Standards Committee
Meeting Agenda
Wednesday, March 1, 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.

TITLE	Champion
None.	

New Business

<table>
<tr><th>TITLE</th><th>Champion</th></tr>
<tr><td>1st time to Committee.
Update of various Structure Directives (SD). The following SDs are included with brief summary of updates:
SD110-Project Design Criteria. Update to line and grade criteria.
SD1040-Structrural System Selection. Removed sections which are duplicated in SD 1041 through 1044.
SD1041-Steel Superstructure Type. 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.
SD1042-Concrete Superstructure Type. Revised text for more generality in superstructure type by span length.
SD1043-Abutment Types. Minor terminology updates.
SD1044-Pier Type. Terminology updates, such as FCM to NSTM
SD1073-Rehabilitation Techniques. 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.
SD2034-Fatigue Critical. Terminology update, such as FCM to NSTM
SD2045-Croncrete Superstructures. Remove strand diameter requirement to provide greater flexibility to the Designer.
</td><td>B. Neeley</td></tr>
</table>

Next Meeting Date: Wednesday, May 3, 2023.
Deadline for submissions: April 7, 2023.

Adjournment

Standards Committee
Meeting Minutes
January 4, 2023

Call to Order: The meeting was called to 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 11-16-2022 Meeting were approved without objection.

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

I. Eight proposed drawings and revisions to WVDOH Standard Details Book – Volume 3. These drawings are for steel superstructures and would be new 3300 Section were briefly explained, no discussion.

- ***a.*** *Sheet # 3300GN1 – Steel Standard Beam Notes 1 of 2*
- ***b.*** *Sheet # 3300GN2 - Steel Standard Beam Notes 2 of 2*
- **c.** *Sheet # 3320SB1 – Composite Steel Beam Sheet 1 of 6*
- **d.** *Sheet # 3320SB2 – Composite Steel Beam Sheet 2 of 6*
- **e.** *Sheet # 3320SB3 – Composite Steel Beam Sheet 3 of 6*
- **f.** *Sheet # 3320SB4 – Composite Steel Beam Sheet 4 of 6*
- **g.** *Sheet # 3320SB5 – Composite Steel Beam Sheet 5 of 6*
- **h.** *Sheet # 3320SB6 – Composite Steel Beam Sheet 9 of 6*

All eight proposed WVDOH Standard Details-Volume 3 were approved at the meeting. Vote 5-0.

II. *Structure Directive (SD) 2048 – Adjacent Box Beams*. The SD updates the Standard Bridge Plans references and approval requirements. There was a brief explanation, no discussion.

SD-2048 was approved at the meeting. Vote 5-0.

III. *Structure Directive (SD) 2150 – Load Rating of New Bridge Design*. The SD is an update the load rating requirements; to follow more closely with AASHTO Manual for Bridge Evaluation.

Adding a sentence to 2105.2 was suggested and reviewed, requiring analysis if any portion of the structure on longitudinal slope. Discussion ensued..

SD-2150 was approved, as noted, with added sentence to 2105.2. Vote 5-0.

IV. *Sign Fabrication Detail, R40-1- Your Highway Taxes at Work (Fed / State).* Revision to the Funding Source sign fabrication. There was a brief explanation and discussion.

R40-1 was approved at the meeting. Vote 5-0.

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

V. None.

Next Meeting: The next meeting is on Wednesday, March 1, 2023. Deadline for submissions February 9, 2023.

Adjournment: The meeting was adjourned.

Manuals Committee
Meeting Minutes
January 4, 2023

Call to Order: The meeting was called to 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. None.

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

II. **2023 Bridge Load Rating Manual (BLRM) for In-Service Bridges.** The manual describes the polices and procedure for load rating and posting of public road bridges. It is an update to the 2020 BLRM. It is a clean copy showing the proposed manual. Stephen Johnson and Craig Iser are the champions of this item.

Item was discussed. Approval expected at the next meeting.

III. **WVDOH Tunnel Inspection Manual.** The manual gives guidance and requirements of tunnel inspection and report requirements to meet state and federal code. It is a clean copy showing the proposed manual. Stephen Johnson and James Bennett are the champions of this item.

Item was discussed. Approval expected at the next meeting.

Next Meeting: The next meeting is on Wednesday, March 1, 2023. Deadline for submissions February 9, 2023.

Adjournment: The meeting was adjourned.

