Standards Committee
Meeting Agenda
Wednesday, May 3, 2023, at 9:00am
Meeting Location: 1334 Smith Street, Charleston, WV in Lower-Level Conference
Also meeting virtually via Google Meet. E-mail distribution includes instruction.

Call to Order

Roll Call of Attendees

Approval of Minutes of 1-4-2023 Meeting

Unfinished Business – Standards discussed at last Committee meeting.

<table>
<thead>
<tr>
<th>TITLE</th>
<th>Champion</th>
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<tbody>
<tr>
<td><strong>2nd time to Committee.</strong></td>
<td>B. Neeley</td>
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<tr>
<td>Update of various Structure Directives (SD). The following SDs are included with brief summary of updates:</td>
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<tr>
<td>1) <strong>SD110-Project Design Criteria.</strong></td>
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<tr>
<td>Update to line and grade criteria.</td>
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<tr>
<td>2) <strong>SD1040-Structural System Selection.</strong></td>
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<tr>
<td>Removed sections which are duplicated in SD 1041 through 1044.</td>
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<tr>
<td>3) <strong>SD1041-Steel Superstructure Type.</strong></td>
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<tr>
<td>Changed Fracture Critical Member (FCM) to Nonredundant Steel Tension Member (NSTM) to reflect changes in National Bridge Inspection Standards terminology. Also, revised text for more generality in superstructure type by span length.</td>
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<tr>
<td>4) <strong>SD1042-Concrete Superstructure Type.</strong></td>
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<tr>
<td>Revised text for more generality in superstructure type by span length.</td>
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<td>5) <strong>SD1043-Abutment Types.</strong></td>
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<tr>
<td>Minor terminology updates.</td>
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<tr>
<td>6) <strong>SD1044-Pier Type.</strong></td>
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<tr>
<td>Terminology updates, such as FCM to NSTM</td>
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<tr>
<td>7) <strong>SD1073-Rehabilitation Techniques.</strong></td>
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<tr>
<td>Terminology updates, such as FCM to NSTM. Revised Dye Penetrate to NDT for greater flexibility in repair. Revision to 1073.6 to reflect what I think was the original goal of the section.</td>
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<tr>
<td>8) <strong>SD2034-Fatigue Critical.</strong></td>
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<tr>
<td>Terminology update, such as FCM to NSTM</td>
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<td>9) <strong>SD2045-Concrete Superstructures.</strong></td>
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<tr>
<td>Remove strand diameter requirement to provide greater flexibility to the Designer.</td>
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No updates to these SDs.

Approval is expected in May
## New Business

<table>
<thead>
<tr>
<th>TITLE</th>
<th>Champion</th>
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<tbody>
<tr>
<td><strong>1st time to Committee.</strong>&lt;br&gt;Twenty (20) proposed drawings and revisions to WVDOH Standard details Book – Volume 2. The sheets are listed below. A summary of each sheets revisions are included in the meeting packet; and proposed sheets have revised areas highlighted yellow.</td>
<td></td>
</tr>
<tr>
<td>a) <em>TE1-3A Roadside Sign Supports Steel Beam Type</em></td>
<td>Susan Hathaway, CDM Smith</td>
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<tr>
<td>b) <em>TE1-3B Roadside Sign Supports Steel Beam Type</em></td>
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<td>c) <em>TE1-3C Roadside Sign Supports Steel Beam Type</em></td>
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<tr>
<td>d) <em>TE2-1A Bridge or Retaining Wall Sign Mounting, Type K 1 &amp; 2 Supports</em></td>
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<tr>
<td>e) <em>TE2-1B Bridge or Retaining Wall Sign Mounting, Type K 1 &amp; 2 Supports</em></td>
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<tr>
<td>f) <em>TE2-2 Bridge or Retaining Wall Sign Mounting, Type L Pipe Post Mount</em></td>
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<tr>
<td>g) <em>TE2-3 Barrier Wall Sign Support Bracket Type D</em></td>
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<tr>
<td>h) <em>TE3-1 Overhead Sign Support – Steel Two Tube Span (TTS)</em></td>
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<tr>
<td>i) <em>TE3-2 Overhead Sign Support – Steel One Tube Span (OTS)</em></td>
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<tr>
<td>j) <em>TE4-3A Overhead Sign Support – Steel Double Arm Cantilever</em></td>
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<tr>
<td>k) <em>TE4-3B Overhead Sign Support – Butterfly Cantilever</em></td>
<td></td>
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<tr>
<td>l) <em>TE4-4A Overhead Sign Support – Single Arm Cantilever (Heavy)</em></td>
<td></td>
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<tr>
<td>m) <em>TE4-4B Overhead Sign Support – Single Arm Cantilever (Light)</em></td>
<td></td>
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<tr>
<td>n) <em>TE4-5 Overhead Sign Support – Steel Common Details</em></td>
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<tr>
<td>o) <em>TE5-1A Overhead Sign Support Box Truss Span</em></td>
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<tr>
<td>p) <em>TE5-1B Overhead Sign Support Box Truss Span</em></td>
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<tr>
<td>q) <em>TE9-1 Sign Clamps for Tubular Supports</em></td>
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<tr>
<td>r) <em>TEL41 Junction Box Details Type A</em></td>
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<tr>
<td>s) <em>TES-31 Pedestrian Push Buttons (PPB)</em></td>
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<tr>
<td>t) <em>TEM-2 Typical Pavement Markings (Sheet 2 of 2)</em></td>
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### Design Directive (DD) 202 – Field and Office Reviews for Initial Engineer, Preliminary Engineering, and Final Design

The DD updates Appendix A, which lists the resource agencies.

Next Meeting Date: Wednesday, July 5, 2023.

Adjournment
Standards Committee  
Meeting Minutes  
March 1, 2023

Call to Order: The meeting was called to order by Acting Chair Steve Boggs shortly after 9:00 AM.

Attendees: See Attendee List for a list of attendees.

Minutes: Minutes of the 1-4-2023 Meeting were approved without objection.

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

I. None.

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

II. Structure Directive (SD). Update of various Structure Directives (SD). The following SDs were introduced and discussed at the meeting:

1) SD110-Project Design Criteria.
2) SD1040-Structural System Selection.
3) SD1041-Steel Superstructure Type.
4) SD1042-Concrete Superstructure Type.
5) SD1043-Abutment Types.
6) SD1044-Pier Type.
7) SD1073-Rehabilitation Techniques.
8) SD2034-Fatigue Critical.
9) SD2045-Concrete Superstructures.

Hope to approve the nine updated SDs at the next meeting.

Next Meeting: The next meeting is on Wednesday, May 3, 2023. Deadline for submissions April 7, 2023.

Adjournment: The meeting was adjourned.
Manuals Committee
Meeting Minutes
March 1, 2023

Call to Order: The meeting was called to order by Acting Chair Steve Boggs shortly after conclusion of Standards Committee meeting.

Attendees: See Attendee List for a list of attendees.

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

I. 2023 Bridge Load Rating Manual (BLRM) for In-Service Bridges. The manual describes the policies and procedure for load rating and posting of public road bridges. It is an update to the 2020 BLRM. The proposed manual was briefly discussed.

   The manual was approved at the meeting. Vote 5-0.

II. WVDOH Tunnel Inspection Manual. The manual gives guidance and requirements of tunnel inspection and report requirements to meet state and federal code. The proposed manual was briefly discussed.

   The manual was approved at the meeting. Vote 5-0.

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

III. WVDOH Consultant Services Manual. This is an update of the 2011 manual; it includes four new chapters and revisions to the other chapters for consistency with current WVDOH policies and procedures.

   The manual was introduced by Amy Staud, HDR at the meeting. There was brief discussion. Hope to approve at the next meeting.

Next Meeting: The next meeting is on Wednesday, May 3, 2023. Deadline for submissions April 7, 2023.

Adjournment: The meeting was adjourned.
**March Standards and Manuals Committee Meeting**  
**Wednesday, March 1, 2023**  
**Attendee List**

**Virtual Meeting Attendees**
1. Brayack, Daniel  
   WVDOH – MCS&T Division
2. Conley-Rinhart, Laura  
   WVDOH – Technical Support Division
3. Cummings, John  
   WVDOH – MCS&T Division
4. Elkins, Jerry  
   HNTB
5. Farley, Paul  
   WVDOH – MCS&T Division
6. Hoover, Kimberly  
   WVDOH – Operations Division
7. Lough, Eric  
   WVDOH – Operations Division
8. Mance, Mike  
   WVDOH – MCS&T Division
9. Mongi, Ahmed  
   HDR
10. Moran, Tim  
    WVDOH – Operations Division
11. Smith, Yvonne  
    FHWA
12. Thaxton, Andrew  
    WVDOH – MCS&T Division
13. Varney, Billy  
    TRC

**In Person Meeting Attendees**
1. Adkins, Janie  
   WVDOH – Technical Support Division
2. Boggs, Steve  
   WVDOH – Technical Support Division
3. Crane, John  
   Contractors Association of West Virginia
4. Crum, Matt  
   WVDOH – Contract Administration Division
5. Hanson, Cal  
   ADS Pipe
6. Long, Travis E  
   WVDOH – Technical Support Division
7. Scites, RJ  
   WVDOH – Engineering Division
8. Staud, Amy  
   HDR
9. Whitmore, Ted  
   WVDOH – Traffic Engineering

**TOTAL ATTENDEES: 24**
All designs shall be in accordance with the latest edition of the AASHTO LRFD Bridge Design Specifications (Governing Specifications), including all interim specifications and the West Virginia Division of Highways Standard Specifications, Roads and Bridges (Standard Specifications) including the latest supplemental specifications.

See Design Directive (DD) 600 information that is for 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 shall be determined by the Project Manager or Consultant as applicable. If a Consultant is designing the project, then the line and grade will be determined by the Consultant. The Bridge Designer shall coordinate with the Project Manager to establish line and grade that can accommodate the proposed structural system. See SD 1040 for more information.

