X1	B4	B2	Area (IN <sup>2</sup> )	X <sub>c</sub> (IN)	I <sub>xx</sub> (IN <sup>4</sup> )			
II	12	18	6	3	15	6	6	-	3	6	6	369	15.8	50,980			
III	16	22	7	4 1/2	19	7 1/2	7	-	4 1/2	7 1/2	7	560	20.3	125,390			
IV	20	26	8	5	23	9	8	-	6	9	8	789	24.7	260,730			
IV MOD	36	26	4	2	34	9	8	11	3	9	8	860	28.8	384,248			
IV MOD	36	26	4	2	34	9	8	11	3	9	8	908	31.6	491,660			
IV MOD	36	26	4	2	34	9	8	11	3	9	8	956	34.4	615,361			
IV MOD	36	26	4	2	34	9	8	11	3	9	8	1,004	37.3	756,222			
IV MOD	36	26	4	2	34	9	8	11	3	9	8	1,052	40.2	915,113			
V	42	28	5	3	43	10	8	13	4	10	8	1,013	31.9	521,160			
VI	42	28	5	3	43	10	8	13	4	10	8	1,085	36.4	733,320			

Figure 1042.B

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.C.

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

**Table of Approximate Maximum Span Lengths (FT)**  
**TABLE 1042.C**

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 IN Type F barriers, no sidewalks and utilizing concrete with a release strength ( $f'_{ci}$ ) of 6000 PSI and a final strength ( $f'_c$ ) of 8000 PSI.

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

Using post-tensioned drop-in spans can increase span lengths for prestressed concrete beams. The drop-in segments will be field spliced with pier segments and then the entire girder will be post-tensioned. At the field splice locations, temporary shoring towers or strongbacks may be required.

**1042.5-SEGMENTAL CONCRETE BOXES**

Segmental concrete boxes are an economical solution for bridges with span lengths over 100 FT 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.

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#### **1042.6-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 prestressed concrete beams, 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.

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

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**STRUCTURE DIRECTIVE 1042  
CONCRETE SUPERSTRUCTURE TYPES  
Date of Implementation  
First Edition**

Concrete superstructure types should be considered for any span length ranging from 20 to 650 FT or more. Generally, the following table, Table 1042.A can be used as a guide for selecting concrete superstructure types.

SPAN LENGTH (FT)	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 6000 PSI (5500 PSI for WVDOH Standard Box Beams) at release ( $f'_{ci}$ ) with a minimum final compressive strength of 8000 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 10000 PSI and a release strength of up to 9000 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.

### **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 30 FT in length.

### **1042.2-BOX BEAMS**

For short span bridges of 100 FT 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.
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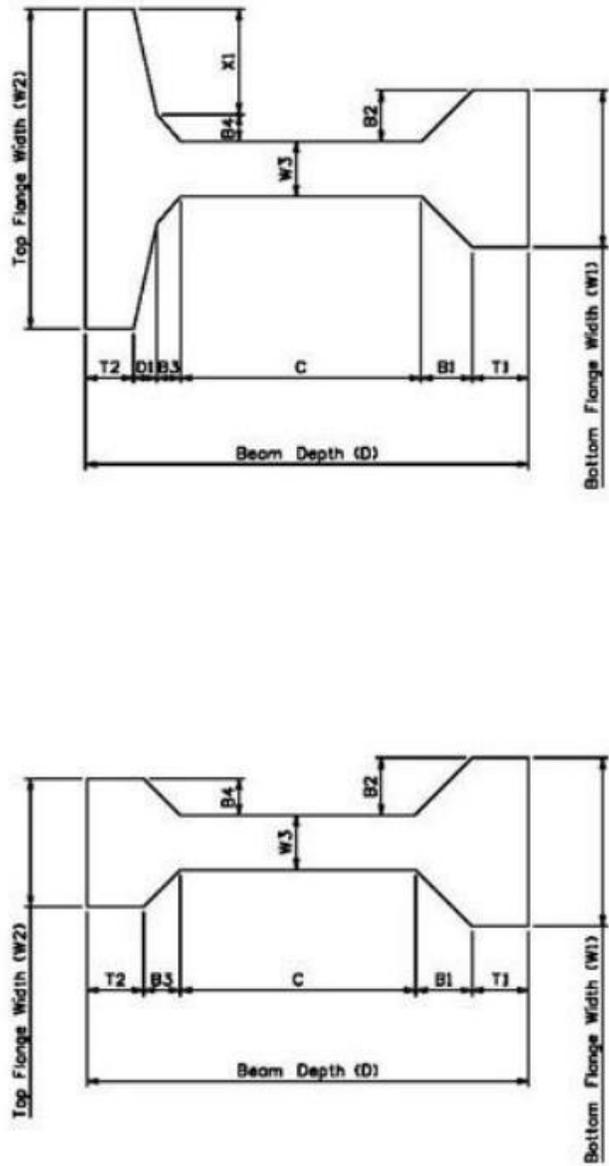
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WVDDT-DDH STANDARD PRESTRESSED CONCRETE I-BEAM SECTIONS



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

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**STRUCTURE DIRECTIVE 1043  
ABUTMENT TYPES  
Date of Implementation  
First Edition**

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 drilled caissons.

The maximum exposed face should generally be 30 FT, 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 or aesthetics are not a governing factor. Otherwise, where walls greater than 30 FT are required, a stepped (terraced) wall configuration shall be used.

#### **1043.2-PEDESTALS**

The beam seat is supported on columns/drilled caissons 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 drilled caissons.

#### **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 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 S2090 for limitations on the use of semi-integral abutments.

#### **1043.6-GEOSYNTHETIC REINFORCED SOIL-INTEGRATED BRIDGE SYSTEM ABUTMENT (GRSIBS)**

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 8 IN 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 Director of Engineering Division.

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 U-shaped wingwalls may be tapered parallel to the roadway slope to match the finished grade.

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

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**STRUCTURE DIRECTIVE 1044  
PIER TYPES  
Date of Implementation  
First Edition**

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 100 to 150 FT, the use of a compression strut at mid-height, similar to the pier cap, shall be investigated. 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 has to 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**

Steel pier caps are fracture critical. If used, the design shall allow for reasonable access to the interior for future maintenance, inspection, and repair.

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

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<p><b>STRUCTURE DIRECTIVE 1045 FOUNDATION TYPES Date of Implementation First Edition</b></p>
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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 discretion of the Geotechnical Engineer.

**1045.1-SPREAD FOOTING**

Spread footings have been found to be economical for depths to 20 FT. 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 Director of Engineering Division.

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 Bridge Project Manager, 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 10 FT. See SD 2120.3 for further discussion.

For integral abutments, single-line piling systems shall be used, predrilled 15.0 FT deep using 1.0 FT diameter for soil or 2.0 FT 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-DRILLED CAISSONS**

Drilled caissons 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, drilled caissons may eliminate the need of caisson caps, for certain configurations such as single or multiple column piers.

Drilled caissons shall be designed using soil-structure intersection 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, but are not limited to, pre-installation core holes, providing a test hole for shafts 10.0 FT and larger in diameter and cross hole sonic logging (CSL) testing.

Results from the CSL testing may show inadequate structural integrity and continuity. Further investigations such as core drilling for each of the unacceptable caissons would then be required, causing construction down time and added expenses.

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

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**STRUCTURE DIRECTIVE 1050  
ADDITIONAL DESIGN CONSIDERATIONS  
Date of Implementation  
First Edition**

**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 moderately curved bridges, the use of straight steel girders or prestressed concrete beams with spacings satisfying the minimum and maximum deck overhang requirements shall be investigated.

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. General desirable traits include:

- A. Uniform superstructure depth.
- B. Homogenous girder material throughout the structure.
- C. Symmetric span arrangements.
- D. Orientation of substructure units should be aesthetically consistent with the features they cross and the overall bridge layout.

For additional aesthetic requirements, see the Governing Specifications.

**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, 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 WVDOH 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.
- 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. The 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.
- F. Effects on the safety of vehicles crossing the structure, especially where speeds are high.

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

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**STRUCTURE DIRECTIVE 1060**  
**MISCELLANEOUS STRUCTURES**  
**Date of Implementation**  
**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 such as box culverts and culverts of other configurations may be designed as precast or cast-in-place, or the contract plans may leave the option to the contractor.

All cast-in-place culverts 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 16 FT 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 3 FT 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**

**DRAFT**

**STRUCTURE DIRECTIVE 1070  
BRIDGE REHABILITATION  
Date of Implementation  
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

DRAFT

**STRUCTURE DIRECTIVE 1071**  
**BRIDGE INSPECTION**  
**Date of Implementation**  
**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*.

- 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, 2nd edition 2010 (2011, 2013 and 2014 Interims) or current FHWA adopted version.
- B. U. S. Department of Transportation Bridge Inspector's Reference Manual 2002 (2006 and 2012 Revisions)
- C. NBIS.

**DRAFT**

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

<p><b>STRUCTURE DIRECTIVE 1072 BRIDGE WIDENING Date of Implementation First Edition</b></p>
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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**

DRAFT

<p><b>STRUCTURE DIRECTIVE 1073 REHABILITATION TECHNIQUES Date of Implementation First Edition</b></p>
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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. Fracture-critical 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 to aid in arresting further propagation. Dye penetrant 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. Dye penetrant 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.

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 must be taken to improve the expected fatigue life of the detail unless a cumulative damage fatigue analysis yields adequate life. 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.

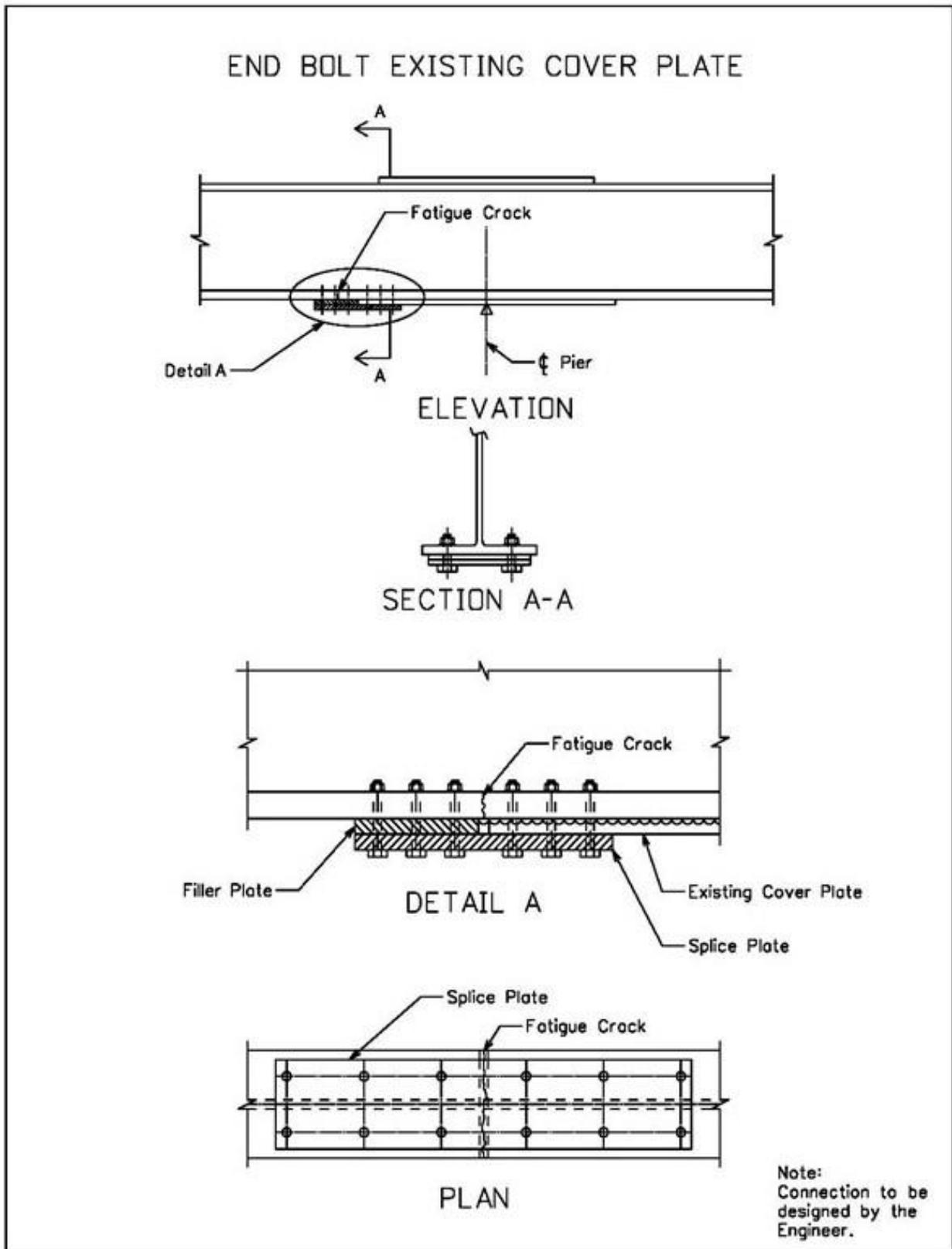
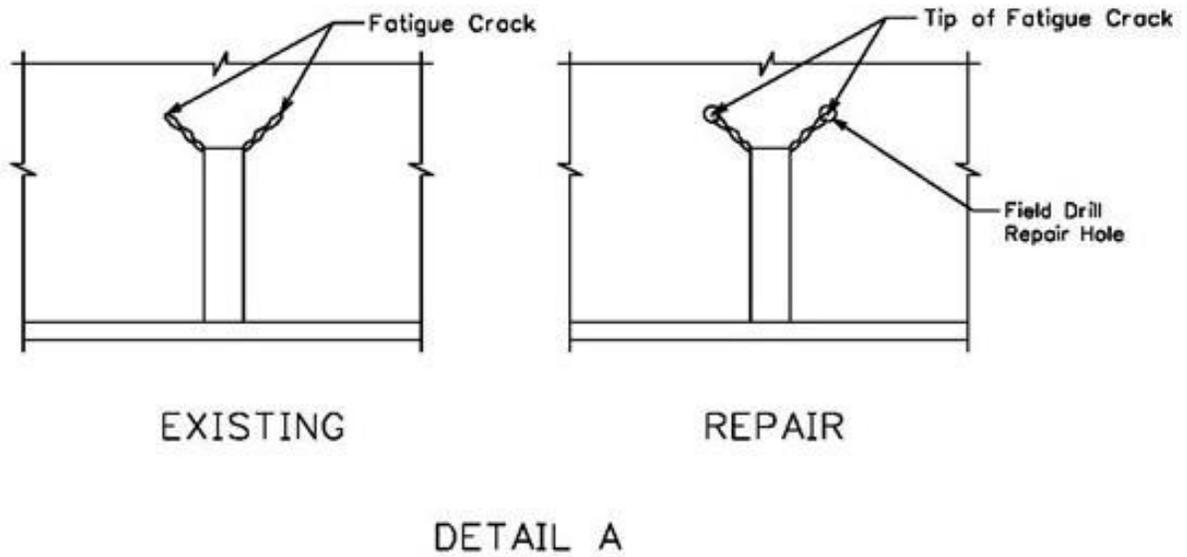
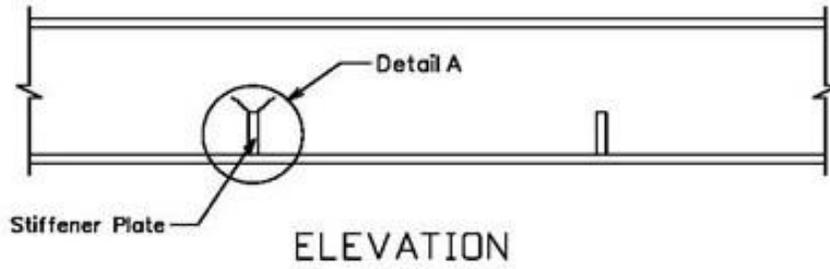


Figure 1073.A

### TYPICAL WEB FATIGUE CRACK REPAIR DETAIL



Note:  
Hole diameter to be  
determined by the  
Engineer.

Figure 1073.B

#### **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 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.

#### **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 removal beyond half the depth of the member, 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 large areas must be repaired and 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 of the Standard 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**

When a specialized concrete overlay (SCO) is used on a deck greater than 7.0 IN thick, the deteriorated concrete shall be removed by rotomilling to 1.0 IN above the rebar followed by hydro-demolition. Conventional concrete removal, such as rotomilling and the use of pavement breakers shall not be utilized for slabs less than 7.0 IN thick. For slabs, 6.5 IN to 7.0 IN 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 IN.

### **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 WVDOH'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. Light rust or surface scaling of non-contact surfaces.
- B. Loss of lubrication.
- C. Debris and dirt accumulation on the bearing seat.
- D. Minor tilting and displacement of bearing components.
- E. Rusted masonry and keeper plates.
- F. Missing nuts or deteriorated anchor bolts.

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

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STRUCTURE DIRECTIVE 1074  
STEEL COATINGS  
Date of Implementation  
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 75 FT 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) (Section 688.2.9 of the Standard Specifications):** 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:** Traffic control: How will the field coating operations affect traffic control?

- A. Public awareness: Is the structure located in an area where color is important to the citizens of the area?
- B. 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.
- C. 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.
- D. 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.
- E. 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.
- F. 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.
- G. 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?
- H. 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.
- I. 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 displeasing 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 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% (5000 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 5000 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 5000 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 WVDODH 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.
- A. 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.
- C. 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.
- D. 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.*
- E. 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.
- F. 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.
- G. 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 Standard Specifications.

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

DRAFT

**STRUCTURE DIRECTIVE 1080  
COAST GUARD BRIDGE PERMITS  
Date of Implementation  
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) 5393900, 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**

**DRAFT**

**STRUCTURE DIRECTIVE 1090  
ALTERNATE BRIDGE DESIGN  
Date of Implementation  
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  
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**STRUCTURE DIRECTIVE 1100**  
**RAILROAD CONSIDERATIONS**  
**Date of Implementation**  
**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.

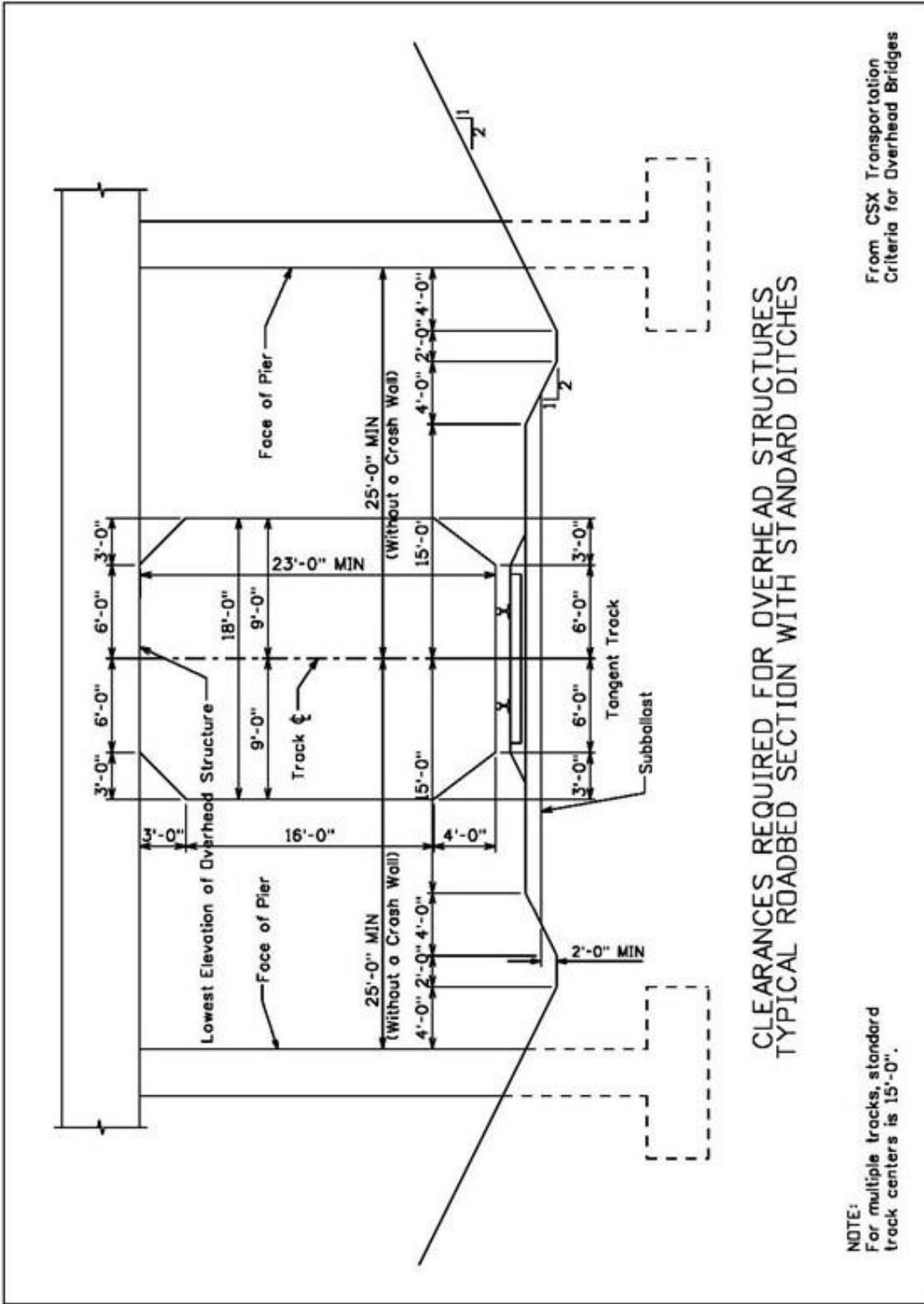
**1100.1-APPROVAL**

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 25 FT 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. Crash walls for piers 12 FT to 25 FT clear from the centerline of track shall have a minimum height of 6 FT (10 FT for Norfolk Southern) above top of rail. Piers less than 12 FT clear from the centerline of track shall have a minimum crash wall height of 12 FT above the top of the rail.

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CLEARANCES REQUIRED FOR OVERHEAD STRUCTURES  
TYPICAL ROADBED SECTION WITH STANDARD DITCHES

NOTE:  
For multiple tracks, standard  
track centers is 15'-0".

From CSX Transportation  
Criteria for Overhead Bridges

### **Figure 1100.A**

The crash wall shall be at least 2.5 FT thick and at least 12 FT 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 4 FT 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 25 FT 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.

### **1100.3-CLEARANCES**

Minimum vertical clearance above the high rail shall be 23 FT. 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 25 FT. The absolute minimum on all new construction is 12 FT. 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 Standard 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.

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**STRUCTURE DIRECTIVE 1110  
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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 bridge. Approval of the Span Arrangement Report on any project shall not limit or preclude the taking of additional borings required by the WVDOH 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. All proposed computer software to be used during the “Combined TS&L” phase and final design phases of the bridge project.
- G. Deck drainage, superstructure joint, and bearing device requirements.
- H. Special environmental, aesthetic, and utility considerations.
- I. A description of the assumed foundation type used for cost estimates and geotechnical data.
- J. A preliminary total structure cost estimate for each span arrangement studied.
- K. Note proposed right of way limits, construction easements, and future maintenance operations.
- L. 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. However, more than two alternates may be advanced.
- M. Unit prices shall be submitted for approval for all pay items that may be used during the plan development process.

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.
- 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, 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.

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 both steel and concrete superstructures 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 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 walls 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, drilled caissons, 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 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 WVDOH for approval prior to submission of the PS&E Plans for comments and approval. At this stage, the plans and design shall be 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 IN x 17 IN 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 IN x 17 IN 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.

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**STRUCTURE DIRECTIVE 2010  
DESIGN CRITERIA  
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**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.

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**STRUCTURE DIRECTIVE 2012**  
**LOADS AND LOAD FACTORS**  
**Date of Implementation**  
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**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. When girder or beam spacing 14 FT 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 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. 6 IN 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 Director of Engineering Division. A "critical" bridge shall be designed based on a 2500-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:  $-30^{\circ}$  to  $120^{\circ}$  F
- B. Concrete:  $0^{\circ}$  to  $80^{\circ}$  F
- C. Wood:  $0^{\circ}$  to  $75^{\circ}$  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:

- $\eta_i$  = The load modifier factor is a function of ductility factor  $\eta_D$ , redundancy factor  $\eta_R$  and importance factor  $\eta_I$
- $\gamma_i$  = 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
- $\phi$  = resistance factor determined per the Governing Specifications

The ductility factor  $\eta_D$ , redundancy factor  $\eta_R$  and importance factor  $\eta_I$  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  $\eta_I$ .
  1.  $\eta_I = 0.95$  for bridges that meet ALL of the following criteria:
    - a) Length less than 50 FT.
    - b) Design ADT less than 500.
    - c) Detour less than 10 miles or capability of erecting a temporary crossing in less than 2 days.
  2.  $\eta_I = 1.05$  for bridges that meet ANY of the following criteria:
    - a) Total bridge construction cost that exceeds 20 million dollars.
    - b) Design ADT greater than 50,000.
    - c) Detour length greater than 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, using computer software that will take into account 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  $10^\circ$ , the bending moments shall be reduced in accordance with AASHTO Table 4.6.2.2e-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.

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**STRUCTURE DIRECTIVE 2013  
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**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 4000 PSI.
- D. Concrete compressive strengths for precast beams shall be no more than 6000 PSI at release ( $f'_{ci}$ ) with a final compressive strength of 8000 PSI ( $f'_c$ ). Precast beams may be designed using high strength concrete with a release strength of up to 9000 PSI and a final strength of up to 10000 PSI with approval of the Director of Engineering Division.
- E. Class B Concrete with a compressive strength of 3000 PSI should be used for abutments, wingwalls and most piers. Class B Modified concrete with a compressive strength of 4000 PSI may be used for piers, if required for strength. Concrete strengths greater than 4000 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.

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**STRUCTURE DIRECTIVE 2014  
CURVED SUPERSTRUCTURE DESIGN CRITERIA  
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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:

<b>Number of Beams</b>	<b>Angle for One Span</b>	<b>Angle for 2 or more Spans</b>
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.

Hybrid sections are not allowed for curved girders.

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**STRUCTURE DIRECTIVE 2014  
CURVED SUPERSTRUCTURE DESIGN CRITERIA  
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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:

<b>Number of Beams</b>	<b>Angle for One Span</b>	<b>Angle for 2 or more Spans</b>
2	2°	3°
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**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.

Hybrid sections are not allowed for curved girders.

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**STRUCTURE DIRECTIVE 2020  
DECKS  
Date of Implementation  
First Edition**

**2020.1-CONCRETE DECKS**

Concrete decks shall be used in all bridge decks.

**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 IN 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 8 FT. 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).

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**STRUCTURE DIRECTIVE 2021  
DESIGN OF CONCRETE DECK SLABS  
Date of Implementation  
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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 8.0 IN.
- B. 1.0 IN clearance at the bottom of the slab.
- C. 2.5 IN clearance at the top of the slab.
- D. 2.0 IN clearance elsewhere.

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

- A. The minimum allowable thickness is 8.5 IN.
- B. 2 IN composite SCO.
- C. 1.0 IN clearance at the bottom of the slab.
- D. 3.0 IN clearance at the top of the slab.
- E. 2.0 IN clearance elsewhere.

The top ¼ IN 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 6 FT.

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 WVDOH Standard 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 2 FT 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 Director of Engineering Division 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 Director of Engineering Division.

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 3500 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 FT.

All bridges with spans greater than 350 FT 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 3.0 IN 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 5 FT or the shoulder width, whichever is greater. If the design speed is less than 40 MPH, spread is generally limited to 5 FT, or the shoulder width plus 2 FT into the traveled way, whichever is greater. If a parking lane is present, then the spread will be limited to 8 FT.

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 8 IN. If a barrier curb is required, the curb height should not be less than 6 IN. 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.

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**STRUCTURE DIRECTIVE 2022**

**BARRIERS**

**Date of Implementation**

**First Edition**

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

- A. 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 3500 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 12 FT 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 IN 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 IN 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 IN 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 IN (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, typically on the wingwalls or approach slabs.

**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.

These provisions are also applicable to bridges designed in accordance with LFD. The following live load provisions will supersede the live load provisions of Article 2.5.2.6.2 when designing bridges in accordance with the AASHTO Standard Specifications for Highway Bridges:

- A. Continuous composite barriers and other structural appurtenances shall be used in the service and fatigue limit states. They shall not be utilized for other load cases without the approval of the Director of Engineering Division.
- B. To ensure satisfactory performance for strength and serviceability criteria, deflection control of bridges will be evaluated during design in accordance with AASHTO 2.5.2.6.2. The principles suggested by AASHTO 2.5.2.6.2 for deflection control and evaluation will be implemented in the design of bridges. Particular emphasis is placed on considering the entire bridge cross section, including the entire width of the roadway and the structurally continuous portions of railings, sidewalks and median barriers, as effective for stiffness of compositely designed structures.
- C. The live load portion of Load Group III of Table 3.22.1A shall be used, including impact.
- D. The live load will be the HS25, 125% of HS20 as specified in Article 3.7.6, applied as specified in Article 3.11 for maximum positive moment.
- E. The provisions for reduction in load intensity of Article 3.12 apply.

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**STRUCTURE DIRECTIVE 2023  
CONSTRUCTION SEQUENCES  
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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.

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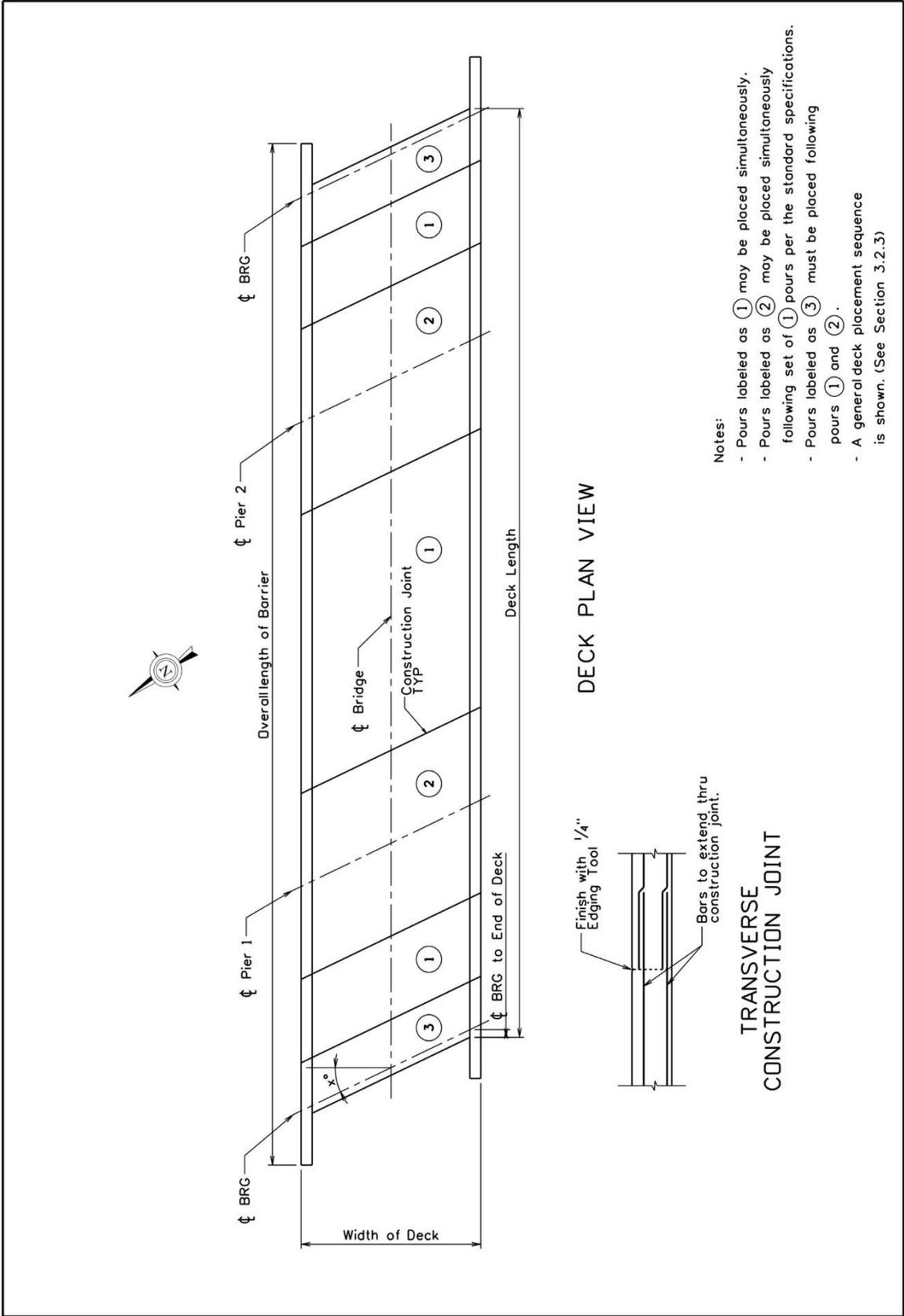


Figure 2023.A

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**STRUCTURE DIRECTIVE 2030  
STEEL SUPERSTRUCTURE  
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The Designer is not limited to the span-to-depth ratios provided by the Governing Specifications. The design of steel bridges will be based on economics and aesthetics.

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

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<p><b>STRUCTURE DIRECTIVE 2031 MATERIAL PROPERTIES Date of Implementation First Edition</b></p>
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**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 Director of Engineering Division.

**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.

Hybrid sections are not allowed for curved girders. Frequently, for curved girders the use of HPS 70W steel in negative flexure regions is used in conjunction with Grade 50 steel in positive flexure regions.

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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.

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<p><b>STRUCTURE DIRECTIVE 2032 DESIGN GUIDELINES Date of Implementation First Edition</b></p>
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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  $\frac{7}{16}$  IN. Increment the web thickness by  $\frac{1}{16}$  IN 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.

