MICHIGAN DESIGN MANUAL
BRIDGE DESIGN - CHAPTER 7: LRFD

CHAPTER 7 - LRFD

DESIGN CRITERIA - NEW AND RECONSTRUCTION PROJECTS

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CHAPTER 7- LRFD

DESIGN CRITERIA - NEW AND RECONSTRUCTION PROJECTS

7.01

GENERAL

7.01.01

Design Specifications (8-20-2009)

In general, bridges in Michigan carrying vehicular traffic are designed according to the current edition of the LRFD Bridge Design Specification published by the American Association of State Highway and Transportation Officials (AASHTO). The exceptions to changes in AASHTO requirements are presented in this Design Manual.

The AASHTO specifications are also applied in the design of pedestrian bridges and major structures such as retaining walls and pumphouses. LRFD Guide Specifications for the Design of Pedestrian Bridges and Guide for the Development of Bicycle Facilities will aid in the design of other modes of transportation. (12-16-2019)

Bridges carrying railroads are designed according to the current specifications of the American Railway Engineering and Maintenance-of-Way Association Specifications (AREMA).

7.01.02

Design Method (8-20-2009)

The design of all structural elements shall satisfy Service Limit State and/or Strength Limit State requirements of the AASHTO LRFD Bridge Design Specifications.

7.01.03

Design Stresses (5-27-2025)

Concrete: Grade 3500, 3500HP* f'_c = 3000 psi
Concrete: Grade 4000, f'_c = 3500 psi
Concrete: Grade 4500, 4500HP* f'_c = 4000 psi
Steel Reinforcement f_y = 60,000 psi
Steel Reinforcement:
Stirrups for Prestressed Beams
(including stainless steel (SD) bars)
f_y = 60,000 psi
Stirrups for 17" & 21" Box Beams
(including stainless steel (SD) bars)
f_y = 40,000 psi
Structural Steel:
AASHTO M270
Grade 36 F_y = 36,000 psi
Structural Steel (including H-Piles, splices and pile points):
AASHTO M270
Grade 50 F_y = 50,000 psi
Grade 50W F_y = 50,000 psi
Structural Steel Pins:
ASTM A276
UNS Designation
S20161 or S21800 F_y = 50,000 psi
Temp Support Hanger Rods:
ASTM A193 Grade B7 (AISI 4140)
2 ½" and under F_u =125,000psi
F_y =105,000 psi
Over 2 ½" to 4" F_u =115,000psi
F_y = 95,000 psi
Over 4" to 7" F_u =100,000psi
F_y = 75,000 psi
Prestressed Concrete ** f'_c = 6000 - 8000 psi
Prestressed Concrete Compressive
Strength at Release f'_{ci} = 7000 psi (max)
Prestressing Strands f_{pu} = 270,000 psi
Foundation Piling (Steel Shells):
ASTM A252
Grade 3 Modified F_y = 50,000 psi
Foundation Piling (Timber) F_{CO} = 900 psi
High Strength Bolts:***
Organic zinc rich primer (Class B)
(Type 4 coating system) F_s = 32,000 psi

* Use Grade 3500HP and 4500HP on all MDOT projects. Grade 3500 and 4500 may be used on Local Agency projects if desired by the Owner. (4-22-2024)
** See Subsection 7.02.03.
*** Value of F_s is Design Slip Resistance for Slip-Critical Connections with faying surfaces coated.

7.01.04

Design Loading (6-27-2022)

The design loading is as specified in AASHTO LRFD 3.6.1.2 of with the exception that the design tandem as specified in 3.6.1.2.3 shall be replaced with a single 60 kip load.

A. Interstate and Trunklines (6-27-2022)

Vehicular live loading on the roadways of bridges designated HL-93 Mod, shall consist of 1.2 times the combination of the:

- Design truck or single 60 kip load
- Design lane load

Where 90% of two design trucks are combined with 90% of the effect of a lane load for both negative moment and pier reactions per 3.6.1.3 a 1.2 multiplier shall be applied to the resulting moment or load. Each design lane under consideration shall be occupied by either the design truck or single 60 kip load, coincident with the lane load, where applicable. The loads shall be assumed to occupy 10.0 ft. transversely within a design lane.

The design truck and design lane load are specified in AASHTO LRFD 3.6.1.2.2 and 3.6.1.2.4.

B. Local Roads and Streets (6-27-2022)

Structures carrying local roads or streets are to be designed according to county or city standards. The minimum design load acceptable for streets or primary county roads is HL-93 Mod loading as specified in this entire section. (8-6-92)

The load modifying factor, η (eta), for operational importance, may be considered for less important roads (AASHTO LRFD 1.3.2.1).

7.01.04 (continued)

C. Pedestrian and Bicycle (Nonmotorized) Bridges

Pedestrian and bicycle (nonmotorized) bridges shall be designed according to the current AASHTO LRFD Bridge Design Specifications 3.6.1.6. and current edition of the Guide Specifications for Design of Pedestrian Bridges. The assumed live load is 90 LBS/SFT. Consideration shall also be given to maintenance vehicles with regard to design loadings and horizontal clearances. For Clear Bridge Width, w, greater than 10'-0", use an H10 truck. For w between 7'-0" and 10'-0", use an H5 truck. Where vehicular access is prevented by permanent physical methods (bollards, gates, etc.) or for w less than 7'-0" the bridge does not need to be designed for a maintenance vehicle.]
(8-20-2009) (11-28-2011) (5-25-2015)

D. Railroad Bridges

Railroad bridges are designed according to the current AREMA Specifications, with the Cooper loading established by the railroad company.

E. Section combined with 7.01.04 C. (5-25-2015)

F. Deck Replacement, Bridge Widening or Lengthening

When an existing deck is to be replaced or the structure is to be widened or lengthened, the proposed reconstruction should be designed according to LRFD where practicable. In cases where LRFD cannot be used, the design method shall be approved by the MDOT Chief Structure Design Engineer.
(5-27-2020)

G. Ice Force on Piers

All piers that are subjected to the dynamic or static force of ice shall be designed according to the current AASHTO LRFD Bridge Design Specifications. (8-20-2009)

7.01.04 (continued)

Design Loading

H. Future Wearing Surface

All new bridges and bridge replacements shall be designed for a future wearing surface load of 25 LBS/SFT. (5-6-99)

I. Stay In Place Forms (6-27-2022)

Generally, corrugations for stay-in-place forms are filled with polystyrene foam. If corrugations are not required to be filled with foam, a design load of 15 LBS/SFT should be added.