January Specification Committee Meeting
Wednesday, January 4, 2023
Attendee List

Virtual Meeting Attendees

1.	Allison, Vince	WVDOH – MCS&T Division
2.	Brown, Phil	WVDOH – MCS&T Division
3.	Cummings, John	WVDOH – MCS&T Division
4.	Elkins, Jerry	HNTB
5.	Farley, Paul	WVDOH – MCS&T Division
6.	Iser, Craig	WVDOH – Operations Division
7.	McGlumphy, Kevin	Associated Asphalt
8.	Mongi, Ahmed	HDR
9.	Moran, Tim	WVDOH – Operations Division
10.	Rinhart Conely, Laura	WVDOH – Technical Support
11.	Tennant, Robby	WVDOH – District 2
12.	Thaxton, Andrew	WVDOH – MCS&T Division
13.	Varney, Billy	TRC
14.	Wagner, Kayla	Mid Atlantic Maintenance
15.	Williams, Steve	Michael Baker

In Person Meeting Attendees

1.	Adkins, Janie	WVDOH – Technical Support Division
2.	Boggs, Steve	WVDOH – Technical Support Division
3.	Bennett, James	WVDOH – Operations Division
4.	Crane, John	Contractors Association of West Virginia
5.	Hypes, Jarred	WVDOH – Engineering
6.	Johnson, Stephen	WVDOH – Operations Division
7.	Long, Travis E	WVDOH – Technical Support Division
8.	Neeley, Joseph	WVDOH – District 1
9.	Scites, RJ	WVDOH – Engineering Division
10.	Smith, Shawn A	WVDOH – Contract Administration Division
11.	Whitmore, Ted	WVDOH – Traffic Engineering

TOTAL ATTENDEES: 26

DRAFT

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

STRUCTURE DIRECTIVE 1010
PROJECT DESIGN CRITERIA
January 25, 2023
Supersedes May 4, 2022
~~First Edition~~

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

DRAFT

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

STRUCTURE DIRECTIVE 1040
STRUCTURAL SYSTEM SELECTION
January 17, 2023
Supersedes May 4, 2022
~~*First Edition*~~

1040-STRUCTURAL SYSTEM SELECTION

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

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

The substructure consists of abutments, ~~and~~ piers 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.~~

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

~~Table 1040.A~~

~~The superstructure should be designed such that the structure has redundant load paths and is not considered fracture critical. Some designs, especially truss and tied arch designs, are generally, by their very nature, fracture critical. As defined in the Governing Specifications, a Fracture Critical Member (FCM) is a "*Component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function*". The Designer is to declare at Span Arrangement or TS&L if the structure is fracture critical. Design calculations, welding procedures, and material specifications can be incorporated into the project to make the use of these superstructure types acceptable.~~

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

~~High performance steel should also be considered when determining viable superstructure alternatives. It has been found to not only provide cost savings but also increase the serviceability of a structure. For a more detailed discussion, see the 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:~~

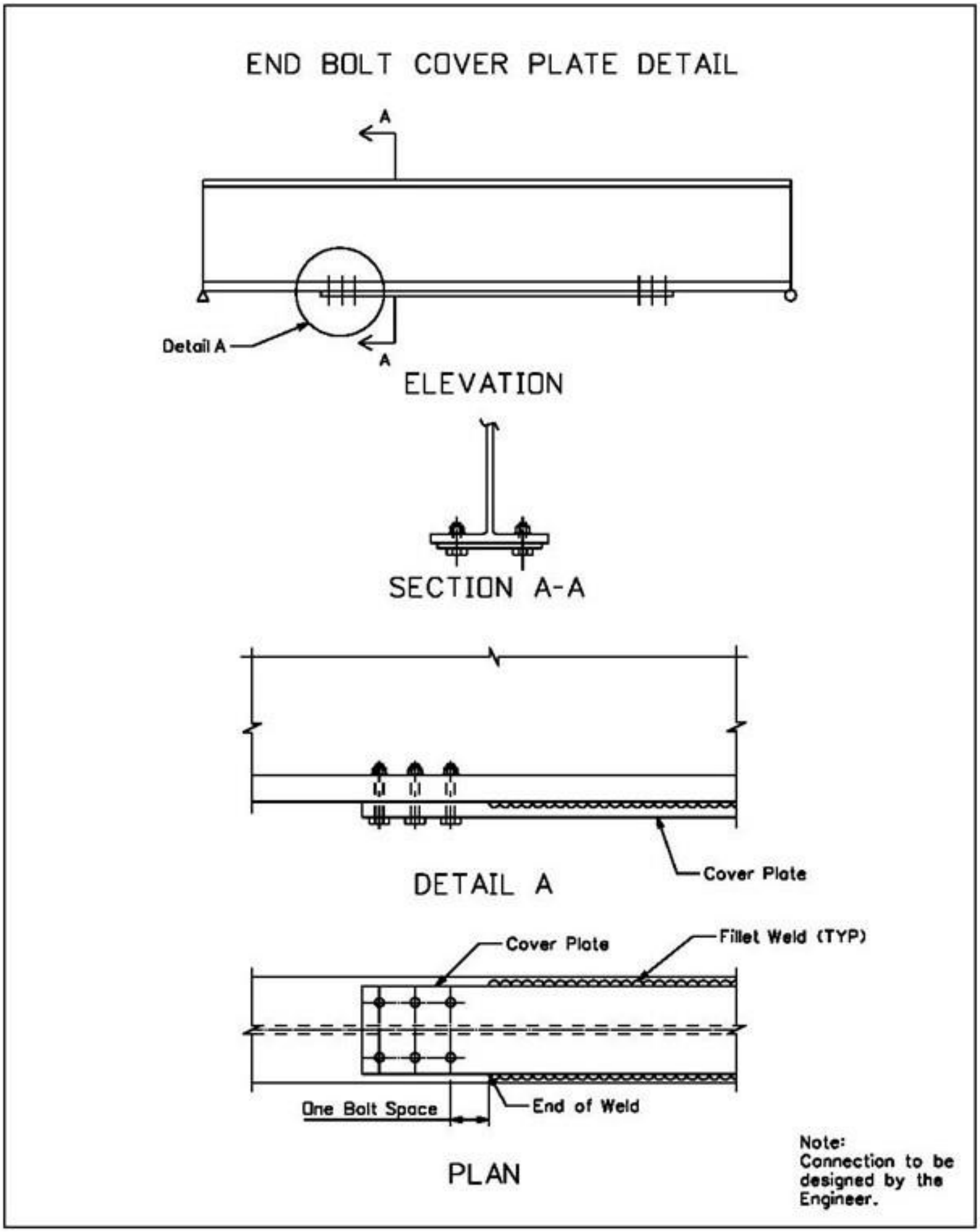
- ~~A. Wet environments.~~
- ~~B. Industrial areas where concentrated chemical fumes may drift directly onto the structure.~~
- ~~C. Grade separations resulting in "tunnel-like" conditions.~~
- ~~D. Low level water crossings.~~
- ~~E. Other locations as determined by the Bridge Project Manager.~~