### **2032.3-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.

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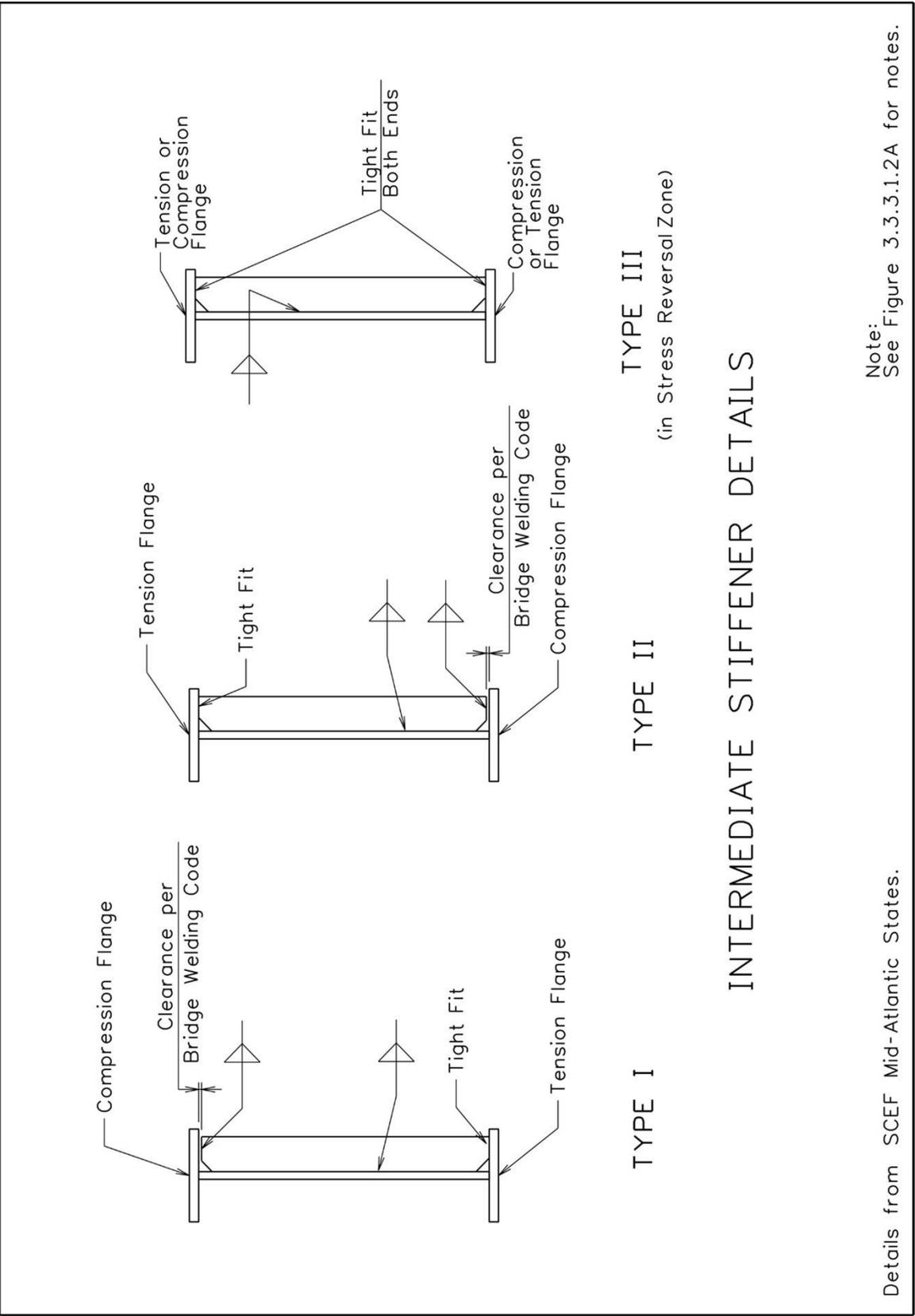
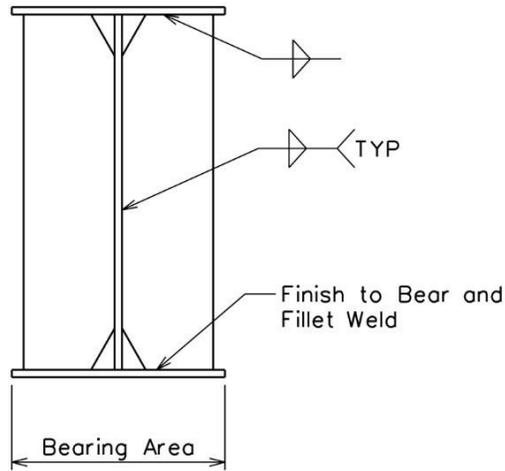


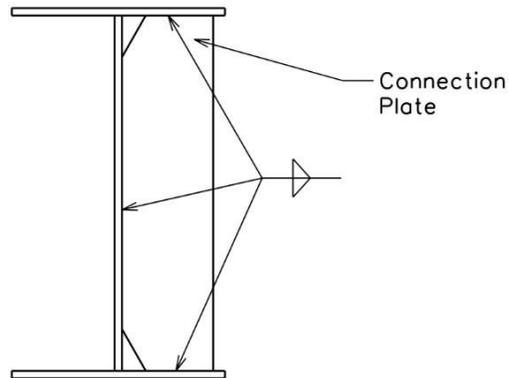
Figure 2032.A

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### STIFFENER DETAILS



### BEARING STIFFENER



### CONNECTION PLATE

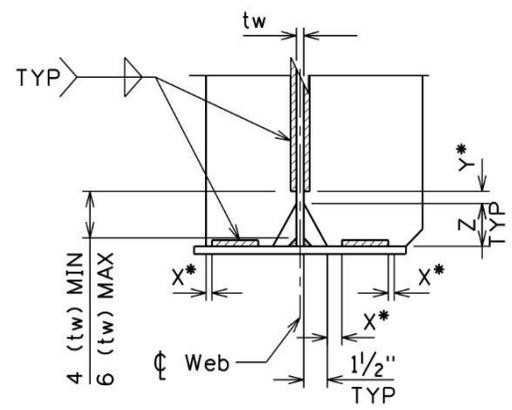
- Notes:
1. Stiffener size must be shown on plans.
  2. Fillet weld size shall be shown on plans unless minimum weld size as per AASHTO/AWS D1.5 is to be used.
  3. If a bearing stiffener is used as a connection plate for cross frames, fillet welds are required.
  4. When longitudinal stiffeners are 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**

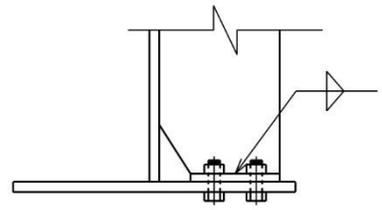
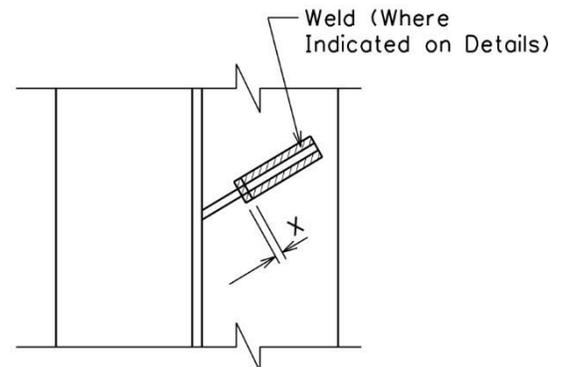
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### STIFFENER DETAILS

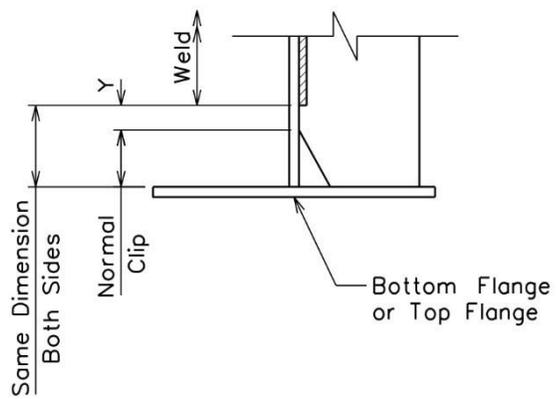


- X = 1/4" ± 1/8"
- Y = 1/2" ± 1/4"
- Z = 2 1/2" for 1/2" web  
3" for 9/16" web  
4" for 3/4" web
- \* 0" for groove weld

#### 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-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.

#### **2032.5-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 Director of Engineering Division.

#### **2032.6-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.

#### **2032.7-FLANGE PLATES**

The minimum flange plate size is  $\frac{3}{4}$  IN thick by 10 IN 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. Roughly, 700 LBS of steel must be saved to offset the cost of a transition butt weld. 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.

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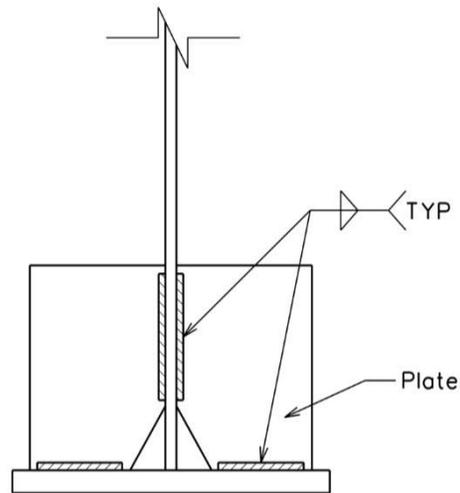
**2032.8-ERECTION REQUIREMENTS**

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.

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## STIFFENER DETAILS



## AUXILIARY BEARING STIFFENER

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

Figure 2032.D

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**STRUCTURE DIRECTIVE 2033  
FATIGUE  
Date of Implementation  
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.

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**STRUCTURE DIRECTIVE 2034  
FATIGUE CRITICAL  
Date of Implementation  
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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 fracture critical members (FCM) is unavoidable and approved by the WVDOH, the FCM 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: 2002, *Bridge Welding Code*, (Bridge Welding Code).

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**STRUCTURE DIRECTIVE 2035**  
**DIAPHRAGMS, CROSSFRAMES AND LATERAL BRACING**  
**Date of Implementation**  
**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 IN.

Cross frames and diaphragms shall be oriented perpendicular to the girder webs (radial for curved bridges). 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.

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<p><b>STRUCTURE DIRECTIVE 2036 CONNECTION DESIGN Date of Implementation First Edition</b></p>
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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  $\frac{1}{4}$  IN for materials up to and including  $\frac{3}{4}$  IN thick, and  $\frac{5}{16}$  IN for materials over  $\frac{3}{4}$  IN. Fillet weld sizes should generally be limited to a maximum of  $\frac{5}{16}$  IN 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  $\frac{7}{8}$  IN 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.

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- 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.

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<p><b>STRUCTURE DIRECTIVE 2037 NON-DESTRUCTIVE TESTING (NDT) REQUIREMENTS Date of Implementation First Edition</b></p>
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**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.

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**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
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<p><b>STRUCTURE DIRECTIVE 2038 COMPOSITE CONSTRUCTION Date of Implementation First Edition</b></p>
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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. 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 7/8 IN 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  
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<p><b>STRUCTURE DIRECTIVE 2039 WEATHERING STEEL Date of Implementation First Edition</b></p>
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**2039.1-GENERAL GUIDELINES**

Weathering steel for bridge components is available as AASHTO M 270 Grade 50W and HPS 70W in plates up to 4 IN thick, 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 FT 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 10 FT over standing water and 8 FT 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.

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**STRUCTURE DIRECTIVE 2040  
STEEL CURVED GIRDER BRIDGES  
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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  
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**STRUCTURE DIRECTIVE 2045  
CONCRETE SUPERSTRUCTURES  
Date of Implementation  
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Concrete compressive strengths for precast beams shall be no more than 6000 PSI at release ( $f'_{ci}$ ) with a final compressive strength of 8000 PSI ( $f'_c$ ). Precast beams may be designed using high strength concrete with a release strength of up to 9000 PSI and a final strength of up to 10000 PSI.

AASHTO girders shall be designed utilizing straight or straight and draped prestressing strands. These strands shall be AASHTO M 203, Grade 270, 0.5 IN or 0.5 IN special, seven-wire, low-relaxation strands. For high performance concrete, 0.6 IN strands may be used for economy. Strand properties are shown in Table 2045.A.

<b>Diameter</b>	<b>Area</b>	<b>Ultimate Strength</b>	<b>Applied Prestressing</b>
0.5 IN	0.153 IN <sup>2</sup>	41.3 KIPS/strand	31.0 KIPS/strand
0.5 IN (Special)	0.167 IN <sup>2</sup>	45.1 KIPS/strand	33.8 KIPS/strand
0.6 IN	0.217 IN <sup>2</sup>	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 FT to 30 FT range.

All reinforcing bars are to be tied at all intersections except where spacing is less than 12 IN 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 girder spans shall be designed for the dead and live loads carried by the composite action of the slab and girders. Multi-span girders shall be designed as continuous for live load purposes.

In a situation where two girders 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. This makes fabrication easier and reduces confusion during construction.

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**STRUCTURE DIRECTIVE 2047  
PRESTRESSED CONCRETE BEAMS  
Date of Implementation  
First Edition**

**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.

Grouping beam designs will help maximize casting beds and minimize variations in material and strand patterns.

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 3 IN, 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 25% of the strands may be debonded to satisfy the allowable stress limits, but debonded strands shall not exceed 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 IN maximum (AASHTO 5.10.10.2).

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

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<p><b>STRUCTURE DIRECTIVE 2048 ADJACENT BOX BEAMS Date of Implementation First Edition</b></p>
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Prestressed box beam designs for spans ranging from 20 FT to 94 FT are included in the Standard Bridge Plans by the WVDOH. Design and Assembly Details and Design Tables are included for box beams ranging in size from 17 IN to 42 IN in height. Also included in the Standard Bridge Plans are Design and Assembly Details (BR-B100 thru BR-B104) 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 12 IN plank beams for spans ranging from 10 FT to 22 FT. As of this printing, the box beams and plank beams in the Standard Bridge Plans have been designed using the AASHTO Standard Specifications (LFD) and are to be used only with the permission of the Bridge Project Manager.

Adjacent box beams are to be placed so that a  $\frac{3}{4}$  IN 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 BR-B103 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 posttensioned using 1 IN 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" and "Approval Signature" 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. A commitment has been made that there will be no alteration to the standard sheets when they are incorporated into plans for specific bridges. The WVDOH has also implemented a statewide purchasing contract that will be utilized for the purchase of beams and slabs for bridges that will be constructed by state forces. This contract will be in effect for one year or longer and the contract prices are based on supplying beams and slabs per the design and details contained in the standard plans.

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 absolutely necessary in order to adapt the plans to some unusual situation, then approval to alter the plan sheets must be obtained from the Director of Engineering Division. Exception to this is permitted where the design must be

modified in accordance with SD 2048.3, Odd Length Beams, to accommodate odd span lengths not listed on the standard plan sheets.

### **2048.2-DESIGN LOADING**

The beams shown on the standard sheets were designed using the LFD Methodology. Any designs using standard sheets shall be checked against LRFD loading.

The design assumes a two-lane structure having a deck width of 33.67 FT out to out. The design loading is AASHTO HS-25 plus 50 PSF of wearing surface, 30 PLF for each guardrail barrier and a diaphragm load based upon the number required. Any increase in these loading conditions, such as additional wearing surface, may cause an overstressed condition in the beam at service loading. Dimensioning or detailing that would cause loading to exceed the above design loading will not be permitted.

### **2048.3-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 check the design for span lengths other than those shown on the standard plan design data chart. It is requested that Engineering Division check designs prepared by consulting engineering firms or by district offices prior to extensive plan development. It is also requested that district offices include span length information in TS&L submittals so that the design can be checked as part of the TS&L approval. If changes are required from the standard design data and strand pattern, the Engineering Division will provide the necessary data. For projects designed by district offices, the Engineering Division will provide the data with the approved TS&L. The Designer should then add the revised data to the design data table of the particular beam sheet. Existing data should not be erased, but new data should be entered into a blank column in the table. If a revised strand pattern is required, the necessary revisions are to be made to the "Typical Beam Reinforcing Section".

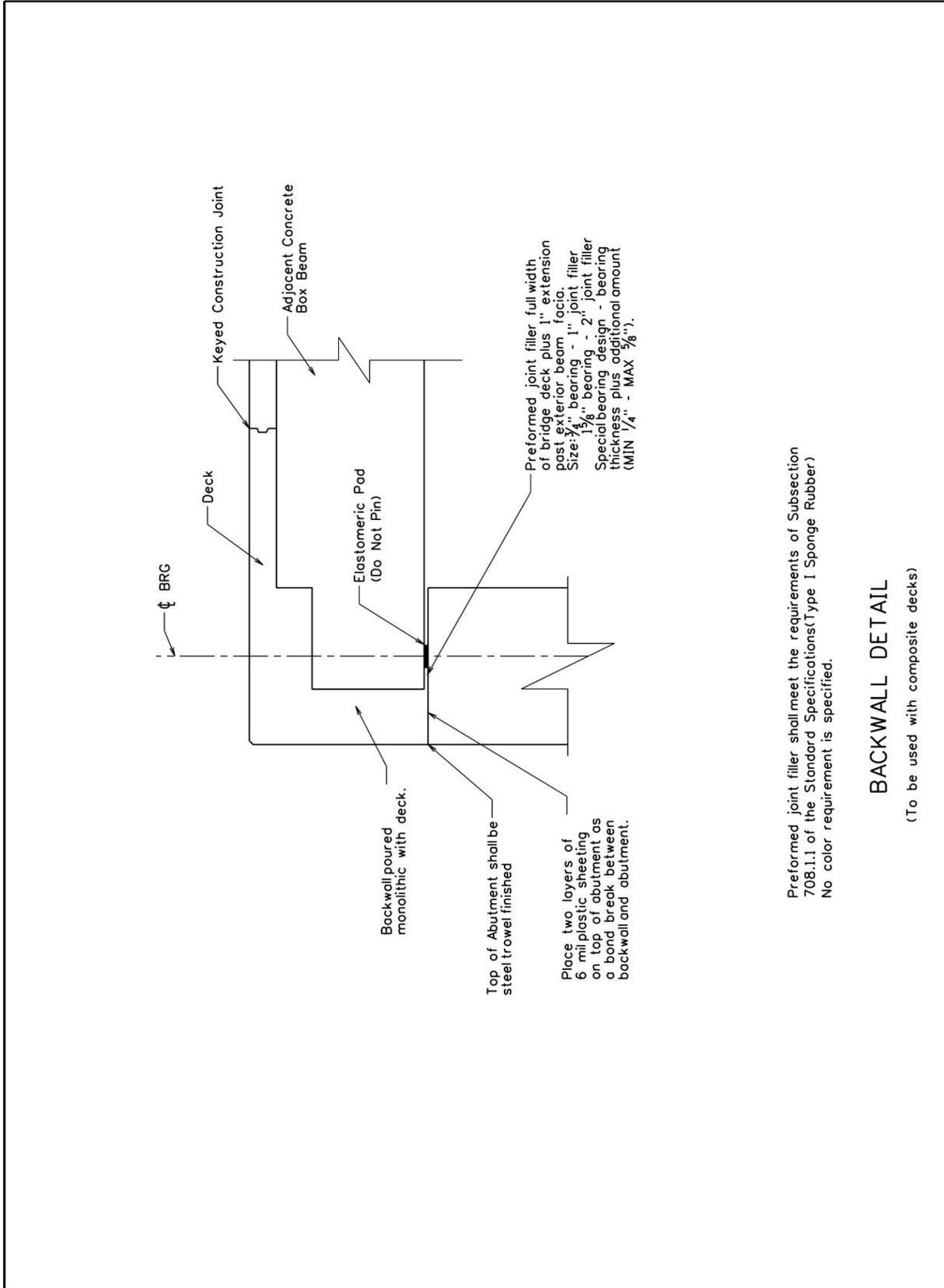
### **2048.4 - Additional Design Information**

A bituminous concrete wearing surface is appropriate on bridges with a low ADT (average daily traffic). On higher ADT bridges, a concrete deck, reinforced with a single mat of reinforcing steel, should be incorporated into the design. 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.

Adjacent box or plank beam bridges without composite decks will generally not be approved for situations where extremely high heavy truck traffic or heavy load service is anticipated. This commonly occurs on haul roads, due to the failure of longitudinal shear keys on bridges built in accordance with the WVDOH Standard Plans. This matter should be addressed during the early planning and TS&L stages of plan development.

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 Plan Sheet BR-B100. If a composite deck with backwall is used, see Figure 2048.A.

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 Plan Sheet BR-B100. If a composite deck with backwall is used, see Figure 2048.A.



Preformed joint filler shall meet the requirements of Subsection 708.1.1 of the Standard Specifications (Type I Sponge Rubber). No color requirement is specified.

**BACKWALL DETAIL**

(To be used with composite decks)

Figure 2048.A

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

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**STRUCTURE DIRECTIVE 2049  
SEGMENTAL CONCRETE STRUCTURES  
Date of Implementation  
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 FT and 180 FT.
- 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 FT to 450 FT for precast construction. For a span range of 450 FT to 700 FT, cast-in-place construction with a form traveler can be used. For spans greater than 600 FT, 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  $\frac{1}{4}$  IN 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 FT.

**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 6500 PSI
- B. Precast segmental superstructure (including CIP joints) 6500 PSI

Clear concrete cover for the superstructure shall be as follows:

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

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 FT.
- D. Anchor set 0.375 IN.

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 FT. |
| D. Anchor set           | 0.063 IN.      |

**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  $12^\circ$  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**

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**STRUCTURE DIRECTIVE 2050  
MISCELLANEOUS SHAPES/TYPES  
Date of Implementation  
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

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**STRUCTURE DIRECTIVE 2055**  
**TIMBER SUPERSTRUCTURES**  
**Date of Implementation**  
**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 100 FT 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 IN to 2 IN 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 20 FT to 100 FT (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 35 FT. 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**

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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 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.

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

**STRUCTURE DIRECTIVE 2060  
BEARINGS  
Date of Implementation  
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.

Where the potential for slip exists between an elastomeric bearing pad and the beam seat, use epoxy grit to increase the coefficient of friction between the surfaces in contact.

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  $\frac{1}{2}$  IN.

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.

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

**STRUCTURE DIRECTIVE 2070  
EXPANSION DEVICES  
Date of Implementation  
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 Inches:** When appropriate, structures with a total thermal movement of 2 IN or less shall be designed in accordance with SD 2090.1, Integral Abutments.

**2070.1.2-Thermal Movement From Two to Four Inches:** When semi-integral abutments cannot be used, strip seal expansion devices shall be used for a thermal movement range of 2 IN to 4 IN. 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 Inches:** Finger plate expansion devices with a drainage trough may be utilized for thermal movements greater than 4 IN. The minimum thickness of a finger plate shall be 2 IN. 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.

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

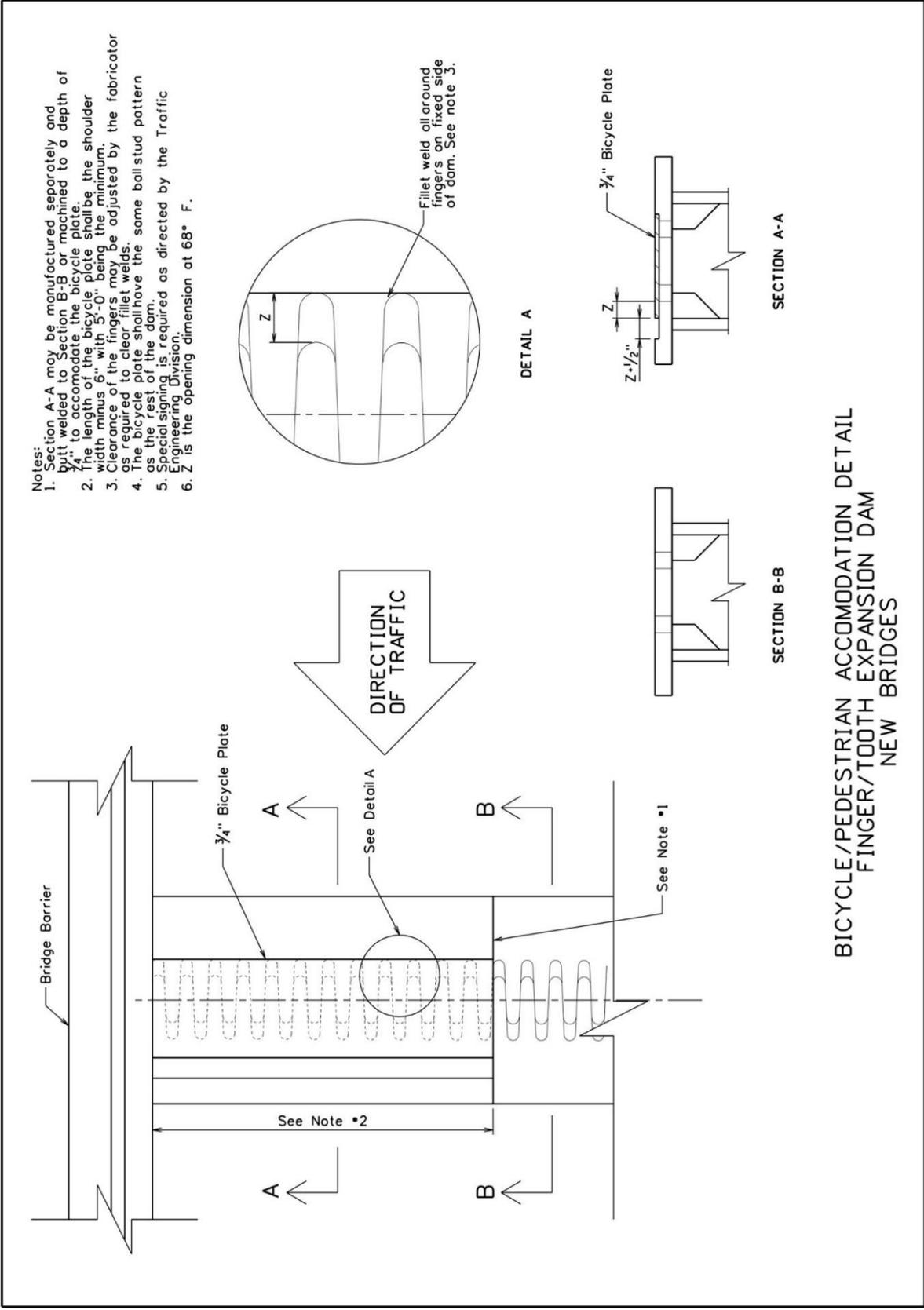
**2070.2-LONGITUDINAL JOINTS:**

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.

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- Notes:
1. Section A-A may be manufactured separately and butt welded to Section B-B or machined to a depth of  $\frac{3}{4}$ " to accommodate the bicycle plate.
  2. The length of the bicycle plate shall be the shoulder width minus 6" with 5'-0" being the minimum.
  3. Clearance of the fingers may be adjusted by the fabricator as required to clear fillet welds.
  4. The bicycle plate shall have the same boltstud pattern as the rest of the dam.
  5. Special signing is required as directed by the Traffic Engineering Division.
  6. Z is the opening dimension at 68° F.

Figure 2070.A

BICYCLE/PEDESTRIAN ACCOMMODATION DETAIL  
FINGER/TOOTH EXPANSION DAM  
NEW BRIDGES

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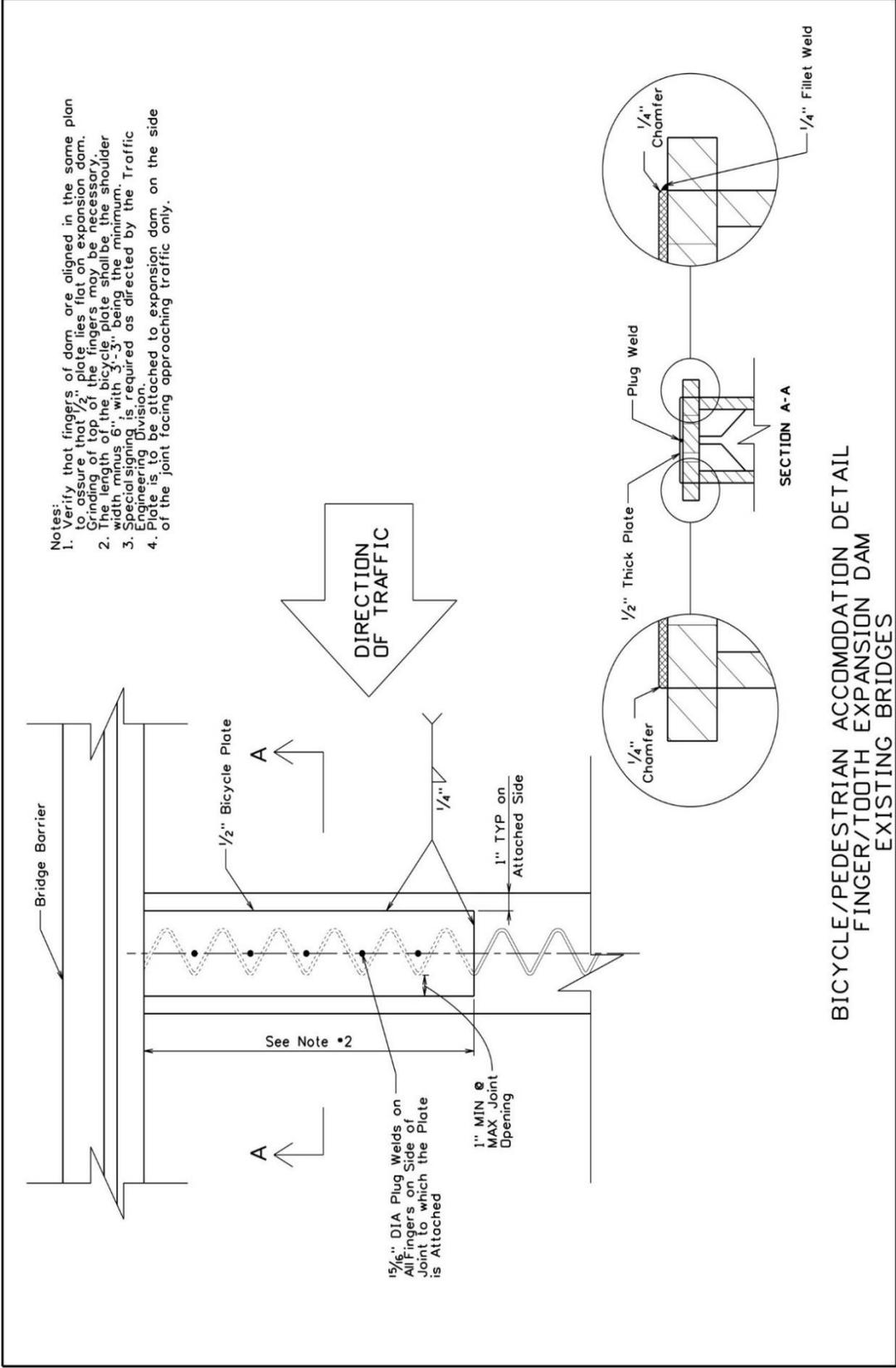


Figure 2070.B

BICYCLE/PEDESTRIAN ACCOMMODATION DETAIL  
FINGER/TOOTH EXPANSION DAM  
EXISTING BRIDGES

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

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<p><b>STRUCTURE DIRECTIVE 2080 ABUTMENTS Date of Implementation First Edition</b></p>
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**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 3000 PSI should be used in most abutments. The minimum reinforcing throughout the abutment and wingwalls shall be No. 5 bars spaced at 1 FT. 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.

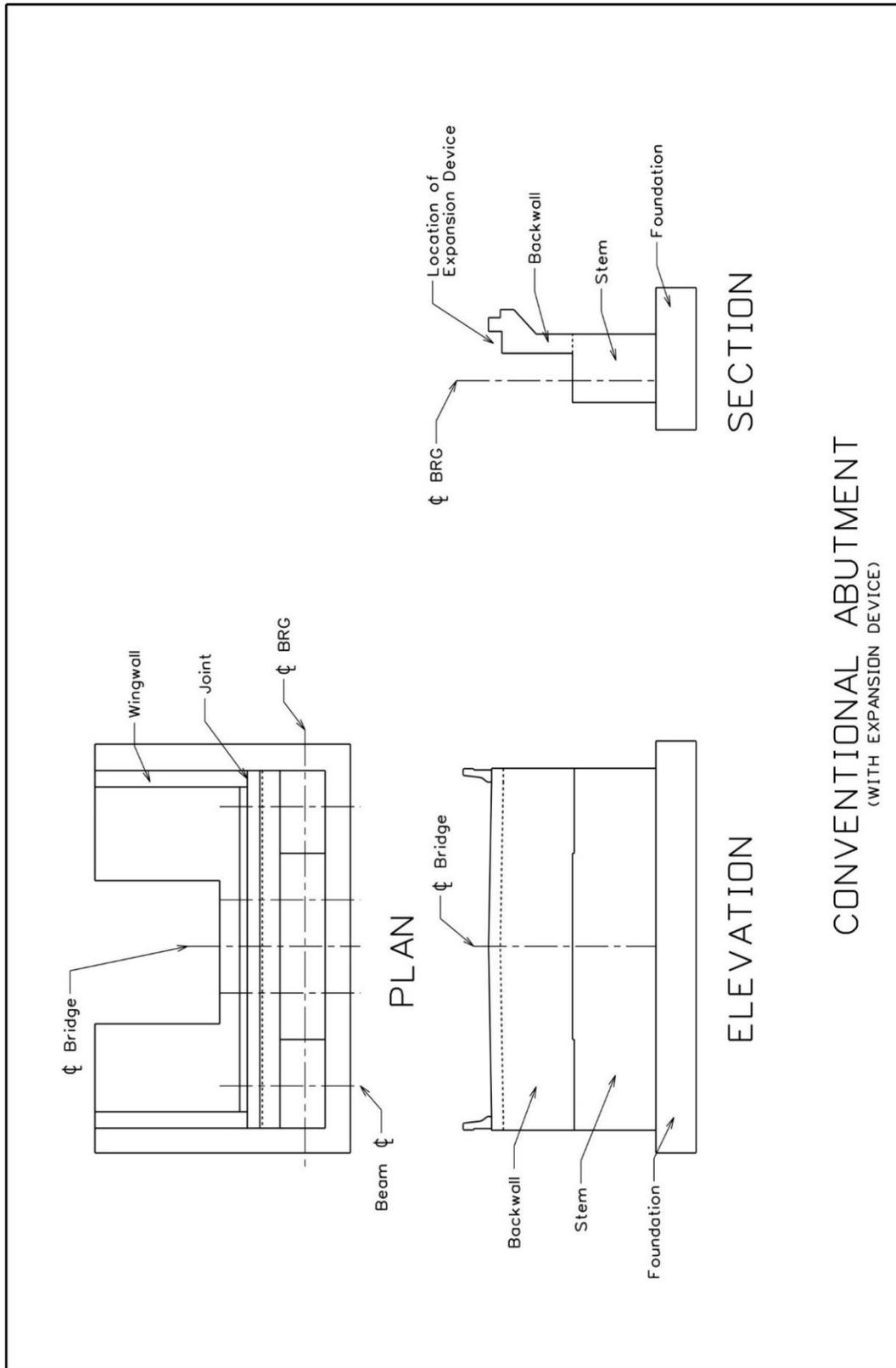


Figure 2080.A

CONVENTIONAL ABUTMENT  
(WITH EXPANSION DEVICE)

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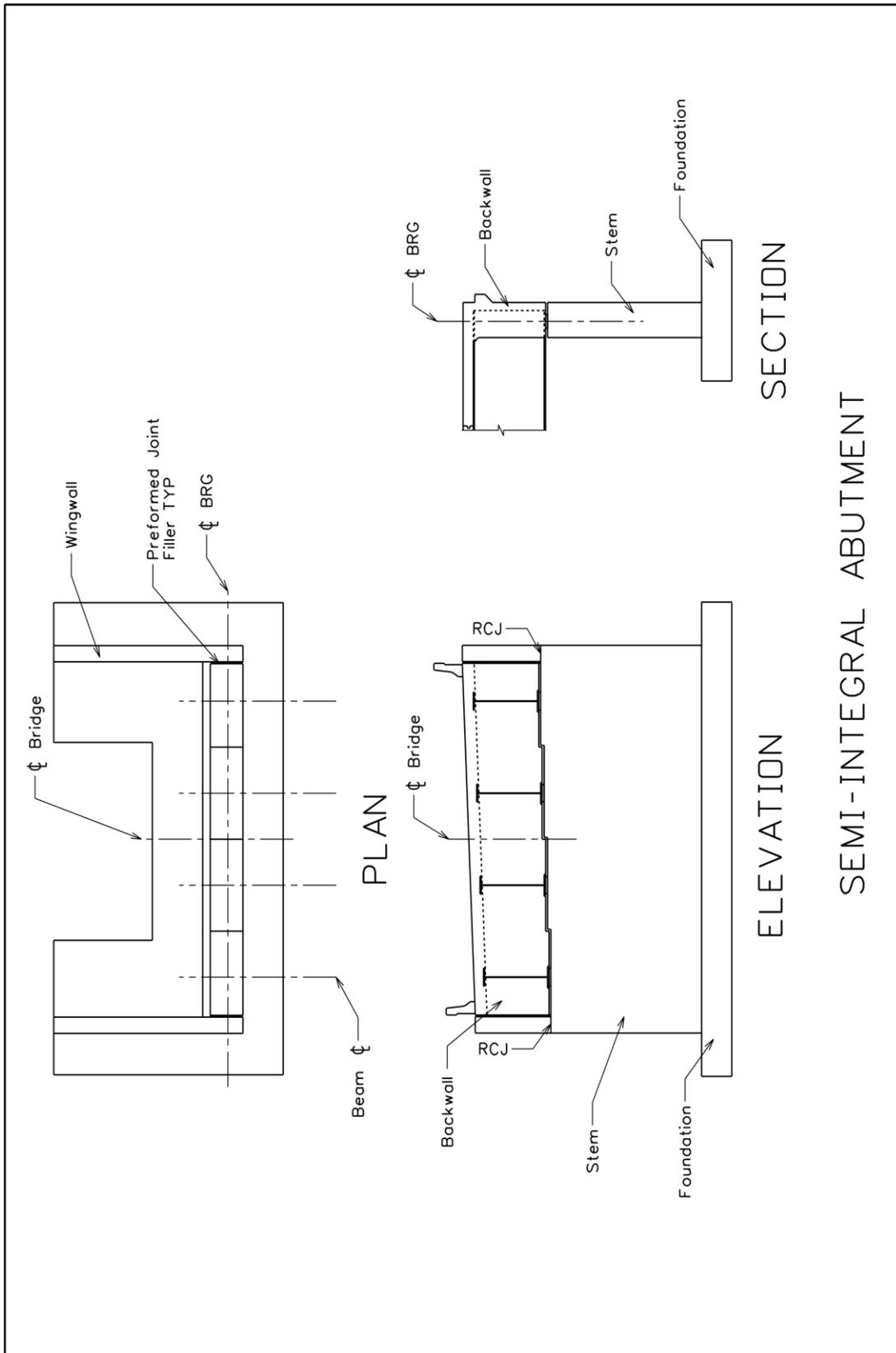