J. Barrier Loads

For purposes of beam design, the barrier dead load can be distributed equally to all beams. (AASHTO Std. Specs 17th edition 3.23.2.3.1.1 & AASHTO LRFD 4.6.2.2) However, when calculating superstructure loads on the substructure, particularly for cantilevered pier caps, 75% of the barrier dead load should be applied with the fascia beam load. The remaining 25% of the barrier load should be applied with the first interior girder load. (8-20-2009)

7.01.04 (continued)

Design Loading

K. Vehicle Collision Force (7-24-2023)

Account for the AASHTO LRFD vehicle collision force in the design of all new bridges, bridge replacements, and pier replacements.

Locate the pier outside of the clear zone as defined in Section 7.01.11 of the MDOT Road Design Manual where possible. The clear zone used to determine the location of the pier must account for future roadway widening where applicable.

If a pier cannot be located outside of the clear zone design a multi-column pier with a base wall. Design the base wall with the minimum dimensions specified in MDOT Bridge Design Guide 5.22.01 and to meet the requirements outlined in Section 3.6.5.1 of the AASHTO LRFD Bridge Design Specifications.

Alternatively, a reinforced solid wall pier may be designed with the following minimum dimensions to meet the requirements outlined in Section 3.6.5.1 of the AASHTO LRFD Bridge Design Specifications:

1. The minimum width of the solid wall is 3'-0".

2. The minimum cross-sectional area of the wall is 30.0 square feet measured in the horizontal plan. Generally, a 10'-0" minimum length based on a width of 3'-0".

A pier base wall with the minimum dimensions specified in MDOT Bridge Design Guide 5.22.01 and a solid wall pier with the minimum dimensions specified above are components with adequate structural resistance and do not need to be designed to withstand the vehicle collision force required by the AASHTO LRFD Bridge Design Specifications. (11-27-2023)

For situations where the above criteria cannot be satisfied, design the pier to withstand the full vehicle collision force required by the AASHTO LRFD Bridge Design Specifications.

7.01.04 (continued)

Design the pier footing or the pile group or drilled shafts supporting the pier to withstand the vehicle collision force using the appropriate limit states and load combinations. (11-27-2023)

The vehicle collision force may be redirected or absorbed with Type C single face concrete barrier in accordance with MDOT Standard Plan R-54-Series if the pier cannot be located outside of the clear zone. Locate the Type C single face concrete barrier relative to the pier to meet the requirements outlined in Section 3.6.5.1 of the AASHTO LRFD Bridge Design Specifications. Provide appropriate barrier end treatments in accordance with the MDOT Road Design Manual. If the Type C single face concrete barrier encroaches on the required lane or shoulder widths for the roadway under the bridge shielding the pier shall not be considered. Design the pier with a base wall, as a reinforced solid wall pier, or to withstand the full vehicle collision force required by the AASHTO LRFD Bridge Design Specifications. If the pier is designed to withstand the full vehicle collision force design the pier footing or the pile group or drilled shafts supporting the pier to withstand the vehicle collision force using the appropriate limit states and load combinations.
(11-27-2023)

Site and project specific conditions must be considered by the Bridge Engineer when determining the option for accounting for the vehicle collision force. This may include, but is not limited to foundation limitations, the estimated cost of each option, and the construction schedule for the project.

New bridges, bridge replacements, and pier replacements shall not be exempted from the application of the AASHTO LRFD vehicle collision force.

7.01.04 (continued)

Design Loading

K. Vehicle Collision Force

Where existing piers are to be widened, design the widened portions of the pier to account for the vehicle collision force. See Section 12.08.08 of the MDOT Bridge Design Manual for guidance on accounting for the vehicle collision force at existing piers to remain in place. If the existing portion of the pier is being protected with single face concrete barrier (R-49-Series) extend the concrete barrier to protect the proposed portion of the pier as well.

A Local Agency has the discretion to define their policy for accounting for the AASHTO LRFD vehicle collision force in the design of bridges within their inventory in accordance with Section 3.6.5 of AASHTO LRFD. In the absence of published guidance from a Local Agency the applicability of the AASHTO LRFD vehicle collision force shall be determined using the same criteria that is used for classifying bridges under MDOT jurisdiction.

Bridges spanning over railroad right-of-way shall meet the requirements outlined in the AREMA Manual for Railway Engineering or local railroad company guidelines.

7.01.05

Fatigue Resistance

Determine nominal fatigue resistance using a structure design life of 75 years and the truck ADTT averaged over the design life. Add note 8.05 P. providing this information on the General Plan of Structure sheet. Design according to AASHTO LRFD Bridge Design Specifications 3.6.1.4 & 6.6.1.
(8-20-2009) (8-23-2021)

7.01.06

Deflection

A. Deflection Limits (6-27-2022)

Deflection limits shall be as specified in the current AASHTO LRFD Bridge Design Specifications 2.5.2.6.2.

The live load shall be taken from AASHTO LRFD 3.6.1.3.2.

B. Cantilever Deflection Computation

In computing the live load plus dynamic load allowance deflection of cantilevers of composite anchor span, the gross section of the anchor span is to be used. The length of the composite section for this analysis is to be assumed to extend from the bearing line to the point of dead load contraflexure.
(5-27-2020)

7.01.07

Temperature Range

A. The temperature range used to determine thermal forces and movements shall be in conformance with AASHTO "cold climate" temperature range per AASHTO LRFD 3.12.2.

B. The type of structure used in determining the temperature range, per AASHTO, shall be defined by the material of the main supporting members of the superstructure or substructure being considered.