~~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.~~



~~**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~~ $\cancel{^{7}/_{16}}$ ~~IN. Increment the web thickness by a minimum of~~ $\cancel{^{1}/_{16}}$ ~~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.~~

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

~~**Table 1040.C**~~

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

~~Concrete compressive strengths for commonly used precast beams shall be no less than 6000 PSI (5500 PSI for WVDOH Standard Box Beams) at release (f'_{ci}) with a minimum final compressive strength of 8000 PSI (f'_{c}).~~

~~High strength concrete (HSC) should also be considered when determining possible concrete superstructure alternatives. Precast beams may be designed using high strength concrete with a final compressive strength of up to 10000 PSI and a release strength of up to 9000 PSI. HSC allows engineers to design structures with smaller beams when clearance criteria needs to be met, reduce dead loads for more cost efficient substructures, and increase span lengths over conventional concrete.~~

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

~~**1040.2.1 Slab Bridges:** This superstructure type consists of a reinforced concrete slab with the main reinforcing parallel to the direction of traffic. This type of structure may be economical for very short span bridges, generally less than 30 FT in length.~~

~~**1040.2.2 Box Beams:** For short span bridges of 100 FT or less, prestressed concrete box beams may be considered an economical solution.~~

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

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

~~Note: All bridges, including adjacent box beam bridges, on routes designated as coal haul roads and/or subject to heavily loaded trucks shall have composite reinforced concrete decks.~~

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

~~**1040.2.3 Prestressed Concrete Beams**: AASHTO Type I, II, III, IV or Type IV Modified prestressed concrete beams should be considered for bridges with spans from 25 FT to 145 FT. The maximum span length is based on the haul capacity for a particular project site and shall be verified with a prestressed concrete beam supplier familiar with the project location. For continuous spans, the bridge system shall be designed simply supported for dead load and continuous for live load and superimposed dead load only. The Designer should minimize the number of beam lines. Prestressed concrete beam bridges should have a minimum of three beam lines.~~

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

~~Approximate Maximum Span Lengths (FT)~~

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

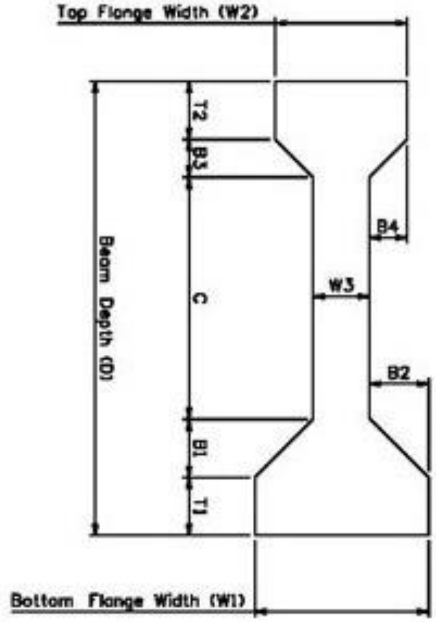
~~NOTE: These values are approximate and should be used for preliminary design purposes only. These values shall not be used for final design. The designs were based on single span (simply supported) bridges with 32 IN Type F barriers, no sidewalks and utilizing concrete with a release strength (f'_{ci}) of 6000 PSI and a final strength (f'_c) of 8000 PSI.~~