Figure 2080.B

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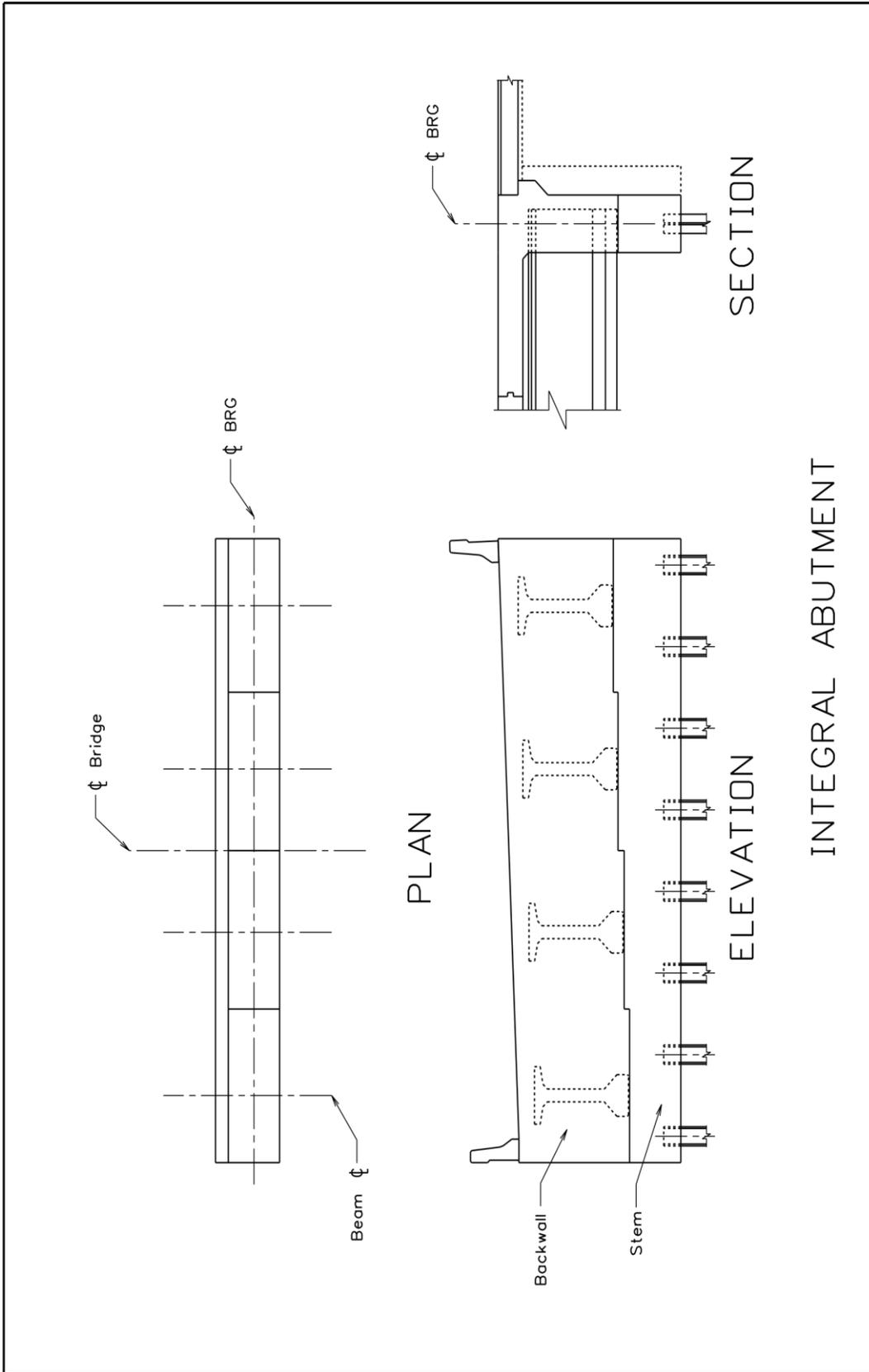


Figure 2080.C

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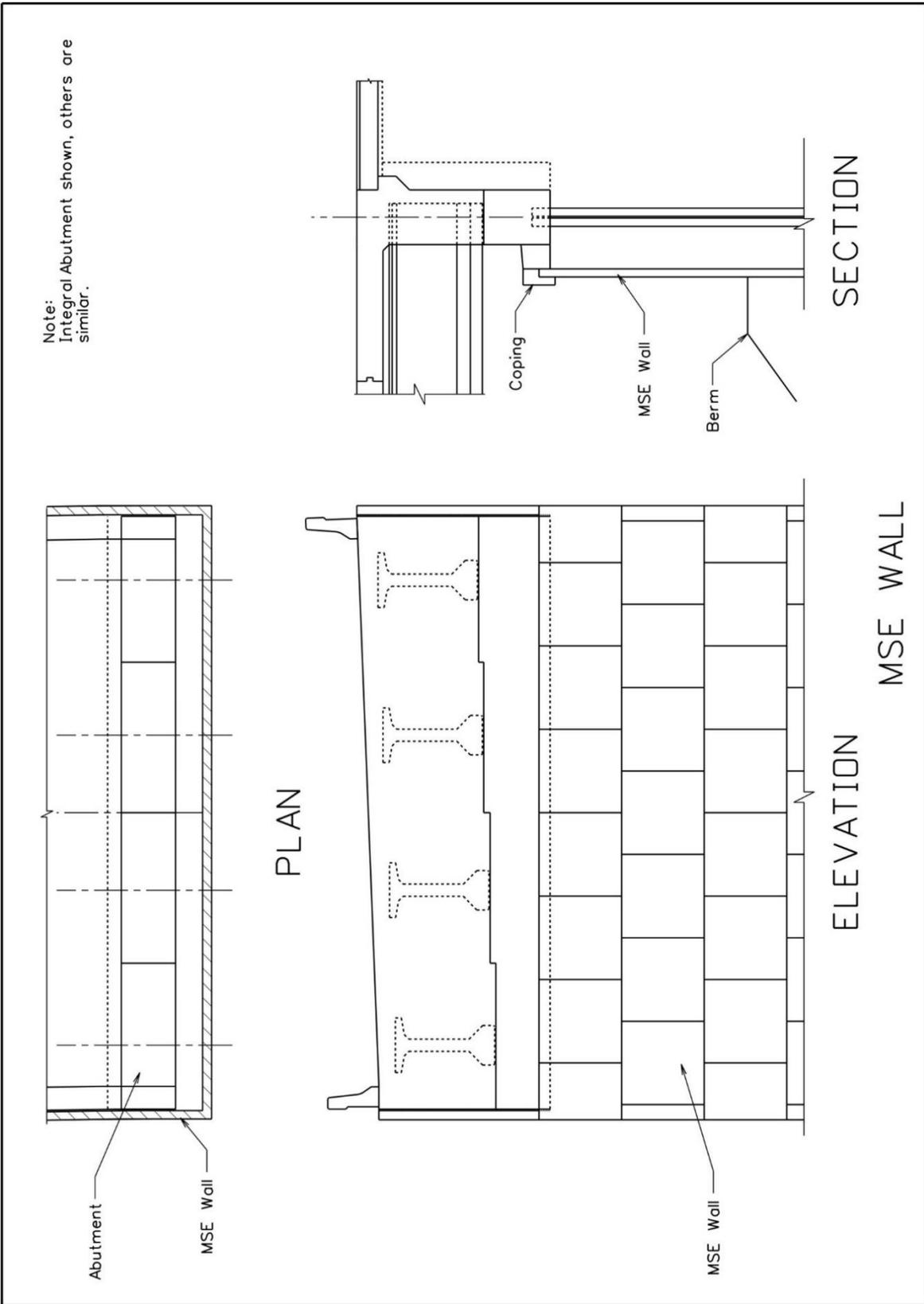


Figure 2080.D

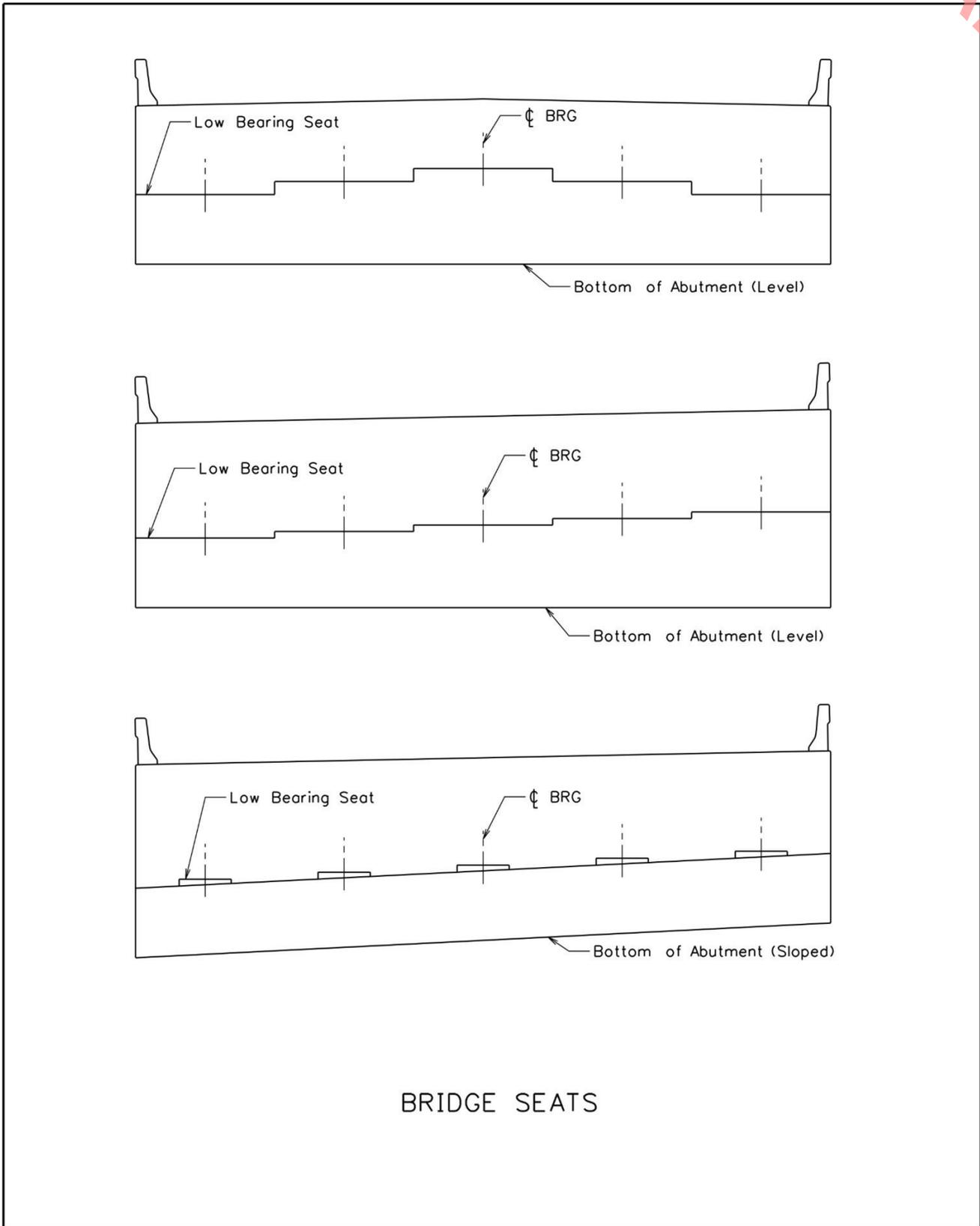


Figure 2080.E

When the beam longitudinal grades are less than 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 4 IN or more or when the standard reinforcing is not sufficient to prevent cracking or spalling.

### **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 1 FT. 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 1 FT 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

Wingwalls where  $q > 90^\circ$  (right wingwall in Figure 3.8.5):

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

Wingwalls where  $q \leq 90^\circ$  (left wingwall in Figure 3.8.5):

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

Wingwalls shall be designed as horizontal cantilevers when supported by the abutment or as vertical cantilevers if they are supported by a foundation. The minimum thickness of a wingwall supporting a barrier shall be the base thickness of the barrier in question and designed for strength; otherwise, it shall be 1 FT.

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.

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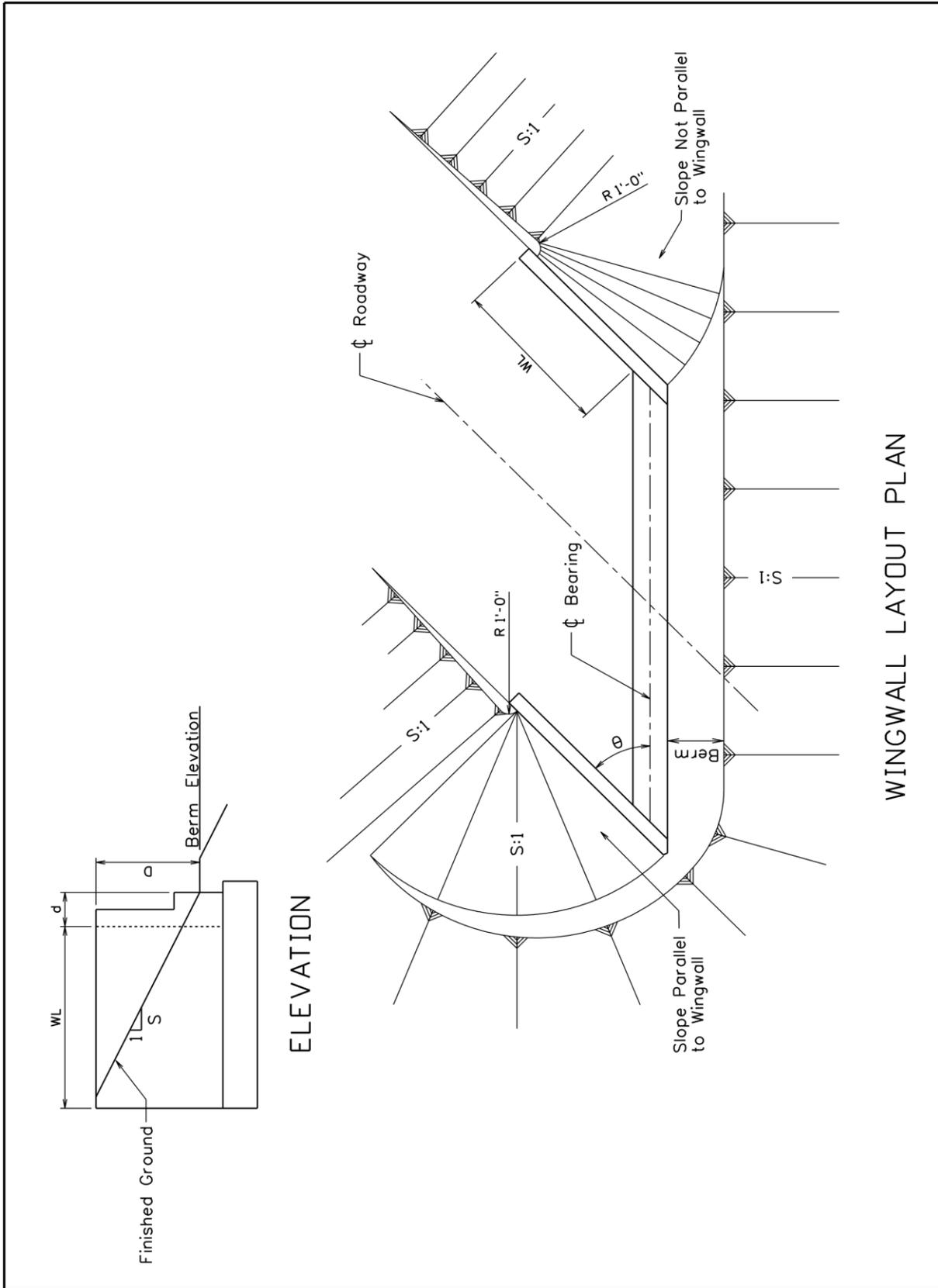


Figure 2080.F

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

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**STRUCTURE DIRECTIVE 2090  
JOINTLESS BRIDGE ABUTMENTS  
Date of Implementation  
First Edition**

Fully integral and semi-integral abutments shall be used whenever possible to eliminate deck expansion joints. See Figures 2090.A thru 2090.E. Integral abutments shall be used when the anticipated movement is 2 IN or less and the skew is 30° or less. If the grade exceeds 5%, the lower grade abutment for a single span bridge shall be fixed and for a multi-span configuration, the pier(s) or lower abutment shall be fixed. Semi-integral abutments may be used for instances that are not appropriate for integral abutments. Both types are assumed to be pinned and shall be designed in accordance with the following guidelines.

**2090.1-INTEGRAL ABUTMENTS**

Approach slabs are required for all integral abutments having a total anticipated thermal movement exceeding ½ IN, or those conditions described in SD 2140. The approach slab shall be anchored to the abutment by reinforcing steel bars. The approach slab shall not be anchored to the wingwalls and to reduce friction, filter fabric shall be placed over the base course prior to placing the approach slab.

Provide expansion joints for utilities, sidewalks, concrete barriers, guardrail and other roadway features that pass over integral abutments onto the approach roadway.

Construct a Type H joint (Standard Detail Sheet PVT2) between the approach slab and pavement to accommodate thermal movement when using flexible approach pavement. Rigid approach pavements require a Type B joint (Standard Sheet PVT1) between the approach slab pavement for movements up to ¼ IN and a Type J joint (Standard Sheet PVT5) for movements greater than ¼ IN.

To reduce cracking in integral abutments, a closure pour consisting of the backwall and an adjacent “X” FT minimum of deck shall not be placed until all other deck pours have been placed. The distance “X” is equal to 0.5 FT + the effective slab length (distance between beam quarter points, in feet) measured from the front face of the abutment.

To reduce the effects of passive earth pressure, use loose (non-compacted) select material for backfilling when thermal movements at integral abutments exceed ½ IN. The design must provide for adequate drainage of the backfill.

Abutment diaphragms or cross frames are not required for superstructures with integral abutments. If required for construction stability, temporary bracing may be placed adjacent to the abutment and removed after the concrete has cured. Sufficient clearance between the abutment and temporary bracing shall be maintained to provide adequate room for the construction of the abutment backwall. Bearing stiffeners are required on steel superstructures.

The beam seat shall be sloped parallel to the beam grade for integral abutments.

Integral abutments shall be designed using a single row of piling. Wingwalls requiring more support than that available from the integral abutment shall be structurally isolated.

The following parameters apply to integral abutments:

- A. Piling shall be a single row and aligned so that the webs are parallel to the centerline of bearing.
- B. Piling shall be embedded into the abutment at least 1.0 FT unless the analysis requires more.
- C. The distance from the side of any pile to the nearest edge of the abutment shall be greater than 9 IN.
- D. Piling lengths of 10 FT (minimum) to 15 FT shall be predrilled to the top of rock. Piling lengths greater than 15 FT shall be predrilled a minimum of 15 FT. Pre-drilling is in accordance with Section 616 of the Standard Specifications. Pile points are permitted to facilitate pile driving but are not considered a substitute for pre-drilling integral abutment piling.
- E. Wingwalls supported by the abutment shall be limited to 6 FT for straight wings and 12 FT for U-wings.

#### **2090.2-SEMI-INTEGRAL ABUTMENTS**

Semi-integral abutments may be used where foundation sites rule out the use of an abutment on a single row of piles, while retaining full integrity with the superstructure.

Consideration must be given to the following:

- A. When full height U-shaped wingwalls are used, provisions shall be made to allow for thermal expansion of the superstructure without interference from the wingwalls.
- B. The Designer must account for these items:
  1. Uplift resulting from the span arrangement.
  2. Buoyancy.
  3. Excessive grade; greater than 5%.
  4. Potential roadway settlement.
- C. Seal between the abutment seat and cap to retain the backfill and for waterproofing. Add a full-length curb to the top of the semi-integral stem to help retain the backfill when the bearing height exceeds 1 ½ IN. See Figure 2090.E.

When the beam longitudinal grade is less than 2%, the beam seat shall be sloped parallel to the beam grade. Otherwise, the beam seats shall be level to true elevation.

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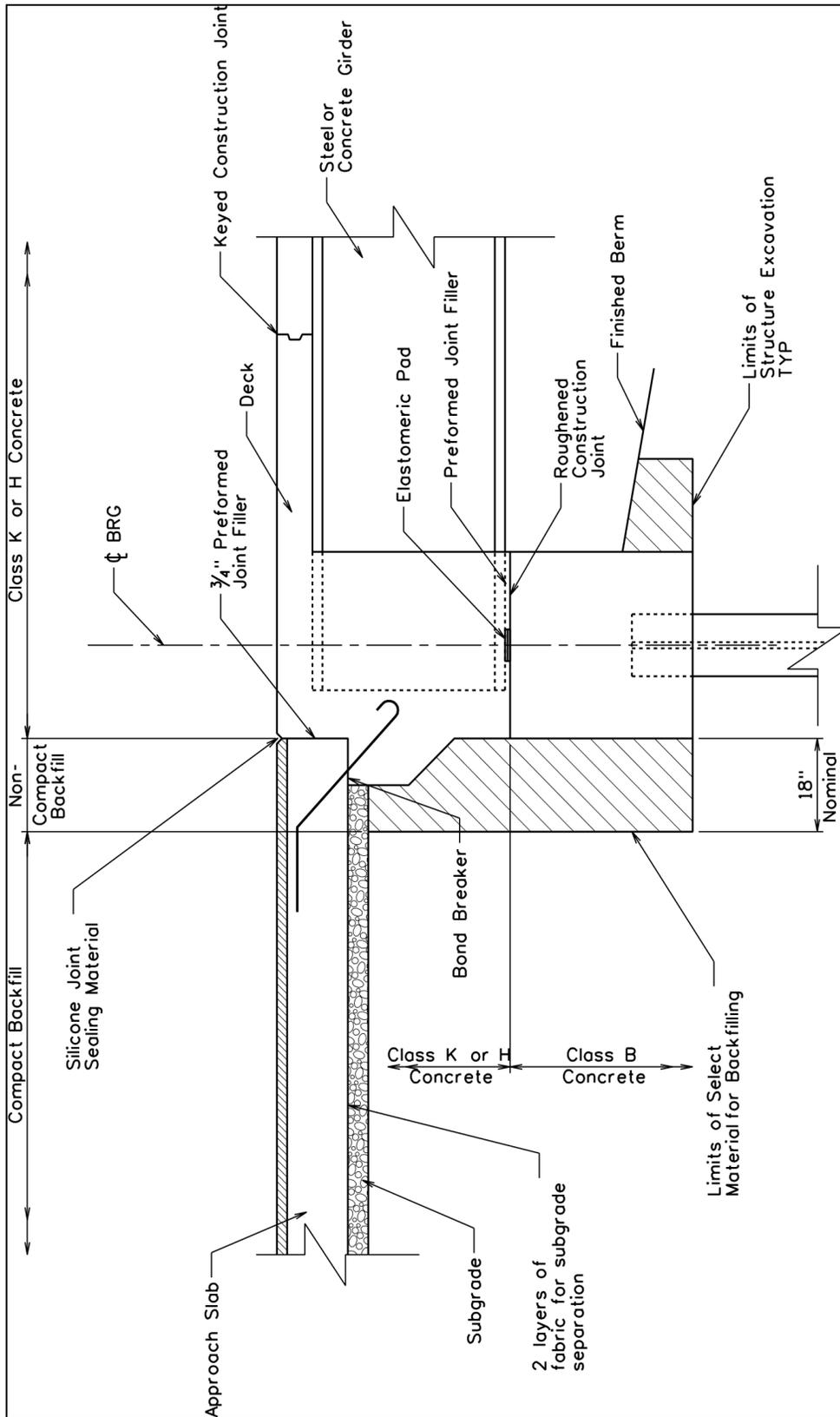


Figure 2090.A

INTEGRAL ABUTMENT FOR  
DECK-GIRDER SYSTEM  
(W/ APPROACH SLAB)

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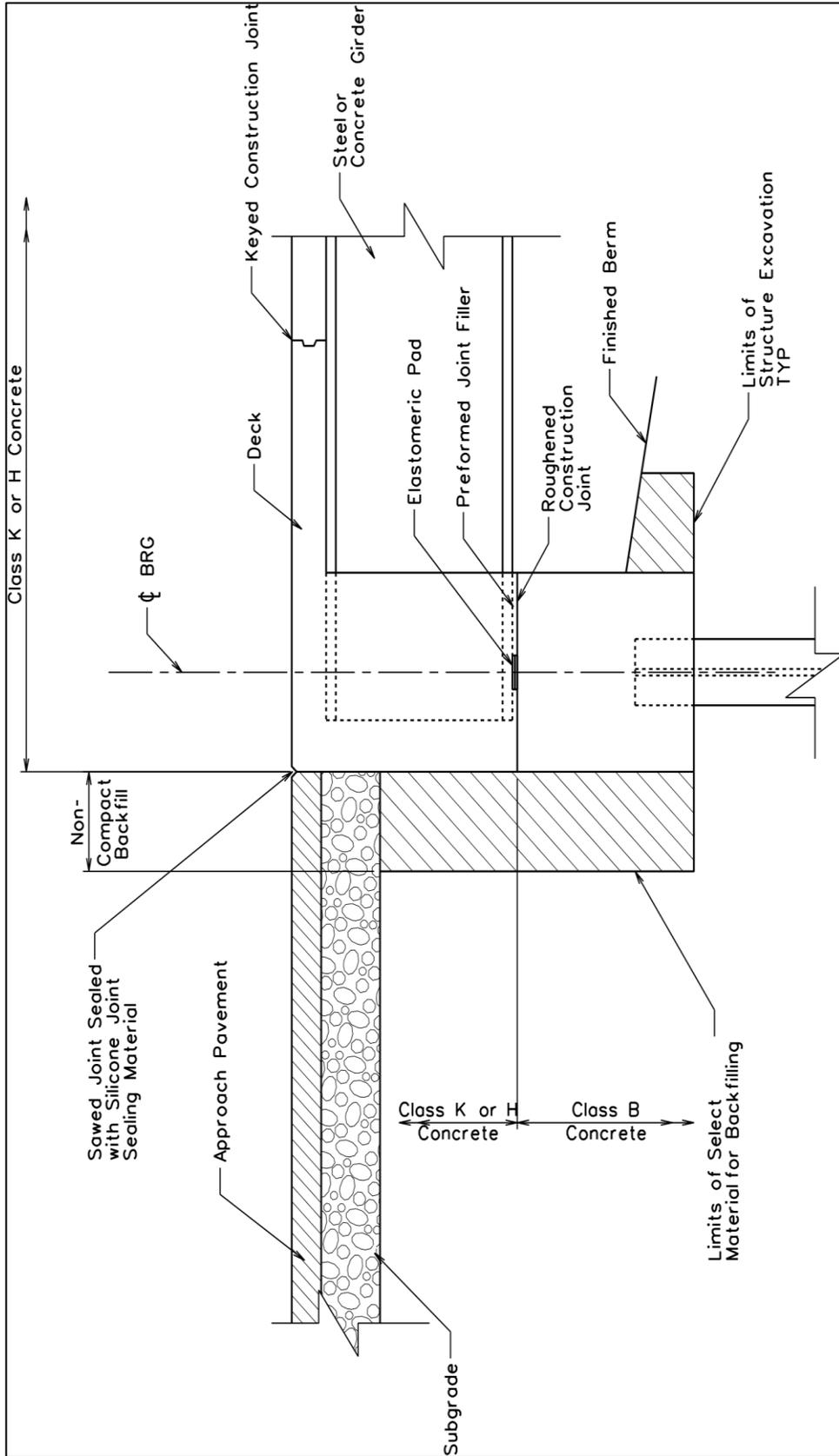
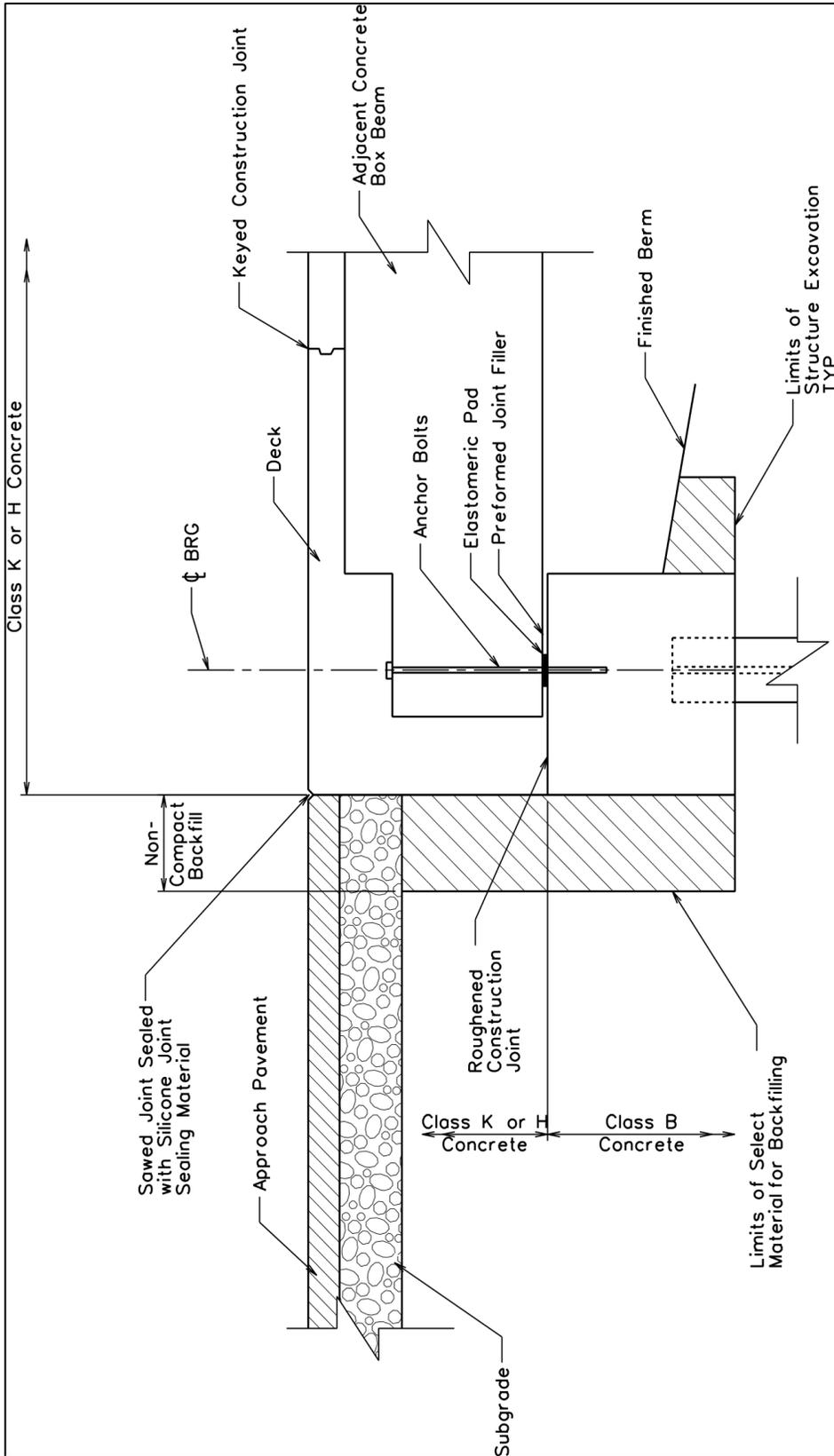


Figure 2090.B

INTEGRAL ABUTMENT FOR  
DECK-GIRDER SYSTEM  
(WITH-OUT APPROACH SLAB)

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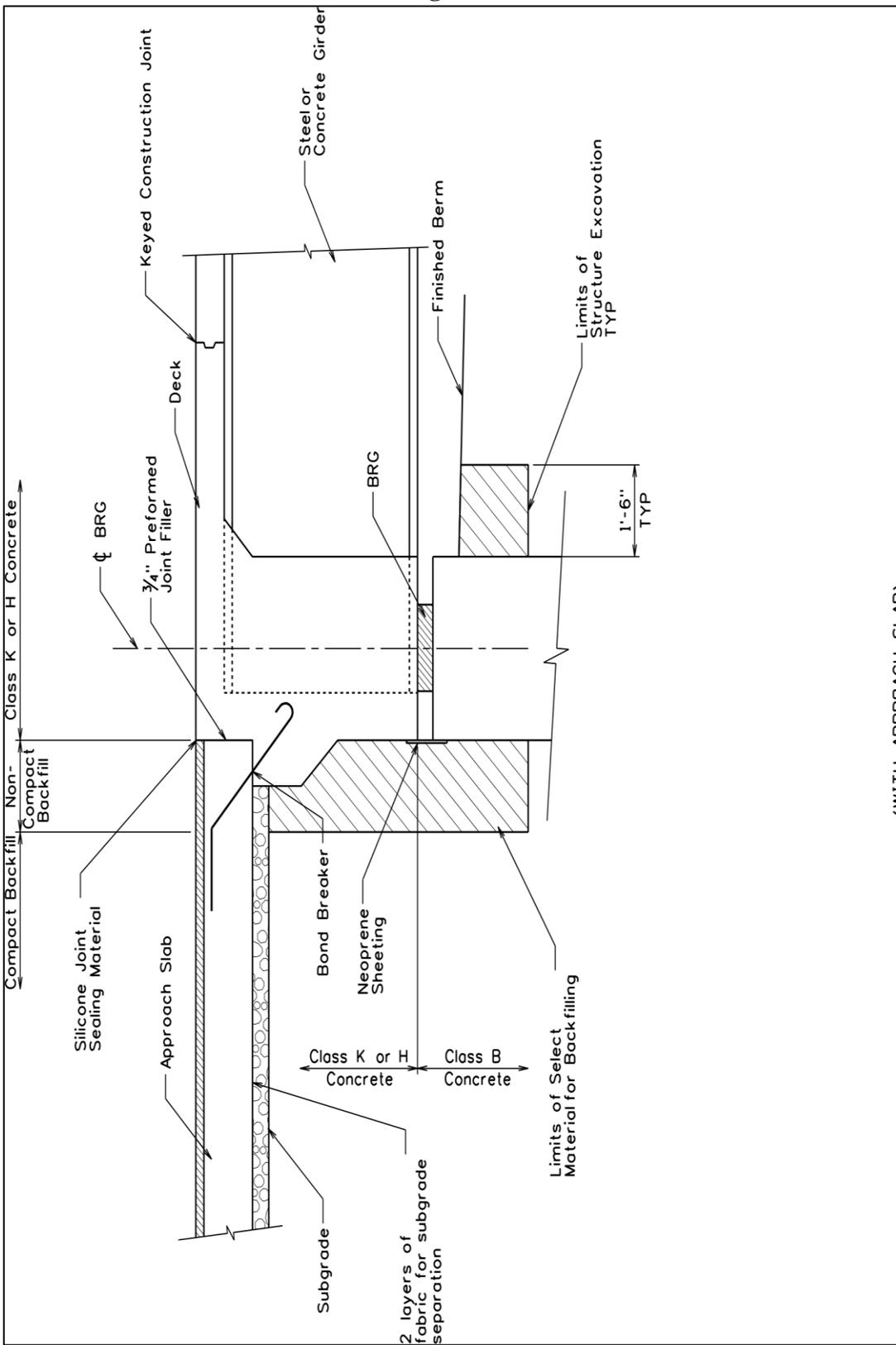


INTEGRAL ABUTMENT - OR  
ADJACENT CONCRETE BOX BEAMS  
(WITHOUT APPROACH SLAB)

Figure 2090.C

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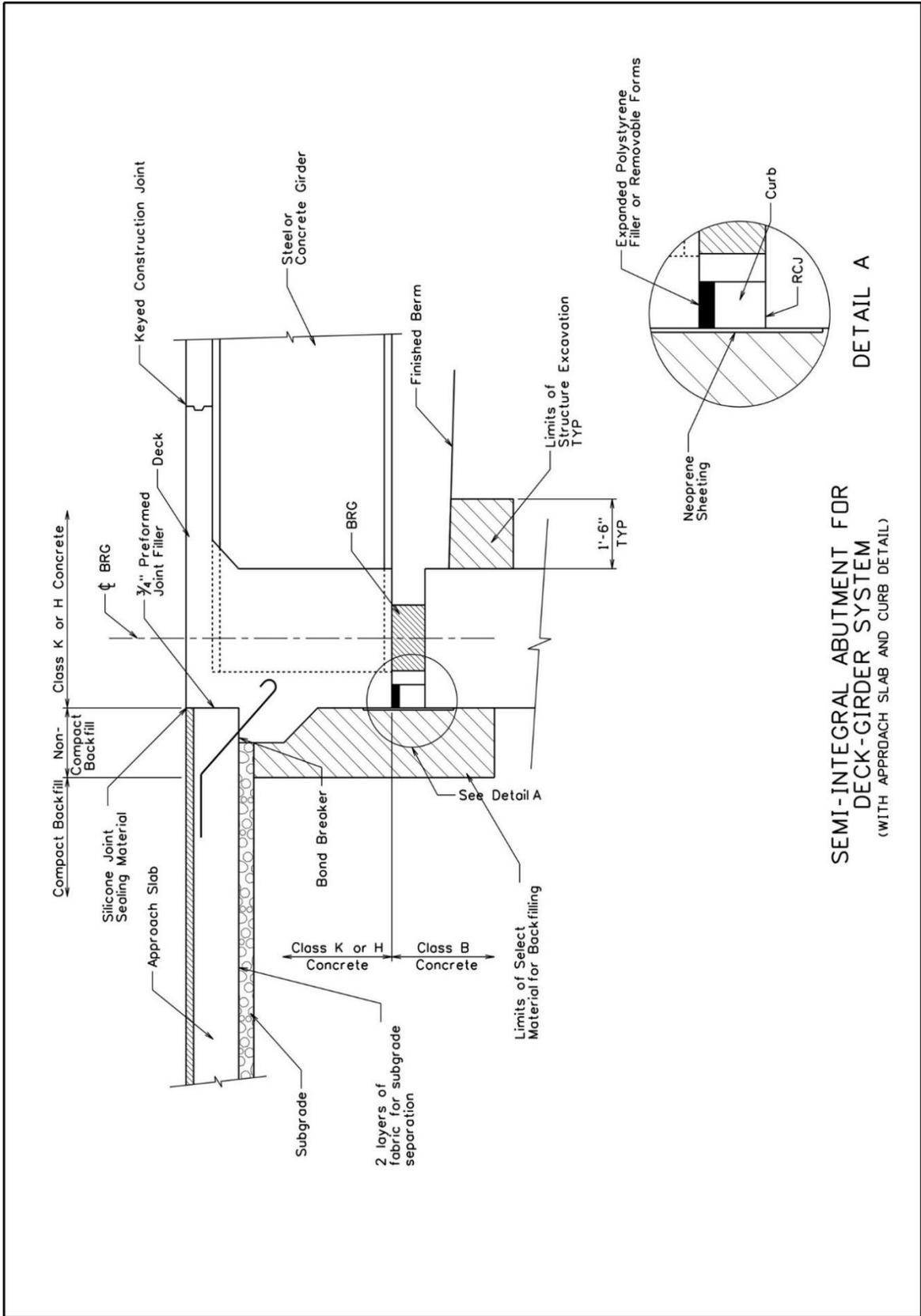
Figure 2090.D



(WITH APPROACH SLAB)

SEMI-INTEGRAL ABUTMENT FOR  
DECK-GIRDER SYSTEM  
(WITH A PROACH SL-3)

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SEMI-INTEGRAL ABUTMENT FOR DECK-GIRDER SYSTEM (WITH APPROACH SLAB AND CURB DETAIL)

DETAIL A

Figure 2090.E

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

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**STRUCTURE DIRECTIVE 2100  
RETAINING WALLS  
Date of Implementation  
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.