7.01.08

Vertical Clearance

A. Requirements

The desired vertical bridge underclearances should be provided as indicated in the following table. If the desired underclearances cannot be provided, then the minimum underclearances shall be met. Where it is considered not feasible to meet these minimums, a design exception shall be requested from the Engineer of Road Design and subsequently to the FHWA (approvals designated in the Risk Based Project Involvement Stewardship and Oversight (RBPI S&O) plan) and from MDOT Bureau of Bridges and Structures, Chief Structure Design Engineer on "MDOT Oversight" projects (see Section 12.03 also). See the vertical clearance design exception matrix in Appendix 12.02.01. Requests to further reduce the underclearance of structures with existing vertical clearance less than indicated in the following table should be made only in exceptional cases.
(12-27-2021)

7.01.08 (continued)

Vertical Clearance

A. Requirements

VERTICAL CLEARANCE REQUIREMENT TABLE (8-20-2009) (6-22-2015)

Route Classification Under the Structure	All Construction (Desired)	New Construction (Min *)	Road 4R Construction (Min *)	Bridge 4R Construction (Min *)	3R Construction (Min *)
Freeways	16'-3"	16'-0"	16'-0"	16'-0"	16'-0" ***
NHS Arterials (Local & Trunkline)	16'-3"	16'-0"	Maintain Existing** and 14'-6" Min	16'-0"	Maintain Existing** and 14'-0" Min
Non NHS Arterials (Local & Trunkline)	16'-3"	14'-6"	Maintain Existing** and 14'-6" Min	Maintain Existing** and 14'-6" Min	Maintain Existing** and 14'-0" Min
Collectors, Local Roads & Special Routes(1)	14'-9"	14'-6"	Maintain Existing** and 14'-6" Min	Maintain Existing** and 14'-6" Min	Maintain Existing** and 14'-0" Min

3R = Rehabilitation, Restoration, Resurfacing

* Minimum Vertical Clearance must be maintained over complete usable shoulder width.

** Existing vertical clearances greater than or equal to the minimums shown may be retained without a design exception. Vertical clearance reductions that fall below the minimums for new construction require a design exception. (6-22-2015)

*** Existing vertical clearances may be retained (or increased) without a design exception unless a pattern of high load hits exist. Vertical clearance reductions below the standard (table value) require design exceptions. (5-27-2020)

(1) Special Routes are in Highly Urbanized Areas (where little if any undeveloped land exists adjacent to the roadway) where an alternate route of 16'-0" is available or has been designated. Bridges located over Special Routes in Highly Urbanized Areas can be found on the MDOT website. (5-23-2022)

Ramps and roadways connecting a Special Route and a 16’-0” route require a vertical clearance minimum of 14’-6” (14'-9" desired). Ramps and roadways connecting two 16’-0” routes require a vertical clearance minimum of 16’-0” (16'-3" desired). (8-20-2009)

4R = Reconstruction

Information on the NHS systems can be obtained by contacting the Statewide Planning Section, Bureau of Transportation Planning or found on the MDOT website at: http://www.michigan.gov/mdot/programs/highway-programs/nfc . (5-23-2022)

Pedestrian bridges are to provide 1'-0" more underclearance than that required for a vehicular bridge. For Freeways (Interstate and non-Interstate), including Special Route Freeways, the desired underclearance shall be 17’-3” (minimum 17'-0"). (8-20-2009)

A vertical underclearance of 23'-0" is required for highway grade separations over railroads when constructing a new bridge or removing the existing superstructure. For preventative maintenance, rehabilitation and deck replacement projects the existing railroad vertical underclearance does not need to be increased unless requested by the Railroad. (11-28-2011)

Clearance signs are to be present for structures with underclearance of 16’-0” or less (show dimensions 2” less than actual). See MDOT Traffic and Safety Sign Design, Placement, and Application Guidelines for additional information and guidelines. (5-23-2022)

7.01.08 (continued)

Vertical Clearance

A. Requirements

For shared use paths (pedestrian and bicycle), the vertical clearance to obstructions, including overhead fencing, shall be a minimum of 8'-6" (10'-0" desired). However, vertical clearance may need to be greater to permit passage of maintenance and emergency vehicles. In undercrossings and tunnels, 10'-0" is desirable for vertical clearance. See AASHTO's Guide for the Development of Bicycle Facilities. (9-2-2003)

B. Interstate Vertical Clearance Exception Coordination (5-23-2022)

In addition to normal processing of design exceptions, all proposed design exceptions pertaining to vertical clearance on Eisenhower Interstate System (Interstate) routes including shoulders, and all ramps and collector distributor roadways of Interstate to Interstate interchanges will be coordinated with the Surface Deployment and Distribution Command Transportation Engineering Agency (SDDCTEA). All exceptions to the 16'-0" vertical clearance standard on Eisenhower Interstate System are coordinated with SDDCTEA. The Eisenhower Interstate System designated routes can be found at the FHWA NHS Maps website (https://www.fhwa.dot.gov/planning/national_highway_system/nhs_maps/michigan/index.cfm). This requirement does not apply to Special Routes (1).

7.01.08 (continued)

MDOT (or its Consultant) is responsible for coordinating exceptions on all projects regardless of oversight responsibilities. MDOT will send a copy of all requests, and responses, to the FHWA. Michigan Interstate Vertical Clearance Exception Coordination, MDOT Form 0333, is available from MDOT web site.

Requests for coordination shall be emailed to:
usarmy.scott.sddc.mbx.tea-hnd@mail.mil

Contact with inquiries:
Douglas E. Briggs, P.E., 618-220-5229
douglas.e.briggs.civ@mail.mil
or
Jamie Todt, P.E., 618-220-5216
jamie.l.todt.civ@mail.mil

Physical mailings:
Highways for National Defense
ATTN: SDDCTEA
1 Soldier Way
Scott AFB, IL 62225

Fax: 618-220-5125

MDOT (or its Consultant) shall verify SDDCTEA receipt of the request. If no comments are received within ten working days, it may be assumed that the SDDCTEA does not have any concerns with the proposed design exception.

7.01.09

Longitudinal Deck Grades (9-23-2024)

Provide longitudinal grades to facilitate deck surface drainage. While it is desired to design bridges with steeper longitudinal grades the minimum grade (or minimum projected tangent grade for vertical curves) is 0.5%. Providing longitudinal grades at or near the minimum grade may necessitate the need for deck drainage with drains and downspouts. Generally, it will be necessary to perform bridge deck drainage design calculations in accordance with MDOT Drainage Manual. In addition, close attention to drainage is critical for sag and crest vertical curves when the K value (rate of grade change) is greater than 167 where,

K=L/A
L= Length of vertical curve, feet
A= Algebraic difference in grades, percent

Consider alignments that locate vertical curves outside the limits of the structure where the desirable minimum longitudinal grades can be achieved. When a vertical curve must be located on the bridge, avoid placing the high point (or low point) of the vertical curve on the bridge.