1010.3-EXISTING PROJECT RELATED INFORMATION

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

1010.4-HIGHWAY DRAINAGE, HYDROLOGY 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.
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, piers and bents 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 900 FT or more. Generally, the following table, Table 1040.A, can be used as a guideline for selecting steel superstructure types.

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

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 WVDOH’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, however four is desired.

Continuous spans shall be used for multi-span bridges. The ratio of the length of the end spans to the intermediate spans should preferably be 0.75.
END BOLT COVER PLATE DETAIL

ELEVATION

SECTION A-A

DETAIL A

PLAN

Note:
Connection to be designed by the Engineer.

Figure 1040.B
**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 girder lines, in some cases, may increase the total weight of the steel. However, the savings realized through fabrication of fewer girders, fewer cross frames and bearings, as well as savings realized through shorter erection time will often offset an increase in raw steel cost. Three girder lines is the minimum unless the system is structurally redundant and not fracture critical, however four is desired.

B. Consultation with fabricators and erectors is recommended to assess the fabrication and erection costs of the girders.

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

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

Limit girder spacing to 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 or where aesthetics play an important part in the bridge type. The main structural elements of a typical bridge truss consist of stringers, floor beams, top chord, bottom chord, vertical and diagonal members of the main longitudinal trusses, lateral bracings and sway bracings. Chord members carry the bending moment while the diagonals carry the shear. Axial loads are the predominant forces in all truss members.

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

The stringers can be designed similar to steel rolled beam bridge members. The floor beams are generally plate girders with variable plate sizes. Generally, the truss members are composite box sections made of welded plates and the bracing members are rolled W, T or channel shapes. The use of high-performance steel shall be investigated in the span arrangement study for main truss members, stringers, and floor beams.

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 700 FT or more. Generally, the following, Table 1040.C, can be used as a guide for selecting concrete superstructure types.

<table>
<thead>
<tr>
<th>SPAN LENGTH (FT)</th>
<th>SUPERSTRUCTURE TYPE</th>
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<tbody>
<tr>
<td>up to 30</td>
<td>Slab Bridges</td>
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<tr>
<td>20 to 100</td>
<td>Box Beams</td>
</tr>
<tr>
<td>35 to 165</td>
<td>I-Girders</td>
</tr>
<tr>
<td>165 to 300</td>
<td>Post Tensioned I-Girders (Drop-In)</td>
</tr>
<tr>
<td>100 to 180</td>
<td>Segmental Concrete Boxes (Span-By-Span)</td>
</tr>
<tr>
<td>150 to 450</td>
<td>Segmental Concrete Boxes (Precast)</td>
</tr>
<tr>
<td>450 to 700</td>
<td>Segmental Concrete Boxes (Cast-In-Place)</td>
</tr>
<tr>
<td>500+</td>
<td>Cable Stayed</td>
</tr>
</tbody>
</table>

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 ($f_{ci}$).

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 beam lines.
The Engineer or Design of Record should verify availability of shapes from multiple fabricators.

<table>
<thead>
<tr>
<th>Approximate Maximum Span Lengths (FT)</th>
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<tbody>
<tr>
<td>Beam Spacing (FT)</td>
</tr>
<tr>
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<tr>
<td>I</td>
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<td>II</td>
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<td>III</td>
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<td>IV</td>
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<td>V</td>
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<td>VI</td>
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<table>
<thead>
<tr>
<th>Type IV Modified</th>
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<tbody>
<tr>
<td>60 -IN</td>
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<tr>
<td>66 -IN</td>
</tr>
<tr>
<td>72 -IN</td>
</tr>
<tr>
<td>78 -IN</td>
</tr>
<tr>
<td>84 -IN</td>
</tr>
</tbody>
</table>

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 ($f'_c$) of 6000 PSI and a final strength ($f''_c$) of 8000 PSI.

**Table 1040.D**

**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 and beam post tensioned as specified within the contract plans. 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.

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.

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 rock socketed drilled shafts. 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 shaft 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 rock socketed drilled shafts.

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 (GRS-IBS): GRS-IBS Abutments were initially developed by FHWA and can provide an economic alternative to other abutment types especially where adjacent box beams are used and scour is not considered to affect the foundations. The GRS-IBS abutment type 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. All GRS-IBS bridge locations shall be approved by the State Bridge Engineer.

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

1040.5 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 Rock Socketed Drilled Shafts: Rock socketed drilled shafts provide superior scour protection versus traditional steel piling, greater resistance against high lateral and uplift loads, and accommodation of site concerns associated with the pile driving process (vibrations,
interference due to battered piles, etc.), and in some cases exclude the need of cofferdams. In addition, rock socketed drilled shafts may eliminate the need of caisson caps, for certain configurations such as single or multiple column piers.

......Rock socketed drilled shafts shall be designed using soil structure 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 testing by the Division of: pre-installation core holes, wet or dry hole condition, plumbness, shaft sideward and bottom cleanliness, and concrete inspection. Results from the testing may require remedial action from the Contractor.
Steel superstructures should be considered for any span length ranging from twenty (20) feet to nine hundred (900) feet or more. Generally, Table 1041.A can be used as a guideline for selecting steel superstructure types.

<table>
<thead>
<tr>
<th>SPAN LENGTH (Feet)</th>
<th>SUPERSTRUCTURE TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 to 100</td>
<td>Rolled Beams</td>
</tr>
<tr>
<td>60 to 130</td>
<td>Rolled Beams with Cover Plates</td>
</tr>
<tr>
<td>80 to 400</td>
<td>Welded Plate Girders</td>
</tr>
<tr>
<td>200 to 400</td>
<td>Box Girders</td>
</tr>
<tr>
<td>400 to 900</td>
<td>Truss</td>
</tr>
<tr>
<td>500+</td>
<td>Cable-Stayed</td>
</tr>
<tr>
<td>650+</td>
<td>Tied Arch</td>
</tr>
</tbody>
</table>

Table 1041.A

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

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

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

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

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

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

**1041.1-ROLLED BEAMS**

Rolled beams should be considered for any span lengths ranging from twenty (20) up to one hundred (100) feet. With cover plates, the span range of rolled beams can be extended to 130 feet. However, only end bolted cover plates shall be used. See Figure 1041.B. The Designer shall determine the availability of any rolled section considered, including lengths and grade of steel.

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

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

**1041.2-PLATE GIRDERS**

Plate girders should be considered for any span lengths ranging from eighty (80) feet up to 400 feet. The Designer shall carefully evaluate the bridge cross section to ensure appropriate girder spacing. Substantial cost savings may be realized early in the design process. The following shall be considered during the span arrangement study:

A. Use of wider girder spacing to eliminate girder lines, in some cases, may increase the total weight of the steel. However, the savings realized through fabrication of fewer girders, fewer cross frames and bearings, as well as savings realized through shorter erection time will often offset an increase in raw steel cost. Three girder lines is the minimum unless the system is structurally redundant and not fracture critical, however four is desired.

B. Consultation with fabricators and erectors is recommended to assess the fabrication and erection costs of the girders.

Generally, continuous spans shall be used for multi-span bridges. The ratio of the length of the end spans to the intermediate spans should preferably be 0.75. If the end span to intermediate span ratio is small, anchored end spans shall be used to eliminate any uplift problems at the abutments. Configurations experiencing subject to uplift shall be approved by the State Bridge Engineer. The Bridge Designer should also consider the economics of a system designed span by span (i.e., simply supported for dead load and continuous for live load).
Figure 1041.B

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

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

Optimize the girder weight by investigating various web depths.

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

1041.3-BOX GIRDERS

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

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

1041.4-TRUSSES

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

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

The stringers can be designed similar to steel rolled beam bridge members. The floor beams are generally plate girders with variable plate sizes. Generally, the truss members are composite box sections made of welded plates and the bracing members are rolled W, T, or channel
shapes. The use of high-performance steel shall be investigated in the span arrangement study for main truss members, stringers, and floor beams.

1041.5-CABLE STAYED

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

1041.6-TIED ARCH

Tied arch bridges can also be used for medium and long spans, (650 FT to 1700 FT) over five hundred (500) feet. 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.
Concrete superstructure types should be considered for any span length ranging from twenty (20) feet to seven hundred (700) feet or more. Generally, the following table, Table 1042.A can be used as a guide for selecting concrete superstructure types.

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

Table 1042.A

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

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

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

1042.1-SLAB BRIDGES

This superstructure type consists of a reinforced concrete slab with the main reinforcing parallel to the direction of traffic. This type of structure may be economical for very short span bridges, generally less than thirty (30) feet in length.
1042.2-BOX BEAMS

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

Three basic cross-sectional configurations are commonly used. They are:
A. Adjacent box beams with or without a hot-laid bituminous concrete (HLBC) wearing surface.
B. Adjacent box beams with a composite reinforced concrete deck.
C. Spread box beams with a composite reinforced concrete deck.

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

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

1042.3-PRESTRESSED CONCRETE BEAMS

Prestressed concrete beams should be considered for bridges with spans from twenty-five (25) feet to 145 feet. The maximum span length is based on the haul capacity and availability for a particular project site and shall be verified with a prestressed concrete beam supplier familiar with the project location. For continuous spans, the bridge system shall be designed simply supported for dead load and continuous for live load and superimposed dead load only. The same prestressed concrete beam type is desired for all spans. The Designer should minimize the number of beam lines. Prestressed concrete beam bridges should have a minimum of three stringer lines.