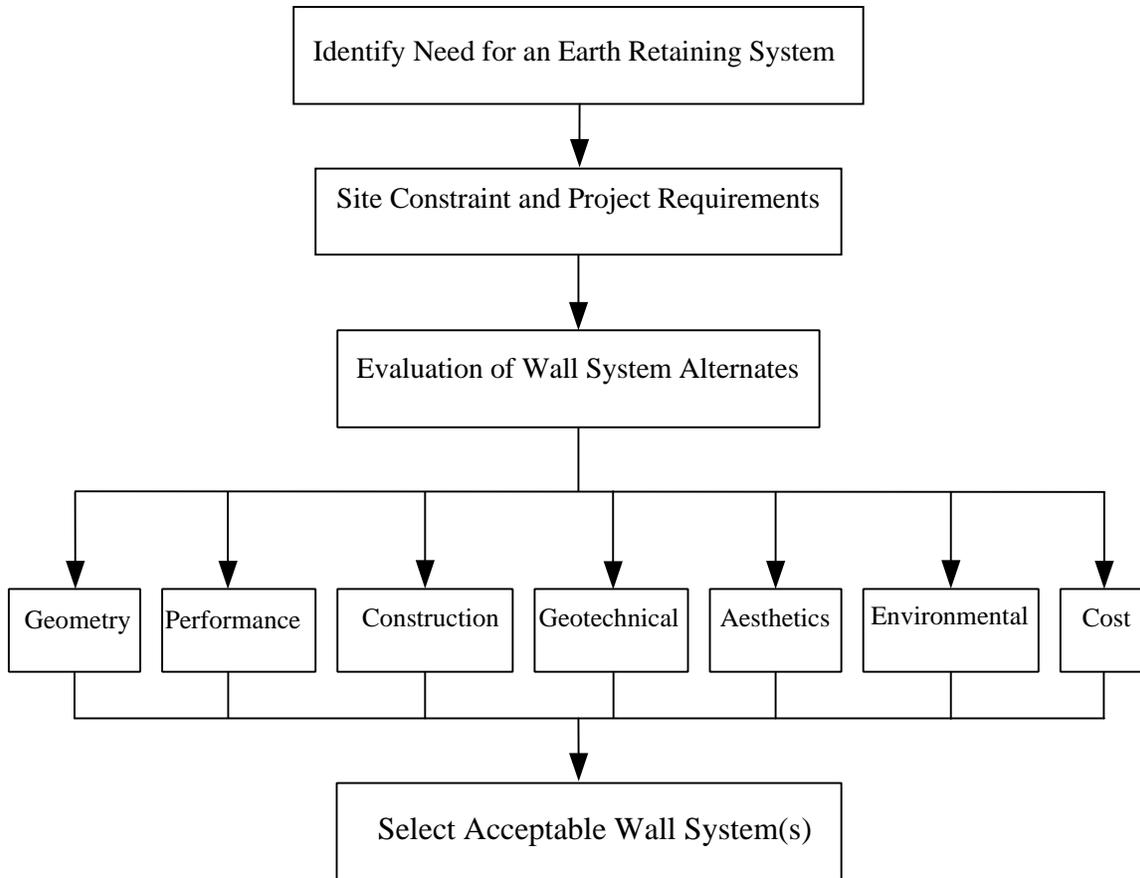
The selection and design of the appropriate retaining wall system is an integral part of the structural design of highway projects. Proper application of retaining system type and the use of proprietary retaining systems requires guidance and advance approval of the wall types and their manufacturers.

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.



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

**STRUCTURE DIRECTIVE 2101  
RETAINING WALL SYSTEM SELECTION PROCESS  
Date of Implementation  
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 WVDOH 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 FT	Large	Fair	<ul style="list-style-type: none"> <li>• Rapid construction</li> <li>• Readily available</li> </ul>	<ul style="list-style-type: none"> <li>• Difficult to construct in hard ground or through obstructions</li> </ul>
Soldier Pile/Lagging wall	X	X	Up to 16 FT	Medium	Poor	<ul style="list-style-type: none"> <li>• Rapid construction</li> <li>• Soldier Piles can be drilled or driven</li> </ul>	<ul style="list-style-type: none"> <li>• Difficult to maintain vertical tolerance in hard ground</li> <li>• Potential for ground loss at excavated face</li> </ul>
Tie-Back or Anchored Wall	X	X	15-60 FT (1)	Small – Medium	N/A	<ul style="list-style-type: none"> <li>• Can resist large horizontal pressure</li> <li>• Adaptable to varying site conditions</li> </ul>	<ul style="list-style-type: none"> <li>• Requires skilled labor and specialized equipment</li> <li>• Anchors may require permanent easements</li> </ul>

(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**

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Wall Type (2)	Perm.	Temp.	Cost Effective Height Range	WVDOH Height Range	Differential Settlement Tolerance (1)	Advantages	Disadvantages
Concrete Gravity Wall			3-10 FT		1/500	<ul style="list-style-type: none"> <li>• Durable</li> <li>• Requires smaller quantity of select backfill as compared to MSE walls</li> <li>• Concrete can meet aesthetic requirements</li> </ul>	<ul style="list-style-type: none"> <li>• Deep foundation support may be necessary</li> <li>• Relatively long construction time</li> </ul>
Concrete Crib Wall	X		6-35 FT		1/300	<ul style="list-style-type: none"> <li>• Does not require skilled labor or specialized equipment</li> <li>• Rapid construction</li> </ul>	<ul style="list-style-type: none"> <li>• Difficult to make height adjustments in field</li> </ul>
Concrete Cantilever Wall	X		6-30 FT		1/500	<ul style="list-style-type: none"> <li>• Durable</li> <li>• Requires smaller quantity of select backfill as compared to MSE walls</li> <li>• Concrete can meet aesthetic requirements</li> </ul>	<ul style="list-style-type: none"> <li>• Deep foundation support may be necessary</li> <li>• Relatively long construction time</li> </ul>
Concrete Counterforted Wall	X		30-60 FT		1/500	<ul style="list-style-type: none"> <li>• Durable</li> <li>• Requires smaller quantity of select backfill as compared to MSE walls</li> <li>• Concrete can meet aesthetic requirements</li> </ul>	<ul style="list-style-type: none"> <li>• Deep foundation support may be necessary</li> <li>• Relatively long construction time</li> </ul>
MSE Wall (Precast Facing)	X		10-65 FT	0 – 60 FT	1/100	<ul style="list-style-type: none"> <li>• Does not require skilled labor or specialized equipment</li> <li>• Flexibility in choice of facing</li> </ul>	<ul style="list-style-type: none"> <li>• Requires use of select backfill</li> <li>• Subject to corrosion in aggressive environment (metallic reinforcement)</li> </ul>
MSE Wall (Wire Face)	X	X	6-50 FT	0 – 60 FT	1/60	<ul style="list-style-type: none"> <li>• Does not require skilled labor or specialized equipment</li> <li>• Flexibility in choice of facing</li> </ul>	<ul style="list-style-type: none"> <li>• Facing may not be aesthetically pleasing</li> <li>• Geosynthetic reinforcement is subject to degradation in some environments</li> </ul>
MSE Wall (Mod. Block)	Class I	X	6-23 FT	0 – 40 FT	1/200	<ul style="list-style-type: none"> <li>• Does not require skilled labor or specialized equipment</li> <li>• Flexibility in choice of facing</li> <li>• Blocks are easily handled</li> </ul>	<ul style="list-style-type: none"> <li>• Requires use of select backfill</li> <li>• Subject to corrosion in aggressive environment (metallic reinforcement)</li> <li>• Positive reinforcement connection to blocks is difficult to achieve</li> </ul>
	Class II			0 – 20 FT			
	Class III			0 – 10 FT			
	Class IV			0 – 5 FT			

(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.

**Table 2101.B**

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<b>Modular Block Walls have the following Classifications</b>	
<b>Class</b>	<b>Wall Type</b>
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.C – System selection chart for FILL walls**

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

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**STRUCTURE DIRECTIVE 2102  
PERMANENT RETAINING WALLS  
Date of Implementation  
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 drilled caissons 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 30 FT 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 WVDOH website for a list of approved MSE wall vendors:  
<http://www.wvdot.com/10%5Fcontractors/aspl/626.htm>.

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 70% of the wall height. Shorter reinforcement lengths, but no less than 8 FT, 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 IN 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 30 FT 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  
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**STRUCTURE DIRECTIVE 2103  
TEMPORARY RETAINING WALLS  
Date of Implementation  
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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  
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**STRUCTURE DIRECTIVE 2110**

**PIERS**

**Date of Implementation  
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Refer to SD 1044 for a general description of various types of piers.

Class B Concrete with a compressive strength of 3000 PSI should be used for most piers. Class B Modified concrete with a compressive strength of 4000 PSI may be used, if required for strength.

For the strength and extreme event limit states, the Designer is strongly encouraged to use the strut and tie model for thick concrete elements (e.g., footings, pile caps and pier caps). Use this method to determine internal force effects near supports and points of concentrated loads.

Moment redistribution shall be used where appropriate.

**2110.1-PIER CAPS**

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 inches from the face of the pier cap. Instances of pier caps being wider than the column, necessitated by skewed bearings and dual bearings such as those found on prestressed concrete beam superstructures are acceptable.

Pier cap depths shall be determined by strength and clearance requirements. The minimum size is 3 FT vertically by 2.5 FT horizontally and must extend 1 FT 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 4 FT beyond the face must be haunched. A 2:1 haunch ratio is preferred. The ends of the pier caps may be either plane vertically or shaped (i.e., cylindrical to mirror the columns).

Minimum horizontal reinforcement shall be #5 bars spaced at 12 IN. Rebar shall be placed to avoid anchor bolts.

Beam seats shall be stepped and finished level. The sloped stem option, per SD 2080, is preferred when steps exceed 4 IN.

**2110.2-PIER COLUMNS**

The minimum longitudinal column diameter or stem thickness shall be 3 FT. Column tapers or section changes shall not be used unless a detailed study proves that they are cost-efficient. Generally, these forming costs are very high. Consider hollow shafts for piers in excess of 100 FT.

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.

**2110.3-PIER FOUNDATIONS**

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

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
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<p><b>STRUCTURE DIRECTIVE 2120 FOUNDATIONS Date of Implementation First Edition</b></p>
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**2120.1-GENERAL**

All substructures are to be founded upon bedrock; whether by spread footings, piles or drilled caissons. 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 3 FT and shall be keyed into rock a minimum of 1 FT. Allowable bearing pressures shall be used to size the footing.

For grade separation structures, the top of the footing shall be a minimum of 1 FT and preferably 2 FT below the finished ground line. The top of the footing should be at least 1 FT 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 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 40 FT.
- B. HP 12x53 for lengths between 40 and 80 FT.

C. HP 14x73 for lengths over 80 FT.

The design may require larger piles, for the above lengths, based on vertical and/or lateral loads. The minimum piling length shall be 10 FT. The pile embedment is a minimum of 1 FT 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 3 FT. 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-DRILLED CAISSON FOUNDATIONS**

Drilled Caissons 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 Standard 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 8 IN crushed rock with confinement curbing. Confinement curbing is reinforced concrete curbing a minimum of 6 IN wide by 2 FT 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 3 FT 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 10 FT, 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**

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<p><b>STRUCTURE DIRECTIVE 2130</b> <b>SCOUR</b> <b>Date of Implementation</b> <b>First Edition</b></p>
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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 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:

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- 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

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**STRUCTURE DIRECTIVE 2140**  
**APPROACH SLABS**  
**Date of Implementation**  
**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 20 FT 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 12 IN of concrete topped with 2 IN of hot laid bituminous skid resistant overlay (14 IN total thickness).

**2140.2.1-Reinforcement:** Longitudinal reinforcement shall consist of No. 5 bars spaced at 12 IN (top) and No. 8 bars spaced at 6 IN (bottom).

Transverse reinforcement shall consist of No. 5 bars spaced at 12 IN (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 IN and a Type J joint (Standard Sheet PVT5) for movements greater than 0.25 IN.

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**

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**STRUCTURE DIRECTIVE 2150  
LOAD RATING OF NEW BRIDGE DESIGN  
Date of Implementation  
First Edition**

Load and Resistance Factor Rating (LRFR) is consistent with the LRFD Specifications in using a reliability-based limit states philosophy and extends the provisions of the LRFD 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.

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). This document provides guidance to load rating engineers for performing and submitting load rating calculations and serves as a supplement to the MBE to describe WVDOT specific load rating requirements.

Each bridge shall be load rated at inventory and operating levels for AASHTO's HL93 loading as presented in the Governing Specifications on all routes. In addition, an analysis shall be completed for each West Virginia Legal Loads 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). The axle configurations and loads for the WV Legal Trucks and the CRTS Trucks are shown in Figure 2150.A.

# VEHICLE LIVE LOADS

NOTE : ALL AXLE WEIGHTS IN KIPS

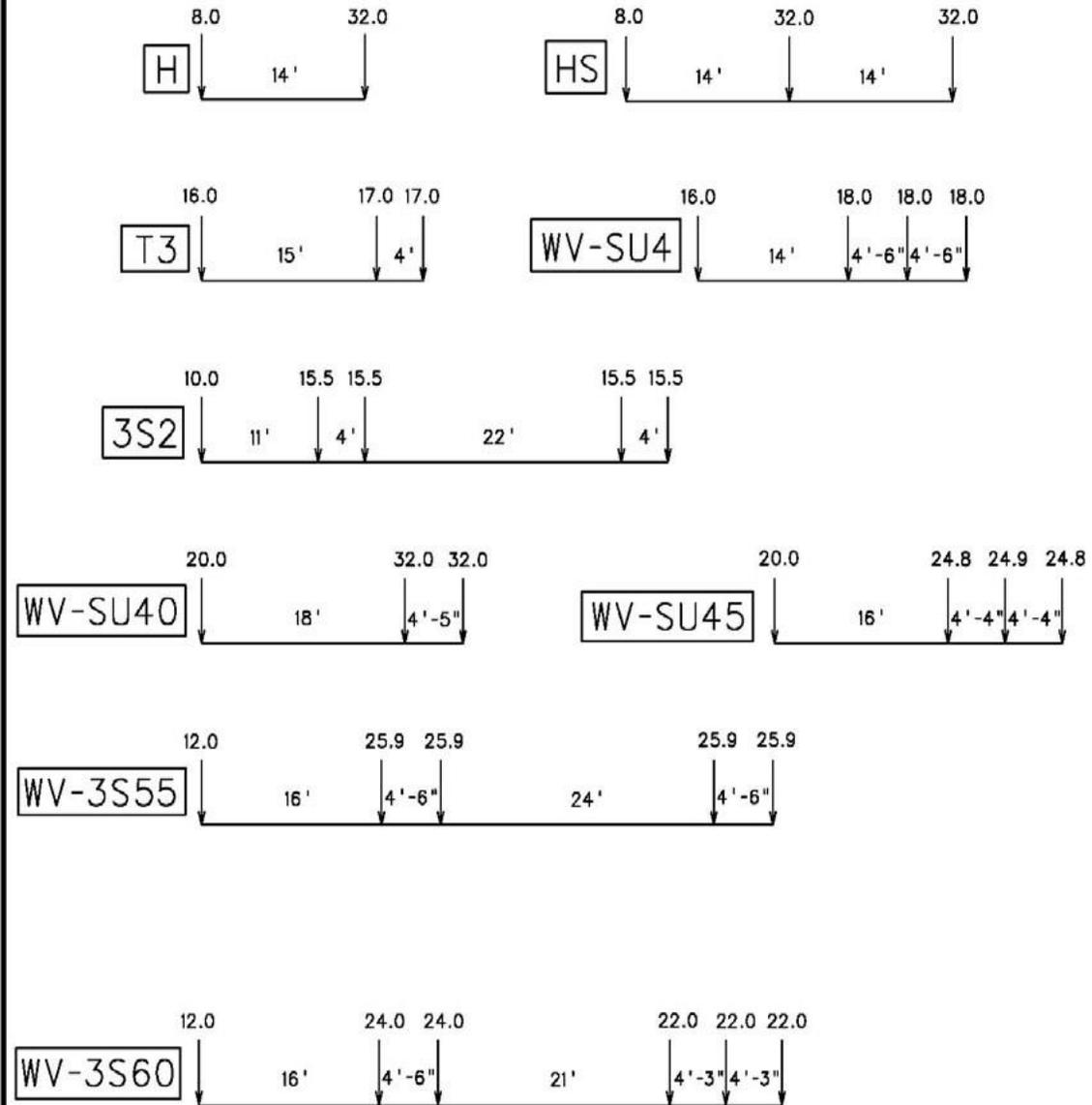


Figure 2150.A

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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 inventory ratings for new or replacement bridge designs are shown in Table 2150.B.

Route	HL93 (RF)	H (Tons)	Type 3 (Tons)	WV SU4 (Tons)	HS (Tons)	3S2 (Tons)	SU40* (Tons)	SU45* (Tons)	3S55* (Tons)	3S60* (Tons)
Interstate	1.00	20	27	29	37	40	-	-	-	-
65,000 lbs.	1.00	22	33	36	36	36	-	-	-	-
80,000 lbs.	1.00	22	33	39	41	44	-	-	-	-
CRTS	1.00	22	33	39	41	44	42	48	58	63

\* Required for CRTS Routes Only

**Table 20150.B-Target Inventory Ratings**

If the rating of bridges designed using the LRFD Specifications is less than the target value, and the design is found to be adequate, the Bridge Project Manager in coordination with the evaluation section 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 4.6.4.

A request for load rating shall be submitted to Operations Division 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, dead load moments and shears, and live load moments and shears at critical locations in each span and at all supports.
- B. Superstructure framing plan, typical cross section, girder elevation, and bridge general notes sheets.
- C. The PS&E date of the project.

If requested, the Designer shall also be required to submit to OM 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. Inventory and Operating 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.

**2150.1-RATING COMPUTATIONS**

The load rating shall be computed using the following general rating equation (see MBE 6A.4.2.1):

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)}$$

- RF = Rating Factor C=Capacity
- DC = Dead load effect due to structural components and attachments
- DW = Dead load effects due to wearing surface and utilities
- P = Permanent loads other than dead loads (secondary prestressing effects, etc.)
- LL = Live load effect of the Rating Vehicle
- IM = Dynamic load allowance
- $\gamma_{DC}$  = LRFD load factor for structural components and attachments
- $\gamma_{DW}$  = LRFD load factor for wearing surfaces and utilities
- $\gamma_P$  = LRFD load factor for permanent loads other than dead loads=1.0
- $\gamma_{LL}$  = Evaluation live load factor for the Rating Vehicle

Load factors shall be determined from MBE Table 6A.4.2.2-1

**2150.1.1-For Strength Limit States:**

$$C = \phi_C \phi_S \phi_R R_n$$

Where the following lower limit shall apply:

$$\phi_C \phi_S \geq 0.85$$

**2150.1.2-For All Non-Strength Limit States:**

$$C = f_R$$

- $\phi_C$  = Condition Factor  $\phi_S$  = System Factor
- $\phi$  = AASHTO LRFD Resistance Factor
- $R_n$  = Nominal member resistance (as built or as inspected)
- $f_R$  = Allowable stress specified in the LRFD code

**2150.2-LRFR LIMIT STATES FOR EVALUATION**

Strength limit state is used for checking the ultimate capacity of structural members and is the primary limit state utilized for determining posting needs. Service and fatigue limit states are utilized to limit stresses, deformations, and cracking under regular service conditions. In LRFR, Service and Fatigue limit state checks are optional in the sense that a posting or permit decision does not have to be dictated by the result. These serviceability checks provide valuable information for the engineer to use in the decision process. LRFR limit states for evaluation are shown in Table 2150.C below. Evaluation at the strength limit state is the only required check during the LRFR

analysis on all new or replacement bridges. Evaluation at the service and fatigue limit states will not be required unless specified as part of the initial scope of work.

Bridge Type	Limit State	Design	Legal
		HL93	H, Type3, WV SU4, HS, 3S2, CRTS, Lane Load Models
Steel	Strength I	x	x
	Strength II		
	Service II	xx	xx
Reinforced Concrete	Strength I	x	x
	Strength II		
	Service I		
Prestressed Concrete (non-segmental)	Strength I	x	x
	Strength II		
	Service III	xx	
Timber	Strength I	x	x
	Strength II		

x Required evaluation on all new or replacement bridges

xx Optional evaluation required only if specified during initial scope of work meeting

**Table 2150.C**

For non-segmental prestressed concrete bridges, LRFR provides a limit state check for cracking of concrete (SERVICE III) by limiting concrete tensile stresses under service loads. Service III need not be checked for design load Operating Ratings as it is a design level check.

Service I and Service III limit states are mandatory for load rating of segmental concrete box girder bridges. See MBE 6A.5.14.

A new SERVICE I load combination for reinforced concrete components and prestressed concrete components has been introduced in LRFR to check for possible inelastic deformations in the reinforcing steel during heavy permit load crossings. See MBE 6A.5.4.2.2.2. This check shall be applied to permit load checks and sets a limiting criterion of 0.9Fy in the extreme tension reinforcement. Limiting steel stress to 0.9Fy is intended to ensure that there is elastic behavior and that cracks that develop during the passage of overweight vehicles will close once the vehicle is removed. It also ensures that there is reserve ductility in the member.

Steel structures shall satisfy the overload permanent deflection check under the SERVICE II load combination for design load and legal load ratings. Maximum steel stress is limited to 95% and 80% of the yield stress for composite and non-composite compact girders respectively. See MBE 6A.6.4.2.2. Service II checks for permit loads are recommended but optional. During an overweight permit review the actual truck weight is available, so a 1.0 live load factor is specified.

A tabulation of rating examples are included in Appendix A of the MBE.

### **2150.3-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 that the depth of fill is such that live load effects can be neglected then the structure would have an infinite safe load capacity for HL93, WV Legal Loads, and CRTS Trucks 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 that is constructed on a longitudinal slope to determine the out of plane load effects on the structure in the final condition.

Calculations shall be submitted to the Bridge Project Manager for approval prior to fabrication of any primary structural elements.

### **2150.4-LOAD RATING OF GUSSET PLATES**

Load rating of gusset plates will be performed in accordance with *FHWA Gusset Guidance - Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges*, FHWA-IF-09-014, February 2009.

- A. When load rating gusset plates with unknown material properties, member strength should be obtained from the current version of the MBE.
- B. When checking the Limiting Slenderness Ratio (see FHWA Gusset Guidance 3.5) the unsupported edges of gusset plates should be evaluated in accordance based the following guidelines:

$$\text{Compression Edges} \quad -t^1 \leq 1.648\sqrt{(F_y - E)}$$

$$\text{Tension Edges} \quad -t^1 \leq 2.06\sqrt{(F_y - E)}$$

- C. All gusset plates rated using LFR will have the optional 0.9 reduction factor applied to the ratings as specified in the FHWA Gusset Guidance. This reduction factor is used to give the same reliability as the values obtained by LRFR ratings that uses the system factor to account for the non-redundant members.

### **2150.5-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 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 Load and Resistance Factor Rating.

## 2150.6-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 distribution 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). Do not use AASHTO distribution factors for the line girder analysis.

- A. The conversion factors for refined analyses shall be computed using the following equation:

$$CF = \frac{\text{Moment (refined analysis)}}{\text{Moment (line girder analysis)}}$$

- B. Use of conversion factors

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 a presumed Girder 3 at mid-span of span 2, the equivalent refined moment can be calculated as follows:

### **Girder 3, Location: Span 2.5**

$$\begin{aligned} CF &= 1.026 \text{ (presumably listed in the table on the original plans)} \\ M_{(LG)} &= 3175.8 \text{ K-FT (live load moment from line girder analysis)} \\ M_{(refined)} &= 3175.8 \text{ K-FT (1.026)} \\ &= 3258.4 \text{ K-FT (equivalent refined live load moment)} \end{aligned}$$

## 2150.7-LOAD RATING PLAN SHEETS

The required information for the plan sheet submittal is located in SD 3042.10.18. Example: plan sheets are also available for reference on the WVDOH website.

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

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**STRUCTURE DIRECTIVE 2160  
GUARDRAIL  
Date of Implementation  
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 25 FT, be flared and as strong as the bridge guardrail. The flares should be 4 FT wherever possible.

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

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<p><b>STRUCTURE DIRECTIVE 2170 BRIDGE IDENTIFICATION PLATE Date of Implementation First Edition</b></p>
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Section 104.8 of the Standard 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  
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**STRUCTURE DIRECTIVE 2180  
CONDUIT ON BRIDGES  
Date of Implementation  
First Edition**

Conduit shall be placed in the parapet for all bridge replacement and rehabilitation projects. For standard 32 IN tall parapet 2-1 ½ IN 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  
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**STRUCTURE DIRECTIVE 2190  
FUTURE INSPECTIONS  
Date of Implementation  
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 54 IN deep shall have rail hand-holds in their inside web faces. Webs deeper than 7 FT 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 6 FT.
  - 2. Bridge length exceeds 1500 FT.
- 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 8 FT 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  
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**STRUCTURE DIRECTIVE 2200  
BRIDGE DEMOLITION/DISMANTLING PLAN  
Date of Implementation  
First Edition**

All bridge projects requiring the demolition/dismantling of structures shall be in accordance with Section 203 of the Standard 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**

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**STRUCTURE DIRECTIVE 2210  
TEMPORARY STRUCTURES  
Date of Implementation  
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**

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**STRUCTURE DIRECTIVE 2220  
SALVAGEABLE MATERIALS  
Date of Implementation  
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

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<p><b>STRUCTURE DIRECTIVE 3000</b> <b>MISCELLANEOUS PLAN NOTES</b> <b>Date of Implementation</b> <b>First Edition</b></p>
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**3000.1-MANDATORY PLAN NOTES**

**3000.1.1-Weathering (Unpainted) Steel Bridges:** *NOTE: All references in these notes are to the WVDOT, DOH Standard Specifications, Roads and Bridges, Adopted 2010 as amended by the Supplemental Specifications dated January 1, 2013, (or latest edition).*

**3000.1.2-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.

High strength fasteners shall meet Section 709.24 and shall be black (uncoated) Type 3 (weathering steel). The high strength fasteners used in regions of the structure that require painting shall be Type 1 or 3 and shall be mechanical galvanized.

**3000.1.3-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 Standard Specifications, PAINTING STEEL 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.

**3000.1.4-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.

**3000.1.5-Handling And Storing Steel Members:** [This note has been added to the Standard Specifications.]

**3000.1.6-Final Cleanup Of Structural Steel Surfaces:** [This note has been added to the Standard Specifications.]

**3000.1.7-Protection Of Concrete Substructure:** [This note has been added to the Standard Specifications.]

### **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.

**3000.2.2-Steel Stud Shear Connectors: Steel Stud Shear Connectors** [This note has been added to the Standard Specifications.]

### **3000.3-Typical Plan Notes:**

**3000.3.1-Elastomeric Bearing with Load Plate:** [This note has been added to the Standard Specifications.]

**3000.3.2-Strip Seals:** [This note has been added to the Standard Specifications.]

**3000.3.3-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.

**3000.3.4-Retaining Walls:** [This note has been added to the Standard Specifications.]

**3000.3.5-Deck Slab Overhang Form:** [This note has been added to the Standard Specifications.]

**3000.3.6-Erection Requirements:** [This note has been added to the Standard Specifications.]

**3000.3.7-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.

**3000.3.8-Asbestos:** [This note has been added to the Standard Specifications.]

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

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**STRUCTURE DIRECTIVE 3010  
STANDARD DRAWINGS  
Date of Implementation  
First Edition**

**3010.1-STANDARD DETAILS AND DRAWING FILES**

A number of standard details and drawings are located on the WVDOH server. These are updated as necessary and are available upon request. For a listing of the standard bridge details, refer to this website: [http://www.wvdot.com/engineering/StandardDetails/Vol3/toc\\_sd3.htm](http://www.wvdot.com/engineering/StandardDetails/Vol3/toc_sd3.htm).

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 at this website: [http://www.wvdot.com/engineering/StandardDetails/Vol3/toc\\_sd3.htm](http://www.wvdot.com/engineering/StandardDetails/Vol3/toc_sd3.htm).

**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.

**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.

DETAIL	UNIT	PRECISION
Stationing	Hundred feet (e.g., 150.25 FT = 1+50.25)	Two decimal places (.01)
Length	FT-IN	Two decimal places (1/8 IN)
Finished Ground Elevation	FT	Two decimal places (.01)
Finished Concrete*	FT	Two decimal places (.01)
Beam Seat Elevation	FT	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	FT-IN	1/8 IN
Clear Cover	IN	1/2 IN
Structural Steel	FT-IN	1/16 IN

DETAIL	UNIT	PRECISION
Reinforcing Steel	FT-IN	1 IN straight, ¼ IN bent
Deep Foundation Tip Elevation	FT	1 FT
Pile Batter/Cut-Fill Slope	Ratio (e.g., 1:1)	--
Core Boring Elevation	FT	One decimal place (.1)

\* Except Beam Seat

**Table 3010.A**

**3010.4-SHEET SIZE AND FORMAT**

All contract plans are published electronically as .pdfs, where B is 11 IN x 17 IN and D is 22 IN x 34 IN. The border for the D size sheet is 21 IN x 32 IN providing a 0.5 IN border on the top, bottom, and right sides with a 1.5 IN left border for binding.

**3010.5-SCALES**

Preferred scales and text sizes for full size sheets are shown in Table 3010.B.

SCALE	TEXT HEIGHT
1 ½ IN = 1 FT	0.10 FT
1 IN = 1 FT	0.15 FT
¾ IN = 1 FT	0.20 FT
½ IN = 1 FT	0.30 FT
⅜ IN = 1 FT	0.40 FT
¼ IN = 1 FT	0.60 FT
⅓ IN = 1 FT	0.80 FT
⅛ IN = 1 FT	1.20 FT
1/16 IN = 1 FT	2.35 FT
1 IN = 10 FT	1.50 FT
1 IN = 20 FT	3.00 FT
1 IN = 30 FT	4.50 FT
1 IN = 50 FT	7.50 FT
1 IN = 100 FT	15.00 FT
1 IN = 200 FT	30.00 FT

**Table 3010.B**

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

DRAFT

**STRUCTURE DIRECTIVE 3020**  
**ESTIMATE OF QUANTITIES**  
**Date of Implementation**  
**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**

DRAFT

**STRUCTURE DIRECTIVE 3030  
STANDARD SPECIFICATIONS AND SPECIAL PROVISIONS  
Date of Implementation  
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 Bridge Project Manager for guidance in preparation of special provisions.

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

**DRAFT**

**STRUCTURE DIRECTIVE 3041  
DRAWING SEQUENCE  
Date of Implementation  
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**

DRAFT

**STRUCTURE DIRECTIVE 3042  
GENERAL PLAN AND EVALUATION  
Date of Implementation  
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 caisson) 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/caissons.
  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. Drilled caisson 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 drilled caissons 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 drilled caissons.
  5. Depth of drilled caisson rock sockets.
  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 drilled caissons size.
  8. Embedment of piles or drilled caissons.
  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 caisson or pile load capacity and the maximum design load per drilled caisson 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. Drilled caisson layout for caisson supported footings.
- B. Elevation View (Looking ahead station).
1. Bridge seat elevations.
  2. Bottom of footing elevation, if on piles or drilled caissons.
  3. Depth of drilled caisson rock sockets.
  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).