~~Table 1040.D~~

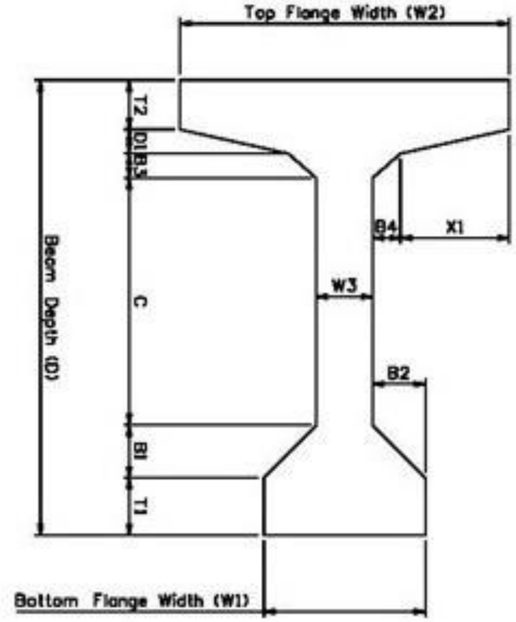
~~1040.2.4 Post-Tensioned I-Beams (Drop-In):~~ ~~Using post-tensioned drop-in spans can increase span lengths for prestressed concrete beams. The drop-in segments will be field spliced and beam post-tensioned as specified within the contract plans. At the field splice locations, temporary shoring towers 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.~~

WVDOT-DOH STANDARD PRESTRESSED CONCRETE I-BEAM SECTIONS



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



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

Beam Designation	Top Flange Width (IN) W2	Bottom Flange Width (IN) W1	Depth (IN) D	Depth (IN)						Flange (IN)			Web Thickness (IN) W3	Basic Beam Properties		
				T2	D1	B3	C	B1	T1	X1	B4	B2		Area (IN^2)	Y_b (IN)	I_{xx} (IN^4)
II	12	18	36	6	-	3	15	6	6	-	3	6	6	369	15.8	50,980
III	16	22	45	7	-	4½	19	7½	7	-	4½	7½	7	560	20.3	125,390
IV	20	26	54	8	-	6	23	9	8	-	6	9	8	789	24.7	260,730
IV MOD	36	26	60	4	2	3	34	9	8	11	3	9	8	860	28.8	384,248
IV MOD	36	26	66	4	2	3	40	9	8	11	3	9	8	908	31.6	491,660
IV MOD	36	26	72	4	2	3	46	9	8	11	3	9	8	956	34.4	615,361
IV MOD	36	26	78	4	2	3	52	9	8	11	3	9	8	1,004	37.3	756,222
IV MOD	36	26	84	4	2	3	58	9	8	11	3	9	8	1,052	40.2	915,113
V	42	28	63	5	3	4	33	10	8	13	4	10	8	1,013	31.9	521,180
VI	42	28	72	5	3	4	42	10	8	13	4	10	8	1,085	36.4	733,320

~~Figure 1040 E~~

~~**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.~~
~~F. Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS).~~

~~**1040.3.1 Wall Abutment:** This type of abutment, also known as a full height abutment, may be used when right-of-way is critical or the site does not permit a longer bridge with sloping embankments. Span lengths can be reduced using a wall type abutment. The footing may transfer loads by direct bearing (spread footing) or it may be supported on piles or 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. FHWAHRT-11-026, is followed.~~

~~**1040.3.7 Wingwalls:** Wingwalls are walls on either side of an abutment used to retain the roadway embankment. Wingwalls can be constructed of cast-in-place concrete or MSE walls and shall be designed as retaining walls. They shall be sufficiently sized to prevent the roadway embankment from spilling onto the abutment seats or into the clear area under the bridge.~~

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

~~**1040.4 PIER TYPES**~~

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

~~**1040.4.1 Cap-and-Column Type Piers:** Cap-and-column type piers have two or more circular or rectangular columns connected on top with a cap (a reinforced concrete beam that supports the superstructure). Generally, the pier cap ends will be cantilevered. For columns greater than 100 FT to 150 FT, the use of a compression strut at mid-height, similar to the pier cap, shall be investigated. The individual columns will be supported on an appropriate foundation.~~

~~**1040.4.2 T-Type or Hammerhead or Wall Type Piers:** T-type or Hammerhead piers have a deep rectangular tapered beam carrying the superstructure supported on a single wide rectangular or oval column in the middle. For wall type piers, the width of the rectangular column will be very close to the length of the pier cap. The single column will be supported on an appropriate foundation. In some situations, the feasibility of using a single large circular column instead of a wide rectangular or oval column has to be investigated during the preliminary design phase of the project.~~

1040.4.3 Post-Tensioned Concrete/Integral Pier Caps: To satisfy the vertical clearance requirement beneath a pier cap, a post-tensioned or integral pier cap shall be investigated.