The design of all structures that utilize prestressed concrete I-beam sections will be accomplished using beam sections locally available. Beam sections shown in Figure 1042.B represent a historical list of available shapes of prestressed concrete beams.
Figure 1042.B

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

### Approximate Maximum Span Lengths (Feet)

<table>
<thead>
<tr>
<th>Beam Spacing (Feet)</th>
<th>14</th>
<th>12</th>
<th>10</th>
<th>8</th>
<th>6</th>
</tr>
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<tbody>
<tr>
<td>I</td>
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<td>30</td>
<td>35</td>
<td>40</td>
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<tr>
<td>VI</td>
<td>105</td>
<td>115</td>
<td>120</td>
<td>130</td>
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</tr>
<tr>
<td>Type IV Modified</td>
<td>60-IN</td>
<td>85</td>
<td>95</td>
<td>100</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td>66-IN</td>
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<td>120</td>
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<tr>
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<td>72-IN</td>
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<tr>
<td></td>
<td>78-IN</td>
<td>110</td>
<td>115</td>
<td>125</td>
<td>130</td>
</tr>
<tr>
<td></td>
<td>84-IN</td>
<td>115</td>
<td>125</td>
<td>130</td>
<td>135</td>
</tr>
</tbody>
</table>

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

### Table 1042.B

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

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

#### 1042.5-SEGMENTAL CONCRETE BOXES

Segmental concrete boxes are an economical solution for bridges with span lengths over 100 FT-300 feet and where repetition of the box fabrication can be achieved. There are three methods of construction for segmental concrete: span-by-span, balanced cantilever, and cast-in-place. Each offers advantages in different situations.

#### 1042.6-CABLE STAYED

Cable-stayed bridges are competitive for medium and long spans (500 feet to 1,500 feet) over 500 feet. The superstructure, consisting of a concrete deck on prestressed concrete beams, is supported at several intermediate points by cables radiating from one or more towers.
Abutments are structures positioned at the beginning and end of a bridge, which support the superstructure and approach roadway and retain 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.

1043.1 - WALL ABUTMENT
This type of abutment, also known as a full height abutment, may be used when right-of-way is critical, or the site does not permit a longer bridge with sloping embankments. Span lengths can be reduced using a wall type abutment. The footing may transfer loads by direct bearing (spread footing) or it may be supported on piles or rock socketed drilled shaft.

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

1043.2 - PEDESTALS
The beam seat is supported on columns/drilled shafts or pedestals resting on individual footings. This configuration is useful for meeting unique construction problems, e.g., widely varying elevations of competent rock.

1043.3 - STUB ABUTMENT
Stub abutments are relatively short abutments that resemble wall type abutments. These abutments are generally placed on the approach embankment and are supported on rock, piles or rock socketed drilled shafts.

1043.4 - INTEGRAL ABUTMENT
Integral abutments are generally short abutments supported on a single row of piling. These abutments, like stub abutments, are generally placed on approach embankments and are well suited for bridges with limited thermal movements. The ends of the bridge beams are cast directly into the abutments, thereby eliminating the need for bridge deck expansion devices.
This abutment type can be used in combination with Mechanically Stabilized Earth (MSE) walls to provide the benefits of a wall type abutment while satisfying the preference for using jointless bridges.

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

1043.5-SEMI-INTEGRAL ABUTMENT

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

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

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

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

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

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

1043.7-WINGWALLS

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

U-shaped or turned-back wingwalls are commonly used in embankment situations and straight wings are used in cut sections. Flared wingwalls between these extremes can also be appropriate based on-site conditions. The Designer must study the existing and proposed surfaces to determine which type of wingwalls best fits the site. Wingwalls with a tapered bottom surface shall be avoided due to compaction difficulties beneath the wall. The top surface of U-shaped wingwalls may be tapered parallel to the roadway slope to match the finished grade.
Piers are intermediate supports in a multi-span bridge system. All feasible pier types must be considered in the preliminary phases of the project.

**1044.1-CAP-AND-COLUMN TYPE PIERS**

Cap-and-column type piers have two or more circular or rectangular columns connected on top with a cap (a reinforced concrete beam that supports the superstructure). Generally, the pier cap ends will be cantilevered. For columns greater than one hundred (100) to 150 feet, the use of a compression strut at mid-height, similar to the pier cap, shall be investigated. Partial height walls may be necessary where vehicle collision or debris buildup is possible. 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 should be investigated during the preliminary design phase of the project.

**1044.3-POST-TENSIONED CONCRETE/INTEGRAL PIER CAPS**

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

**1044.4-STEEL PIER CAPS AND BENTS**

Most steel pier caps and bents are fracture critical Nonredundant Steel Tension Members (NSTM) and should be avoided. If used, the design shall allow for reasonable access to the interior for future maintenance, inspection, and repair. Steel pier caps and bents shall be designed for redundancy unless otherwise approved by the State Bridge Engineer.
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 Nonredundant Steel Tension Members require special assessment because their failure would be expected to result in bridge collapse. All repairs shall consider the dead load that exists in original members and the original members shall not be stressed beyond their original allowable inventory stress level. All steel repairs shall be in accordance with the Steel Structures section of the Governing Specifications. All repairs to welds on steel members shall be in accordance with the AASHTO/AWS D1.5M/D1.5: current version, Bridge Welding Code.

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

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

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

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

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

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

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

**1073.1.4-Section Loss:** Cover plates are an effective means for restoring section loss in a member. The member must be analyzed to ensure its original or target capacity can be attained with the addition of cover plates. Details of repairs are largely up to the Designer’s creativity. The Designer must consider the fatigue characteristics of the repairs they design. If excessive deterioration exists, then replacement of the member may be required. The Designer must consider “locked-in” forces and differences in supplementary cover plate material properties.
Figure 1073.B

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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 lose bars shall be carefully undercut to a depth that permits a minimum of one inch of new concrete around the reinforcement bars. Sections with deteriorated bars should be re-evaluated and capacities restored, when necessary. The area of concrete removal should be large enough to allow for adequate bar splicing. The exposed area of concrete should be cleaned. Where concrete deterioration requires substantial removal 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 623, of the Specifications.

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

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

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

1073.2.1-Concrete Decks: Most repairs needed in bridge decks are associated with increased traffic, heavier vehicles, deicing chemicals, and geometric deficiencies as a result of the initial construction. Common problems are cracking, spalling, chloride contamination, potholing, and
delaminating. Cracking in the deck can be repaired as described in the previous section. Minor spalling, potholes, etc. may be temporarily repaired with patches. Patches cannot be considered a permanent solution. Eventually, a bridge deck becomes a composition of patches with no end to the repair process. As the patching process is repeated to repair more damaged areas, an overlay will be needed to serve as a wearing surface and a moisture barrier.

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

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

**1073.2.2-Deck Overlays:** 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 one (1.0) inch 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 seven (7.0) inches thick. For slabs, 6.5 inches to 7.0 inches thick, special consideration must be given to methods of removal of the deteriorated concrete, such as hydro-demolition, so that damage of the remaining slab is minimized. A specialized concrete overlay will not be considered an acceptable method for deck retrofit for any bridge deck where the original slab thickness is less than 6.5 inches.

**1073.3-ADDITIONAL REHABILITATION ISSUES**

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

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

**1073.4-TIMBER**

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

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

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

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

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

1073.5-DECK JOINTS

The following describes rehabilitation techniques associated with commonly used types of expansion joints. It is the 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-Heavy rust or surface scaling of non-contact surfaces.
B. Loss of lubrication.
C. Debris and dirt accumulation on the bearing seat.
D. Minor—Significant tilting and displacement of bearing components at mild temperatures.
E. Heavily rusted masonry and keeper plates.
F. Missing nuts—Heavily or-deteriorated anchor bolts rods and nuts.

Bearings requiring replacement are ones that are severely deteriorated, suffered loss of function, and exhibit signs of imminent structural instability. The following can be used as a guideline in the choice of bearing replacement:
A. The ability of the bearing to provide the same functions as the existing in terms of load transfer and movement.
B. Compatibility with the environment.
C. Dimensions of new bearing, particularly the height.
D. Structural compatibility of the bearing with other bridge components.

1073.7-HISTORICAL STRUCTURES
Historic structures that are scheduled for rehabilitation shall adhere to the United States Department of Interior’s Standards for the Treatment of Historic Properties. These standards can
be obtained from the Technical Support Division, Environmental Section, of the WVDOH. The Designer shall work closely with the WVDOH on historic rehabilitation projects.
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) nonredundant steel tension members (NSTM) is unavoidable and approved by the WVDOH, the FCM-NSTM should be clearly designated on the contract drawings with the appropriate tension zones indicated and shall be fabricated according to Section 12 of AASHTO/AWS D1.5M/D1.5: 2002 current version, Bridge Welding Code, (Bridge Welding Code).
Concrete compressive strengths for precast beams shall be no more than 8,000 PSI at release ($f'_{ci}$) with a final compressive strength of 10,000 PSI ($f'_{c}$). Precast beams may be designed using high strength concrete with approval of the State Bridge Engineer.