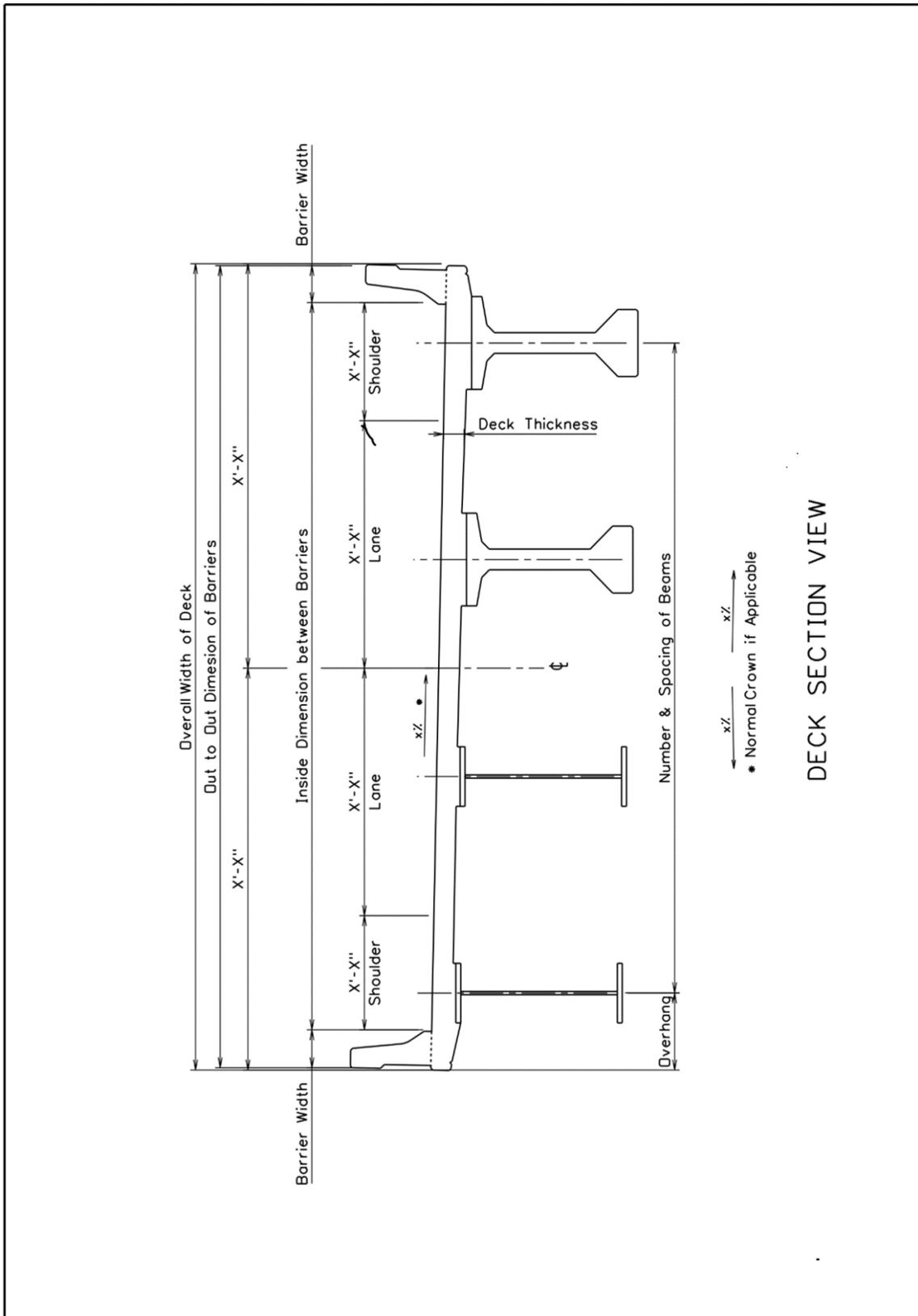
- N. Bar marks, number of bars and bar location (top and/or bottom).
- O. The minimum lap lengths for the bar sizes being used in tabular form.
- P. Special reinforcement details for sharply skewed bridges.
- Q. Deck drain locations.
- R. Location of appurtenances.

Sample Deck Reinforcing Plan sheets are available from the WVDOH.

**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.



DECK SECTION VIEW

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 IN edging tool. See SD 2022.2, Figure 2022.A.

**3042.10.11-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 FT). 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.

**3042.10.12-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.

**3042.10.13 -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 Standard Specifications”.

**3042.10.14-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.

**3042.10.15-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.

**3042.10.16-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.

**3042.10.17-Existing Bridge Plans:** Sheets from the existing bridge plans, if available, shall be attached.

**3042.10.18-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 ( $DL_1$ ), composite dead load ( $DL_2$ ) 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  
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**STRUCTURE DIRECTIVE 3043  
REINFORCING SCHEDULES  
Date of Implementation  
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).

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**STRUCTURE DIRECTIVE 3050  
BASIC LAYOUT – RETAINING WALLS  
Date of Implementation  
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 drilled caissons.
- D. Design loads.
- E. Excavation.
- F. Safety factors utilized in the wall design.
- G. Wall design parameters and notes.
- H. Allowable bearing pressure pile/drilled caisson 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 drilled caisson 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. Drilled caisson 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 25 FT 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 drilled caisson footings and the estimated pile or drilled caisson 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 drilled caisson size.
  7. Embedment of piles or drilled caissons.
  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. Solder 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.

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- 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 drilled caisson tip elevation.
- E. Station and offset of each boring.

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**STRUCTURE DIRECTIVE 3060  
SHOP DRAWINGS – REQUIRED INFORMATION  
Date of Implementation  
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. Copies are available at their website, [www.steelbridge.org](http://www.steelbridge.org).

A list of Material Procedures (MP's) is also available at [www.wvdot.com](http://www.wvdot.com), 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.

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<p><b>STRUCTURE DIRECTIVE 3061 SHOP DRAWINGS – APPROVAL PROCEDURES Date of Implementation First Edition</b></p>
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**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.

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**STRUCTURE DIRECTIVE 3070  
CONTRACT PLAN REVISIONS  
Date of Implementation  
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  
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**STRUCTURE DIRECTIVE 4000  
COMPUTER DESIGN SOFTWARE  
Date of Implementation  
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.

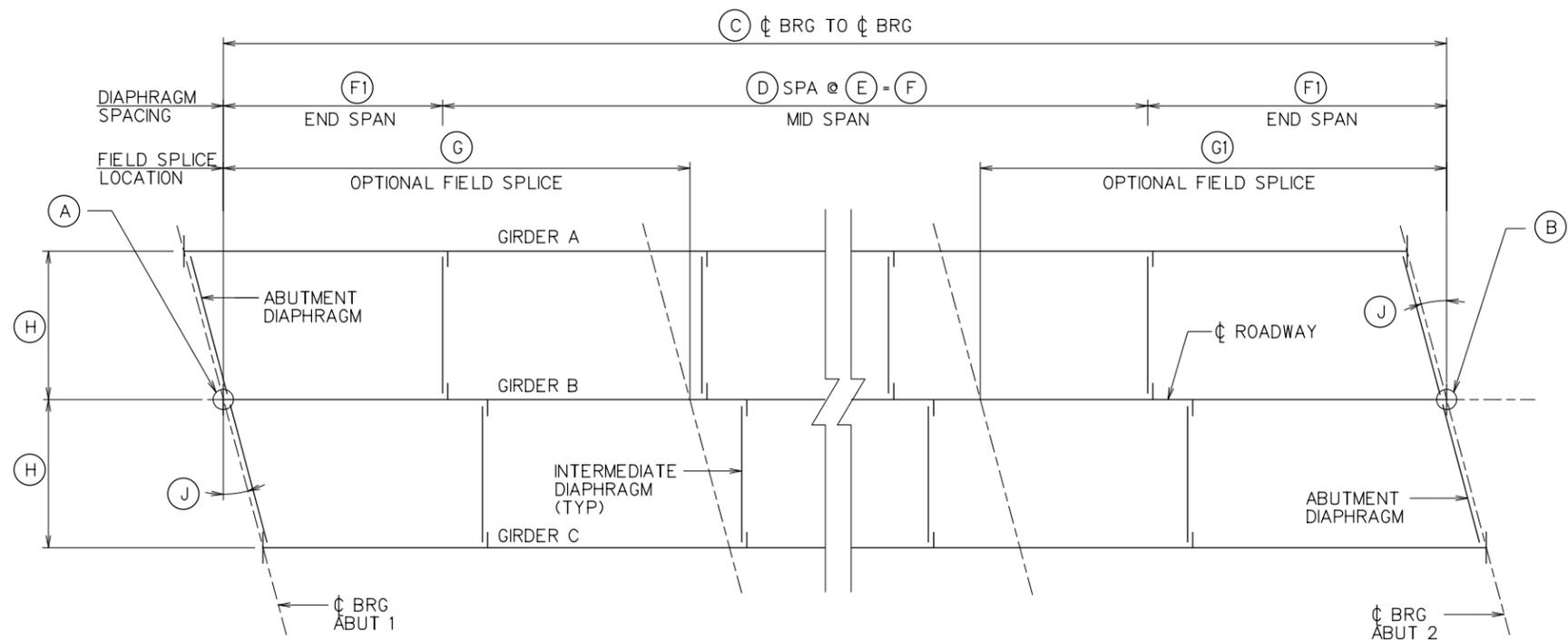
**4000.1-HYDRAULIC PROGRAMS**

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.

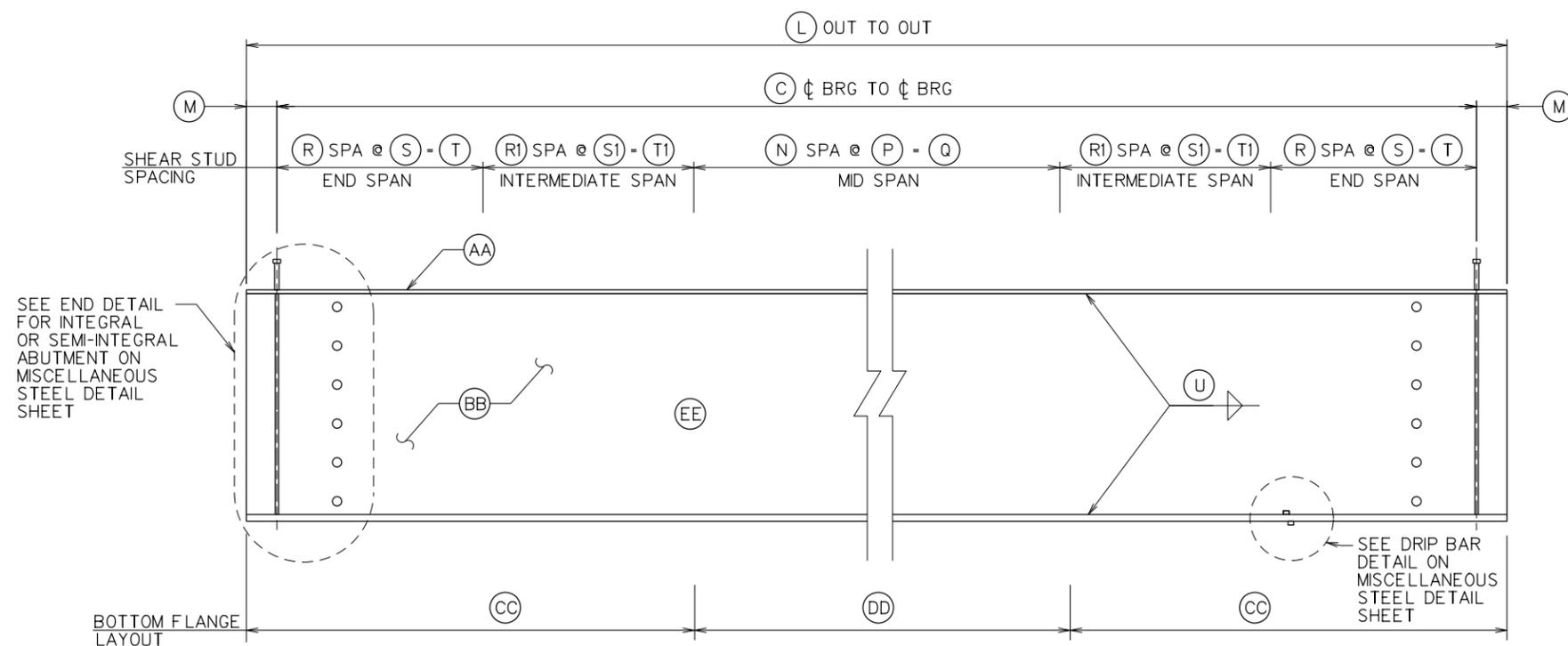
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FRAMING PLAN

FRAMING PLAN CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
A	WP 1 (STATION)	_____
B	WP 2 (STATION)	_____
C	BEARING TO BEARING LENGTH	_____
D	NUMBER OF MID SPAN DIAPHRAGM SPACES	_____
E	MID SPAN DIAPHRAGM SPACING	_____
F	MID SPAN DIAPHRAGM LENGTH	_____
F1	END SPAN DIAPHRAGM LENGTH	_____
G	LOCATION OF FIELD SPLICE FROM WP 1	_____
G1	LOCATION OF FIELD SPLICE FROM WP 2	_____
H	GIRDER SPACING	_____
J	SKEW	_____

GIRDER CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
L	OVERALL GIRDER LENGTH	_____
M	BEGINNING OFFSET	_____
N	NUMBER OF MID SPAN SHEAR STUD SPACES	_____
P	SHEAR STUD SPACING	_____
Q	LENGTH OF MID SPAN SHEAR STUDS	_____
R	NUMBER OF END SPAN SHEAR STUD SPACES	_____
R1	NUMBER OF INTERMEDIATE SPAN SHEAR STUD SPACES	_____
S	END SPAN SHEAR STUD SPACING	_____
S1	INTERMEDIATE SPAN SHEAR STUD SPACING	_____
T	LENGTH OF END SPAN SHEAR STUD	_____
T1	LENGTH OF INTERMEDIATE SPAN SHEAR STUD	_____
U	WELD	_____



ELEVATION

STEEL GIRDER DIMENSIONS				
CODE	DESCRIPTION	PLATE DIM.	LENGTH	F <sub>y</sub> (KSI)
AA	TOP PLATE	_____	_____	_____
BB	WEB PLATE	_____	_____	_____
CC	BOTTOM PLATE END SPAN	_____	_____	_____
DD	BOTTOM PLATE MID SPAN	_____	_____	_____
EE	W BEAM	_____	_____	_____
QUANTITIES				
NUMBER OF SHEAR STUDS PER GIRDER				_____

PRINT DATE  
2-FEB-2022 14:45

NOT TO SCALE

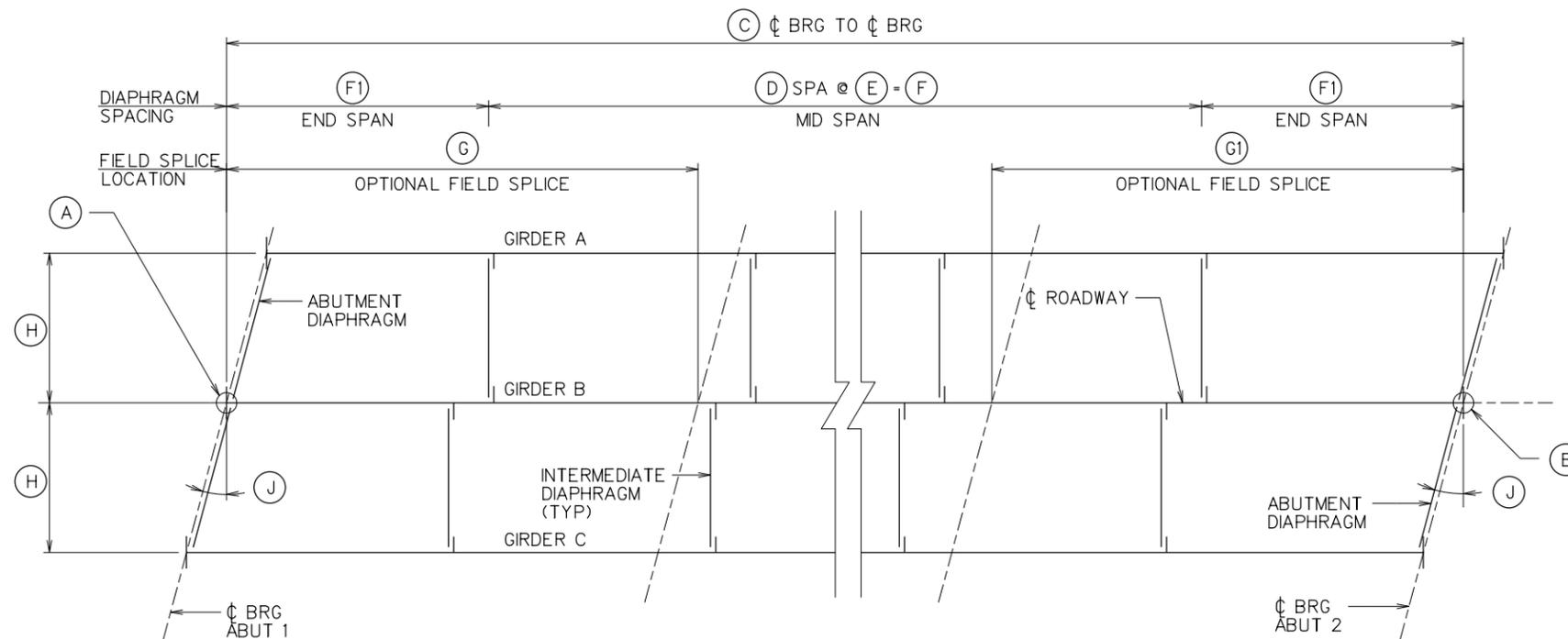
NO.	REVISION	DATE	BY

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

DESIGNED	DATE	CHECKED	DATE
DRAWN	DATE	REVIEWED	DATE

STANDARD BRIDGE PLANS  
**FRAMING STEEL**  
**3 GIRDER RF SKEW STAGGER**  
SHEET NUMBER 1000FR2A

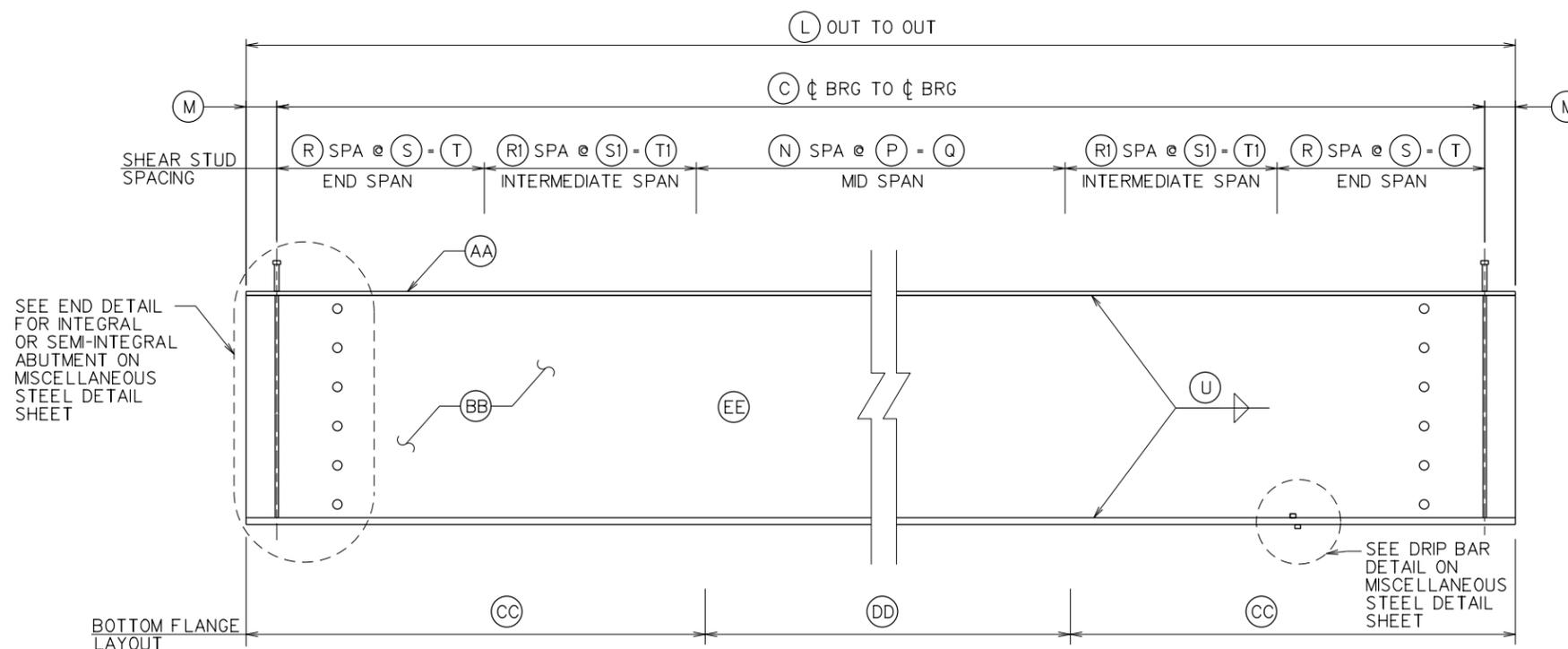
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**FRAMING PLAN**

FRAMING PLAN CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
A	WP 1 (STATION)	_____
B	WP 2 (STATION)	_____
C	BEARING TO BEARING LENGTH	_____
D	NUMBER OF MID SPAN DIAPHRAGM SPACES	_____
E	MID SPAN DIAPHRAGM SPACING	_____
F	MID SPAN DIAPHRAGM LENGTH	_____
F1	END SPAN DIAPHRAGM LENGTH	_____
G	LOCATION OF FIELD SPLICE FROM WP 1	_____
G1	LOCATION OF FIELD SPLICE FROM WP 2	_____
H	GIRDER SPACING	_____
J	SKEW	_____

GIRDER CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
L	OVERALL GIRDER LENGTH	_____
M	BEGINNING OFFSET	_____
N	NUMBER OF MID SPAN SHEAR STUD SPACES	_____
P	SHEAR STUD SPACING	_____
Q	LENGTH OF MID SPAN SHEAR STUDS	_____
R	NUMBER OF END SPAN SHEAR STUD SPACES	_____
R1	NUMBER OF INTERMEDIATE SPAN SHEAR STUD SPACES	_____
S	END SPAN SHEAR STUD SPACING	_____
S1	INTERMEDIATE SPAN SHEAR STUD SPACING	_____
T	LENGTH OF END SPAN SHEAR STUD	_____
T1	LENGTH OF INTERMEDIATE SPAN SHEAR STUD	_____
U	WELD	_____



**ELEVATION**

STEEL GIRDER DIMENSIONS				
CODE	DESCRIPTION	PLATE DIM.	LENGTH	F <sub>y</sub> (KSI)
AA	TOP PLATE	_____	_____	_____
BB	WEB PLATE	_____	_____	_____
CC	BOTTOM PLATE END SPAN	_____	_____	_____
DD	BOTTOM PLATE MID SPAN	_____	_____	_____
EE	OR W BEAM	_____	_____	_____
QUANTITIES				
NUMBER OF SHEAR STUDS PER GIRDER				_____

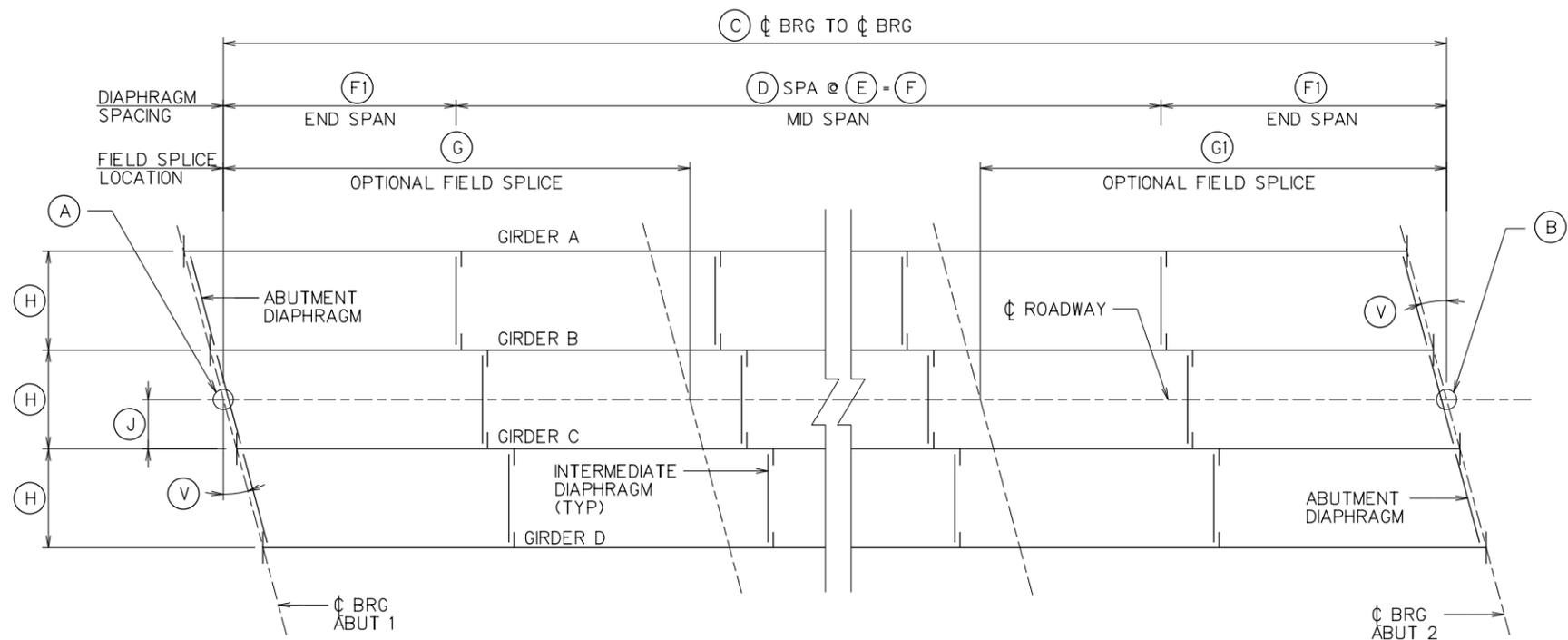
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2-FEB-2022 14:47

NOT TO SCALE

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION DIVISION OF HIGHWAYS				DESIGNED _____	DATE _____	CHECKED _____	DATE _____
				DRAWN _____	DATE _____	REVIEWED _____	DATE _____
NO.	REVISION	DATE	BY				

STANDARD BRIDGE PLANS  
**FRAMING STEEL**  
**3 GIRDER LF SKEW STAGGER**  
SHEET NUMBER 1000FR3A

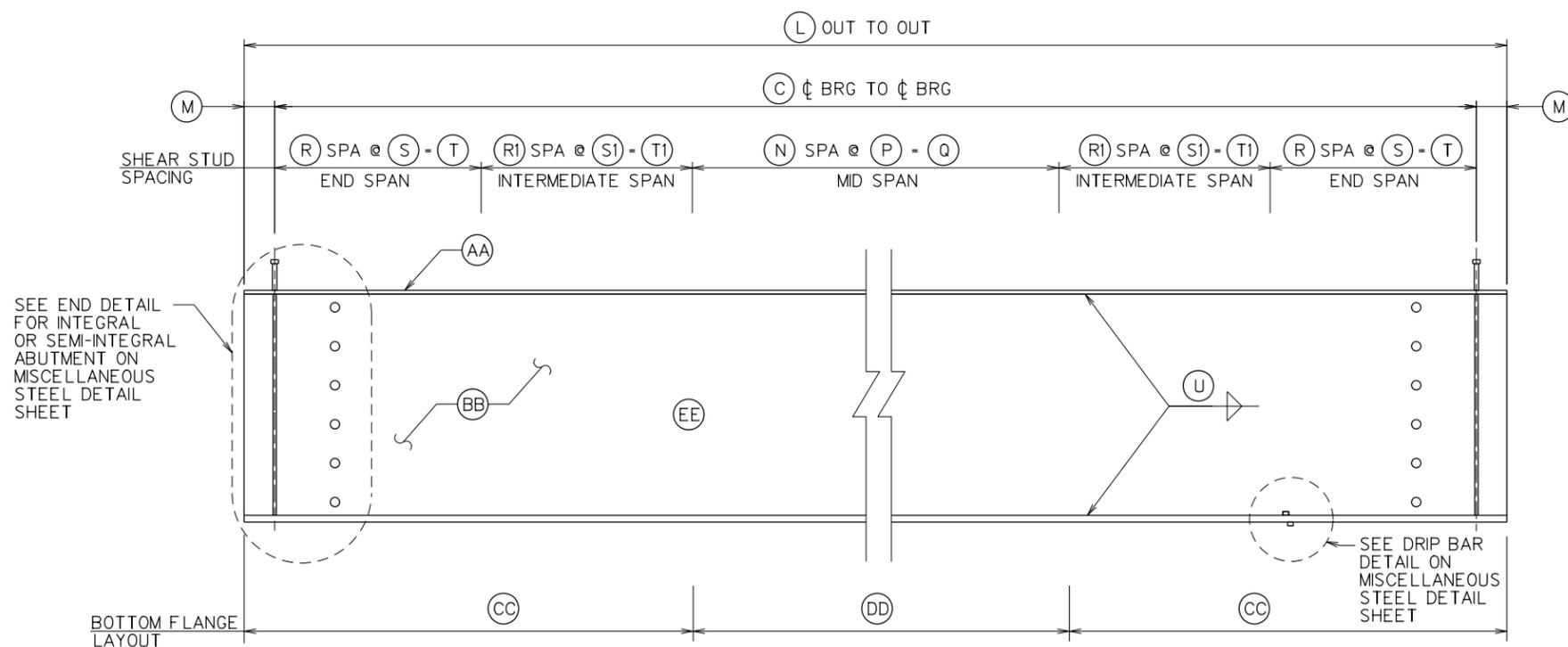
DRAFT



FRAMING PLAN

FRAMING PLAN CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
A	WP 1 (STATION)	_____
B	WP 2 (STATION)	_____
C	BEARING TO BEARING LENGTH	_____
D	NUMBER OF MID SPAN DIAPHRAGM SPACES	_____
E	MID SPAN DIAPHRAGM SPACING	_____
F	MID SPAN DIAPHRAGM LENGTH	_____
F1	END SPAN DIAPHRAGM LENGTH	_____
G	LOCATION OF FIELD SPLICE FROM WP 1	_____
G1	LOCATION OF FIELD SPLICE FROM WP 2	_____
H	GIRDER SPACING	_____
J	OFFSET FROM $\phi$	_____
V	SKEW	_____

GIRDER CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
L	OVERALL GIRDER LENGTH	_____
M	BEGINNING OFFSET	_____
N	NUMBER OF MID SPAN SHEAR STUD SPACES	_____
P	SHEAR STUD SPACING	_____
Q	LENGTH OF MID SPAN SHEAR STUDS	_____
R	NUMBER OF END SPAN SHEAR STUD SPACES	_____
R1	NUMBER OF INTERMEDIATE SPAN SHEAR STUD SPACES	_____
S	END SPAN SHEAR STUD SPACING	_____
S1	INTERMEDIATE SPAN SHEAR STUD SPACING	_____
T	LENGTH OF END SPAN SHEAR STUD	_____
T1	LENGTH OF INTERMEDIATE SPAN SHEAR STUD	_____
U	WELD	_____



ELEVATION

STEEL GIRDER DIMENSIONS				
CODE	DESCRIPTION	PLATE DIM.	LENGTH	F <sub>y</sub> (KSI)
AA	TOP PLATE	_____	_____	_____
BB	WEB PLATE	_____	_____	_____
CC	BOTTOM PLATE END SPAN	_____	_____	_____
DD	BOTTOM PLATE MID SPAN	_____	_____	_____
EE	W BEAM	_____	_____	_____
QUANTITIES				
NUMBER OF SHEAR STUDS PER GIRDER				_____

PRINT DATE  
2-FEB-2022 14:48

2-FEB-2022 14:48

NOT TO SCALE

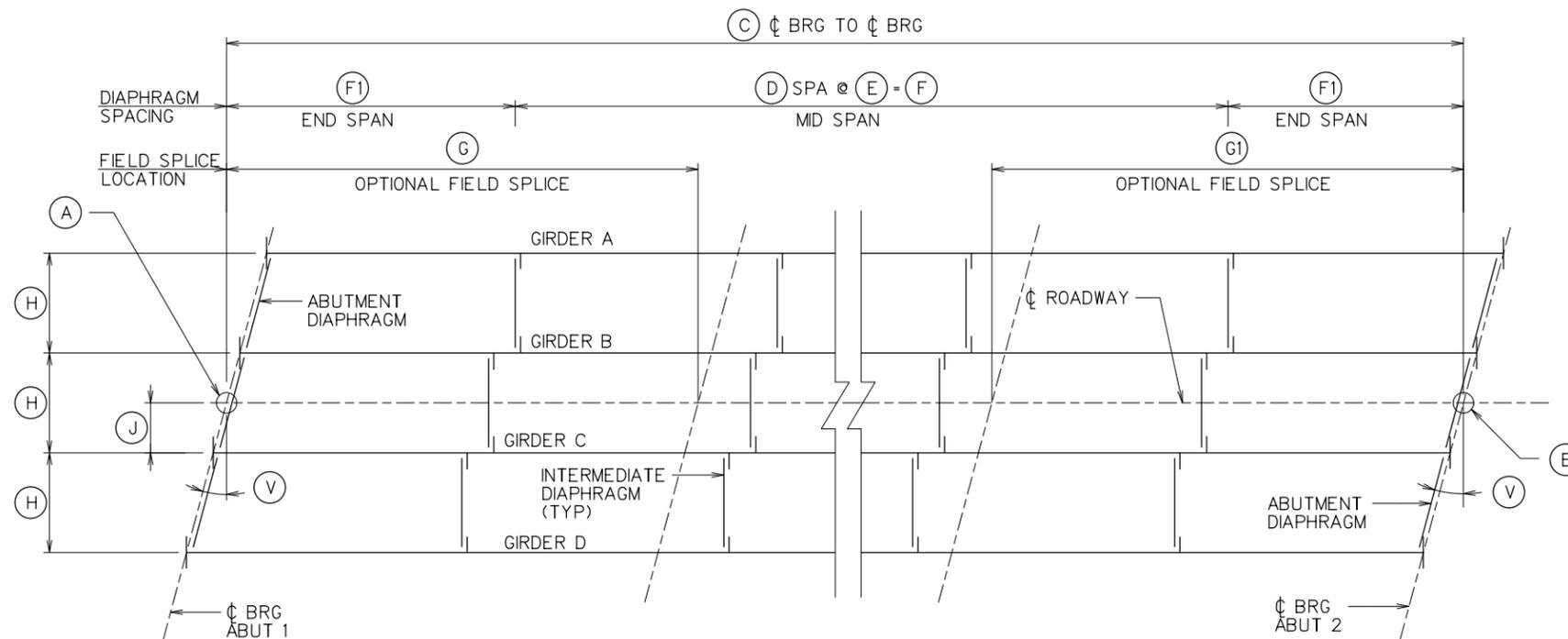
NO.	REVISION	DATE	BY

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

DESIGNED	DATE	CHECKED	DATE
DRAWN	DATE	REVIEWED	DATE


STANDARD BRIDGE PLANS  
**FRAMING STEEL**  
**4 GIRDER RF SKEW STAGGER**  
SHEET NUMBER 1000FR5A

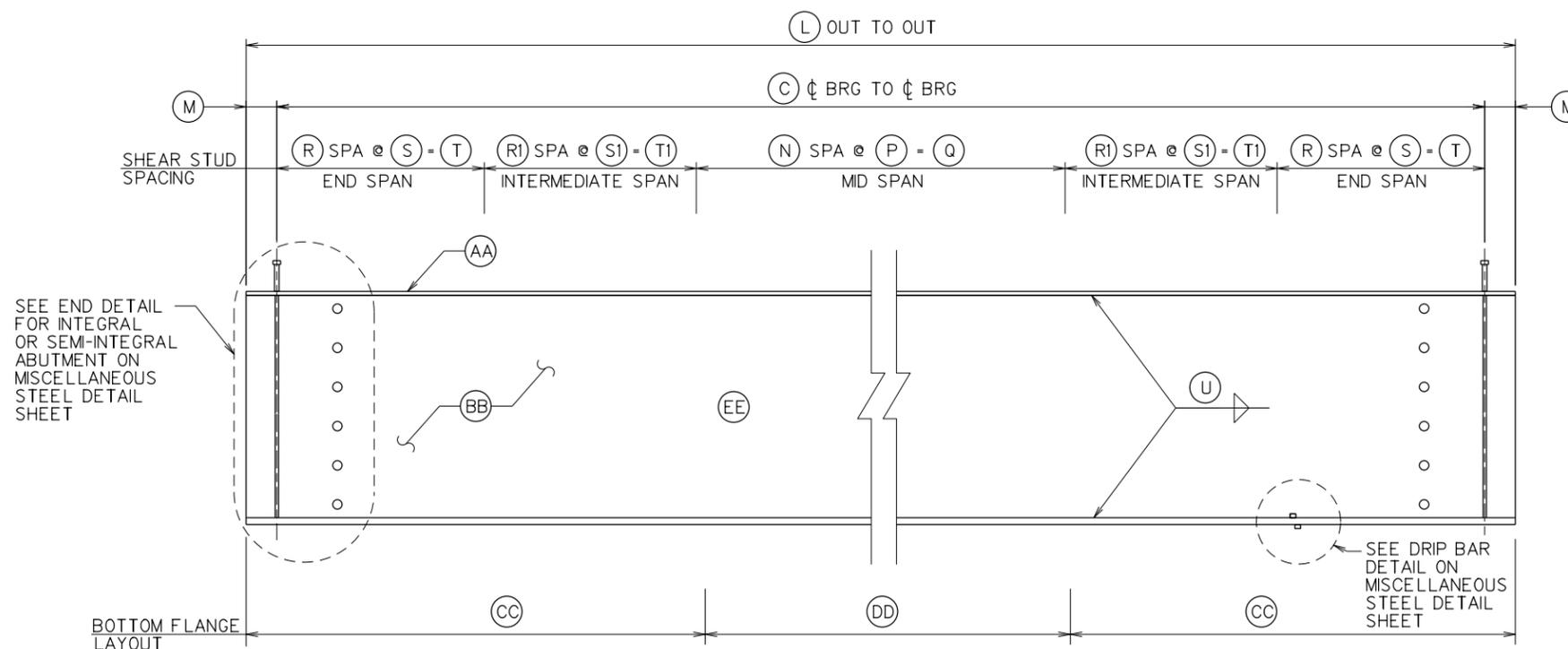
DRAFT



FRAMING PLAN

FRAMING PLAN CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
A	WP 1 (STATION)	_____
B	WP 2 (STATION)	_____
C	BEARING TO BEARING LENGTH	_____
D	NUMBER OF MID SPAN DIAPHRAGM SPACES	_____
E	MID SPAN DIAPHRAGM SPACING	_____
F	MID SPAN DIAPHRAGM LENGTH	_____
F1	END SPAN DIAPHRAGM LENGTH	_____
G	LOCATION OF FIELD SPLICE FROM WP 1	_____
G1	LOCATION OF FIELD SPLICE FROM WP 2	_____
H	GIRDER SPACING	_____
J	OFFSET FROM $\phi$	_____
V	SKEW	_____