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

1040.5 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 sidewall and bottom cleanliness, and concrete inspection. Results from the testing may require remedial action from the Contractor.~~

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WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

STRUCTURE DIRECTIVE 1041
STEEL SUPERSTRUCTURE TYPES
January 25, 2023
Supersedes May 4, 2022
~~*First Edition*~~

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

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

~~**Table 1041.A**~~

The superstructure should be designed such that the structure has redundant load paths and ~~is not considered fracture critical~~ does not contain Nonredundant Steel Tension Members (NSTM). Some designs, especially truss and tied arch designs, ~~are generally,~~ by their very nature, ~~fracture critical~~ 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~~ 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 length~~s 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 length~~s 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).

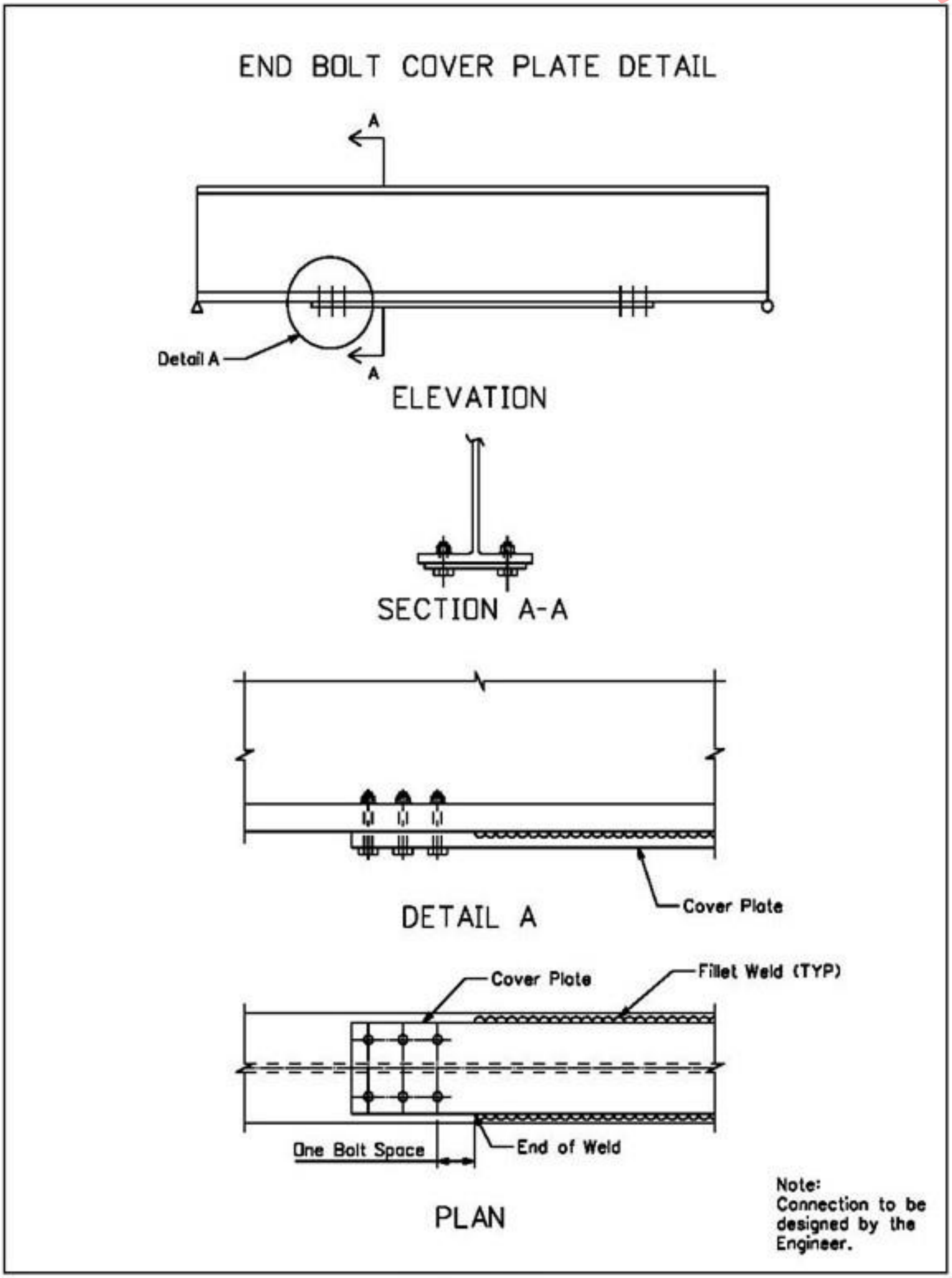

Figure 1041.B

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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.