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

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Area</th>
<th>Ultimate Strength</th>
<th>Applied Prestressing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5 Inch</td>
<td>0.153 IN$^2$</td>
<td>41.3 KIPS/strand</td>
<td>31.0 KIPS/strand</td>
</tr>
<tr>
<td>0.5 Inch (Special)</td>
<td>0.167 IN$^2$</td>
<td>45.1 KIPS/strand</td>
<td>33.8 KIPS/strand</td>
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<tr>
<td>0.6 Inch</td>
<td>0.217 IN$^2$</td>
<td>58.6 KIPS/strand</td>
<td>44.0 KIPS/strand</td>
</tr>
</tbody>
</table>

Table 2045.A

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

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

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

In a situation where two or more girder beams of the same size require a slightly different number of strands, resulting from differences in design loadings (i.e., interior and exterior beams), use the greater number of strands if possible. This makes fabrication easier and reduces confusion during construction.
<table>
<thead>
<tr>
<th>Sheet</th>
<th>Revision #</th>
<th>Date</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>TE1-3A</td>
<td>1</td>
<td>4/2022</td>
<td>Returned hinge plate dimensions to the previous standard. Revised T3 &amp; D3 for S4 and W6 supports</td>
</tr>
<tr>
<td>TE1-3B</td>
<td>1</td>
<td>4/2022</td>
<td>Removed note allowing use of W10X22 behind guardrail or on a bench.</td>
</tr>
<tr>
<td>TE1-3C</td>
<td>1</td>
<td>4/2022</td>
<td>Returned to previous foundation sizes.</td>
</tr>
<tr>
<td>TE2-1A</td>
<td>1</td>
<td>11/11/2022</td>
<td>Increased the base plate to allow for more fasteners due to structural loads. Revised Note 1 to remove reference to three post option and to limit sign width to 6'. Revised Note 3 to clarify loads for fabricator to specify anchors. Added note to verify that the concrete structure mounting to will support these loads.</td>
</tr>
<tr>
<td>TE2-1B</td>
<td>1</td>
<td>11/11/2022</td>
<td>This option for three posts is not to be used. Loads are too great for typical bridge parapet.</td>
</tr>
<tr>
<td>TE2-2</td>
<td>1</td>
<td>11/11/2022</td>
<td>Increased the base plate to allow for more fasteners due to structural loads. Increased the number of required anchor bolts. This bracket is only allowed for Pipe Post Types 6 &amp; 9. Note 4 was added to clarify loads for fabricator to specify anchors for retrofit option.</td>
</tr>
<tr>
<td>TE2-3</td>
<td>1</td>
<td>4/2022</td>
<td>Clarified the tubular steel post length to be based on TP3-1A.</td>
</tr>
<tr>
<td>TE3-1</td>
<td>1</td>
<td>2/22/2023</td>
<td>Increased the support bracket from W10X77 to W14X90 and revised associated dimensions. Added weld to flange portion of support bracket. Increased thickness of stiffener plate. Increased foundation diameter from 4'-6&quot; to 5'-0&quot; and number of vertical bars (See notes for TE3-2). Shifted Detail 4 and Coping Detail to make room for revision info.</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>6/2022</td>
<td>Added camber information to the sheet.</td>
</tr>
<tr>
<td>TE3-2</td>
<td>2</td>
<td>2/22/2023</td>
<td>Increased the support bracket from W10X77 to W14X90. For TTS option, increased bolt circle (B) to 43&quot; to meet the 1 1/2&quot; clearance requirement between the nut and the post, which required the foundation diameter to increase. Decreased the S and F dimensions which previously were excessive. For the OTS option, increased bolt circle (B) to 26&quot; to meet the 1 1/2&quot; clearance requirement between the nut and the post. Increased the S and F dimensions. Increased the splice plate thickness (C) and added center hole diameter for base plate for both TTS and OTS types.</td>
</tr>
<tr>
<td>TE4-3A</td>
<td>1</td>
<td>2/22/2023</td>
<td>Added NPS to column headings and changed post to NPS in Member Size Chart. Revised plate dimensions and added max plate hole diameter to Footing Table. Revised end and box flange plates, revised bolt numbers for DAC-32 &amp; DAC-40, and added Hole Diameter column to Box Connection Table.</td>
</tr>
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<td>Description</td>
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<tr>
<td>TE4-3B</td>
<td>2/22/2023</td>
<td>Revised rules for allowable sign area and secondary arm member sizes. Original version allowed max sign area for each arm, but that is incorrect.</td>
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<tr>
<td>TE4-4A</td>
<td>11/18/2022</td>
<td>Revised foundation diameter and embedment depth; moved diameter information to new column in Footing Table. Added design soil parameters and 2:1 max slope to Elevations. Added 'W' dimension to the Box Connection Table for SACH-15, 25 &amp; 35 designs. Increased post sizes for SACH-45 &amp; 55 designs in the Member Size Chart and corrected arm sizes in the Box Connection Table. Added base plate hole diameter column to Footing Table. Added camber information.</td>
<td></td>
</tr>
<tr>
<td>TE4-4B</td>
<td>9/2020</td>
<td>Added Moment Arm Calculation to aid in determining proper design for multiple smaller signs. Revised bolt hole diameter in the Footing Table. Added 'W' dimension and 'X' dimension (TE4-5) to the Box Connection Table. Added camber angle.</td>
<td></td>
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<tr>
<td></td>
<td>5/2022</td>
<td>Revised Arm A and Arm B lengths and added Arm B diameters in Member Size chart.</td>
<td></td>
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<tr>
<td>TE4-5</td>
<td>11/18/2022</td>
<td>Revised reinforcing based on footing size changes on TE4-4A.</td>
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</tr>
<tr>
<td>TE5-1A</td>
<td>12/19/2022</td>
<td>Added camber information. Revised max span section length.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2/22/2023</td>
<td>Revised chord member sizes. Deleted inappropriate weld.</td>
<td></td>
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<tr>
<td>TE5-1B</td>
<td>2/28/2022</td>
<td>Revised foundation reinforcement. Added Detail 8 to clarify reinforcing. Revised/Added bolt projection information. Clarified footing depth/embedment depth on Foundation Detail.</td>
<td></td>
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<tr>
<td></td>
<td>12/19/2022</td>
<td>Updated Chord Splice Table to match changes made to TE5-1A. Deleted 8&quot; post from Base Plate Table.</td>
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<tr>
<td>TE9-1</td>
<td>4/2022</td>
<td>Added saddle to all U-bolt connections on Type 2 and Type 3 details. Added Note 8 about including a saddle between the tube and the zee bar.</td>
<td></td>
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<tr>
<td>TEL-41</td>
<td>4/2022</td>
<td>Added note 5 to clarify requirements for size and number of hubs.</td>
<td></td>
</tr>
<tr>
<td>TES-31</td>
<td>3/2020</td>
<td>Revised sign number for pedestrian push buttons.</td>
<td></td>
</tr>
<tr>
<td>TEM-2 (2 of 2)</td>
<td>4/2022</td>
<td>Specified in notes 7 &amp; 8 that markings that are to be used for turn movements in the middle of the intersection are to be Type V.</td>
<td></td>
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</tbody>
</table>
**BASE CONNECTION DATA TABLE (IN.)**

| POST SIZE | BOLT SIZE | A | B | C | D | E | T1 | T2 | W | R | 01 | F | G | H | J | L | K | M | N | P | T3 | D2 | D3 | BOLT DIA |
|-----------|-----------|---|---|---|---|---|----|----|----|---|----|----|---|---|---|---|---|---|---|----|----|----|------------|
| S4 x 7.2 | 1/4" x 2 1/4" | 15 | 10 | 10 | 3/4 | 3/4 | 3/4 | 3/4 | 1/2 | 1/2 | 1/2 | 3/4 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 |
| W6 x 12  | 1/4" x 2 1/4" | 6  | 6  | 6  | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 |
| W8 x 18  | 1/4" x 2 1/4" | 6  | 6  | 6  | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 |
| W10 x 22 | 1/4" x 2 1/4" | 6  | 6  | 6  | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 |

**HINGE PLATE DATA TABLE (IN.)**

| POST SIZE | BOLT SIZE | A | B | C | D | E | T1 | T2 | W | R | 01 | F | G | H | J | L | K | M | N | P | T3 | D2 | D3 | BOLT DIA |
|-----------|-----------|---|---|---|---|---|----|----|----|---|----|----|---|---|---|---|---|---|---|----|----|----|------------|
| S4 x 7.2 | 1/4" x 2 1/4" | 15 | 10 | 10 | 3/4 | 3/4 | 3/4 | 3/4 | 1/2 | 1/2 | 1/2 | 3/4 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 |
| W6 x 12  | 1/4" x 2 1/4" | 6  | 6  | 6  | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 |
| W8 x 18  | 1/4" x 2 1/4" | 6  | 6  | 6  | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 |
| W10 x 22 | 1/4" x 2 1/4" | 6  | 6  | 6  | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 | 1/2 |

**SHIM DETAIL**

1. PROCEDURE FOR ASSEMBLY OF HINGE PLATE:
   - Assemble connection and pre-tighten the bolts in a manner consistent with the snug tightening procedures described in the Research Council on Structural Connections (RCSC) Specification for Structural Joints Design in the Standard Specifications.
   - Fully tighten the bolts by rotating the nuts in accordance with the snug tightening procedure specified in Section 657 of the Standard Specifications.
   - Post shall be cut before galvanizing.

2. PROCEDURE FOR ASSEMBLY OF BASE CONNECTION:
   - Assemble post to stub with bolts and with one flat washer on each bolt between plates.
   - Saw as required to plumb post.
   - Base plate bolts are to be torqued using a "click" type torque wrench meeting the requirements specified in Section 657 of the Standard Specifications.

3. POST SHALL BE SAW CUT BEFORE GALVANIZING.

4. MATERIALS AND FABRICATION SHALL CONFORM TO THE REQUIREMENTS OF THE WEST VIRGINIA DIVISION OF HIGHWAYS SPECIFICATIONS. ALL HOLES SHALL BE DRILLED; ALL PLATE CUTS SHALL BE SAW CUTS. ALL PLATE THICKNESS-Ê TYP.

**FOR ALL SHAPES**
### SUPPORT SIZE SELECTION CHART

<table>
<thead>
<tr>
<th>TOTAL SQUARE FEET OF SIGN AREA PER POLE</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
<th>110</th>
<th>120</th>
<th>130</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5</td>
<td>20</td>
<td>30</td>
<td>40</td>
<td>50</td>
<td>60</td>
<td>70</td>
<td>80</td>
<td>90</td>
<td>100</td>
<td>110</td>
<td>120</td>
<td>130</td>
</tr>
<tr>
<td>S4X7.7</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W6X12</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W8X18</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W10X22</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**POST SELECTION PROCEDURES:**

1. **Determine Total Number of Panels (P):**
   - The total number of panels is determined by the total square footage of the sign area per pole divided by the number of support beams used.

2. **Determine Preliminary Selection of Number of Posts Used:**
   - The preliminary selection is based on the total number of support beams per pole.

3. **Calculate the Total Square Footage of Sign Area Per Pole:**
   - The total square footage per pole is calculated by multiplying the number of support beams per pole by the total number of panels (P).