Structure on 1% or steeper grades should be fixed to the substructure at the lower end of structure where practicable. (9-2-2003)

7.01.10

Temporary Support Systems and Construction Methods

Where construction procedures will require a temporary support system, the plans shall note the loading that will be imposed on the system and the allowable stresses that can be assumed for the supporting soil. (8-6-92)

Where a construction sequence is critical, where there are restrictions on access for construction, or where the method of temporary support is not obvious, the plans shall provide an acceptable system that the contractor may employ. Alternatives may be proposed by the contractor, but these must be reviewed and approved by the Engineer if they are to be substituted.

This review is to insure that:

A. appropriate design specifications and permit limitations have been complied with, and

B. any temporary or permanent stresses imposed on the completed structure are within allowable limits.

C. possible vibration induced damage to existing structures and utilities is identified and mitigated. (11-28-2011)

7.01.11

Clear Zone Considerations

(8-6-92) If possible, substructure units should be located outside the clear zone, as defined by current AASHTO Roadside Design Guide. Where this is not feasible, the unit shall be shielded from impact by errant vehicles.

7.01.12

Sight Distance Considerations

When designers are developing shoulder widths on structures or pier offsets from pavement edges, sight distance should be considered. MDOT policy has set bridge (shoulder) widths 2' (offset) greater than AASHTO widths for safety considerations of the traveling public. Consult with Traffic & Safety Geometric Section for guidance and see Bridge Design Guides 6.05 Series & 6.06 Series. (5-6-99) (9-21-2015)

7.01.13

Concrete QA/QC

The provisions for Concrete QA/QC do not apply to bridge deck overlay mixtures or substructure patching. (12-5-2005)

7.01.14

Skew Policy (12-5-2005)

Skewed cross sections and stresses resulting from them must be considered when designing structures. Where possible, avoid excessive skews by moving abutments back and squaring them off (decreasing skew angle). Where the skew cannot be avoided, the engineer shall perform the necessary analyses to account for the skew.

θ = skew angle = angle measured from line perpendicular to bridge centerline to support reference line = 90° - angle of crossing.

Skew Angle	Design Requirements
$\theta \leq 30^\circ$	Standard design using approximate methods
$30^\circ < \theta \leq 45^\circ$	Special design using refined methods.*
$\theta > 45^\circ$	Use of angles greater than 45° must be approved through Bridge Design. Refined methods of design are also required.*

*Refined methods shall include using finite element methods of analysis to address girder roll, torsion, bearing restraints, bearing rotations, thermal movement direction and amount, cross frame loading, camber detailing and deck edge/end reinforcement.

7.01.15

Shoulder Widths for Work Zone Safety and Mobility (8-20-2009) (6-16-2014)

For 2 lane freeway and interstate new bridge construction and superstructure replacements the standard bridge shoulder widths shall be 14'-10". This will provide increased safety and mobility for future maintenance of traffic. The cross section will provide part width bridge construction with traffic being maintained on two 11 ft. lanes with 1 ft. shy distance on each side. For cross section see Bridge Design Guide 6.05.01A. A design exception will be required when the shoulder width is not met. The Region shall determine the required shoulder width at the scoping of the projects. (9-25-2023)

For shoulder widths for deck replacements, see section 7.02.31. A design exception will be required when the shoulder widths provided on deck replacement projects do not meet those required in section 7.02.31. Shoulders wider than those specified in section 7.02.31 may be required to accommodate corridor mobility needs and to accommodate the future maintenance of traffic needs for projects affecting infrastructure adjacent to the bridge. The Region shall determine the required maintenance of traffic needs for the corridor at the scoping of the projects and will document how the corridor mobility needs have been considered in the Scoping Report for the structure. The corridor mobility needs and the proposed clear roadway width for the deck replacement project will be confirmed during the Scope Verification Meeting for the project and the discussion must be documented in the minutes for the meeting. (9-25-2023)

Designers should layout beam spacing to accommodate future part width reconstruction. Beams may be located under the crown of the bridge deck provided there is no construction joint in the bridge deck at the crown.
(11-27-2023)

Bridge approach guardrail and bridge approach curb and gutter will be affected as a result of the widened shoulders and must be addressed in the design of the approaches. If the increased shoulder width is deemed necessary on reconstruction projects substructure widening may become necessary.

7.01.16

Redundancy
(8-20-2009) (9-17-2012) (3-23-2020)

Any proposed elements, or systems that do not meet AASHTO and FHWA redundancy requirements are prohibited. Bridge superstructures (beam/slab type) must have a minimum of 3 longitudinal beams or girders.

7.01.17

Part Width Construction (11-28-2011)

For existing bridges used to maintain traffic, the structural performance of the in-service portion of the structure shall be evaluated with respect to stage demolition and adjacent construction.

To the extent possible, plans shall show location of existing spread footings with respect to proposed construction.

Unbraced excavations for new substructures shall not extend below the bearing elevation of adjacent spread footing foundations.

Drilled excavations adjacent to in-service spread footing foundations shall be cased to prevent undermining.

For part-width construction of bridges, provide a minimum of 6' between the centerline of temporary sheeting (along the stage line) and the existing substructure sawcut line. This will allow for the width of sheeting and any required whalers and/or tiebacks. (2-26-2018)

7.01.18

Horizontally Curved Girder Bridges
(6-27-2022)

At a minimum, refined analysis shall address primary structural members, including the beams and cross frames of horizontally curved steel beams during all phases of the construction process. Special consideration shall be given to avoid part width construction of structures. At a minimum, refined methods shall address camber detailing, girder stress, cross frame loading, girder roll, and torsional load on the beams/girders.

Shoring (temporarily supporting) may be necessary to prevent deflections during part width construction and maintenance of traffic. Interior girders in the final structure will be exterior girders in a part width situation and shall be designed accordingly.