GIRDER CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
L	OVERALL GIRDER LENGTH	_____
M	BEGINNING OFFSET	_____
N	NUMBER OF MID SPAN SHEAR STUD SPACES	_____
P	SHEAR STUD SPACING	_____
Q	LENGTH OF MID SPAN SHEAR STUDS	_____
R	NUMBER OF END SPAN SHEAR STUD SPACES	_____
R1	NUMBER OF INTERMEDIATE SPAN SHEAR STUD SPACES	_____
S	END SPAN SHEAR STUD SPACING	_____
S1	INTERMEDIATE SPAN SHEAR STUD SPACING	_____
T	LENGTH OF END SPAN SHEAR STUD	_____
T1	LENGTH OF INTERMEDIATE SPAN SHEAR STUD	_____
U	WELD	_____



ELEVATION

STEEL GIRDER DIMENSIONS				
CODE	DESCRIPTION	PLATE DIM.	LENGTH	Fy (KSI)
AA	TOP PLATE	_____	_____	_____
BB	WEB PLATE	_____	_____	_____
CC	BOTTOM PLATE END SPAN	_____	_____	_____
DD	BOTTOM PLATE MID SPAN	_____	_____	_____
OR				
EE	W BEAM	_____	_____	_____
QUANTITIES				
NUMBER OF SHEAR STUDS PER GIRDER				_____

PRINT DATE  
2-FEB-2022 14:50

NOT TO SCALE

NO.	REVISION	DATE	BY

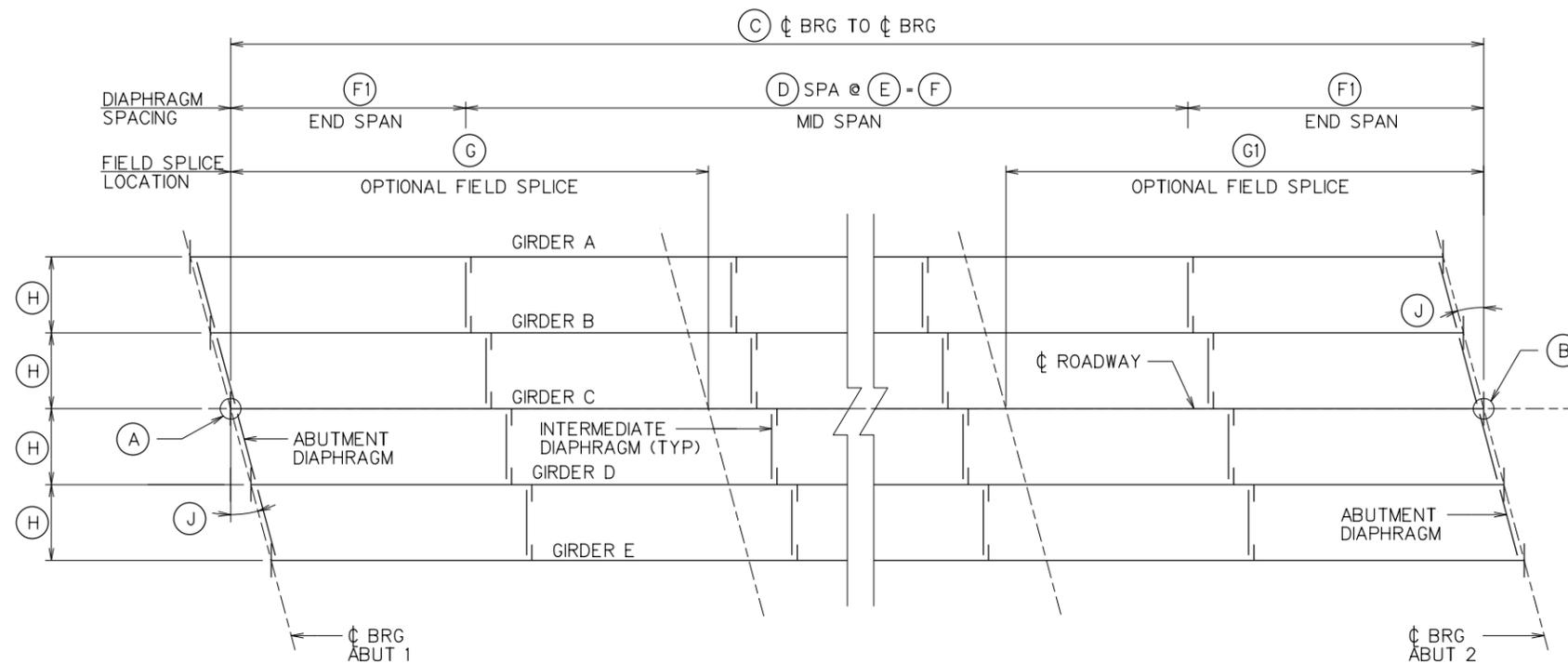
WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

DESIGNED	DATE	CHECKED	DATE
DRAWN	DATE	REVIEWED	DATE

STANDARD BRIDGE PLANS  
**FRAMING STEEL**  
**4 GIRDER LF SKEW STAGGER**  
SHEET NUMBER 1000FR6A

2-FEB-2022 14:50

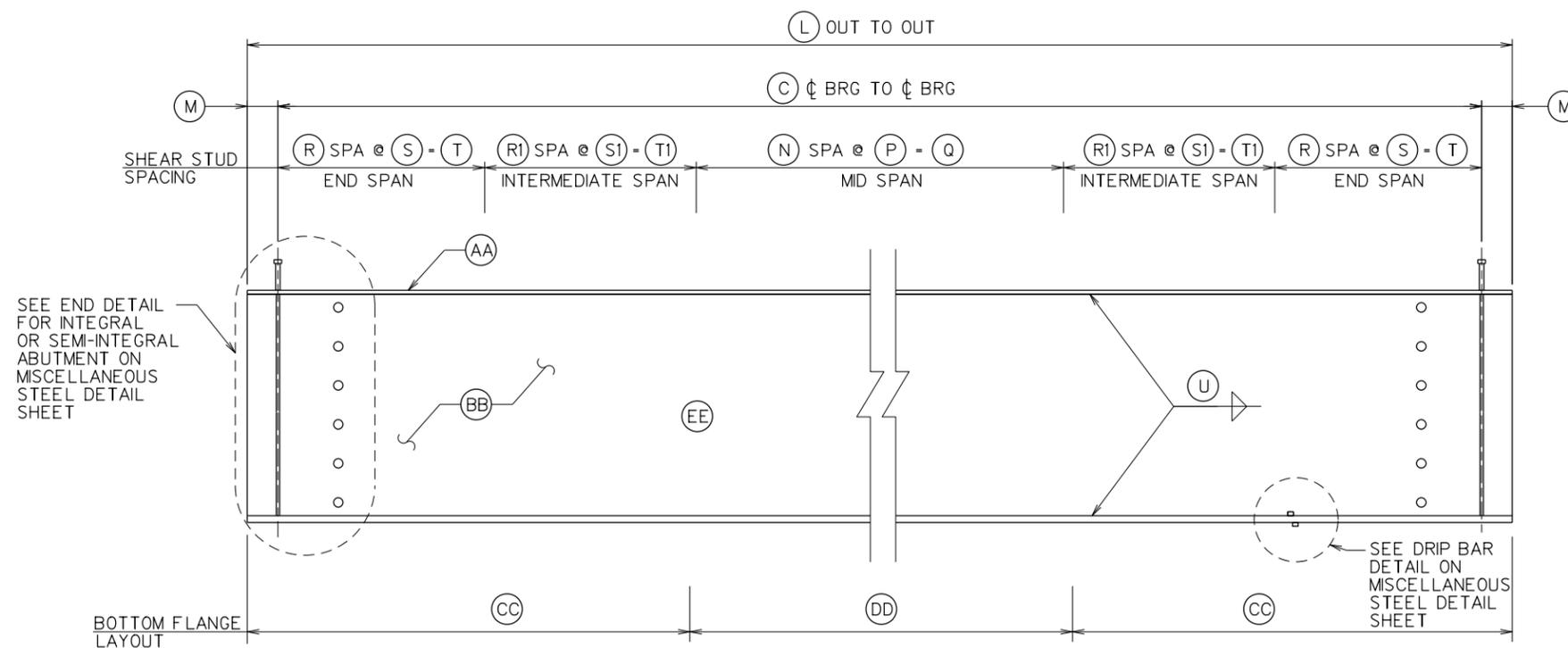
DRAFT



FRAMING PLAN

FRAMING PLAN CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
A	WP 1 (STATION)	_____
B	WP 2 (STATION)	_____
C	BEARING TO BEARING LENGTH	_____
D	NUMBER OF MID SPAN DIAPHRAGM SPACES	_____
E	MID SPAN DIAPHRAGM SPACING	_____
F	MID SPAN DIAPHRAGM LENGTH	_____
F1	END SPAN DIAPHRAGM LENGTH	_____
G	LOCATION OF FIELD SPLICE FROM WP 1	_____
G1	LOCATION OF FIELD SPLICE FROM WP 2	_____
H	GIRDER SPACING	_____
J	SKEW	_____

GIRDER CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
L	OVERALL GIRDER LENGTH	_____
M	BEGINNING OFFSET	_____
N	NUMBER OF MID SPAN SHEAR STUD SPACES	_____
P	SHEAR STUD SPACING	_____
Q	LENGTH OF MID SPAN SHEAR STUDS	_____
R	NUMBER OF END SPAN SHEAR STUD SPACES	_____
R1	NUMBER OF INTERMEDIATE SPAN SHEAR STUD SPACES	_____
S	END SPAN SHEAR STUD SPACING	_____
S1	INTERMEDIATE SPAN SHEAR STUD SPACING	_____
T	LENGTH OF END SPAN SHEAR STUD	_____
T1	LENGTH OF INTERMEDIATE SPAN SHEAR STUD	_____
U	WELD	_____



ELEVATION

STEEL GIRDER DIMENSIONS				
CODE	DESCRIPTION	PLATE DIM.	LENGTH	F <sub>y</sub> (KSI)
AA	TOP PLATE	_____	_____	_____
BB	WEB PLATE	_____	_____	_____
CC	BOTTOM PLATE END SPAN	_____	_____	_____
DD	BOTTOM PLATE MID SPAN	_____	_____	_____
EE	OR W BEAM	_____	_____	_____
QUANTITIES				
NUMBER OF SHEAR STUDS PER GIRDER				_____

PRINT DATE  
2-FEB-2022 14:51

NOT TO SCALE

NO.	REVISION	DATE	BY

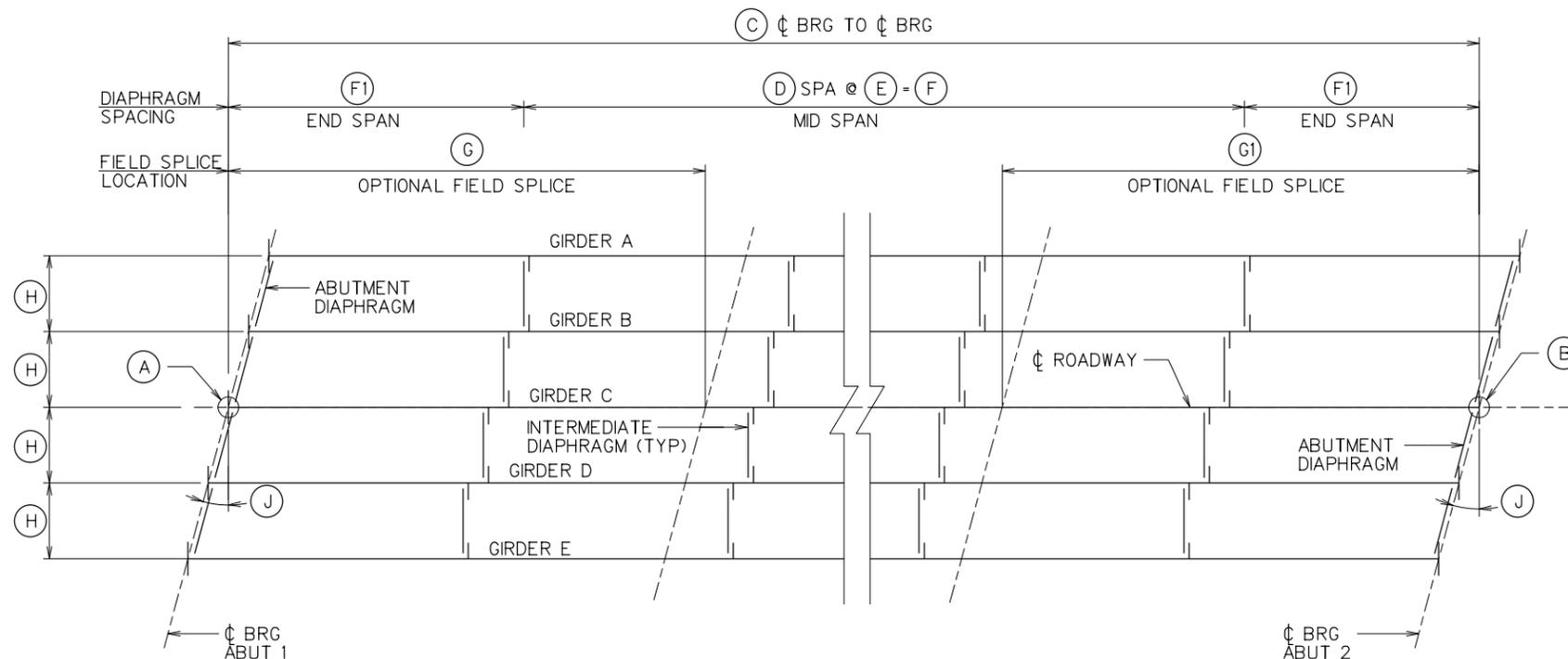
WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

DESIGNED	DATE	CHECKED	DATE
DRAWN	DATE	REVIEWED	DATE

STANDARD BRIDGE PLANS  
**FRAMING STEEL**  
**5 GIRDER RF SKEW STAGGER**  
SHEET NUMBER 1000FR8A

2-FEB-2022 14:51

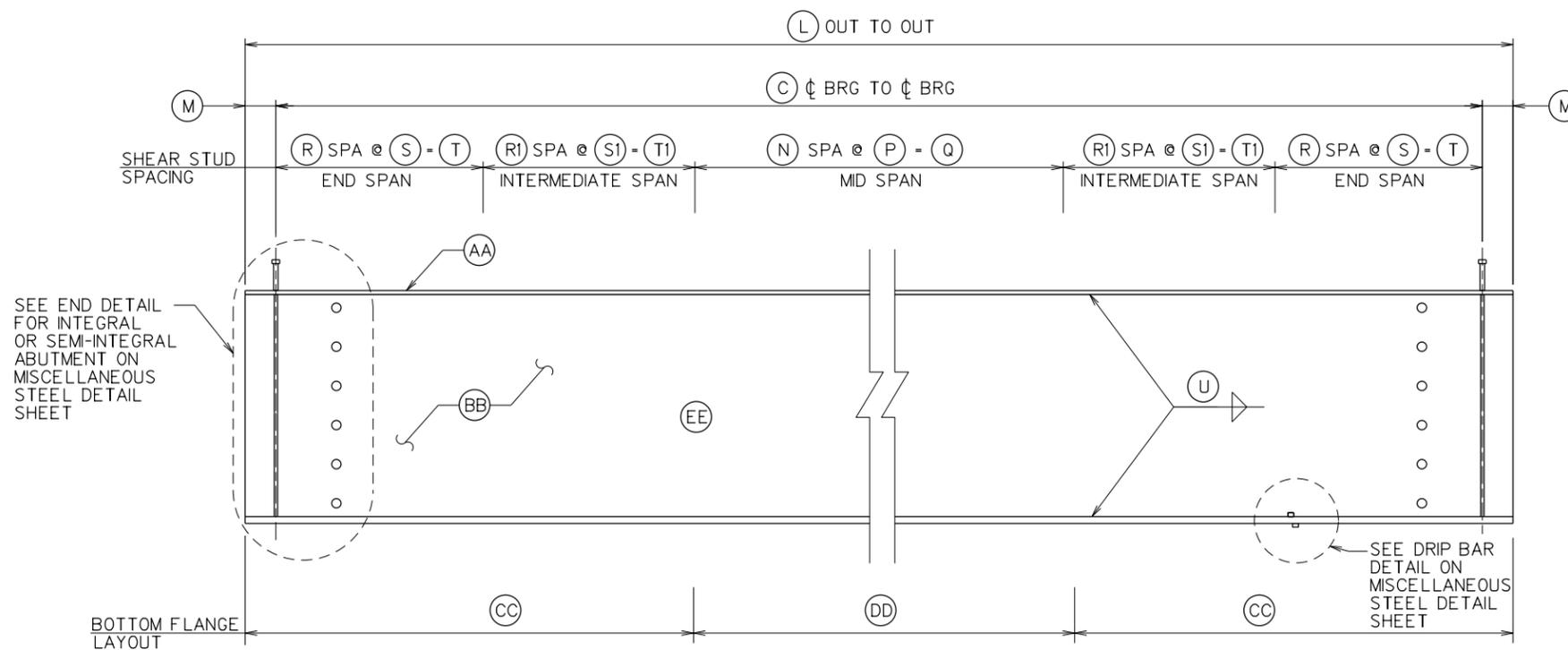
DRAFT



FRAMING PLAN

FRAMING PLAN CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
A	WP 1 (STATION)	_____
B	WP 2 (STATION)	_____
C	BEARING TO BEARING LENGTH	_____
D	NUMBER OF MID SPAN DIAPHRAGM SPACES	_____
E	MID SPAN DIAPHRAGM SPACING	_____
F	MID SPAN DIAPHRAGM LENGTH	_____
F1	END SPAN DIAPHRAGM LENGTH	_____
G	LOCATION OF FIELD SPLICE FROM WP 1	_____
G1	LOCATION OF FIELD SPLICE FROM WP 2	_____
H	GIRDER SPACING	_____
J	SKEW	_____

GIRDER CONTROL POINTS		
CODE	DESCRIPTION	DIMENSION
L	OVERALL GIRDER LENGTH	_____
M	BEGINNING OFFSET	_____
N	NUMBER OF MID SPAN SHEAR STUD SPACES	_____
P	SHEAR STUD SPACING	_____
Q	LENGTH OF MID SPAN SHEAR STUDS	_____
R	NUMBER OF END SPAN SHEAR STUD SPACES	_____
R1	NUMBER OF INTERMEDIATE SPAN SHEAR STUD SPACES	_____
S	END SPAN SHEAR STUD SPACING	_____
S1	INTERMEDIATE SPAN SHEAR STUD SPACING	_____
T	LENGTH OF END SPAN SHEAR STUD	_____
T1	LENGTH OF INTERMEDIATE SPAN SHEAR STUD	_____
U	WELD	_____



ELEVATION

STEEL GIRDER DIMENSIONS				
CODE	DESCRIPTION	PLATE DIM.	LENGTH	F <sub>y</sub> (KSI)
AA	TOP PLATE	_____	_____	_____
BB	WEB PLATE	_____	_____	_____
CC	BOTTOM PLATE END SPAN	_____	_____	_____
DD	BOTTOM PLATE MID SPAN	_____	_____	_____
EE	W BEAM	_____	_____	_____
OR				
QUANTITIES				
NUMBER OF SHEAR STUDS PER GIRDER				_____

PRINT DATE  
2-FEB-2022 14:53

NOT TO SCALE

NO.	REVISION	DATE	BY

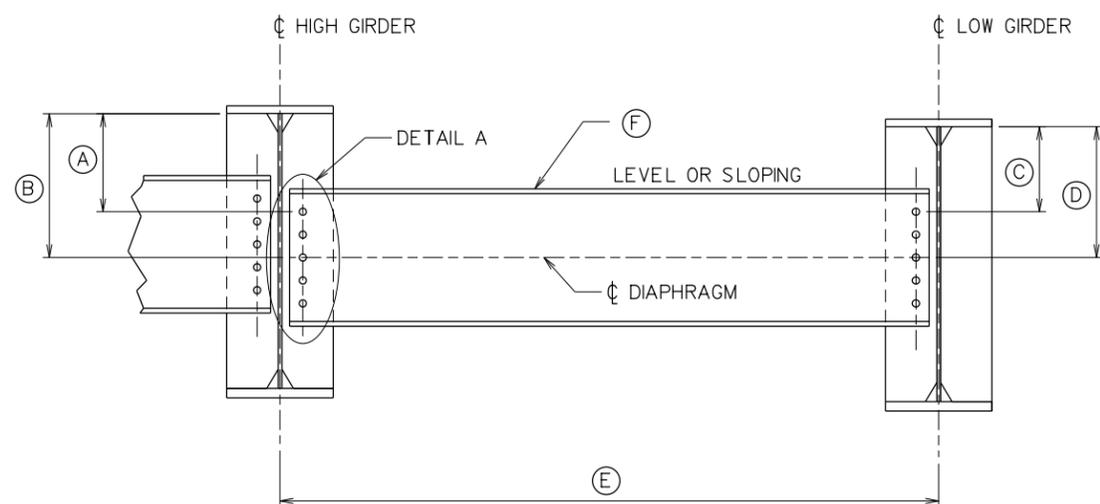
WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

DESIGNED	DATE	CHECKED	DATE
DRAWN	DATE	REVIEWED	DATE

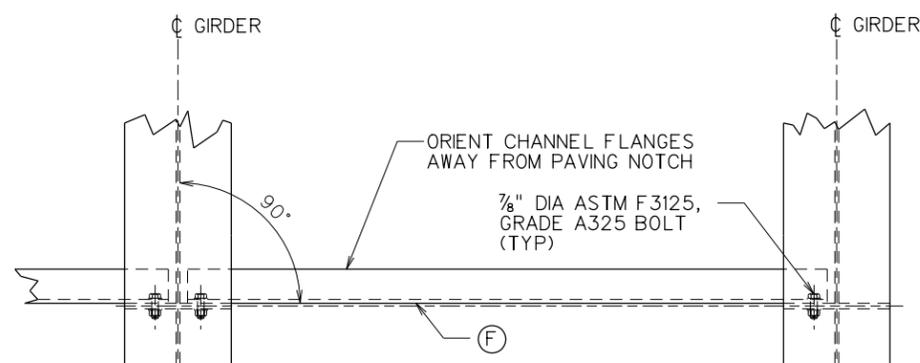
STANDARD BRIDGE PLANS  
**FRAMING STEEL**  
**5 GIRDER LF SKEW STAGGER**  
SHEET NUMBER 1000FR9A

2-FEB-2022 14:53

DRAFT



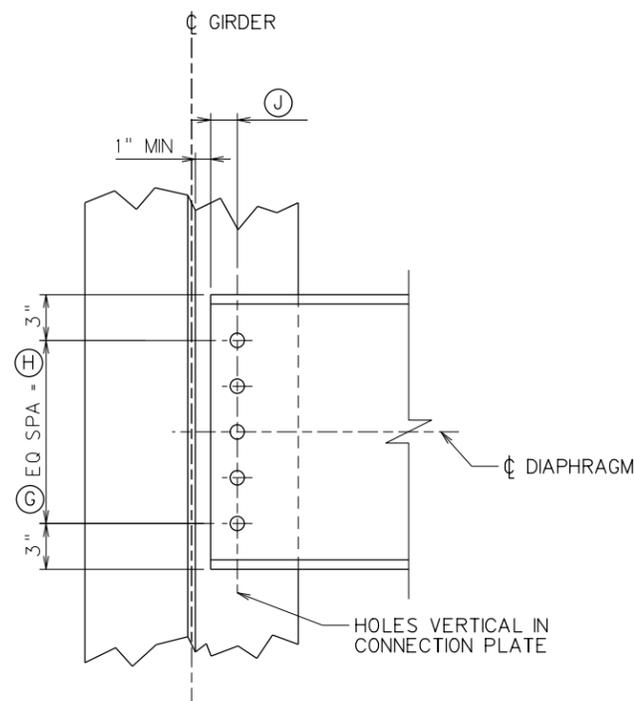
ABUTMENT DIAPHRAGM ELEVATION



ABUTMENT DIAPHRAGM PLAN

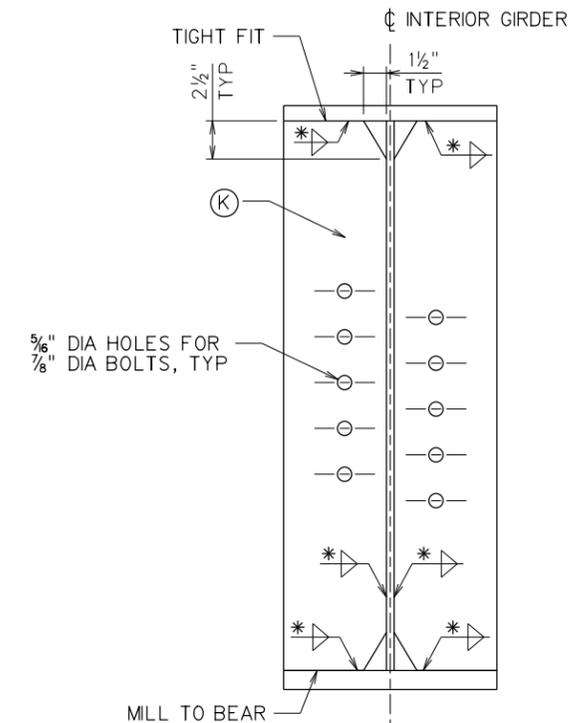
* WELD SIZES	
PLATE THICKNESS	WELD SIZE
< 3/4"	1/4"
≥ 3/4"	5/16"

NOTE: SEE FRAMING PLAN FOR GIRDER DIMENSIONS



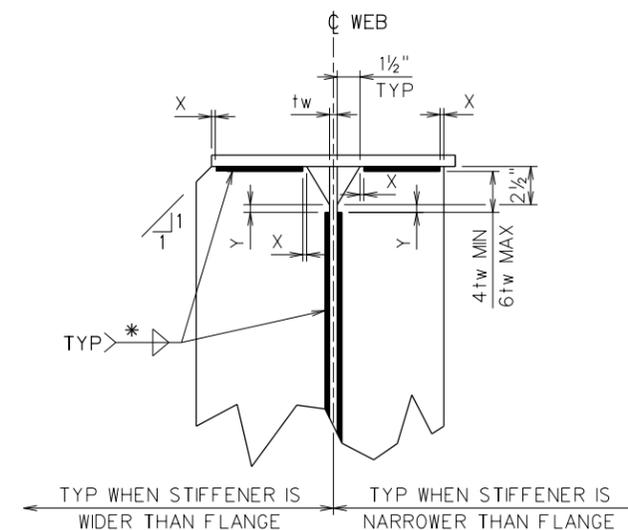
DETAIL A

CONTROL DIMENSIONS		
CODE	DESCRIPTION	DIMENSION
A	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER OF UPPER BOLT	_____
B	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER LINE OF DIAPHRAGM	_____
C	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER OF UPPER BOLT	_____
D	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER LINE OF DIAPHRAGM	_____
E	CENTER TO CENTER GIRDER SPACING	_____
F	DIAPHRAGM-C CHANNEL SIZE	_____
G	NUMBER OF BOLT SPACES	_____
H	DISTANCE OF TOTAL BOLT SPACING	_____
J	MINIMUM BOLT CLEARANCE FROM END	_____
K	PLATE SIZE WxHxt	_____



BEARING STIFFENER

LOOKING AHEAD STATION



WELDING DETAIL AT END OF STIFFENER OR CONNECTION PLATE

NOT TO SCALE

PRINT DATE  
2-FEB-2022 14:55

2-FEB-2022 14:55

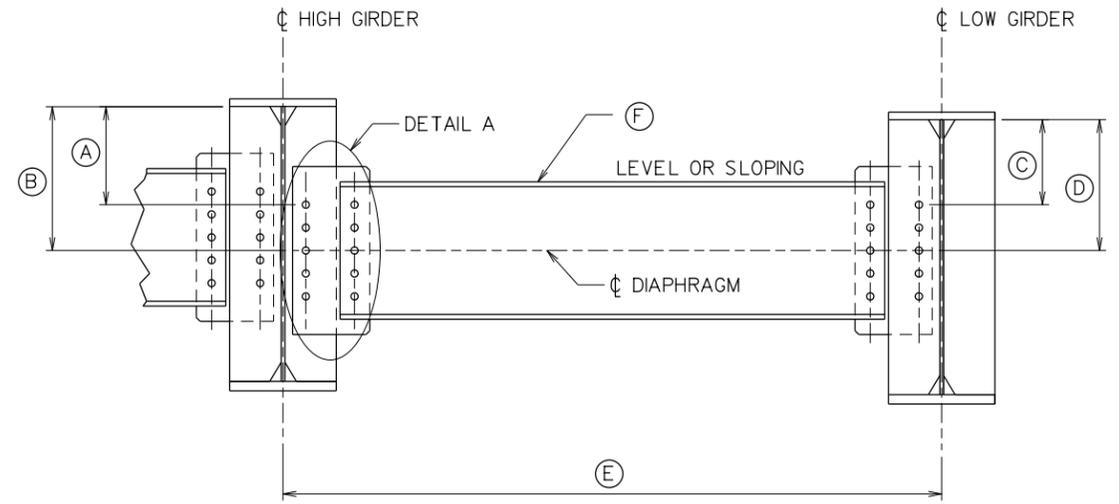
NO.	REVISION	DATE	BY

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

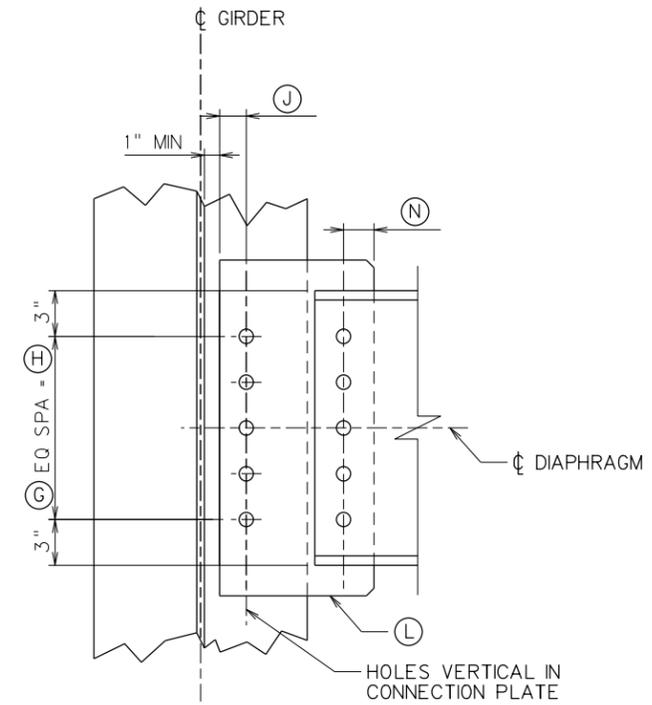
DESIGNED	DATE	CHECKED	DATE
DRAWN	DATE	REVIEWED	DATE

STANDARD BRIDGE PLANS  
ABUTMENT DIAPHRAGM  
C CHANNEL NO SKEW  
SHEET NUMBER 110QD11A

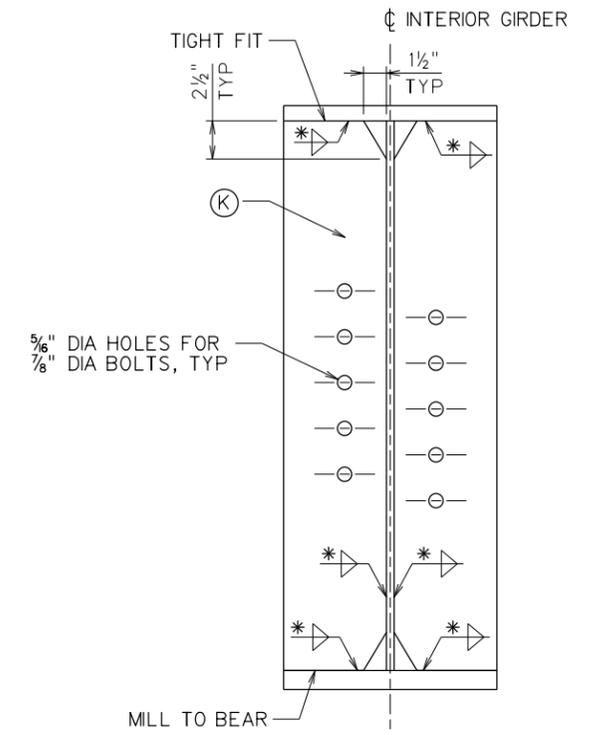
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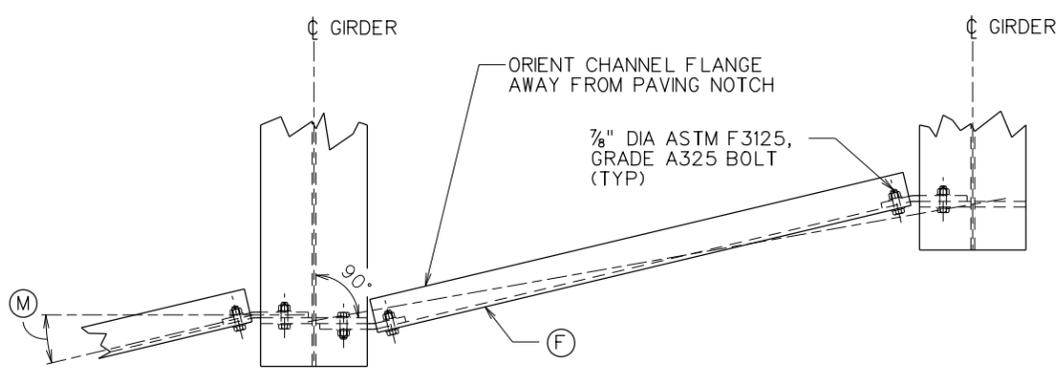
ABUTMENT DIAPHRAGM ELEVATION



DETAIL A

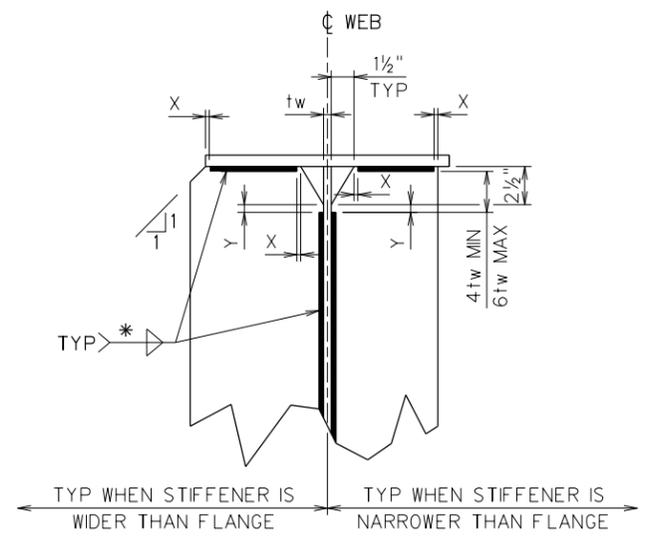


BEARING STIFFENER  
LOOKING AHEAD STATION



ABUTMENT DIAPHRAGM PLAN

CONTROL DIMENSIONS		
CODE	DESCRIPTION	DIMENSION
A	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER OF UPPER BOLT	_____
B	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER LINE OF DIAPHRAGM	_____
C	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER OF UPPER BOLT	_____
D	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER LINE OF DIAPHRAGM	_____
E	CENTER TO CENTER GIRDER SPACING	_____
F	DIAPHRAGM-C CHANNEL SIZE	_____
G	NUMBER OF BOLT SPACES	_____
H	DISTANCE OF TOTAL BOLT SPACING	_____
J	MINIMUM BOLT CLEARANCE FROM END	_____
K	PLATE SIZE WxHxt	_____
L	BENT PLATE SIZE WxHxt	_____
M	ANGLE OF BEND ON CONNECTION PLATE	_____
N	TO CENTER OF HOLE FOR BENT PLATE	_____