4. **Determine Height from Sign Plate to the Center of Pressure (CP):**
   - The height is determined by measuring the vertical distance from the sign plate to the center of pressure (CP) for each support beam.

5. **Calculate the Square Footage of Sign Area Per Support (Total Square Footage Divided by Number of Supports):**
   - The square footage per support is calculated by dividing the total square footage of sign area per pole by the number of support beams used.

6. **Verify That the Selected Posts May Be Used Based on Minimum Required Post Spacing:**
   - The minimum required post spacing is determined by the design considerations and the requirements of the traffic control device.

7. **If Not, Change Number of Posts Used and Repeat Steps 4, 5, 6:**
   - If the selected posts do not meet the minimum required post spacing, change the number of posts and repeat the selection process.

**SUPPORT SPACING REQUIREMENTS:**

- Support beams are to be placed in accordance with the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1994 edition.

- The support spacing shall be determined based on the greater of:
  - The width of the single sign that is attached to all of the assembly supports.
  - The combined overall width of the signs that are attached to the assembly supports and that are attached to all of the assembly supports.

- An example of the use of support beams is provided in the diagram on the next page.

---

**NOTES:**


2. For foundation connections to be used in conjunction with the post selection chart shown, see sheet TE1-5B.

3. For foundation see sheet TE1-5B.

4. For base connections to be used in conjunction with the base selection chart shown, see sheet TE1-7B.

5. For routing, see sheet TE1-7C.

6. For base connections to be used in conjunction with the base selection chart shown, see sheet TE1-7B.

7. For foundation see sheet TE1-5B.

---

**DIAGRAM:**

- The diagram illustrates the support beams and their placement in relation to the center of pressure (CP).

- The dimensions shown in the diagram are based on the requirements for min. required post spacing and the requirements of the traffic control device.

---

**POST SPACING:**

<table>
<thead>
<tr>
<th>NO. OF POSTS</th>
<th>DIM A</th>
<th>DIM B</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.2L</td>
<td>0.8L</td>
</tr>
<tr>
<td>3</td>
<td>0.14L</td>
<td>0.36L</td>
</tr>
<tr>
<td>4</td>
<td>0.1L</td>
<td>0.26L</td>
</tr>
<tr>
<td>5</td>
<td>0.08L</td>
<td>0.2L</td>
</tr>
</tbody>
</table>

**CENTER OF PRESSURE:**

- The center of pressure is the vertical distance measured from the lowest point where the support foundation meets the ground to the center of the sign assembly.
The projection of the stub above ground level is to not extend above a 60 inch wide chord which extends 4 inches above the ground level on each end as shown on detail 1.

**Foundation in Slope**

- The tops of all foundations shall be finished smooth with the concrete sloping slightly downward from the stub to the edge of the footer in order to facilitate drainage.
- If the slope is 4:1 or greater and it is not possible to build up the downhill side of the ground slope in order to allow the top of the foundation to be level, the contractor shall incorporate a form as described in Section 657 of the Standard Specifications.

**Foundation Required Per Post**

<table>
<thead>
<tr>
<th>Beam Size</th>
<th>Dimension W</th>
<th>Dimension H</th>
<th>Cubic Yards of Concrete</th>
<th>Vertical Steel</th>
<th>Stirrup Steel</th>
<th>Stub Length</th>
<th>Stub Projection</th>
</tr>
</thead>
<tbody>
<tr>
<td>S4X7.7</td>
<td>1'-6''</td>
<td>4'-0''</td>
<td>0.3</td>
<td>6#4 @ 12''</td>
<td>#4 @ 12''</td>
<td>1'-6''</td>
<td>3/4''</td>
</tr>
<tr>
<td>W8X12</td>
<td>2'-6''</td>
<td>4'-0''</td>
<td>0.7</td>
<td>6#4 @ 12''</td>
<td>#4 @ 12''</td>
<td>2'-6''</td>
<td>3''</td>
</tr>
<tr>
<td>W8X18</td>
<td>2'-6''</td>
<td>5'-6''</td>
<td>1.0</td>
<td>6#6 @ 12''</td>
<td>#4 @ 12''</td>
<td>2'-6''</td>
<td>3''</td>
</tr>
<tr>
<td>W10X22</td>
<td>2'-6''</td>
<td>6'-0''</td>
<td>1.2</td>
<td>6#8 @ 12''</td>
<td>#4 @ 12''</td>
<td>3'-0''</td>
<td>3/4''</td>
</tr>
</tbody>
</table>

The volume of concrete shown in Table does not include additional concrete that may be required when the foundation is in a slope and must be extended so that the top of the foundation is flush with the uphill side. See detail above.

*For exceptions see Note 1.*
For signs twelve (12) inches or less in actual width to be installed on parapets, the type A barrier wall sign support bracket described in Section 657 of the Standard Specifications shall be specified in lieu of the Type K or L bridge or retaining wall sign mounting brackets. This is provided the allowable loading on the Type A barrier wall sign support bracket will not be exceeded. The type a barrier wall sign support bracket is required to withstand loading which meets or exceeds that which will be generated based on the limits provided for the three (3) lbs per foot U-channel support on the support size selection chart on Sheet TE1-7A. If the type a bracket is specified, the “square tube support,” 2.00x36GA” bid item shall be specified and used for payment of the support.

NOTES:
1. The following guidelines should be followed when selecting the number of supports to be used with the Type K bracket:
   - Signs greater than 26 in. wide should be installed on a minimum of two (2) supports. 36” diamons excluded.
2. Only U-channel supports shall be used with Type K brackets. Refer to Chart on TE1-7A to confirm 36” U-channel will work for the sign to be installed.
3. Anchor size shall be specified by the mount fabricator along with the anchorage. Each anchor shall be designed for a maximum service tensile load of 4,000 lbs and shear service load of 2,000 lbs. A factor of safety of 4 shall be applied to these loads when selecting the anchors. Hole size shall be 1” or larger than anchor diameter. The designer shall verify that the structure (bridge, deck, wall, etc.) has the capacity to support the sign structure loading.
4. All items shown on this detail sheet shall be in accordance with Section 657 of the West Virginia Division of Highways standard specifications, Roads and Bridges, current edition and all current supplemental specifications.

NOTICE:
- Skin size and shape may vary. See Contract Plans.
- Plate “a” is 3/4” x 5” x 15” with 5/8” fillet weld.
- Plate “b” is 3/4” x 5” x 15” with 5/8” fillet weld.
- Holes for eight (8) anchors with nut and washer. See Note 3.

- Holes for eight (8) anchors with nut and washer. See Note 3.
** TYPE K - THREE SUPPORTS **

** SECTION B-B **
- Plate 5⁄8" x 10" x 14"
- Hole for galvanizing
- Holes for four (4) anchors w/HS nut and washer
- See note 3 on TE2-1A

** SECTION C-C **
- 1⁄2" plate 18" "PLATE "a"
- Hole for galvanizing
- Plate 4⁄16" x 3⁄8" x 18" with 3⁄16" fillet weld

** SECTION D-D **
- 3⁄8" plate 18"
- Plate 5⁄8" x 3⁄8" x 18" with 3⁄16" fillet weld
- Plate 5⁄8" x 3⁄8" x 18" with 3⁄16" fillet weld

** DETAIL E **
- Plate "b"
- "PLATE "b"'
- For galvanizing
- Hole for 1" Dia hole of 1" dia hole

** FRONT VIEW **
- Plate "a"
- Plate "b"
- Plate "c"

** ELEVATION **
- Sign post
- Barrier shape may vary
- Sign size and shape varies

** NOTE **
- See notes on TE2-1A

** OBSOLETE **
- See notes on TE2-1A
TYPE L - PIPE POST MOUNT

NEW CONSTRUCTION

- BENT ANCHOR BOLT
  - 2 REQUIRED

- U-TYPE ANCHOR BOLT
  - FULLY GALVANIZED

- HEX NUT FLAT WASHER AND FLAT WASHER (ALL GALV.:

- PIPE POST TYPE SIGN SUPPORT

SECTION A-A

BRACKET TOP VIEW

BRACKET FRONT VIEW

BRACKET SIDE VIEW

FOR BARRIER DIMENSIONS REFER TO STANDARD DETAILS BOOK VOLUME II, SHEET BM6670A.

NOTES:
1. MATERIAL USED TO FABRICATE THE BRACKET, GALVANIZED ANCHOR BOLTS, AND SUPPORT TO BRACKET CONNECTION BOLTS SHALL MEET THE REQUIREMENTS CONTAINED IN THE SPECIFICATIONS.
2. ANY AND ALL MATERIALS, EQUIPMENT, LABOR, INCIDENTALS, ETC. NECESSARY TO COMPLETE THE INSTALLATION SHALL BE BID AS ITEM 057050-001, BRIDGE OR RETAINING WALL BRACKET, TYPE L.
3. TYPE L BRACKET FOR USE WITH PIPE POST TYPES 6 & 9 ONLY. SEE STANDARD SHEET TE2-900 AND TE2-905 FOR PIPE POST DETAILS.
4. EACH ANCHOR SHALL BE DESIGNED FOR A MAXIMUM SERVICE TENSILE LOAD OF 4,200 LBS AND SHEAR SERVICE LOAD OF 1,600 LBS. A FACTOR OF SAFETY OF 4 SHALL BE APPLIED TO THESE LOADS. WHEN SELECTING THE ANCHORS, THE DESIGNER SHALL VERIFY THAT THE STRUCTURE (BARRIER, BRIDGE DECK, WALL, ETC.) HAS ADEQUATE CAPACITY TO SUPPORT THE SIGN STRUCTURE LOADING.