Use refined methods when the skew angle exceeds 30 degrees, the span length of any one span is greater than 150 feet or the radius of the beam/girder is less than 2000 feet (degree of curvature, “D”, is greater than three degrees (3°)).

Constructability Reviews shall be done on all projects especially those with part width construction and curved steel girders. See Chapter 2.

Refined methods include finite element, method, finite strip method, finite difference method, analytical solution to differential equations, and slope deflection method.

7.01.19

Accelerated Bridge Construction (ABC) (6-17-2013)

A. Background and Process

Accelerated Bridge Construction (ABC) techniques, including Prefabricated Bridge Elements & Systems (PBES) and Full Structural Placement Methods, are recognized by the Michigan Department of Transportation (MDOT) and the Federal Highway Administration (FHWA) as important and effective methods to construct or rehabilitate highway structures, while reducing the impact of bridge construction activities on mobility, the economy, and user delay.

ABC may include new technologies in the form of construction and erection techniques, innovative project management, high performance materials, and pre-fabricated structural elements to achieve the overall goals of shortening the duration of construction impacts to the public, encouraging innovation, ensuring quality construction, and expected serviceability of the completed structure.

All major rehabilitation or reconstruction bridge projects should be evaluated at the Scoping Process, see Chapter 6 of the Scoping Manual, to determine if ABC is suitable and provides a benefit; taking into consideration safety, construction cost, site conditions, life cycle cost of the structure, MDOT's mobility policy and user delays, and economic impact to the community during construction.

7.01.19 (continued)

All proposed ABC candidate projects are subject to Statewide Alignment Team Bridge (Bridge Committee) approval. Candidate projects during the scoping phases are to be presented at the monthly Bridge Committee meeting. The Bridge Committee will review candidate projects for further evaluation, and grant approval to pursue ABC techniques and determine availability of Bridge Emerging Technology funding. Once the Bridge Designer is assigned the project they shall determine if the ABC methodology is feasible from a design aspect. Issues shall be discussed with the Bridge Development Engineer, Bridge Field Services Engineer, and subsequently the Bridge Committee. A Scope Verification meeting may be necessary to resolve design and constructability issues (see Section 2.02.14 &.15 of Bridge Design Manual).

If the determination has been made that ABC will be implemented on a specific project, the next step is to choose the methods that are technically and economically feasible.

7.01.19 (continued)

Accelerated Bridge Construction (ABC)

B. Prefabricated Bridge Elements & Systems (PBES)

Prefabricated Bridge Elements & Systems (PBES) can be built on site away from traffic if site conditions warrant, or they can be fabricated off site and shipped to the site. Both methods offer advantages in quality control compared to cast in place construction where schedule or staging dictate the work progression. Non-prestressed reinforced concrete elements can be considered for on-site, or near site fabrication. Prestressed elements must be fabricated in a Prestressed Concrete Institute (PCI) certified plant.

1. Constructability

   Erection of prefabricated elements and the connection details will require special attention being paid to the following:

   a. Dimensional Tolerances:

      (1) Connections between elements must accommodate field erection. This may require staggering, or mechanically splicing connection or closure pour reinforcement or grouted splicers.

      (2) Elements fabricated off site should be test fit or otherwise confirmed to be of the correct dimensions prior to shipping.

      (3) Templates should be used to ensure correct fit-up between prefabricated elements or between a prefabricated element and a cast in place element.

      (4) Connection details should be standardized.

   b. The weight and size of precast elements:

      (1) Need to ensure elements can be erected with contractor's equipment. Typically, PBES element weights should be limited to 40 tons.

      (2) Need to ensure elements can be shipped to the site.

      (3) Need to ensure elements can be erected without long term lane closures.

7.01.19 (continued)

Accelerated Bridge Construction (ABC)

B. Prefabricated Bridge Elements & Systems (PBES)

2. Prefabricated Element Types

   The following prefabricated elements may be considered for use on MDOT bridge projects:

   a. Precast full depth deck panels.

      (1) Panels may be connected by reinforcement splice with closure pours using high strength concrete or ultra-high performance concrete or they may be transverse or longitudinally post tensioned.

      (2) Panels are sensitive to skew and beam camber and haunches.

      (3) Panels using post tensioning may have long term maintenance concerns.

      (4) Riding/wearing/sealing surface should be provided such as epoxy overlay or HMA overlay with waterproofing membrane.

      (5) Dimensional tolerances are very tight.

      (6) Additional geometry control will be required, and should be stated in the plans to be included in the Contractor Staking pay item.

      (7) Match casting may be used to assure proper fit-up when complex geometry is required.

   b. Decked Beam elements. (12-17-2018)

      (1) Two steel beams connected with deck (modular beams).

      (2) Decked Bulb Tee beams.

      (3) Decked prestressed spread box beams.

      (4) These systems rely on full shear and moment capacity joints and closure pours.

      Ultra High Performance Concrete may be used to reduce the lap length of the connection detail.

      (5) Camber control may require pre-loading of erected modular units, or partial post tensioning until all dead load deflections are applied.

      (6) Casting the roadway cross slope and/or vertical alignment curvature on modular units may be difficult, consider variable thickness overlays to develop required geometry.

7.01.19 (continued)

Accelerated Bridge Construction

B. Prefabricated Bridge Elements & Systems (PBES)

c. Pier Elements.

(1) Precast pier caps.

(2) Precast columns.

(3) Precast pile caps.

(4) These systems rely on grouted or mechanical reinforcement splices to develop reinforcement sufficiently to transfer reactions from one element to the next.

(5) Consider multiple smaller caps spanning two columns as opposed to one large cap.

(6) Pier columns that directly support beams without pier caps may be considered.

(7) Pier column voids can be considered to reduce weight. Weight of PBES elements should be limited to 40 tons where possible.

7.01.19 (continued)

d. Abutment and Wall Elements.

(1) Precast abutment panels.

(2) Precast footings.

(3) Precast backwalls and wingwalls.