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

STRUCTURE DIRECTIVE 1042
CONCRETE SUPERSTRUCTURE TYPES
January 17, 2023
Supersedes May 4, 2022
~~First Edition~~

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

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

~~**Table 1042.A**~~

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

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

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

1042.1-SLAB BRIDGES

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

1042.2-BOX BEAMS

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

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

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

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

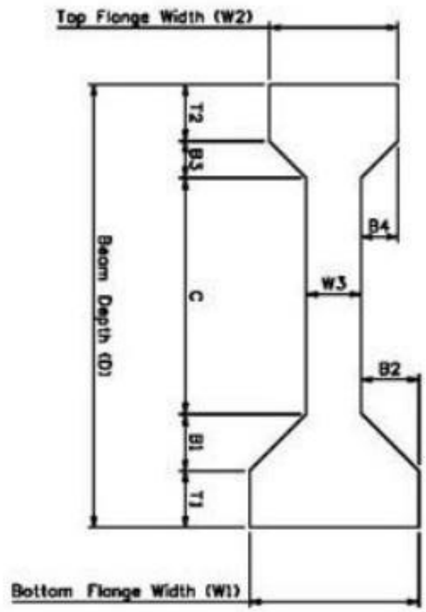
Factors involved in the choice of box beam configuration design should include but are not limited to economics, traffic type and volume, time constraints, and method of construction (whether by contract or state construction crews which generally have limited construction capabilities). The Bridge Designer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

1042.3-PRESTRESSED CONCRETE BEAMS

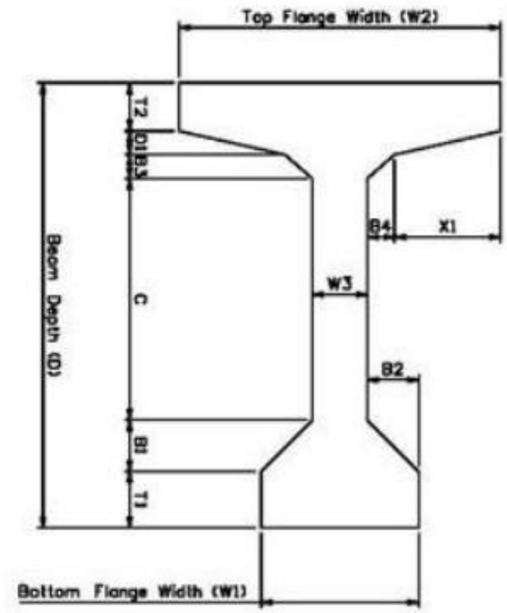
Prestressed concrete beams should be considered for bridges with spans from twenty-five (25) feet to ~~145~~ 160 feet. The maximum span length is based on the haul capacity and availability for a particular project site and shall be verified with a prestressed concrete beam supplier familiar with the project location. For continuous spans, the bridge system shall be designed simply supported for dead load and continuous for live load and superimposed dead load only. The same prestressed concrete beam type is desired for all spans. The Designer should minimize the number of beam lines. Prestressed concrete beam bridges should have a minimum of three stringer lines.

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

WVDOT-DOH STANDARD PRESTRESSED CONCRETE I-BEAM SECTIONS



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



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

Beam Designation	Top Flange Width (IN) W2	Bottom Flange Width (IN) W1	Depth (IN) D	Depth (IN)						Flange (IN)			Web Thickness (IN) W3	Basic Beam Properties		
				T2	D1	B3	C	B1	T1	X1	B4	B2		Area (IN^2)	Y_b (IN)	I_{xx} (IN^4)
II	12	18	36	6	-	3	15	6	6	-	3	6	6	369	15.8	50,980
III	16	22	45	7	-	4½	19	7½	7	-	4½	7½	7	560	20.3	125,390
IV	20	26	54	8	-	6	23	9	8	-	6	9	8	789	24.7	260,730
IV MOD	36	26	60	4	2	3	34	9	8	11	3	9	8	860	28.8	384,248
IV MOD	36	26	66	4	2	3	40	9	8	11	3	9	8	908	31.6	491,660
IV MOD	36	26	72	4	2	3	46	9	8	11	3	9	8	956	34.4	615,361
IV MOD	36	26	78	4	2	3	52	9	8	11	3	9	8	1,004	37.3	756,222
IV MOD	36	26	84	4	2	3	58	9	8	11	3	9	8	1,052	40.2	915,113
V	42	28	63	5	3	4	33	10	8	13	4	10	8	1,013	31.9	521,180
VI	42	28	72	5	3	4	42	10	8	13	4	10	8	1,085	36.4	733,320