RETROFIT

- BENT ANCHOR BOLT
  - FULLY GALVANIZED

- HEX NUT FLAT WASHER AND FLAT WASHER (ALL GALV.:

- PIPE POST TYPE SIGN SUPPORT

FOR SIGNS TWELVE (12) INCHES OR LESS IN ACTUAL WIDTH TO BE INSTALLED ON PARAPETS, THE TYPE A BARRIER WALL SIGN SUPPORT BRACKET DESCRIBED IN SECTION 657 OF THE STANDARD SPECIFICATIONS SHALL BE SPECIFIED IN LIEU OF THE TYPE K OR L BRIDGE OR RETAINING WALL SIGN MOUNTING BRACKETS. THIS IS PROVIDED THE ALLOWABLE LOADING ON THE TYPE A BRACKET WALL SIGN SUPPORT BRACKET WILL NOT BE EXCEEDED. THE TYPE A BARRIER WALL SIGN SUPPORT BRACKET IS REQUIRED TO WITHSTAND LOADING WHICH MEETS OR EXCEEDS THAT WHICH WILL BE GENERATED BASED ON THE UNITS PROVIDED FOR THE THREE (3) LB PER FOOT U-CHANNEL SUPPORT ON THE SUPPORT SIZE SELECTION CHART ON SHEET TE-74A. IF THE TYPE A BRACKET IS SPECIFIED, THE "SQUARE TUBE SUPPORT," 2.00X14GA" BID ITEM SHALL BE SPECIFIED AND USED FOR PAYMENT OF THE SUPPORT.

NOTES:
- FOR BOLT DIAMETER.
- 2.00X14GA" BID ITEM SHALL BE SPECIFIED AND USED FOR PAYMENT OF THE SUPPORT.

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
STANDARD DETAILS

STANDARD DETAIL

PREPARED: 8/2018

3/6/2023
1. Materials used to manufacture anchor bolts, TS post plates, and hardware shall be in accordance with the standard specifications. All components shall be galvanized in accordance with the standard specifications.

2. All signs less than 36 inches in width may be mounted to the TS support without the galvanized steel bar using the standard punching pattern for direct mount types shown on TP1 series standards.

3. Vertical placement of galvanized steel bars shall match the vertical placement of the standard punching pattern shown on the TP1 series standards. The galvanized steel bars may be trimmed as needed to achieve the 3 inch min. edge clearance. Additional holes shall be field punched in the center of the sign for attachment to the steel bars and the TS support.

4. Costs for concrete barrier sign support shall be included in item 67000-001, barrier wall bracket, type D.

5. Every effort shall be made to locate the center of square plate at the midpoint of the space between two joints of the barriers. In no case shall the edge of the base plate be less than 6 inches from joints in barrier.

6. Sign widths and mounting heights shall be in conformance with TP3-1A.

7. Before specifying the use of the type D barrier wall sign support bracket, due consideration shall be given to the use of either the type A or B barrier wall sign support bracket as described in section 657 of the standard specifications. For barrier sections ten (10) inches or wider in width at the top, the type B bracket shall be considered. For barrier sections less than ten (10) inches in width at the top, the type A bracket shall be considered. Both the type A and B barrier wall sign support brackets are required to withstand a loading which meets or exceeds that which will be generated based on the limits provided for the three (30) lb per foot U-channel support on the support size selection chart on sheet TP1-7A. If either the type A or B bracket is specified, the "Square tube support, 2.00X14GA" bid item shall be specified and used for payment of the support.

8. Joint costs for concrete barrier sign support shall be included in item 67000-001, barrier wall bracket, type D.

9. Before specifying the use of the type D barrier wall sign support bracket, due consideration shall be given to the use of either the type A or B barrier wall sign support bracket as described in section 657 of the standard specifications. For barrier sections ten (10) inches or wider in width at the top, the type B bracket shall be considered. For barrier sections less than ten (10) inches in width at the top, the type A bracket shall be considered. Both the type A and B barrier wall sign support brackets are required to withstand a loading which meets or exceeds that which will be generated based on the limits provided for the three (30) lb per foot U-channel support on the support size selection chart on sheet TP1-7A. If either the type A or B bracket is specified, the "Square tube support, 2.00X14GA" bid item shall be specified and used for payment of the support.

10. The barrier wall sign support shall be less than 6 inches from joints in barrier.

11. The edge of the base plate at the midpoint of the space between two joints of the barriers. In no case shall the edge of the base plate be less than 6 inches from joints in barrier.

12. Sign widths and mounting heights shall be in conformance with TP3-1A.

13. Before specifying the use of the type D barrier wall sign support bracket, due consideration shall be given to the use of either the type A or B barrier wall sign support bracket as described in section 657 of the standard specifications. For barrier sections ten (10) inches or wider in width at the top, the type B bracket shall be considered. For barrier sections less than ten (10) inches in width at the top, the type A bracket shall be considered. Both the type A and B barrier wall sign support brackets are required to withstand a loading which meets or exceeds that which will be generated based on the limits provided for the three (30) lb per foot U-channel support on the support size selection chart on sheet TP1-7A. If either the type A or B bracket is specified, the "Square tube support, 2.00X14GA" bid item shall be specified and used for payment of the support.

14. Joint costs for concrete barrier sign support shall be included in item 67000-001, barrier wall bracket, type D.

15. Before specifying the use of the type D barrier wall sign support bracket, due consideration shall be given to the use of either the type A or B barrier wall sign support bracket as described in section 657 of the standard specifications. For barrier sections ten (10) inches or wider in width at the top, the type B bracket shall be considered. For barrier sections less than ten (10) inches in width at the top, the type A bracket shall be considered. Both the type A and B barrier wall sign support brackets are required to withstand a loading which meets or exceeds that which will be generated based on the limits provided for the three (30) lb per foot U-channel support on the support size selection chart on sheet TP1-7A. If either the type A or B bracket is specified, the "Square tube support, 2.00X14GA" bid item shall be specified and used for payment of the support.
THE STRUCTURES ARE DESIGNED IN ACCORDANCE WITH THE AMERICAN STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES, AND TRAFFIC SIGNALS, 6TH EDITION, 2013, USING 90 MPH WIND SPEED AND FATIGUE CATEGORY 1.

2. FOR SECTION A-A, B-B & D-D, SEE TE4-5.

3. FOR FOUNDATION NOTES, SEE TE4-5.

4. FOR FOUNDATION NOTES, SEE TE4-5.

5. Hi-STRENGTH BOLTS SHALL CONFORM TO THE REQUIREMENTS OF THE SPECIFICATIONS, WHETHER ALL HIGH STRENGTH BOLTS IN ACCORDANCE WITH THE SPECIFICATIONS.

6. DETAILS LABELED AS NOT TO SCALE ARE INTENTIONALLY NOT DRAWN TO SCALE FOR VISUAL CLARITY.

7. THE REMOVABLE CAP SHOULD BE A FRICTION CAP FOR REQUIREMENTS AND DETAILS, SEE NOTES ON SHEET TE4-5.

8. IF THE FOUNDATION IS WITHIN OR PROJECTS INTO A CONCRETE OR ASPHALT SURFACE UTILIZED BY PEDESTRIANS, THE GUIDELINES PROVIDED IN SECTION 658 OF THE STANDARD SPECIFICATIONS SHALL BE FOLLOWED IN RESPECT TO PLACEMENT AND FRET HEIGHT. OTHERWISE, ALL FACES OF THE FOUNDATION SHALL BE A MINIMUM OF 18 IN ABOVE GROUND LEVEL. WHEN FOUNDATION IS INSTALLED ON A SLOPE, THE 18 IN. SHALL BE APPLIED TO THE UPHILL FACE.

9. FOR A STRUCTURE WITH ARM LENGTH VARYING FROM THE DESIGN LENGTHS SPECIFIED, SIZE MEMBER DIMENSIONS BASED ON THE NEXT LONGER ARM LENGTH IN THE CHART AND ADJUST PANEL WIDTH (B) ACCORDINGLY WHILE RETAINING THE NUMBER OF PANELS (N).

10. SEE SHEET TE4-3A FOR GROUNDING NOTES.

11. DEPTH OF FOUNDATION IS BASED ON AN ASSUMED SOIL SUCH AS MEDIUM CLAY OR SAND CLAY PROVIDING THAT THE FRICTION ANGLE IS NOT LESS THAN 30 DEGREES.

12. THE SPECIFICATIONS. TIGHTEN ALL HIGH STRENGTH BOLTS IN ACCORDANCE WITH HI-STRENGTH BOLTS SHALL CONFORM TO THE REQUIREMENTS OF THE STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES, AND TRAFFIC SIGNALS, 6TH EDITION, 2013, USING 90 MPH WIND SPEED AND FATIGUE CATEGORY 1.

13. FOR ANCHOR BOLT DETAIL, SEE TE4-5.

14. FOR FOUNDATION NOTES, SEE TE4-5.

15. FOR SECTION A-A, B-B & D-D, SEE TE4-5.

NOTES:

1. BUTTERFLY STYLE CANTILEVER SIGN SUPPORTS ARE MADE UP OF TWO DOUBLE ARM CANTILEVERS ON OPPOSITE SIDES OF ONE SUPPORT POST. THE PRIMARY ARM AND THE SECONDARY ARM. IF DIFFERENT, THE PRIMARY ARM SHALL ALWAYS BE THE LONGER OF THE TWO. POST SIZE AND ARM CONFIGURATION (CHORD, BRACING, AND 'A' DIMENSION, ETC.) SHALL BE DETERMINED BASED ON THE PRIMARY ARM.

2. THE TOTAL AREA OF BOTH ARMS COMBINED SHALL NOT EXCEED THE MAX SIGN AREA LISTED IN THE MEMBER SIZE CHART ON TE4-3A FOR THE PRIMARY ARM.

3. SEE TABLES ON TE4-3A FOR STRUCTURE FABRICATION AND FOUNDATION DETAILS.

4. BUTTERFLY CANTILEVERS SHALL HAVE DESIGN NUMBERS IN THE FORMAT OF BC-XX-YY, WHERE XX = LENGTH OF PRIMARY ARM AND YY = LENGTH OF SECONDARY ARM.