(4) These systems rely on grouted or mechanical reinforcement splices to develop reinforcement sufficiently to transfer reactions from one element to the next

(5) Voids can be considered to reduce weight. Weight of PBES elements should be limited to 40 tons where possible.

e. Precast Approach Slabs

Dimensional tolerances are very tight for all Prefabricated Bridge Elements & Systems (PBES). The tolerance sensitivity required when erecting prefabricated elements may require dual or independent survey contracts to ensure proper fit up, camber, deflections and finished grades.

7.01.19 (continued)

Accelerated Bridge Construction

C. Full Structural Placement Methods

The following full structural placement methods may be considered for use on MDOT bridge projects:

1. Self-Propelled Modular Transport (SPMT):

    a. Computer controlled platform vehicle with movement precision to within a fraction of an inch.

    b. Capable of lifting 165 to 3,600 tons.

    c. Vertical lift range of 36 to 60 inches.

    d. Axle units can be rigidly coupled longitudinally and laterally.

    e. Move costs can be up to $500,000 (mobilization costs are significant, so SPMTs should be considered on corridors where multiple bridges may be moved).

    f. Limited to use on sites with minimal grade changes.

2. Lateral Bridge Slide:

    a. Bridge section is built on temporary supports adjacent to existing substructure.

    b. Bridge section bears on stainless steel, or other low friction surface such as Teflon.

    c. Existing substructure units can be reused, or new units constructed with minimal impact to traffic. Consider converting multiple span bridges into single spans so that proposed substructure units can be constructed in different locations from existing without impacting the operation of the existing structure.

    d. Bridge section is laterally jacked, or rolled into place.

    e. Required jacking forces must overcome static and kinetic friction.

    f. Consideration shall be given for the need to push and pull the bridge to meet movement tolerances. The hydraulic ram or cable with rollers shall be sized to accommodate both movements.

    g. Cost to slide a bridge is approximately $50,000 to $100,000 depending upon size of the bridge, and the number of spans.

    h. Additional stiffeners may be required on beams at point of jacking force application.

    i. Additional reinforcement in concrete elements may be required to control jacking stresses.

    j. Grade raises can be accommodated by casting backwalls and abutment portions on the proposed superstructure, and sliding over proposed sawcut elevations on existing abutments.

    k. Deflections of temporary substructure units must be considered, and the connection from the temporary substructure units to the permanent substructure units must be sufficiently rigid as to allow minimal deflections at the transition.

7.01.19 (continued)

Accelerated Bridge Construction

C. Full Structural Placement Methods

3. Incremental (Longitudinal) Launching:

    a. Bridge section is built near approaches, and then longitudinally launched into place.

    b. Prestressing may be required for concrete elements due to alternating bending moments generated during launch.

    c. Launching trusses, gantries, and hydraulic systems may be considered.

Allowing the contractor to select methods of placement may also lead to additional innovations and acceleration to the project schedule. Depending on the complexity of the overall project, innovative contracting methods may also be used in conjunction with ABC/PBES techniques. Innovative contracting methods are approved on a project by project basis by the MDOT Innovative Contracting Committee, and the MDOT Engineering Operations Committee. For more information see the Innovative Construction Contracting Manual.

The Federal Highway Administration provides additional information about ABC and PBES at the following website:
http://www.fhwa.dot.gov/bridge/abc/index.cfm.

7.01.20

Stream/River Crossing Low Chord Elevation for Navigation (11-25-2024)

Provide for navigation, where practicable (possible or capable of being done), a minimum clearance of 2 feet from the low chord to the design high water elevation. Clearance should conform to Federal requirements based on normally expected flows during the navigation season. Navigation includes using canoes, small boats and wading by fishermen.

7.01.21

Structural Cold Joints & Construction Joints (3-23-2020)

Cold joints and construction Joints in structural members resisting lateral loads must have a shear key, or other mechanical means of force transfer through the interface area, in addition to the fully developed steel reinforcement on both sides of the interface.

For additional information on substructure joint spacing see section 7.03.01 C., 703.02 C., 7.03.03 B.&C. and 7.04.01 B.&E.

7.02

SUPERSTRUCTURE

7.02.01

Structure Type (5-6-99)

Whenever possible, multispan steel structures shall be continuous to avoid having expansion joints over piers. Consideration shall also be given to integral or semi-integral structures. Suspended cantilever design shall be avoided. When simple spans of an existing bridge are being replaced, consideration should be given to replacement with continuous beams and continuous for live load superstructure.

Where supporting members are prestressed concrete beams, decks should be cast continuous over piers where possible. Consideration shall also be given to integral or semi-integral structures.

Beam designs with complex layout may require the contractor to provide provisions and design any falsework required to ensure proper erection of beams. (11-28-2011)

Include the special provision, Complex Steel Erection, Shoring and Falsework, when one of the following situations may occur during the erection of structural members:

A. Construction of continuous spans > 200’.

B. Girders with horizontal curvature.

C. Field assembled suspension, movable bridge, cable-stayed, truss, tied arch, or other non-typical spans.

(11-23-2015)

7.02.02

Beam Spacing (5-6-99) (11-28-2011)

Space all beams so that the center to center distance does not exceed 10'-0". If the spacing is exceeded the designer shall perform an analysis to ensure that the structure meets load rating criteria specified in MDOT Bridge Analysis Guide. Space spread box beams such that the center to center distance is not less than 6’-0”. (8-20-2009)

7.02.03

Beam Material Selection

The following is a guide for beam or girder material selection:

A. Prestressed Concrete (12-27-2021)

Use concrete with 28-day strengths of 6000-8000 psi concrete. For greater strengths, approval is required from MDOT Chief Structure Design Engineer. Strengths greater than 10,000 psi are not allowed.

1. Spread box beams, 36" wide, up to 42” deep
2. Spread box beams, 48" wide, up to 60” deep
3. I-beams (Types I thru IV, 28” to 54” deep)
4. I-beams (Wisconsin type, 70” deep)
5. I-beams (Michigan 1800 Girder, 70.9” deep)
6. Bulb Tee Beams (36” to 72” deep, 49” and 61” top flange)

B. Steel (4-17-2017)

1. Rolled Beams,
   AASHTO M270 Grade 36, 50 or 50W.
2. Welded plate girders
   AASHTO M270 Grade 36, 50 or 50W.

7.02.03 (continued)

Beam Material Selection

C. New Structures

The choice of girder material is generally to be governed by the economics of design and expected span lengths. For structures over depressed freeways or other areas where there is a high concentration of salt spray or atmospheric corrosion, concrete is preferred.