Figure 1042.B

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

~~Approximate Maximum Span Lengths (Feet)~~

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

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

~~Table 1042.B~~

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

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

1042.5-SEGMENTAL CONCRETE BOXES

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

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

STRUCTURE DIRECTIVE 1043
ABUTMENT TYPES
January 17, 2023
Supersedes May 4, 2022
~~First Edition~~

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

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

1043.1-WALL ABUTMENT

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

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

STRUCTURE DIRECTIVE 1044
PIER TYPES
January 17, 2023
May 4, 2022
~~*First Edition*~~

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

1044.1-CAP-AND-COLUMN TYPE PIERS

Cap-and-column type piers have two or more circular or rectangular columns connected on top with a cap (a reinforced concrete beam that supports the superstructure). Generally, the pier cap ends will be cantilevered. For columns greater than 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.

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

STRUCTURE DIRECTIVE 1073
REHABILITATION TECHNIQUES
January 25, 2023
Supersedes May 4, 2022
~~First Edition~~

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

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

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

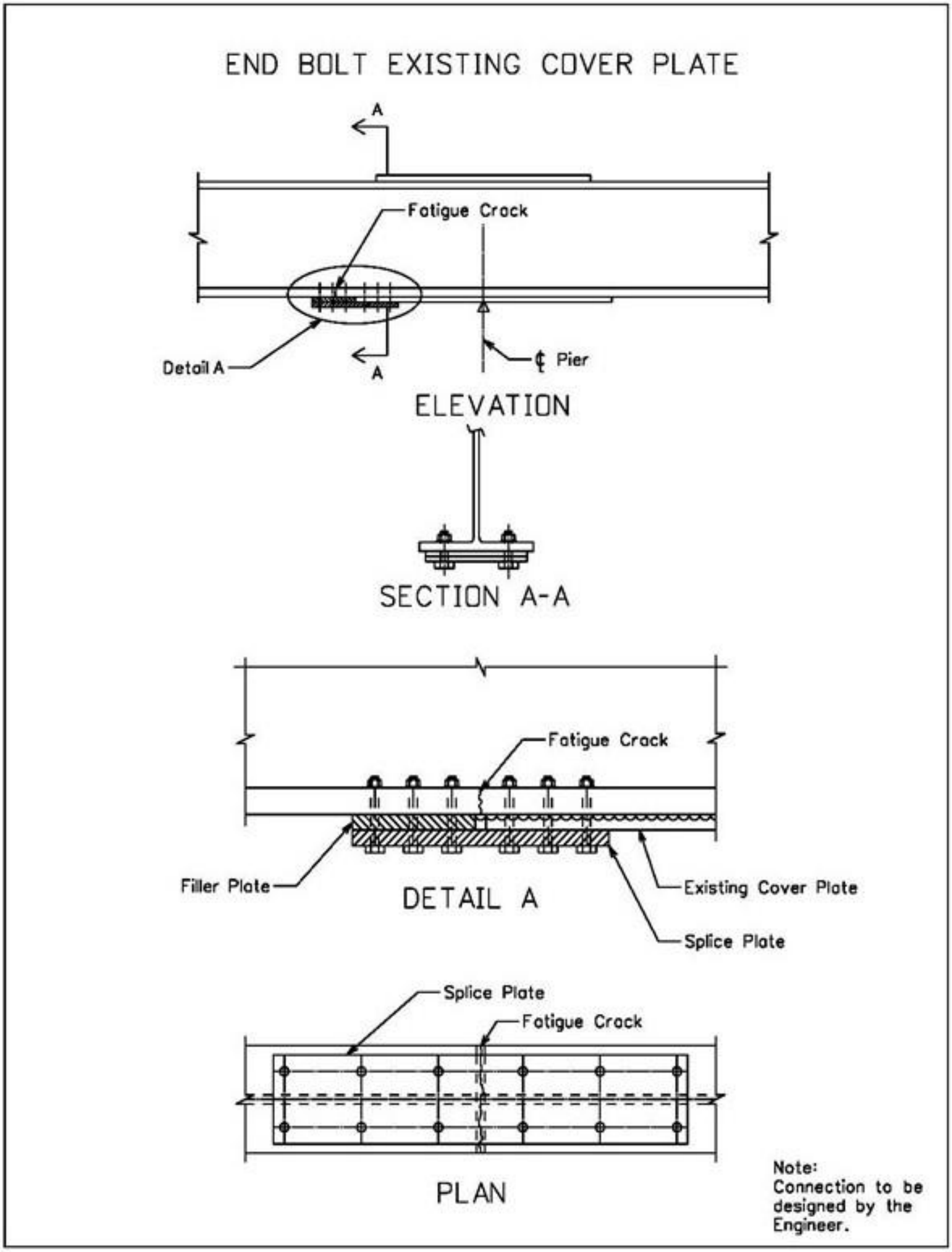


Figure 1073.A

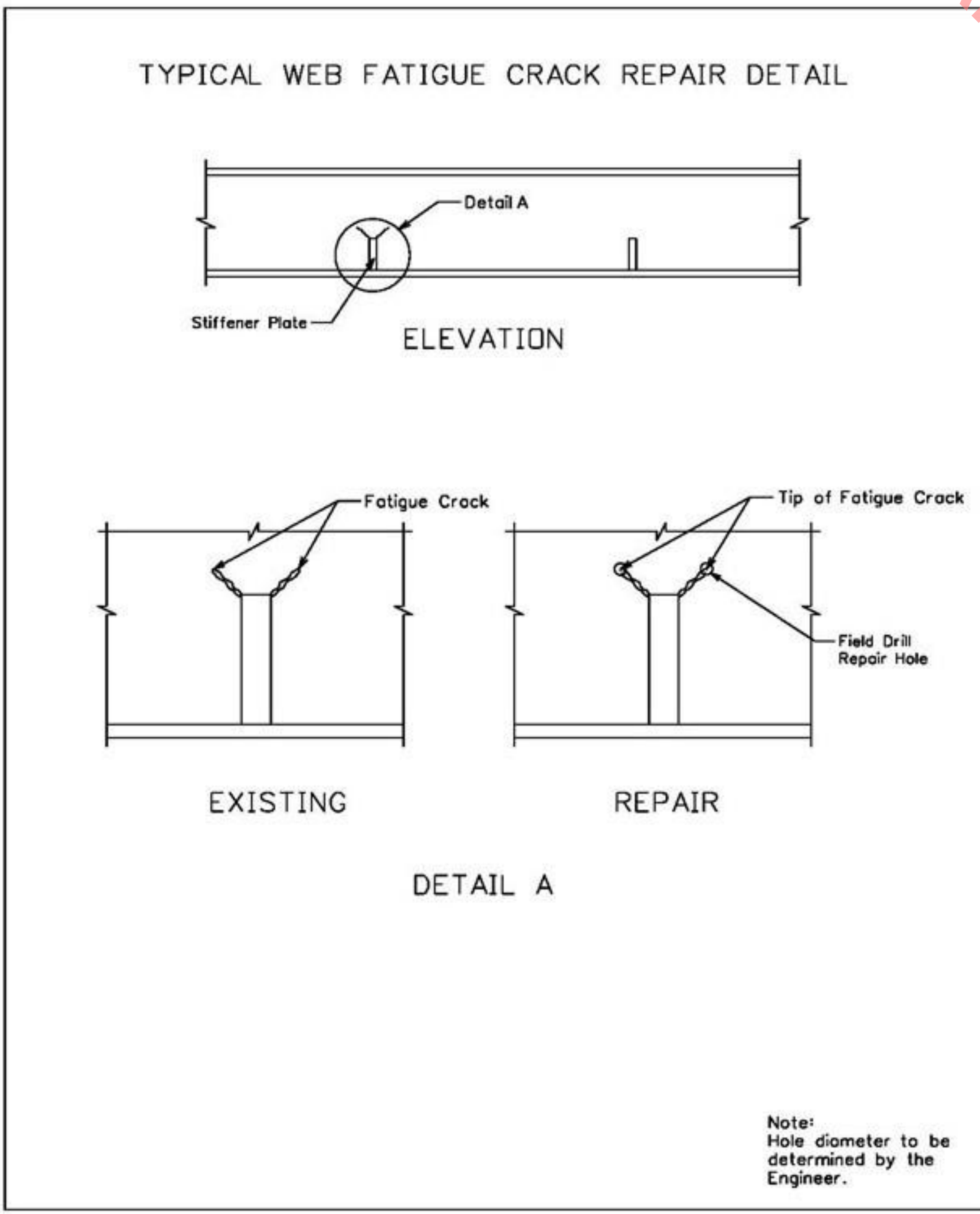


Figure 1073.B

1073.2-CONCRETE

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

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

Initially, all exterior concrete surfaces should be thoroughly examined by means of soundings with hammers to determine loose or defective areas that may exist beyond the visual assessment of deficiencies and deterioration. Defective areas should be removed to a depth necessary to eliminate any loose and disintegrated materials. All exposed reinforcement should be cleaned, care being taken to not damage the steel. Loose reinforcement should be tied back into place and, where necessary, concrete adjacent to 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.

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

STRUCTURE DIRECTIVE 2034
~~FATIGUE CRITICAL~~ NONREDUNDANT STEEL TENSION MEMBERS
January 17, 2023
Supersedes May 4, 2022
~~First Edition~~

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)~~.

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

STRUCTURE DIRECTIVE 2045
CONCRETE SUPERSTRUCTURES
January 17, 2023
Supersedes May 4, 2022
~~First Edition~~

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.

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

Table 2045.A

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

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

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

In a situation where two or more ~~girders~~ 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.