5. FOR EXAMPLE, A BC-32-16 WOULD HAVE A PRIMARY ARM 32 FEET IN LENGTH AND A SECONDARY ARM 16 FEET IN LENGTH IT WOULD HAVE A 30 INCH DIAMETER POST AND BOTH ARMS WOULD HAVE RX400 CHORDS, RX1110 BRACING, AND 'A' DIMENSION OF 6'-0".

6. THE TOTAL AREA OF SIGNS ON BOTH ARMS COMBINED CANNOT EXCEED 430 SF.
NOTES:

1. **TYPE 1 CLAMP**:
   - For use with flat sheet signs only.
   - Must have at least 2 clamps (bands) per sign.
   - Max. area per band to be 9 sq. ft.
   - Max. sign width to be 3 ft. (includes 3 ft. diamond).

2. **TYPE 2 CLAMP**:
   - Used for attachment of signs to vertical tubes.
   - Can be used for flat sheet or extruded panel signs.
   - Flat sheet signs must have extruded ribbing.
   - Use signs (see sheet TE7-1) at upper ZEE to U-channel connection on tapered posts.

3. **TYPE 3 CLAMP**:
   - Used for attachment of signs to horizontal tubes.
   - Can be used for flat sheet or extruded panel signs.
   - Flat sheet signs must have extruded ribbing.

4. Contact between aluminum and galvanized parts shall be prevented with a minimum 1/16 inch thick gasket. Gaskets are not required between stainless steel and aluminum.

5. Signs mounted using TYPE 1 clamps shall be mounted using the standard punching as shown in the TP series if possible. If holes are required to be field punched, the punching shall be approved by the Engineer. The holes shall be punched such that spacing between the holes and from the outermost holes to the edges of the sign are uniform in addition, the hole locations shall be placed such that the attachment hardware will not unnecessarily interfere with the sign message.

6. See sheet TE7-1 regarding details for attachment of flat sheet sign face to extruded rib and for extruded rib dimensioning details.

7. See sheet TE7-1 for extruded sign panel, post clip, and stitch bolt details.

8. All U-bolts to include saddle between tube and ZEE bar.

**ELEVATION**

**TYPE 1**
- Vertical tube mount flat sheet sign with extruded rib shown.
- Contact between aluminum and galvanized parts shall be prevented with a minimum 1/16 inch thick gasket. Gaskets are not required between stainless steel and aluminum.
- Signs mounted using TYPE 1 clamps shall be mounted using the standard punching as shown in the TP series if possible. If holes are required to be field punched, the punching shall be approved by the Engineer. The holes shall be punched such that spacing between the holes and from the outermost holes to the edges of the sign are uniform in addition, the hole locations shall be placed such that the attachment hardware will not unnecessarily interfere with the sign message.

**TYPE 2**
- Used for attachment of signs to vertical tubes.
- Can be used for flat sheet or extruded panel signs.
- Flat sheet signs must have extruded ribbing.

**TYPE 3**
- Used for attachment of signs to horizontal tubes.
- Can be used for flat sheet or extruded panel signs.
- Flat sheet signs must have extruded ribbing.

**SIGN CLAMPS FOR TUBULAR SUPPORTS**

**STANDARD SHEET TE9-1**

- Vertical or horizontal mount.
- Max. 2 bands per sign.
- Max. 30 sq. ft. per band.
- Min. 2 bands per sign.

CONTACT BETWEEN ALUMINUM AND GALVANIZED PARTS SHALL BE PREVENTED WITH A MINIMUM 1/16 INCH THICK GASKET. GASKETS ARE NOT REQUIRED BETWEEN STAINLESS STEEL AND ALUMINUM.

SIGNS MOUNTED USING TYPE 1 CLAMPS SHALL BE MOUNTED USING THE STANDARD PUNCHING AS SHOWN IN THE TP SERIES IF POSSIBLE. IF HOLES ARE REQUIRED TO BE FIELD PUNCHED, THE PUNCHING SHALL BE APPROVED BY THE ENGINEER. THE HOLES SHALL BE PUNCHED SUCH THAT SPACING BETWEEN THE HOLES AND FROM THE OUTERMOST HOLES TO THE EDGES OF THE SIGN ARE UNIFORM IN ADDITION, THE HOLE LOCATIONS SHALL BE PLACED SUCH THAT THE ATTACHMENT HARDWARE WILL NOT UNNECESSARILY INTERFERE WITH THE SIGN MESSAGE.

FLAT SHEET SIGNS MUST HAVE EXTRUDED RIBBING.
- CAN BE USED FOR FLAT SHEET OR EXTRUDED PANEL SIGNS.
- USED FOR ATTACHMENT OF SIGNS TO VERTICAL TUBES.

FLARED LEG BRACKET

- Threaded for 3/8-18 bolt.
- Stitch bolt details.
- USE SHIMS (SEE SHEET TE17-1) AT UPPER ZEE TO U-COMMENT CONNECTION ON TAPERED POSTS.

- Max. sign width to be 3 ft. (includes 3 ft. diamond).
- Max. area per band to be 9 sq. ft.
- Max. sign width to be 3 ft. (includes 3 ft. diamond).

- For use with flat sheet signs only.
- Must have at least 2 clamps (bands) per sign.
- Flat sheet signs must have extruded ribbing.

- Use signs (see sheet TE7-1) at upper ZEE to U-channel connection on tapered posts.

- Contact between aluminum and galvanized parts shall be prevented with a minimum 1/16 inch thick gasket. Gaskets are not required between stainless steel and aluminum.

- Signs mounted using TYPE 1 clamps shall be mounted using the standard punching as shown in the TP series if possible. If holes are required to be field punched, the punching shall be approved by the Engineer. The holes shall be punched such that spacing between the holes and from the outermost holes to the edges of the sign are uniform in addition, the hole locations shall be placed such that the attachment hardware will not unnecessarily interfere with the sign message.

- See sheet TE7-1 regarding details for attachment of flat sheet sign face to extruded rib and for extruded rib dimensioning details.

- See sheet TE7-1 for extruded sign panel, post clip, and stitch bolt details.

- All U-bolts to include saddle between tube and ZEE bar.

- Vertical or horizontal mount.
- Max. 2 bands per sign.
- Max. 30 sq. ft. per band.
- Min. 2 bands per sign.

- Vertical or horizontal mount.
- Max. 2 bands per sign.
- Max. 30 sq. ft. per band.
- Min. 2 bands per sign.
GENERAL NOTES

1. TYPE A BOXES ARE TO BE FABRICATED FROM STEEL 1/8" IN THICKNESS MIN. AND HOT-DIPPED GALVANIZED AFTER ASSEMBLY.

2. REINFORCING STEEL THAT CONFLICTS WITH TYPE A BOX SHALL BE APPROPRIATELY MODIFIED AS SHOWN ON THE BRIDGE PLANS OR AS DIRECTED BY THE ENGINEER.

3. UNUSED CONDUIT TO BE FIELD CAPPED.

4. JUNCTION BOXES SHOULD BE NEMA 3R RATED.

5. HUB SIDE AND NUMBER ARE TO BE AS REQUIRED FOR EACH SPECIFIC APPLICATION.
1. **Location:**
   - The push button must be within accessible reach range of a level landing for use by a wheelchair. The button may be placed up to 10 inches from the level landing area.
   - The optimal location for the push button is between the curb ramp and the edge of the crosswalk line extended farther from the corner. If the optimal location is not possible, the push button needs to be less than 5 feet from the edge of the crosswalk line extended farther from the corner.
   - The push button should be between 1.5 feet and 6 feet, but no further than 10 feet from the edge of the curb, shoulder, or pavement unless otherwise shown in the contract plans.

2. **Push Button Unit:**
   - The push button shall be mounted at a height of 3 ft-6 in above the surface of the sidewalk unless otherwise specified on the contract plans.
   - Tactile arrows on pedestrian push buttons shall be oriented parallel to the direction of travel on the crosswalk controlled by the push button.
   - Push button shall be mounted as per manufacturer's recommendations.
   - Audible pedestrian push buttons shall incorporate a push button with vibration, audible message and tactile relief symbol.
   - The push button shall be a combination pushbutton/sign combination and a model listed in the APL.

3. **Sign:**
   - The sign shall conform to the sign designated as R10-3aL or R as shown in the West Virginia sign fabrication details manual.
   - The sign shall be mounted immediately above the push button and be an integral part of the push button unit.
   - Signs shall be 0.080 in flat sheet aluminum and fabricated according to West Virginia standards for sheeting and design unless otherwise specified on the contract plans.

4. **Stub Post Support:**
   - Use stub post type support when a type A1, C1 or E pole is not within reach range of an accessible level landing area.
   - Push button mounted to PPB shall be within reach range of an accessible level landing area.
   - Mount push button as per manufacturer's recommendations.

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**GENERAL NOTES**

**PPB INSTALLATION**

ON WOOD OR METAL STUB POST

6"X6" WOOD POST WITH PPB

3" CONDUIT POST WITH PPB

**PPB INSTALLATION**

ON TYPE A1, C1 OR E POLE

**PPB LOCATION**

SEE NOTE 1

**NOTE 1:**

A. Use stub post type support when a type A1, C1 or E pole is not within reach range of an accessible level landing area.
B. Push button height to be based on minimum required clearance to PPB.
C. Mount push button as per manufacturer's recommendations.
**General Notes**

1. Broken lines shall be 10 feet in length with 30 feet spaces, unless otherwise specified. The ratio of painted line length to skip length shall be 1 to 3.

2. Stop lines shall be 12 inches in width unless one of the following conditions are met, in which case the width shall be 24 inches:
   - The stop line is on the approach to a separated intersection.
   - The stop line is at the end of an interstate or expressway interchange exit ramp.
   - The posted speed limit of the roadway that the stop line is placed is 35 mph or greater.
   - Stop lines shall be placed 4 feet in advance of and parallel to the nearest crosswalk line. The stop line should be placed at the desired stopping point, but in no case more than 30 feet or less than 3 feet from the nearest edge of the intersecting traveled way.