D. Reconstruction of Existing Structures

The new portions are to be similar in appearance to the existing structure. Current materials and construction procedures are to be used with considerations given to matching the beam deflections of the existing structure. (5-6-99)

E. False Decking

False decking shall be erected prior to deck removal or repair on reconstruction projects and after beam erection on new or reconstruction projects. (12-5-2005)

7.02.04

Structural Steel Grades- Available Thickness (5-6-99)

A. AASHTO M270 Grade 36 up to 8" in thickness.

B. AASHTO M270 Grade 50 up to 4" in thickness.

C. AASHTO M270 Grade 50W (painted) steel up to 4" in thickness may be substituted for Grade 50 steel.

7.02.05

Bearings

A. Sole Plates

Plate thicknesses are to be specified in ¼" increments. For beveled sole plates, this ¼" increment is based on the maximum thickness.

For steel beams, sole plates are to be beveled when the calculated bevel is greater than 1% for curved steel bearings and greater than 0.5% for elastomeric bearings. For requirements for prestressed concrete beams, see Subsection 7.02.18. (8-6-92)

B. Elastomeric Pads

Elastomeric pads (⅛") are required under all steel masonry plates and are to be 1½" longer and wider than the masonry plates.
(10-24-2001)

7.02.05

Bearings (continued)

C. Elastomeric Bearings

Plain bearings shall have a shear modulus, G, of 200 (±30psi), laminated bearings shall have a shear modulus of 100 psi (±15psi). Pads shall be 4" minimum (generally 6") by beam width - 3" with ¾" minimum thickness (increase in ¼" increments). (6-27-2022)

Design steel-reinforced elastomeric bearings with AASHTO LRFD Method A. Method B shall not be used unless approved by MDOT Structural Fabrication Engineer. (11-28-2011) (3-26-2018)

Fabric laminated (cotton-duck or other fiber reinforcement) bearings shall not be used unless approved by MDOT Structural Fabrication Engineer or MDOT Chief Structure Design Engineer.
(3-20-2017) (3-26-2018) (5-27-2020)

All fabrication and material property tolerances must be accounted for in the design of elastomeric bearings. (9-22-2025)

Additional information (polymer type, minimum low-temperature grade, etc.) can be found at MDOT's Elastomeric Bearing Guidance Document. (3-20-2017)

D. Anchor Bolts

Calculated lengths of bridge anchor bolts should be based on a bolt projection of 1" beyond the nut. (5-6-99)

7.02.06

Precamber - Steel Beams

Where dead load deflection, vertical curve offset, and deflection due to field welding (rare occurrence) is greater than ¼", the beams shall have a compensating camber. Camber is to be figured to the nearest ¼" and shall be parabolic.

In certain instances, such as for continuous spans or long cantilevers, reverse camber should be called for in order to obtain uniform haunch depths.

When several beams in a bridge have corresponding camber ordinates which differ only slightly from each other, the Engineer should attempt to average these into one set for all beams.

7.02.07

Moment of Inertia - Composite Beam

The composite moment of inertia shall be used throughout positive moment regions. This moment of inertia is to be used in negative moment regions to compute beam stiffness only.

7.02.08

Multiple Span Design

A. Beam Depth (6-27-2022)

Generally, use the same depth beams for all spans with the longest span controlling the beam depth. Site constraints and conditions may lead to differing beam depths.

B. Composite Design

Composite design shall be used on all spans. Composite design uses the entire deck/slab thickness versus deck/slab stand-alone design which eliminates the top 1 ½" wearing surface. (5-6-99) (8-17-2015)

C. Suspended Spans
The suspended span should be poured first (see Section 7.02.01).

7.02.09

Rolled Beam Design (6-27-2022)

Cover plates for rolled beams are to be designed according to the following information and current AASHTO LRFD Bridge Design Specification 6.10.12.

A. The cover plate width for a new beam should be equal to the beam flange width minus 1½" and for an existing beam should be equal to the beam flange width plus 1½".

B. The minimum cover plate thickness shall be the greater of ⅜" or 1/24 of the plate width.

C. Cover plate steel should be the same as the beam steel or matched as closely as possible.

7.02.10

Plate Girder Design (Welded)

A. Web Plates (6-27-2022)

When necessary, web plate depths are varied in 2" increments and the thickness shall be a minimum of 7/16".

B. Flange Plates

Flange plate widths may be varied to achieve a more economical design when required. The minimum width shall be 12". The minimum thickness shall be ½" when shear connectors are not used and ¾" when shear connectors are welded to the flange in the field.

C. Hybrid Designs

Hybrid designs using a combination of quenched and tempered steel according to ASTM A514 (AASHTO M244) & A852 (AASHTO M313) shall not be used. (5-6-99)

7.02.11

Stiffeners

A. Orientation and Size

Stiffeners are to be set normal to the web; however, when the angle of crossing is between 70º and 90º, the stiffeners may be skewed so that the diaphragms or crossframes may be connected directly to the stiffeners. Minimum thickness shall be 7/16". (9-2-2003)

B. Bearing Stiffeners

In general, bearing stiffeners should be eliminated at abutments with a dependent backwall, and the lower portion of the backwall should be poured and allowed to set before the deck is cast.

C. Bearing Stiffeners at Temporary Supports

To prevent the possibility of web buckling, bearing stiffeners should be provided at temporary supports for all plate girders. They need be placed on one side only; and on the fascia girders, they are to be placed on the inside.

D. Bearing Stiffeners for Rolled Beams

Even though bearing stiffeners are not required by design, if a beam end is under a superstructure transverse joint, two ½" x 4" bearing stiffeners should be provided as a safety factor in the event of corrosion and section loss of the web. (8-6-92)

7.02.12

Welding

All welding details are to be according to AWS specifications, except for minimum fillet weld sizes, which should be as shown in the Standard Specifications. Any intended deviations are to be called to the attention of the MDOT Chief Structure Design Engineer.
(5-6-1999) (5-27-2020)

Plans should show welding details but should not show the size unless a deviation from AWS specifications is intended.