WELDING DETAIL AT END OF STIFFENER OR CONNECTION PLATE

* WELD SIZES	
PLATE THICKNESS	WELD SIZE
< 3/4"	1/4"
≥ 3/4"	5/16"

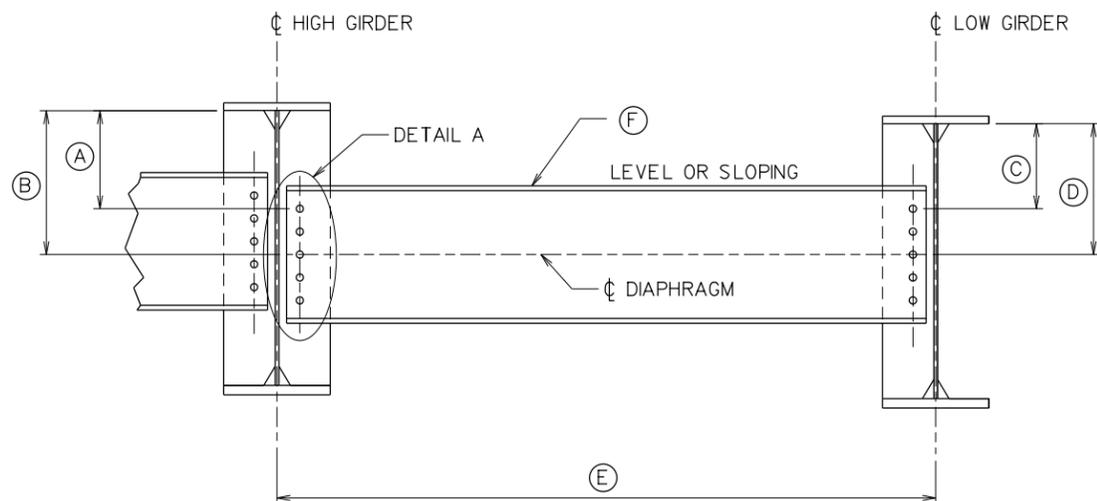
NOTE: SEE FRAMING PLAN FOR GIRDER DIMENSIONS AND SKEW ANGLES

PRINT DATE  
2-FEB-2022 14:57

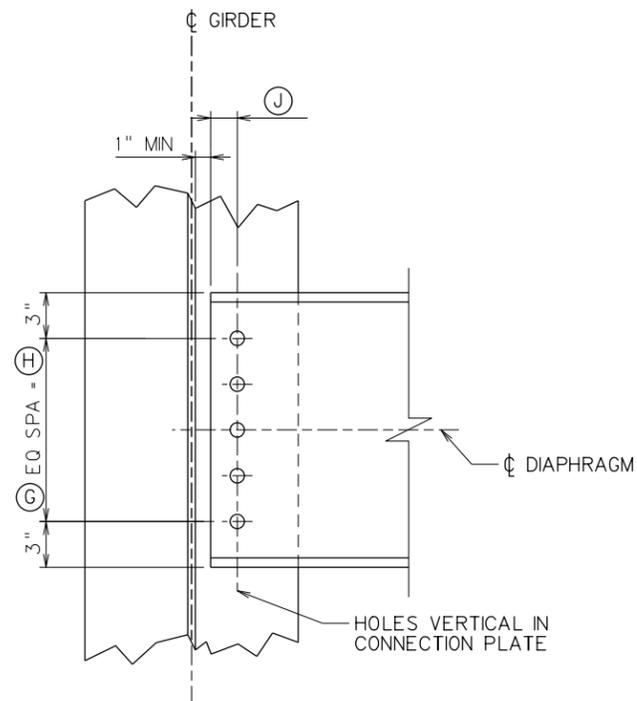
NOT TO SCALE

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION DIVISION OF HIGHWAYS				DESIGNED _____	DATE _____	CHECKED _____	DATE _____
				DRAWN _____	DATE _____	REVIEWED _____	DATE _____
NO.	REVISION	DATE	BY	STANDARD BRIDGE PLANS ABUTMENT DIAPHRAGM C CHANNEL SKEWED SHEET NUMBER 1100D12A			

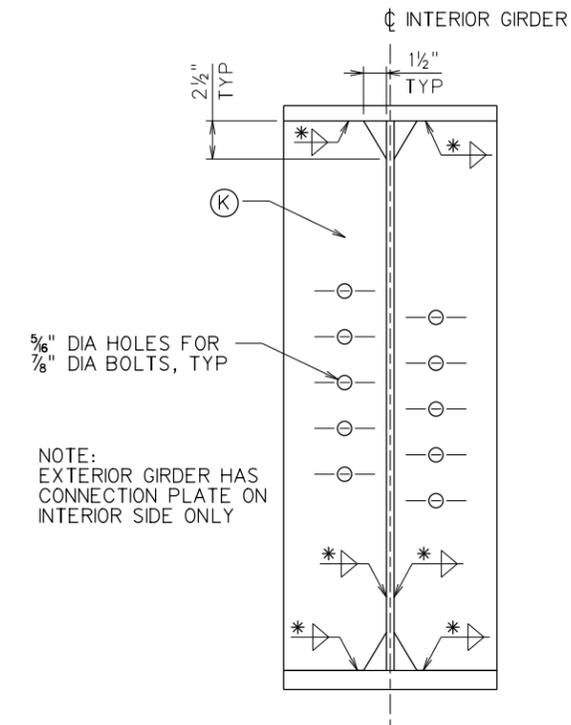
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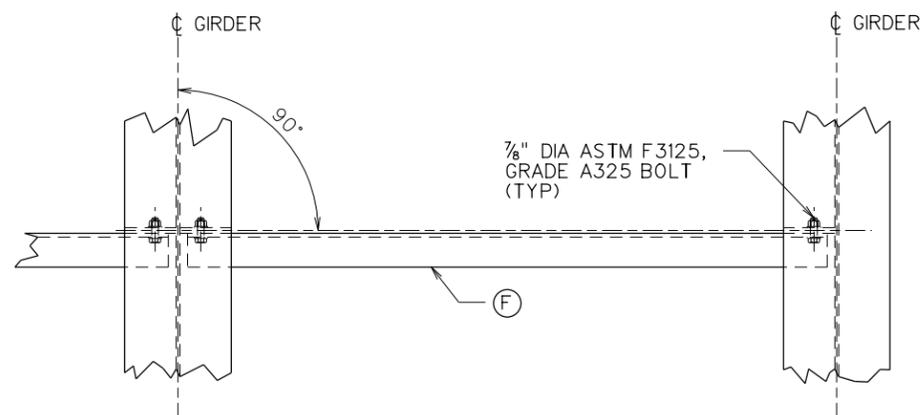
INTERMEDIATE DIAPHRAGM ELEVATION



DETAIL A

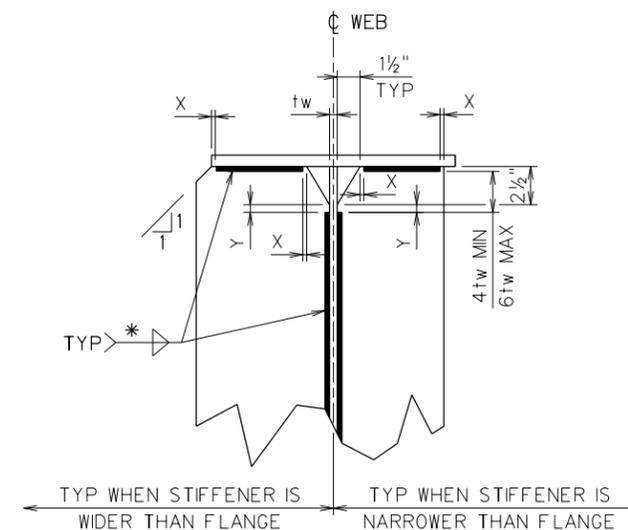


CONNECTION PLATE  
LOOKING AHEAD STATION



INTERMEDIATE DIAPHRAGM PLAN

CONTROL DIMENSIONS		
CODE	DESCRIPTION	DIMENSION
A	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER OF UPPER BOLT	_____
B	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER LINE OF DIAPHRAGM	_____
C	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER OF UPPER BOLT	_____
D	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER LINE OF DIAPHRAGM	_____
E	CENTER TO CENTER GIRDER SPACING	_____
F	DIAPHRAGM-C CHANNEL SIZE	_____
G	NUMBER OF BOLT SPACES	_____
H	DISTANCE OF TOTAL BOLT SPACING	_____
J	MINIMUM BOLT CLEARANCE FROM END	_____
K	PLATE SIZE WxHxt	_____



WELDING DETAIL AT END  
OF STIFFENER OR  
CONNECTION PLATE

* WELD SIZES	
PLATE THICKNESS	WELD SIZE
< 3/4"	1/4"
≥ 3/4"	5/16"

NOTE: SEE FRAMING PLAN FOR GIRDER DIMENSIONS

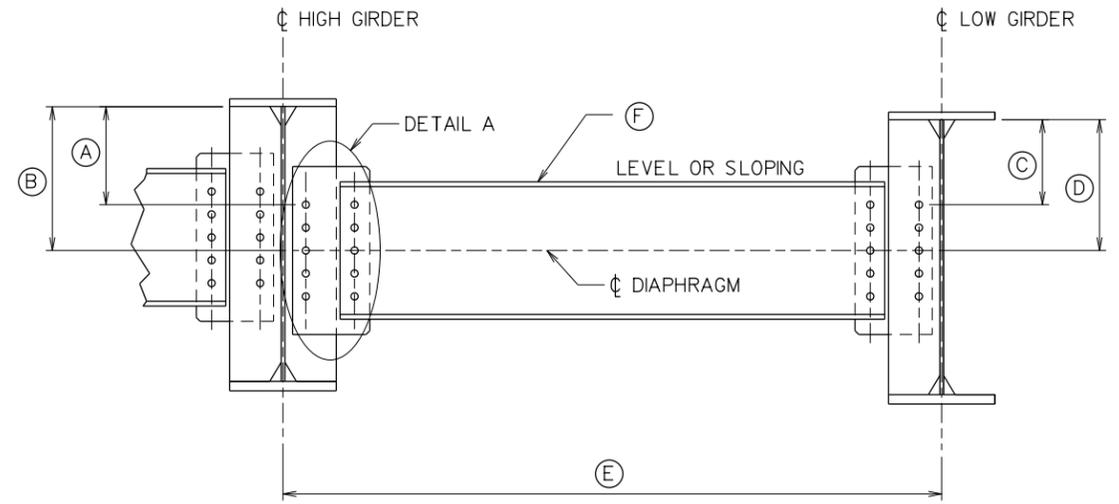
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2-FEB-2022 14:59

NOT TO SCALE

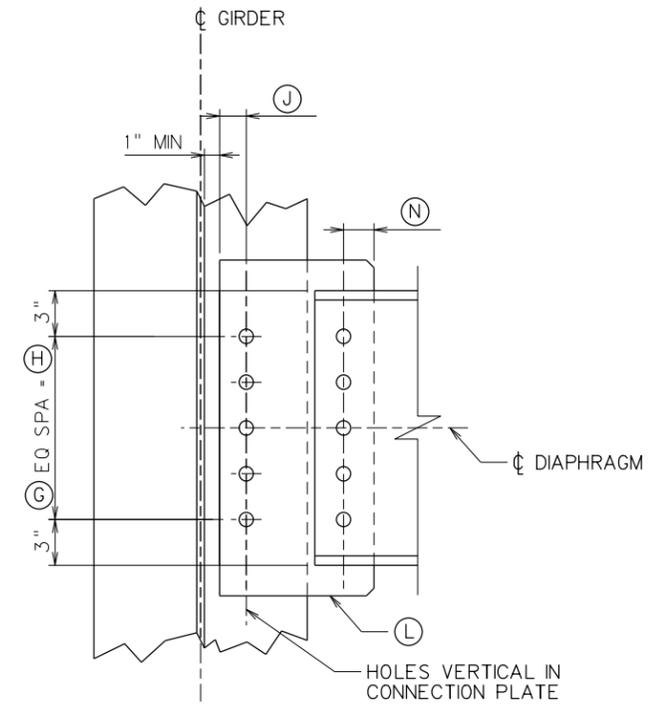
				WEST VIRGINIA DEPARTMENT OF TRANSPORTATION DIVISION OF HIGHWAYS				DESIGNED _____ DATE _____ CHECKED _____ DATE _____				DRAWN _____ DATE _____ REVIEWED _____ DATE _____			
NO.	REVISION	DATE	BY									STANDARD BRIDGE PLANS INTERMEDIATE DIAPHRAGM C CHANNEL SHEET NUMBER 110.0D14			

2-FEB-2022 14:59

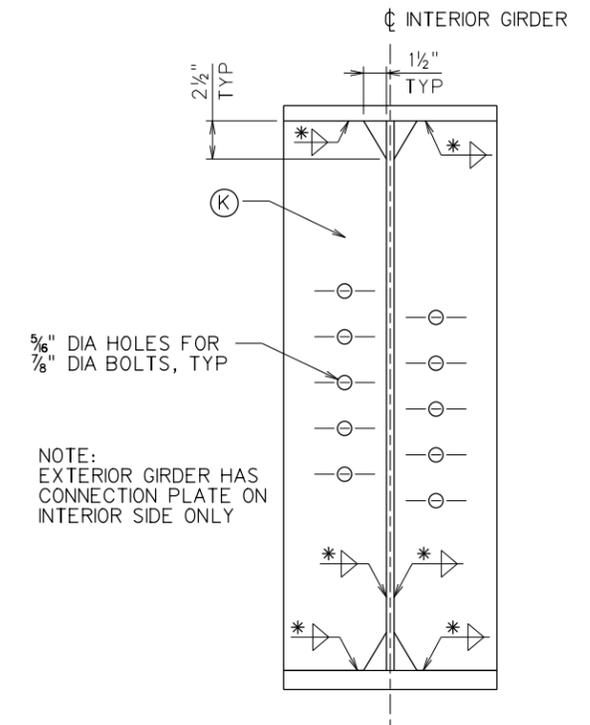
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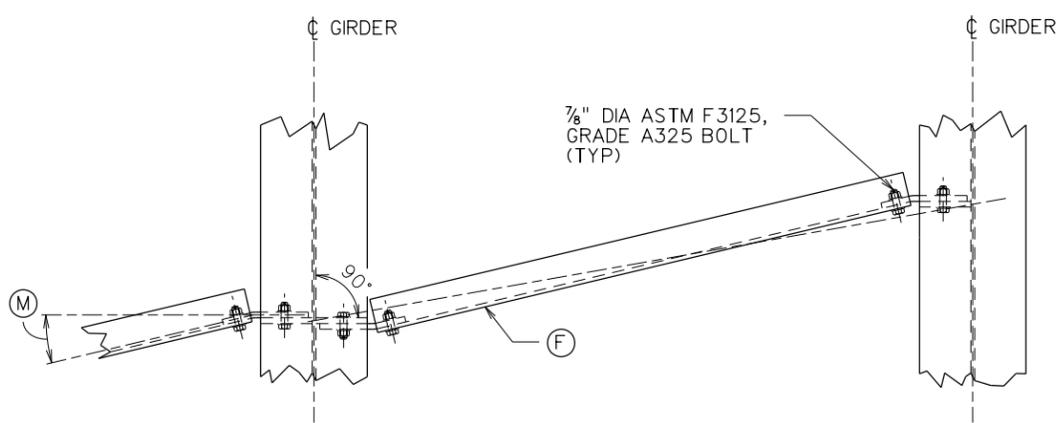
**INTERMEDIATE DIAPHRAGM ELEVATION**



**DETAIL A**

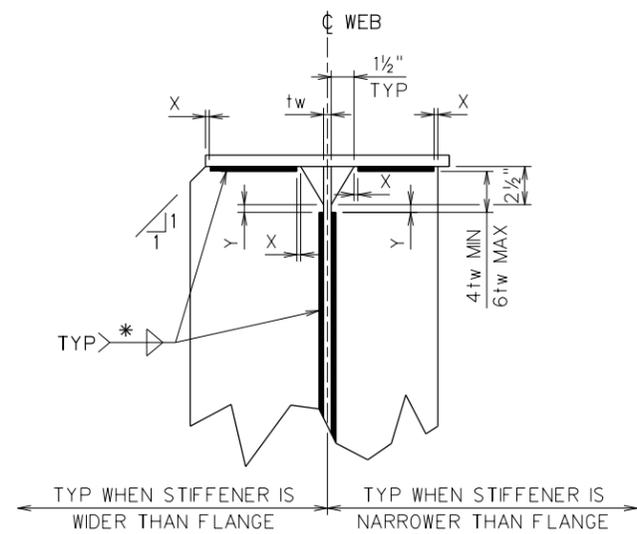


**CONNECTION PLATE**  
LOOKING AHEAD STATION



**INTERMEDIATE DIAPHRAGM PLAN**

CONTROL DIMENSIONS		
CODE	DESCRIPTION	DIMENSION
A	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER OF UPPER BOLT	_____
B	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER LINE OF DIAPHRAGM	_____
C	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER OF UPPER BOLT	_____
D	DISTANCE FROM BOTTOM OF TOP FLANGE TO CENTER LINE OF DIAPHRAGM	_____
E	CENTER TO CENTER GIRDER SPACING	_____
F	DIAPHRAGM-C CHANNEL SIZE	_____
G	NUMBER OF BOLT SPACES	_____
H	DISTANCE OF TOTAL BOLT SPACING	_____
J	MINIMUM BOLT CLEARANCE FROM END	_____
K	PLATE SIZE WxHxt	_____
L	BENT PLATE SIZE WxHxt	_____
M	ANGLE OF BEND ON CONNECTION PLATE	_____
N	TO CENTER OF HOLE FOR BENT PLATE	_____



**WELDING DETAIL AT END OF STIFFENER OR CONNECTION PLATE**

* WELD SIZES	
PLATE THICKNESS	WELD SIZE
< 3/4"	1/4"
≥ 3/4"	5/16"

NOTE: SEE FRAMING PLAN FOR GIRDER DIMENSIONS AND SKEW ANGLES

PRINT DATE  
2-FEB-2022 14:58

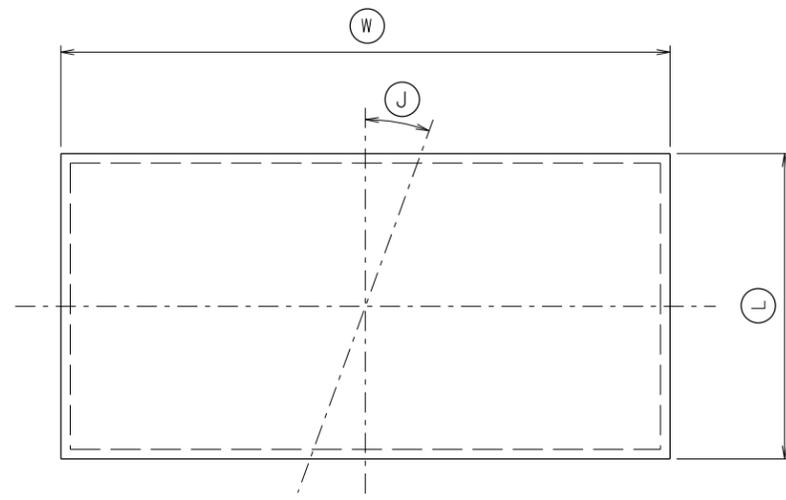
NOT TO SCALE

	WEST VIRGINIA DEPARTMENT OF TRANSPORTATION DIVISION OF HIGHWAYS	DESIGNED _____	DATE _____	CHECKED _____	DATE _____		STANDARD BRIDGE PLANS <b>INTERMEDIATE DIAPHRAGM C CHANNEL SKEWED</b> SHEET NUMBER 1100D14A
NO. _____	REVISION _____	DRAWN _____	DATE _____	REVIEWED _____	DATE _____		
DATE _____	BY _____						

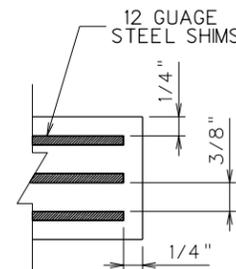
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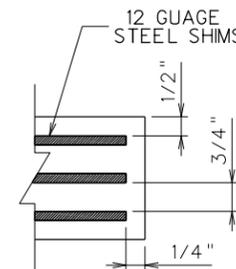
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**BEARING PLAN**



**DETAIL A**  
(TYPE A SHOWN)



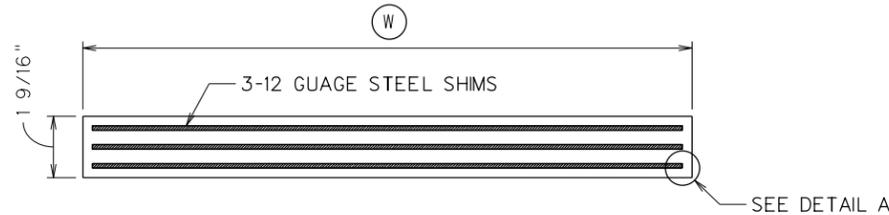
**DETAIL B**  
(TYPE A SHOWN)

LAMINATED BEARING PAD CONTROL DIMENSIONS					
CODE	DESCRIPTION	ABUT 1	PIER 1	PIER 2	ABUT 2
	PAD TYPE				
	NO. OF PADS				
J	SKEW				
L	PAD LENGTH				
W	PAD WIDTH				
	DESIGN REACTION (K)				
	DESIGN ROTATION (RAD)				
	DESIGN MOVEMENT (IN)				

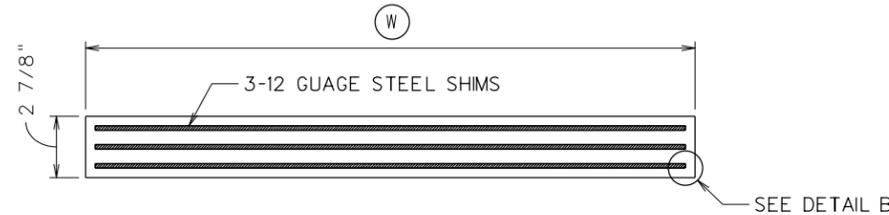
DESIGNS SHALL BE BY AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, METHOD B

**NOTES:**

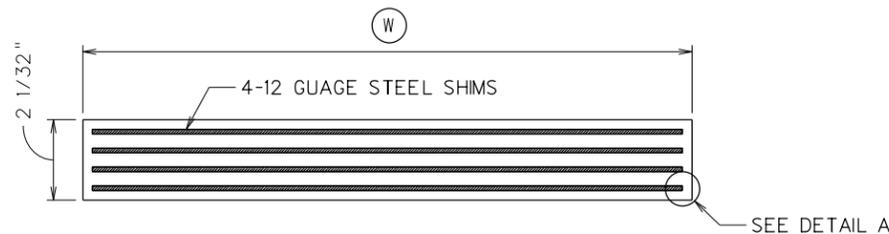
- ALL LAMINATED BEARING PAD SHALL BE OF 60 DUROMETER (HARDNESS) ELASTOMER. STEEL LAMINATE SHALL CONFORM TO ASTM A1011, GRADE 36 OR BETTER.
- LAMINATED BEARINGS SHALL BE SUBJECT TO TESTING REQUIREMENTS IN ACCORDANCE WITH AASHTO LRFD CONSTRUCTION SPECIFICATIONS.
- PRIOR TO SHIPMENT, LAMINATED BEARING ASSEMBLIES SHALL BE FULLY ASSEMBLED, BLOCKED AND SECURED INTO POSITION, AND WRAPPED WITH A WATERPROOFING COVERING. THE BEARING ASSEMBLY SHALL NOT BE UNWRAPPED UNTIL THE BEARING COMPONENTS ARE READY TO BE SET INTO THEIR FINAL POSITION.
- BRIDGE SEATS ON WHICH BEARING PADS WILL BE MOUNTED SHALL BE FINISHED TO A TRULY LEVEL PLANE AT THE EXACT REQUIRED ELEVATION. IF FULL CONTACT IS NOT ACHIEVED AFTER THE DECK IS IN PLACE, FIELD ADJUSTMENTS OR MODIFICATIONS SHALL BE MADE BY THE CONTRACTOR TO ENSURE FULL CONTACT SUBJECT TO THE APPROVAL OF THE ENGINEER.
- EPOXY GRIT COATING SHALL BE APPLIED TO ALL STEEL SURFACES CONTACTING THE BEARING PAD AND EXTEND 1/2" IN ALL DIRECTIONS BEYOND THE PAD'S LIMITS. THE EPOXY GRIT SHALL BE INSTALLED IN ACCORDANCE WITH THE EPOXY MANUFACTURER'S INSTRUCTIONS. ALLOW THE EPOXY TO FULLY CURE FOR THE MINIMUM TIME RECOMMENDED BY THE MANUFACTURER THEN REMOVE ANY LOOSE GRIT BEFORE BEARING INSTALLATION. EPOXY GRIT SHALL MEET THE REQUIREMENTS OF SSPC AB1 ABRASIVE SPECIFICATIONS #1 - MINERAL & SLAG ABRASIVES, TYPE 2 OR BETTER.
- WELDING WHILE THE LAMINATED BEARING PAD IS IN CONTACT WITH METAL IS DISCOURAGED. WHEN WELDING IS REQUIRED, TEMPERATURE INDICATING WAX PENS OR OTHER SUITABLE MEAN SHALL BE USED TO ENSURE THE PAD NOT BE EXPOSED TO TEMPERATURES GREATER THAN 250° F. ANY DAMAGE TO THE PAD DUE TO WELDING WILL BE CAUSE FOR REJECTION.
- ALL BEARINGS SHALL BE MARKED PRIOR TO SHIPPING. THE MARKS SHALL INCLUDE THE BEARING LOCATION AND A DIRECTION ARROW THAT POINTS UP-STATION. ALL MARKS SHALL BE PERMANENT AND SHALL BE VISIBLE AFTER THE BEARING IS INSTALLED.
- ALL DESIGN PARAMETERS REQUIRED WITHIN THE LAMINATED BEARING PAD CONTROL DIMENSIONS TABLE SHALL BE UNFACTORED (SERVICE LIMIT STATE) AND INCLUDE IMPACT.
- A STATIC COEFFICIENT OF FRICTION OF 0.20 SHALL BE USED IN THE DESIGN TO VERIFY LAMINATED BEARINGS ARE NOT SUBJECT TO SLIP UNDER SERVICE AND STRENGTH LIMIT STATES. ADDITIONAL CONSIDERATION MAY BE WARRANTED FOR STEEP GRADES.



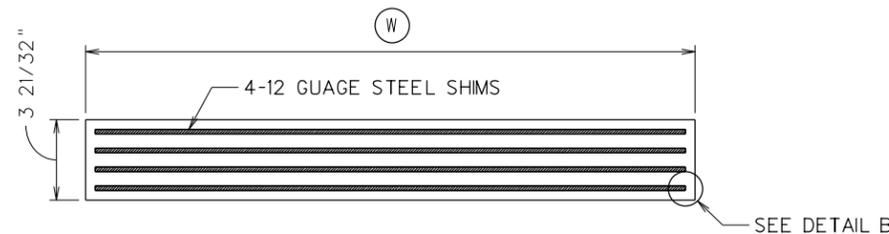
**TYPICAL SECTION - TYPE A1**



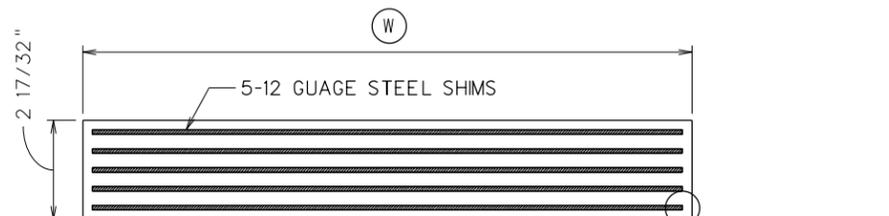
**TYPICAL SECTION - TYPE A2**



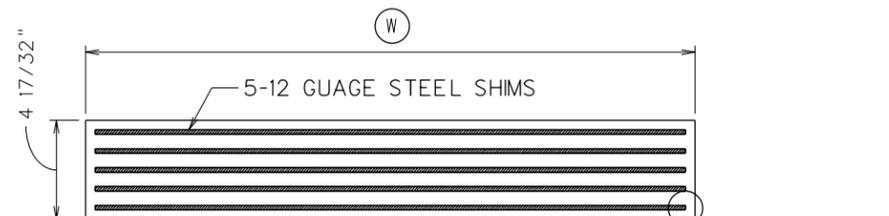
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**TYPICAL SECTION - TYPE B2**



**TYPICAL SECTION - TYPE C1**



**TYPICAL SECTION - TYPE C2**

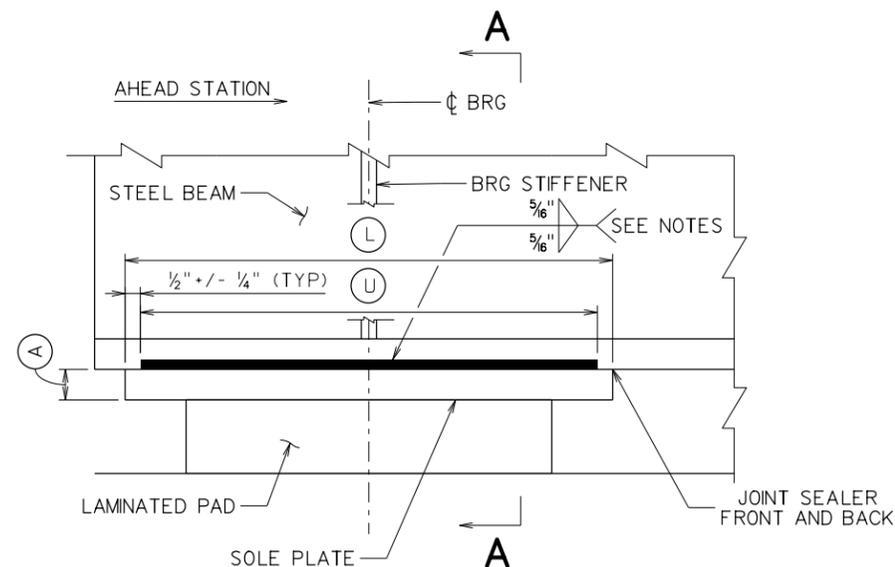
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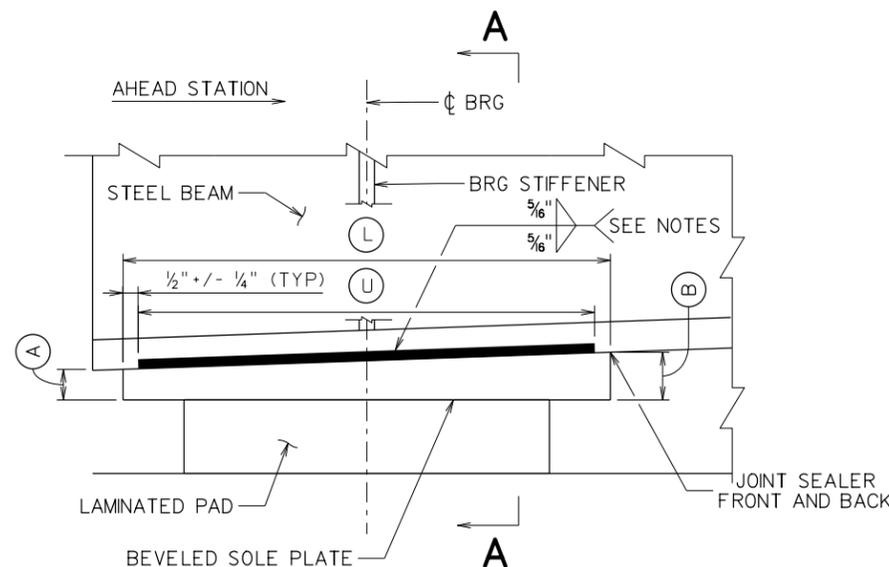
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				WEST VIRGINIA DEPARTMENT OF TRANSPORTATION DIVISION OF HIGHWAYS				DESIGNED _____ DATE _____ CHECKED _____ DATE _____			
								DRAWN _____ DATE _____ REVIEWED _____ DATE _____			
NO.	REVISION	DATE	BY					STANDARD BRIDGE PLANS <b>LAMINATED ELASTOMERIC BEARING DETAILS</b> SHEET NUMBER 18.00B2			

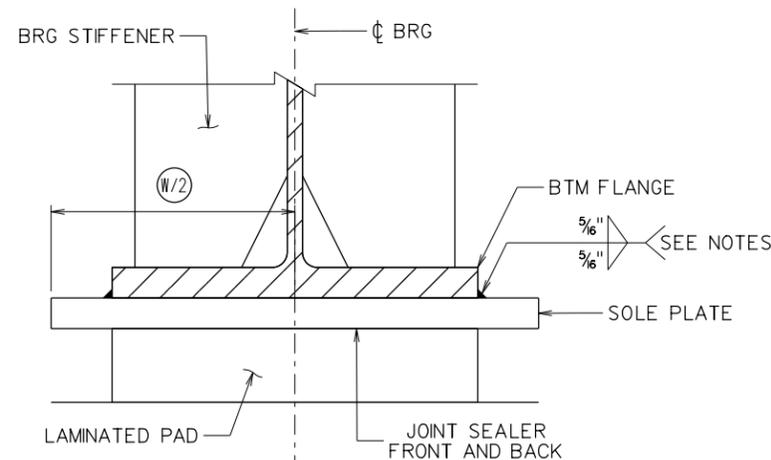
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**NON-GUIDED SIDE ELEVATION**  
(SLOPES < 0.50% ALONG BEAM)



**NON-GUIDED SIDE ELEVATION**  
(SLOPES > 0.50% ALONG BEAM)



**SECTION A-A**

BEARING RESTRAINTS CONTROL DIMENSIONS					
CODE	DESCRIPTION	ABUT 1	PIER 1	PIER 2	ABUT 2
A	PLATE THICKNESS (IN)	_____	_____	_____	_____
B	PLATE THICKNESS (IN)	_____	_____	_____	_____
L	PLATE LENGTH (IN)	_____	_____	_____	_____
W	PLATE WIDTH (IN)	_____	_____	_____	_____
U	WELD LENGTH (IN)	_____	_____	_____	_____

**NOTES:**

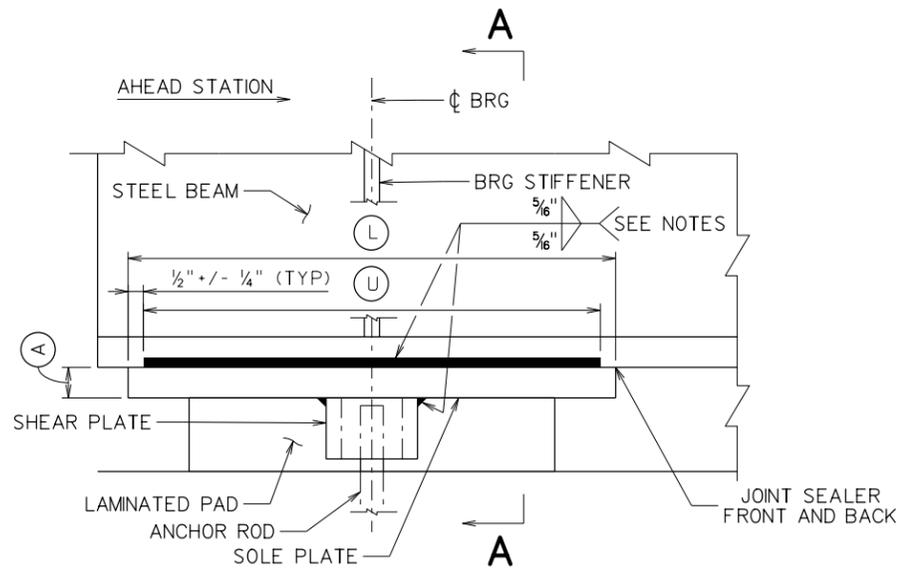
- UNLESS OTHERWISE NOTED, SOLE PLATES SHALL BE MADE OF THE SAME STEEL AS THE ABUTTING BOTTOM FLANGE. WHERE DIFFERING MATERIALS ARE SPECIFIED WELDABILITY OF THE MATERIALS SHALL BE VERIFIED IN ACCORDANCE WITH AWS D1.5 - BRIDGE WELDING CODE.
- SOLE PLATES SHALL HAVE A MINIMUM THICKNESS OF 3/4" AND MINIMUM BEVEL, IF APPLICABLE, OF 0.01 RADIAN.
- EPOXY GRIT COATING SHALL BE APPLIED TO ALL STEEL SURFACES CONTACTING THE BEARING PAD AND EXTEND 1/2" IN ALL DIRECTIONS BEYOND THE PAD'S LIMITS. THE EPOXY GRIT SHALL BE INSTALLED IN ACCORDANCE WITH THE EPOXY MANUFACTURER'S INSTRUCTIONS. ALLOW THE EPOXY TO FULLY CURE FOR THE MINIMUM TIME RECOMMENDED BY THE MANUFACTURER THEN REMOVE ANY LOOSE GRIT BEFORE BEARING INSTALLATION. EPOXY GRIT SHALL MEET THE REQUIREMENTS OF SSPC AB1 ABRASIVE SPECIFICATIONS #1 - MINERAL & SLAG ABRASIVES, TYPE 2 OR BETTER.
- SHOP WELDING OF SOLE PLATES IS PREFERRED. WHERE FIELD WELDING IS NECESSARY, ALL COATINGS SHALL BE REMOVED BY GRINDING IN THE VICINITY OF THE WELD TO A BRIGHT METAL SURFACE AT LEAST 1 INCH IN ALL DIRECTIONS OF THE AREA TO BE WELDED. THE WELD AND DAMAGED AREA OF PAINTED SURFACE SHALL BE CLEANED AND REPAIRED AS SPECIFIED BY THE ENGINEER.
- WELDING WHILE THE LAMINATED BEARING PAD IS IN CONTACT WITH METAL IS DISCOURAGED. IF REQUIRED, TEMPERATURE INDICATING WAX PENS OR OTHER SUITABLE MEAN SHALL BE USED TO ENSURE THE PAD IS NOT EXPOSED TO TEMPERATURES GREATER THAN 250° F. ANY DAMAGE TO THE PAD DUE TO WELDING WILL BE CAUSE FOR REJECTION.
- THE ENGINEER SHALL VERIFY WELDS ARE CAPABLE OF WITHSTANDING ALL SUBJECT FORCE EFFECTS.
- THE DETAILS AND NOTES HEREIN ARE NOT APPLICABLE FOR VULCANIZED BEARINGS. REFER TO AASHTO-NSBA STEEL BRIDGE COLLABORATION G9.1 FOR MORE INFORMATION.