3. Supplemental pavement word and/or symbol markings should be limited to not more than a total of three lines of information (words and/or symbols). They shall be white in color. Letters, symbols, and numerals shall be a minimum of 6 feet in height. The word marking "ONLY" and the arrow shall be used when a movement that would otherwise be legal is to be prohibited. The space between lines shall be at least four times the height of the characters low speeds but not more than ten times the height of the characters under any conditions. Location of supplemental pavement markings shall be as shown or as dimensioned on the plans.

4. The spacing between adjacent yellow centerline markings shall be equal to the line width.

5. All longitudinal markings shall be from the pavement joints as specified in the standard specifications.

6. Normally, the maximum lane width shall be 12 feet. Single lane ramp widths shall be 16 feet.

7. Dual left turn lanes shall be separated by dashed white lines 2 feet long with 6 feet spaces where engineering judgment determines that such additional markings are needed. The width of the dashed line shall be equal to the width of the line that the dashes originate from, unless specified.

8. Left turn movements may be guided by dashed yellow lines 2 feet long with 6 feet spaces, where engineering judgment determines that such additional markings are needed. The width of the dashed line shall be equal to the width of the line that the dashes originate from, unless specified.

9. If the distance between the preceding intersection and the approach intersection is 1 mile or less, the dashed lane line shall be extended back to the preceding intersection. Otherwise, the dashed lane line shall be a distance in advance of the intersection as determined by engineering judgment as being suitable to enable drivers who do not desire to make the mandatory turn to move out of the lane being dropped prior to reaching the queue of vehicles that are waiting to make the turn. The dashed lane line shall begin no closer to the intersection than the most upstream regulatory or warning sign associated with the lane drop.

10. The Type V material used for crosswalk markings shall be enhanced skid resistant material as specified in the specification for Type V materials. Enhanced skid resistant material shall also be used for other Type V markings indicated in the project plans.

**Supplemental Detail**

**Mandatory Turn Lane Markings**

**Typical Intersections Markings**

**Typical Lane-Use Marking Spacing**

**Methodology for Installing (Bending) Stop Lines at Wide Throated Intersections**

**Typical Parallel Crosswalk Line Details**

**Legend**

General Markings:
- 24 inches in width
- 30 feet spaces
- 10 feet in length

Arrows:
- 15' minimum
- Arrow only indicates direction of travel

**Typical Pavement Markings**

**Sheet 2 of 2**

**West Virginia Department of Transportation Division of Highways Standard Details**

**Standard Sheet TEM-2**

**Note:** All lane use markings shown on this sheet are optional as called for on plans.
APPENDIX A

RESOURCE AGENCY
ADDRESS LISTING AND SAMPLE LETTERS
April 19, 2023

A. West Virginia Department of Environmental Protection (WVDEP)

LETTERS AND PLANS

Larry D. Board
WV Dept. of Environmental Protection
Division of Water and Waste Management
601 57th Street SE
Charleston, WV 25304-2345
    Phone: (304)926-0499 Ext. 43763
    Fax: (304)926-0452
    E-Mail: Larry.D.Board@wv.gov

Brad Wright
Department of Environmental Protection
Enforcement
601 57th St. SE
Charleston, WV 25304
    Phone: (304) 926-0499, Ext. 49746
    E-Mail: Brad.M.Wright@wv.gov

Dawn A. Newell
Department of Environmental Protection
Water Quality
601 57th St. SE
Charleston, WV 25304
    Phone: (304) 926-0499, Ext. 41114
    E-Mail: Dawn.A.Newell@wv.gov
    Address Letter to Dawn A Newell, but send it via e-mail to:
    Jackie Thornton
    Jackie.N.Thornton@wv.gov

COPY OF LETTER ONLY

Katheryn Emery-Fultineer, Director
Department of Environmental Protection
Division Of Water and Waste
601 57th St. SE
Charleston, WV 25304
    Phone: (304) 926-0499, Ext 43830
    E-Mail: Katheryn.D.Emery@wv.gov
B. West Virginia Division of Natural Resources (WVDNR)

Entire State

COPY OF ALL LETTERS

Mr. Danny Bennett  
WV Division of Natural Resources  
Environmental Review  
738 Ward Road  
Elkins, WV 26241  
Phone: 304-637-0245  
E-Mail: Danny.A.Bennett@wv.gov

LETTERS AND PLANS

NOTE: Project submittals (coordination packages) shall include the following:

- Four color photographs
- Bankfull Measurement – Plan View
- Bankfull Measurement – Cross Section View
- Counter Sinking Depth – Cross Section View

Ms. Anne Wakeford  
WVDNR-Wildlife Resources  
738 Ward Road  
Elkins, WV 26241  
Phone: (304) 637-0245  
Fax: (304) 637-0250  
Email: Anne.M.Wakeford@wv.gov

WVDNR – District Fisheries Biologists

COPY OF LETTER AND PLANS (NO CROSS-SECTIONS)
a. District 1 (Hancock, Brooke, Ohio, Marshall, Wetzel, Monongalia, Marion, Harrison, Taylor, Preston, Barbour, Tucker counties)  
   Mr. Dave Wellman  
   WV Division of Natural Resources  
   Fisheries Management  
   PO Box 99  
   1110 Railroad Street  
   Farmington, West Virginia 26571  
   Phone: (304) 825-6787  
   Fax: (304) 825-6270  
   E-Mail: David.I.Wellman@wv.gov
b. District 2 (Grant, Pendleton, Mineral, Hampshire, Hardy, Morgan, Berkeley, and Jefferson counties)
   Mr. Brandon Keplinger  
   WV Division of Natural Resources  
   Fisheries Management  
   1 Depot Street  
   Romney, WV 26757  
   Phone: (304) 822-3551  
   Fax: (304) 822-7331  
   E-Mail: Brandon.J.Keplinger@wv.gov

c. District 3 (Clay, Braxton, Nicholas, Lewis, Upshur, Webster, Randolph, and Pocahontas counties)
   Mr. James Walker  
   WV Division of Natural Resources  
   WV State Wildlife Center  
   P. O. Box 38  
   French Creek, WV 26218  
   Phone (304) 924-6211  
   Fax (304) 924-6781  
   E-Mail: James.A.Walker@wv.gov

d. District 4 (Fayette, Greenbrier, Raleigh, Summers, Monroe, Wyoming, McDowell, and Mercer counties)
   Mr. Glenn R Nelson  
   WV Division of Natural Resources  
   2006 Robert C. Byrd Drive Beckley,  
   WV 25801-8320  
   Phone (304) 256-6947  
   Fax (304) 256-6948  
   E-Mail: Glenn.R.Nelson@wv.gov

e. District 5 (Mason, Putnam Kanawha, Cabell, Wayne, Lincoln, Boone, Mingo, and Logan counties)
   Mr. Jeff Hansbarger  
   WV Division of Natural Resources  
   Fisheries Management  
   50 Rocky Branch Road  
   Alum, WV 25003  
   Phone 304-756-1023  
   Fax: 304-756-1055  
   E-Mail: Jeff.L.Hansbarger@wv.gov
f. **District 6 (Tyler, Pleasants, Doddridge, Wood, Ritchie, Jackson, Wirt, Calhoun, Gilmer, and Roane counties)**

   Mr. Nate Taylor  
   WV Division of Natural Resources  
   Fisheries Management  
   3211 Ohio Avenue  
   Parkersburg, WV 26101-2559  
   Phone (304) 420-4550  
   Fax (304) 420-4554  
   E-Mail: Nate.D.Taylor@wv.gov

C. **US Army Corps of Engineers**

   **NOTE:** All Corridor “H” projects and Route “9” projects contact CH (DK) for Agency Distribution List

   **Entire State**

   **LETTERS AND PLANS**

   Mr. Michael E. Hatten, Chief  
   Regulatory Branch  
   Huntington District, Corps of Engineers  
   502 Eighth Street  
   Huntington, WV 25701-2070  
   Phone: (304) 399-5710  
   Fax: (304) 399-5590  
   E-mail: Michael.E.Hatten@usace.army.mil

   cc Susan A. Porter  
   Chief, South/Transportation Branch  
   Regulatory Division  
   USACE, Huntington District, CELRH-RDS  
   502 8th Street  
   Huntington, WV 25701  
   E-mail: Susan.A.Porter@usace.army.mil
D. **US Fish and Wildlife Service - USFWS**

Entire State

*LETTERS AND PLANS*

Jared Varner  
Senior Endangered Species Biologist  
West Virginia Field Office  
U.S. Fish and Wildlife Service  
6263 Appalachian Highway  
Davis, WV 26260  
Phone: 304-866 3858  
E-Mail: FW5_WVFO@fws.gov

E. **Environmental Protection Agency – EPA Region III**

Entire State

*LETTERS AND PLANS*

**NEPA**

Ms. Samantha Beers, Director  
Office of Community, Tribes & Environmental Assessment  
EPA Region III (3RA10)  
1650 Arch St.  
Philadelphia, PA 19103  
Phone: 215-814-2627  
E-Mail: beers.samantha@epa.gov

Barbara Rudnick, P.G. NEPA Program Coordinator  
U.S. EPA Region III  
Office of Communities, Tribes & Environmental Assessment  
1650 Arch Street (3RA10)  
Philadelphia PA 19103  
Phone: 215-814-3322  
E-Mail: Rudnick.Barbara@epa.gov

**404/401 Corps Permits and Water Quality Certification**

Jeff Lapp  
Chief - Wetlands Branch  
U.S. EPA Region III Water Division  
1650 Arch Street(3WD10)  
Philadelphia, PA 19103-2029  
Phone: (215) 814-2717  
E-Mail: lapp.jeffrey@epa.gov