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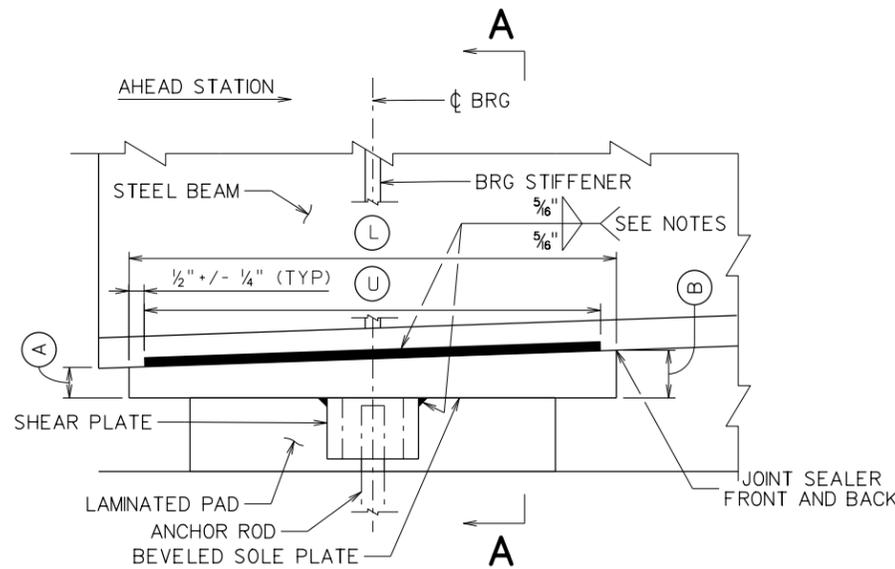
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				WEST VIRGINIA DEPARTMENT OF TRANSPORTATION DIVISION OF HIGHWAYS				DESIGNED _____ DATE _____ CHECKED _____ DATE _____				DRAWN _____ DATE _____ REVIEWED _____ DATE _____			
NO.	REVISION	DATE	BY									STANDARD BRIDGE PLANS <b>STEEL BEAM</b> <b>NON-GUIDED BEARING RESTRAINTS</b> SHEET NUMBER 18.00BR1			

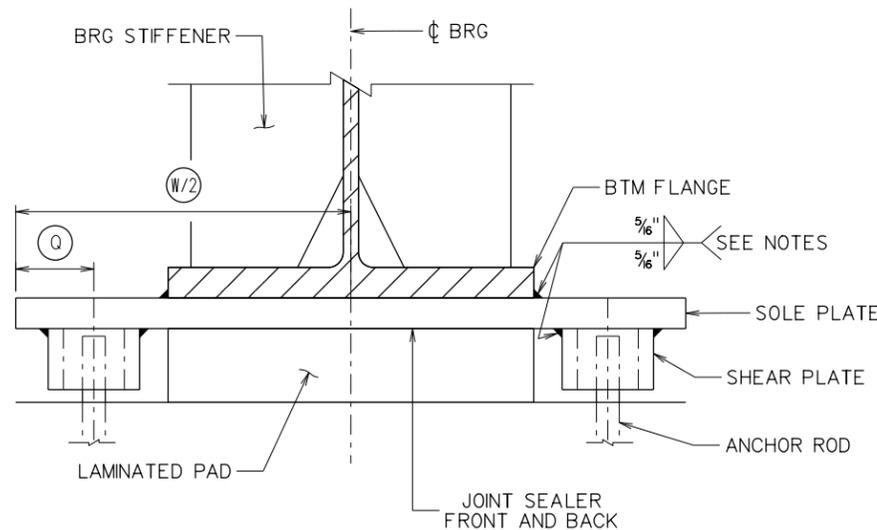
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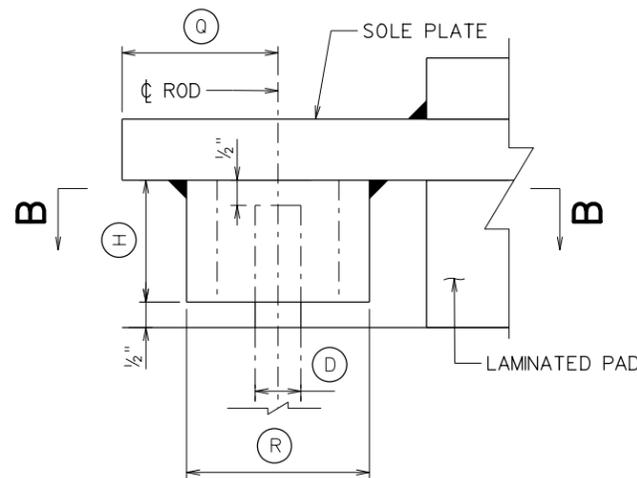
**GUIDED SIDE ELEVATION**  
(SLOPES < 0.50% ALONG BEAM)



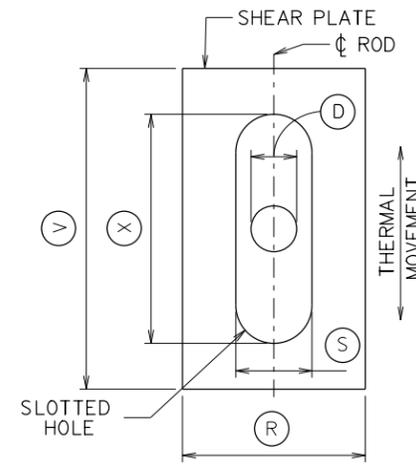
**GUIDED SIDE ELEVATION**  
(SLOPES > 0.50% ALONG BEAM)



**SECTION A-A**



**SHEAR KEY DETAIL**



**SECTION B-B**

BEARING RESTRAINTS CONTROL DIMENSIONS					
CODE	DESCRIPTION	ABUT 1	PIER 1	PIER 2	ABUT 2
A	PLATE THICKNESS (IN)				
B	PLATE THICKNESS (IN)				
L	PLATE LENGTH (IN)				
W	PLATE WIDTH (IN)				
U	WELD LENGTH (IN)				
D	ANCHOR ROD DIAMETER (IN)				
Q	SHEAR PLATE OFFSET (IN)				
H	SHEAR PLATE HEIGHT (IN)				
R	SHEAR PLATE WIDTH (IN)				
S	SLOTTED HOLE WIDTH (IN)				
V	SHEAR PLATE LENGTH (IN)				
X	SLOTTED HOLE LENGTH (IN)				
FACTORED LATERAL LOAD (KIP)					

**NOTES:**

- UNLESS OTHERWISE NOTED, SOLE PLATES SHALL BE MADE OF THE SAME STEEL AS THE ABUTTING BOTTOM FLANGE. WHERE DIFFERING MATERIALS ARE SPECIFIED WELDABILITY OF THE MATERIALS SHALL BE VERIFIED IN ACCORDANCE WITH AWS D1.5 - BRIDGE WELDING CODE.
- SOLE PLATES SHALL HAVE A MINIMUM THICKNESS OF 3/4" AND MINIMUM BEVEL, IF APPLICABLE, OF 0.01 RADIAN.
- EPOXY GRIT COATING SHALL BE APPLIED TO ALL STEEL SURFACES CONTACTING THE BEARING PAD AND EXTEND 1/2" IN ALL DIRECTIONS BEYOND THE PAD'S LIMITS. THE EPOXY GRIT SHALL BE INSTALLED IN ACCORDANCE WITH THE EPOXY MANUFACTURER'S INSTRUCTIONS. ALLOW THE EPOXY TO FULLY CURE FOR THE MINIMUM TIME RECOMMENDED BY THE MANUFACTURER THEN REMOVE ANY LOOSE GRIT BEFORE BEARING INSTALLATION. EPOXY GRIT SHALL MEET THE REQUIREMENTS OF SSPC AB1 ABRASIVE SPECIFICATIONS #1 - MINERAL & SLAG ABRASIVES, TYPE 2 OR BETTER.
- SHOP WELDING OF SOLE PLATES IS PREFERRED. WHERE FIELD WELDING IS NECESSARY, ALL COATINGS SHALL BE REMOVED BY GRINDING IN THE VICINITY OF THE WELD TO A BRIGHT METAL SURFACE AT LEAST 1 INCH IN ALL DIRECTIONS OF THE AREA TO BE WELDED. THE WELD AND DAMAGED AREA OF PAINTED SURFACE SHALL BE CLEANED AND REPAIRED AS SPECIFIED BY THE ENGINEER.
- WELDING WHILE THE LAMINATED BEARING PAD IS IN CONTACT WITH METAL IS DISCOURAGED. IF REQUIRED, TEMPERATURE INDICATING WAX PENS OR OTHER SUITABLE MEAN SHALL BE USED TO ENSURE THE PAD IS NOT EXPOSED TO TEMPERATURES GREATER THAN 250° F. ANY DAMAGE TO THE PAD DUE TO WELDING WILL BE CAUSE FOR REJECTION.
- THE ENGINEER SHALL VERIFY WELDS ARE CAPABLE OF WITHSTANDING ALL SUBJECT FORCE EFFECTS.
- THE DETAILS AND NOTES HEREIN ARE NOT APPLICABLE FOR VULCANIZED BEARINGS. REFER TO AASHTO-NSBA STEEL BRIDGE COLLABORATION G9.1 FOR MORE INFORMATION.
- UNLESS OTHERWISE NOTED, SHEAR PLATES SHALL BE MADE OF THE SAME STEEL AS THE ABUTTING SOLE PLATE. WHERE DIFFERING MATERIALS ARE SPECIFIED WELDABILITY OF THE MATERIALS SHALL BE VERIFIED IN ACCORDANCE WITH AWS D1.5 - BRIDGE WELDING CODE.
- SHEAR PLATES SHALL BE WELDED TO THE SOLE PLATE ALL AROUND. THE ENGINEER SHALL VERIFY WELDS ARE CAPABLE OF WITHSTANDING ALL SUBJECT FORCE EFFECTS.
- THE WIDTH OF THE SLOTTED HOLE SHALL BE AS RECOMMENDED FOR ANCHOR RODS IN AMERICAN INSTITUTE OF STEEL CONSTRUCTION (AISC) UNLESS OTHERWISE NOTED. OVERSIZING HOLE WIDTH SHOULD BE AVOIDED.
- THE LENGTH OF THE SLOTTED HOLE SHALL BE NO LESS THAN THE COMPUTED THERMAL MOVEMENT AT THE BEARING.
- ANCHOR RODS SHALL COMPLY WITH AASHTO M111, GRADE 55 AND BE FULL LENGTH HOT DIPPED GALVANIZED IN ACCORDANCE WITH AASHTO M232. THE ENGINEER SHALL SIZE ANCHOR RODS FOR ALL FORCE EFFECTS. ANCHOR ROD FLEXURE SHALL BE CONSIDERED.

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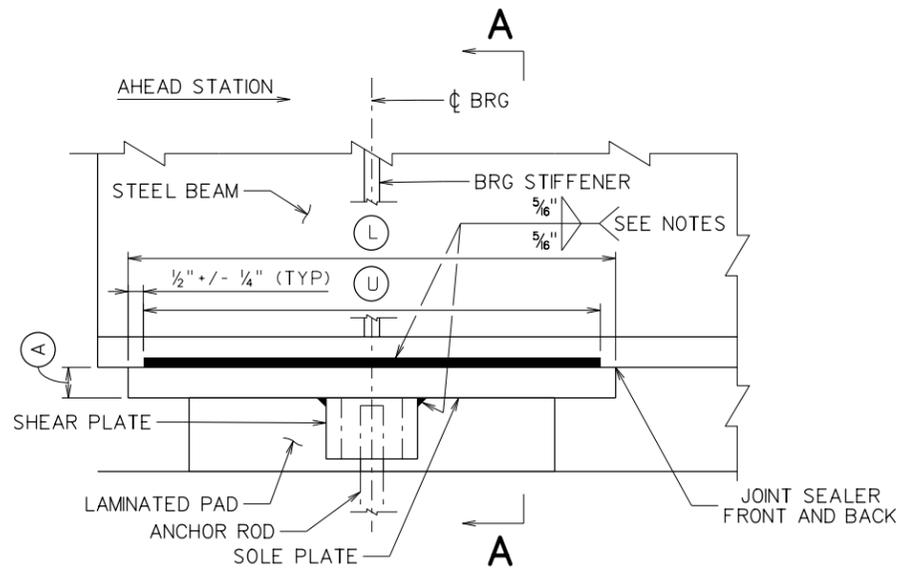
WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

DESIGNED	DATE	CHECKED	DATE
DRAWN	DATE	REVIEWED	DATE

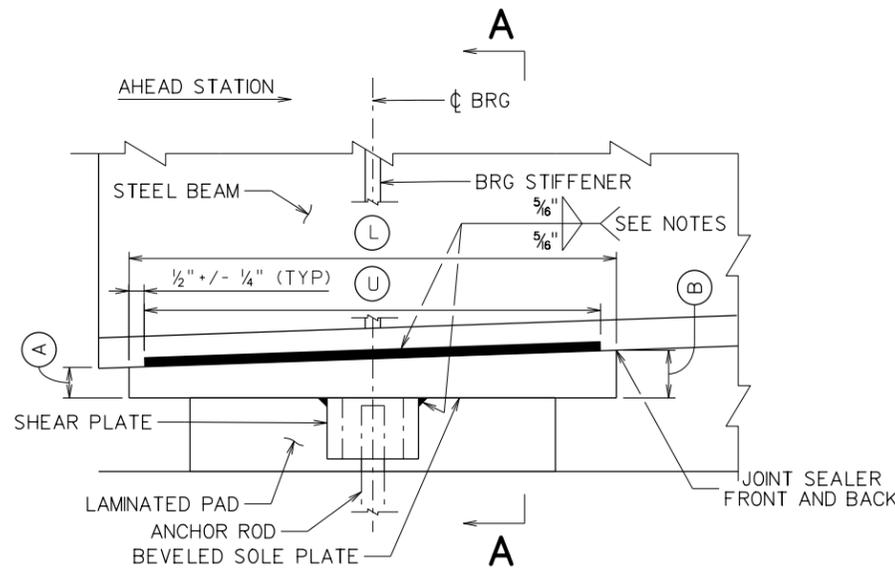

STANDARD BRIDGE PLANS  
**STEEL BEAM**  
**GUIDED BEARING RESTRAINTS**  
SHEET NUMBER 1800BR2

7-FEB-2022 07:40

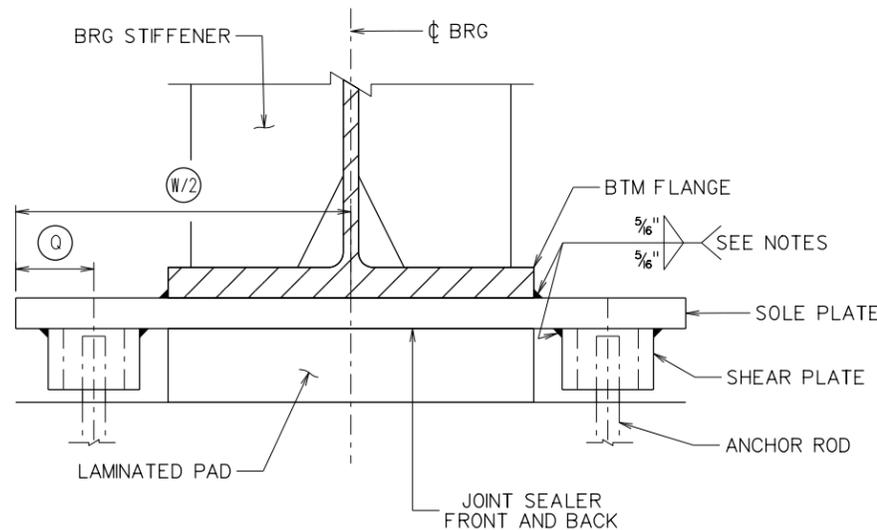
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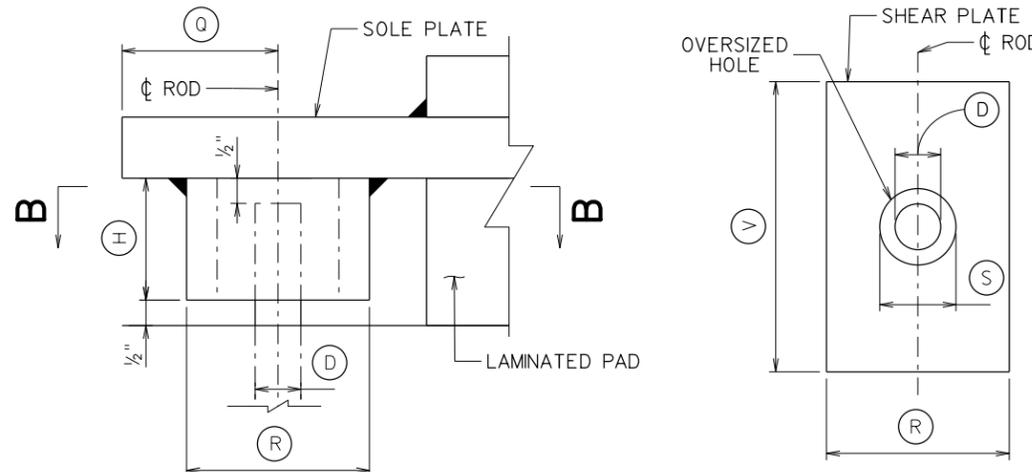
**FIXED SIDE ELEVATION**  
(SLOPES < 0.50% ALONG BEAM)



**FIXED SIDE ELEVATION**  
(SLOPES > 0.50% ALONG BEAM)



**SECTION A-A**



**SHEAR KEY DETAIL**

**SECTION B-B**

BEARING RESTRAINTS CONTROL DIMENSIONS					
CODE	DESCRIPTION	ABUT 1	PIER 1	PIER 2	ABUT 2
A	PLATE THICKNESS (IN)				
B	PLATE THICKNESS (IN)				
L	PLATE LENGTH (IN)				
W	PLATE WIDTH (IN)				
U	WELD LENGTH (IN)				
D	ANCHOR ROD DIAMETER (IN)				
Q	SHEAR PLATE OFFSET (IN)				
H	SHEAR PLATE HEIGHT (IN)				
R	SHEAR PLATE WIDTH (IN)				
S	SHEAR PLATE HOLE DIA (IN)				
V	SHEAR PLATE LENGTH (IN)				
FACTORED LONGITUDINAL LOAD (KIP)					
FACTORED LATERAL LOAD (KIP)					

**NOTES:**

- UNLESS OTHERWISE NOTED, SOLE PLATES SHALL BE MADE OF THE SAME STEEL AS THE ABUTTING BOTTOM FLANGE. WHERE DIFFERING MATERIALS ARE SPECIFIED WELDABILITY OF THE MATERIALS SHALL BE VERIFIED IN ACCORDANCE WITH AWS D1.5 - BRIDGE WELDING CODE.
- SOLE PLATES SHALL HAVE A MINIMUM THICKNESS OF 3/4" AND MINIMUM BEVEL, IF APPLICABLE, OF 0.01 RADIAN.
- EPOXY GRIT COATING SHALL BE APPLIED TO ALL STEEL SURFACES CONTACTING THE BEARING PAD AND EXTEND 1/2" IN ALL DIRECTIONS BEYOND THE PAD'S LIMITS. THE EPOXY GRIT SHALL BE INSTALLED IN ACCORDANCE WITH THE EPOXY MANUFACTURER'S INSTRUCTIONS. ALLOW THE EPOXY TO FULLY CURE FOR THE MINIMUM TIME RECOMMENDED BY THE MANUFACTURER THEN REMOVE ANY LOOSE GRIT BEFORE BEARING INSTALLATION. EPOXY GRIT SHALL MEET THE REQUIREMENTS OF SSPC AB1 ABRASIVE SPECIFICATIONS #1 - MINERAL & SLAG ABRASIVES, TYPE 2 OR BETTER.
- SHOP WELDING OF SOLE PLATES IS PREFERRED. WHERE FIELD WELDING IS NECESSARY, ALL COATINGS SHALL BE REMOVED BY GRINDING IN THE VICINITY OF THE WELD TO A BRIGHT METAL SURFACE AT LEAST 1 INCH IN ALL DIRECTIONS OF THE AREA TO BE WELDED. THE WELD AND DAMAGED AREA OF PAINTED SURFACE SHALL BE CLEANED AND REPAIRED AS SPECIFIED BY THE ENGINEER.
- WELDING WHILE THE LAMINATED BEARING PAD IS IN CONTACT WITH METAL IS DISCOURAGED. IF REQUIRED, TEMPERATURE INDICATING WAX PENS OR OTHER SUITABLE MEAN SHALL BE USED TO ENSURE THE PAD IS NOT EXPOSED TO TEMPERATURES GREATER THAN 250° F. ANY DAMAGE TO THE PAD DUE TO WELDING WILL BE CAUSE FOR REJECTION.
- THE ENGINEER SHALL VERIFY WELDS ARE CAPABLE OF WITHSTANDING ALL SUBJECT FORCE EFFECTS.
- THE DETAILS AND NOTES HEREIN ARE NOT APPLICABLE FOR VULCANIZED BEARINGS. REFER TO AASHTO-NSBA STEEL BRIDGE COLLABORATION G9.1 FOR MORE INFORMATION.
- UNLESS OTHERWISE NOTED, SHEAR PLATES SHALL BE MADE OF THE SAME STEEL AS THE ABUTTING SOLE PLATE. WHERE DIFFERING MATERIALS ARE SPECIFIED WELDABILITY OF THE MATERIALS SHALL BE VERIFIED IN ACCORDANCE WITH AWS D1.5 - BRIDGE WELDING CODE.
- SHEAR PLATES SHALL BE WELDED TO THE SOLE PLATE ALL AROUND. THE ENGINEER SHALL VERIFY WELDS ARE CAPABLE OF WITHSTANDING ALL SUBJECT FORCE EFFECTS.
- THE DIAMETER OF HOLES IN SHEAR PLATE SHALL BE AS RECOMMENDED FOR ANCHOR RODS IN AMERICAN INSTITUTE OF STEEL CONSTRUCTION (AISC) UNLESS OTHERWISE NOTED.
- ANCHOR RODS SHALL COMPLY WITH AASHTO M111, GRADE 55 AND BE FULL LENGTH HOT DIPPED GALVANIZED IN ACCORDANCE WITH AASHTO M232. THE ENGINEER SHALL SIZE ANCHOR RODS FOR ALL FORCE EFFECTS. ANCHOR ROD FLEXURE SHALL BE CONSIDERED.
- THE BEARING RESTRAINTS SHALL BE DESIGNED FOR MAXIMUM FACTORED LONGITUDINAL AND LATERAL LOADS CONCURRENTLY UNLESS OTHERWISE NOTED.

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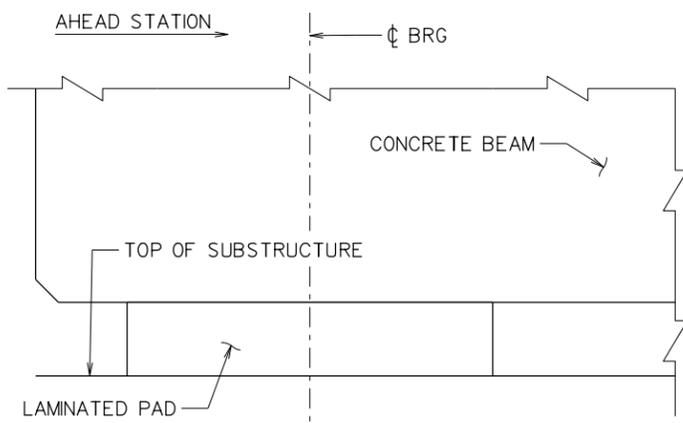
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WEST VIRGINIA DEPARTMENT OF TRANSPORTATION  
DIVISION OF HIGHWAYS

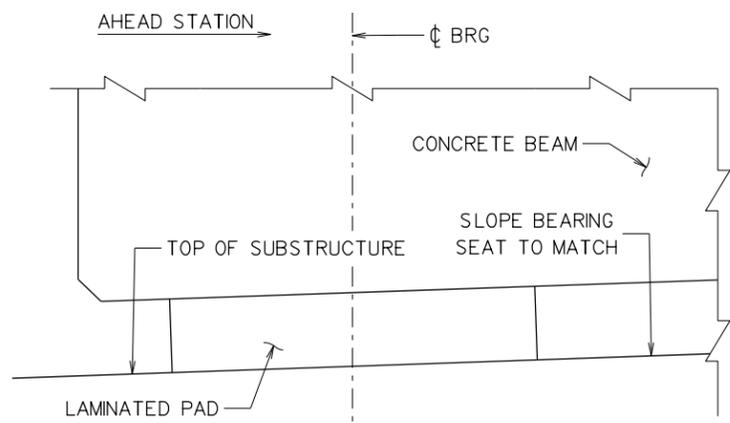
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DRAWN	DATE	REVIEWED	DATE

STANDARD BRIDGE PLANS  
**STEEL BEAM**  
**FIXED BEARING RESTRAINTS**  
SHEET NUMBER 1800BR3

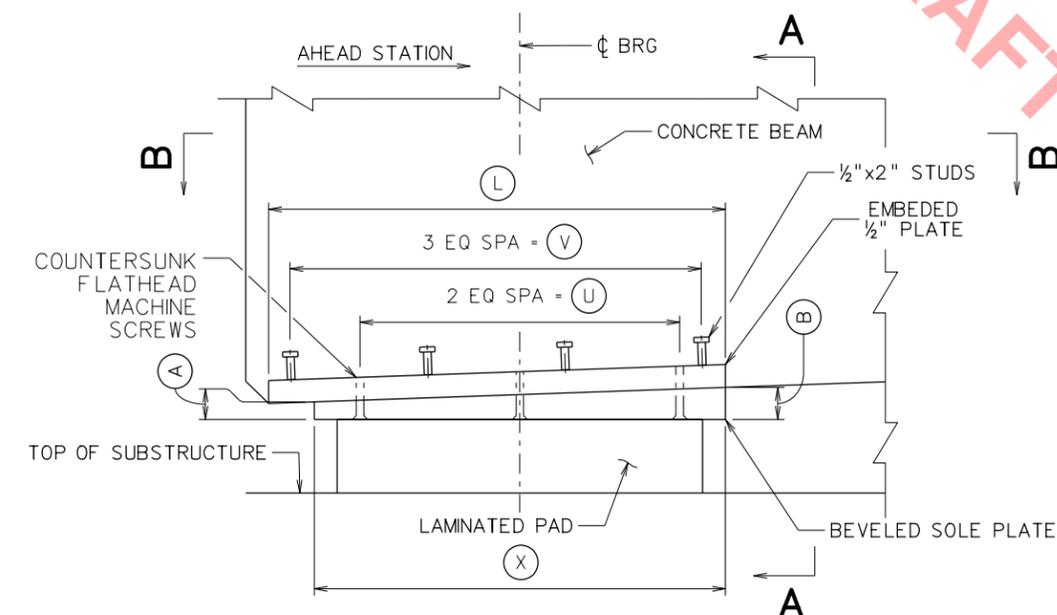
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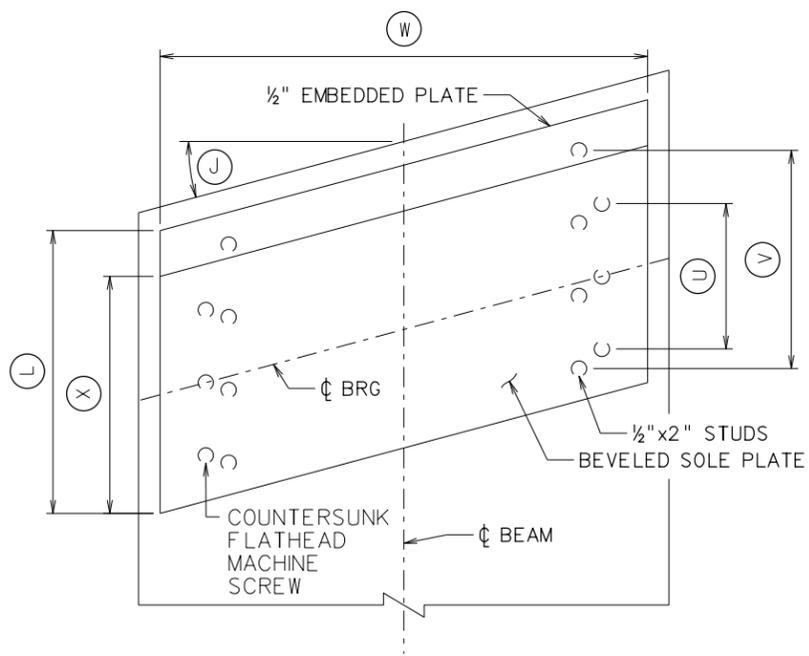
**NON-GUIDED SIDE ELEVATION**  
(SLOPES < 0.50% ALONG BEAM)



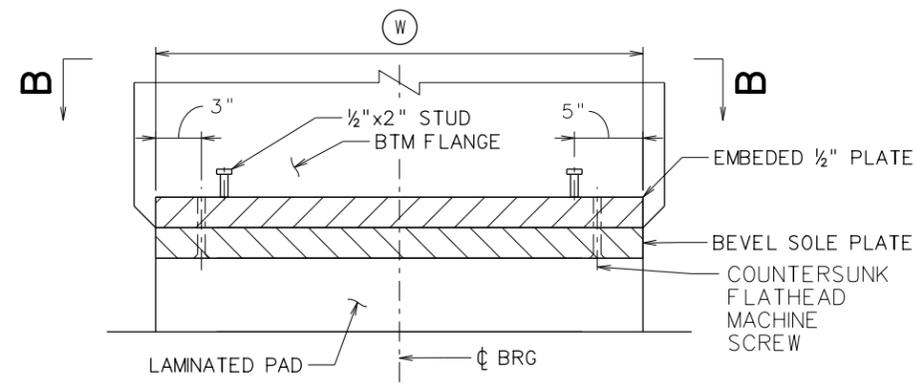
**NON-GUIDED SIDE ELEVATION**  
(SLOPES > 0.50% AND < 2.00% ALONG BEAM)



**NON-GUIDED SIDE ELEVATION**  
(SLOPES > 2.00% ALONG BEAM)



**SECTION B-B**  
(SKEW < 30°)



**SECTION A-A**

BEARING RESTRAINTS CONTROL DIMENSIONS					
CODE	DESCRIPTION	ABUT 1	PIER 1	PIER 2	ABUT 2
A	PLATE THICKNESS (IN)				
B	PLATE THICKNESS (IN)				
J	SKEW (DEG)				
L	BEVEL PLATE LENGTH (IN)				
W	WIDTH OF PLATES (IN)				
U	END TO END OF BOLTS (IN)				
V	END TO END OF STUDS (IN)				
X	EMBEDDED PLATE LENGTH (IN)				

**NOTES:**

- SPECIAL ATTENTION SHOULD BE GIVEN TO DRAINAGE WHEN SLOPING BEARING SEATS TOWARDS A BACKWALL OR OTHER OBSTRUCTION. SLOPING OF BEARING SEATS AT ABUTMENTS IS FORBIDDEN BELOW DECK JOINTS.
- UNLESS OTHERWISE NOTED, BEARING PLATE AND SOLE PLATES SHALL CONFORM TO AASHTO M270 GRADE 50. AFTER FABRICATION AND BEFORE DRILLING COUNTERSUNK HOLES, THE PLATES SHALL BE HOT-DIP GALVANIZED IN ACCORDANCE WITH AASHTO M111.
- BEVELED SOLE PLATES SHALL HAVE A MINIMUM BEVEL OF 0.01 RADIAN.
- TAP PLATES AFTER GALVANIZATION FOR INSTALLATION OF 1/2" DIAMETER ASTM F 3125, GRADE A325S COUNTERSUNK BOLTS. DRILL BOTH PLATES AS AN ASSEMBLED UNIT. THREAD THE EMBEDDED PLATE ONLY. COVER TAPPED HOLES IN EMBEDDED PLATE AS NEEDED TO AVOID INFILTRATION OF CONCRETE DURING BEAM FABRICATION.
- THE POSITION OF SHEAR STUDS MAY BE MODIFIED AS NEEDED WITH THE ENGINEER'S APPROVAL.
- ALL PLATES SHALL BE THOROUGHLY CLEANED OF ALL FOREIGN MATERIAL IMMEDIATELY PRIOR TO INSTALLATION IN ACCORDANCE WITH STANDARD SPECIFICATIONS. TIGHTEN COUNTERSUNK BOLTS SNUG TIGHT.
- EPOXY GRIT COATING SHALL BE APPLIED TO ALL STEEL SURFACES CONTACTING THE BEARING PAD AND EXTEND TO THE EDGES OF THE STEEL PLATES. THE EPOXY GRIT SHALL BE INSTALLED IN ACCORDANCE WITH THE EPOXY MANUFACTURER'S INSTRUCTIONS. ALLOW THE EPOXY TO FULLY CURE FOR THE MINIMUM TIME RECOMMENDED BY THE MANUFACTURER THEN REMOVE ANY LOOSE GRIT BEFORE BEARING INSTALLATION. EPOXY GRIT SHALL MEET THE REQUIREMENTS OF SSPC AB1 ABRASIVE SPECIFICATIONS #1 - MINERAL & SLAG ABRASIVES, TYPE 2 OR BETTER.

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7-FEB-2022 10:24

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	WEST VIRGINIA DEPARTMENT OF TRANSPORTATION DIVISION OF HIGHWAYS	DESIGNED _____	DATE _____	CHECKED _____	DATE _____		STANDARD BRIDGE PLANS <b>CONCRETE BEAM</b> <b>NON-GUIDED BEARING RESTRAINTS</b> SHEET NUMBER 1800BR4	
		DRAWN _____	DATE _____	REVIEWED _____	DATE _____			
NO.	REVISION	DATE	BY					