Plans should also show beam or girder top flange tension zones. Tension zones should be labeled as the longest dimension for all load cases. Generally, welding of lifting lugs, contactor attachments, etc. will not be permitted in tension zones. (6-27-2022)

7.02.13

Field Splices in Plate Girders

A. General

Girder Length	Field Splice
0’ to 125'	None provided.
Over 125' to 160'	Field splice is shown on plans as optional; it is designed and detailed, but not paid for.
Over 160'	Field splice is designed, detailed, and paid for. *

Fabricators that wish to field splice other than as called for on the plans will need prior Bridge Design Project Manager approval.
(5-27-2020)

* Additional steel weight from splices will be added to quantity for “Structural Steel, Plate, Furn and Fab”. (12-22-2011)

B. Location

Field splices are to be located at low-stress areas at or near the point of contraflexure for continuous spans.

7.02.13 (continued)

Where practical avoid locating field splices at locations where inspections and future maintenance work may be difficult including, but not limited to, within the horizontal clearance limits of railroads, directly over the navigable channel of a waterway, or over environmentally sensitive areas. (2-24-2025)

C. Bolts

All high strength bolts are to be hot-dip galvanized. (3-18-2013)

D. Bridge Deck Haunches

Minimum haunches must be maintained along the entire length of the beams. See Section 7.02.19.C for additional information. (9-22-2025)

7.02.14

Diaphragms and Cross-frames

A. Orientation (8-20-2009) (11-25-2024)

Provide diaphragms or cross-frames at abutments, piers and hinge joints. Intermediate diaphragms may be used between beams in curved systems or where necessary to provide torsional resistance and support the deck at points of discontinuity or at angle in girders.

Investigate the need for diaphragms or cross-frames for all stages of assumed construction procedures and the final condition. Diaphragms or cross-frames required for conditions other than the final condition may be specified to be temporary bracing.

7.02.14 (continued)

B. End Diaphragms(11-25-2024)

End diaphragms or cross-frames are required at ends of beams to support the end of slab unless it is supported by other means. Provide diaphragms or cross-frames at the centerline of support for curved girders.

Use steel end diaphragms or cross-frames at independent backwalls. End diaphragms are to be no closer than 2'-0" from the center line of bearing to provide access for painting and inspection. (3-24-2025)

Check edge of slab capacity. If deemed necessary, strengthen with edge beam - a strengthened strip of slab made to act as a vertical frame with the beam ends. Design edge beam in accordance with AASHTO LRFD. (3-24-2025)

7.02.15

Shear Developers

Use shear developers in all steel beam spans. See Section 7.02.31 D. for the treatment of existing shear developers when replacing a deck. (12-27-2022)

A. Type Used (6-27-2022)

Shear developers are to be the stud type shown in Bridge Design Guide 8.07.01. Show details and spacing for ¾" studs on the plans. Generally, shear developers are 12" or less in length. Provide additional longitudinal reinforcement when haunch becomes greater than 6" and longer than 12" shear developers are required.

B. Spacing

1. Standard Bridge Slabs

The spacing is to be constant along the beam as required by the design. Shear developers are not to be used in areas of negative moment. They should extend through the positive moment area and to, or slightly beyond, the point of contraflexure. This point should be determined for the loading condition that will place it closest to the support over which negative moment will occur. In the event of a special case in which shear developers are used in negative moment areas, maximum tensile stress at the point of attachment is not to exceed that which is allowed by the current AWS specifications.

Shear developers (acting as slab ties) shall be placed in at least one half of all spans regardless of contraflexure points and moment orientations. In end spans with all negative moments place shear developers from abutment towards pier at 24" spacing. In interior spans with all negative moments place shear developers in middle half of span at 24" spacing.
(12-5-2005)

7.02.15 (continued)

2. Empirical Bridge Slabs (6-27-2022)

For empirical bridge slabs, the studs shall be placed on the entire length of beams. This includes the negative moment regions. The design of the studs shall be based on the positive moment area as critical. (5-6-99)

A minimum of two shear connectors at 24" shall be provided in the negative moment regions of continuous steel superstructure (AASHTO LRFD 9.7.2.4). Where composite girders are noncomposite for negative flexure, additional shear connectors shall be provided in the region of points of permanent load contraflexure. The additional shear connectors shall be placed within a distance equal to one-third of the effective slab width on each side of the point of permanent load contraflexure.

Field splices should be placed so that they do not interfere with the shear connectors.

When detailing empirical slabs on plans designate them as an "Empirical Slab".
(9-27-2021)

C. Fatigue Life (5-22-2023)

Design shear developers for an infinite fatigue life on all structures regardless of the projected Average Daily Truck Traffic (ADTT). When determining if additional shear developers are needed to meet the interface shear requirements for partial or complete deck replacement projects assume that the existing stud type shear developers to remain in place were initially designed for an infinite fatigue life (over 2,000,000 cycles). A reduction in the fatigue capacity of the existing stud type shear developers to remain in place is not necessary to account for the fatigue cycles the existing shear developers have already experienced.

7.02.16

Lifting Lugs

The contractor will be permitted to use lifting lugs to transport and erect beams, subject to the requirements of the Standard Specifications. Our plans should indicate the tension zones where lifting lugs will not be permitted.

7.02.17

Painting and Galvanizing (6-27-2022)

Structural steel will be painted light gray. AMS-STD-595 color #16440.

The Roadside Development Unit may request color # 15488 or another variety from AMS-STD-595 and obtain Region/TSC concurrence. Indicate the color number on the plan notes. See Section 12.07.06 for information regarding performance warranties.

Galvanizing (without top coat) of steel beams may be practicable if site conditions warrant it.

7.02.18

Prestressed Concrete Design

A. General

1. Strand Selection (7-28-2025)

The design and detail sheets shall specify only ASTM A416 (AASHTO M203) Grade 270 low relaxation strands. Strands shall be 0.6 inches in diameter with a release force of 44,000 pounds.

MDOT has begun using CFRP strands in some locations. If CFRP strands are desired, the provisions of MDOT guidance for Concrete Structures with CFRP Reinforcement shall be followed. CFRP strand use must be approved by the Chief Structure Design Engineer.

2. Draping Strands (7-28-2025)

Draped Strands are pretensioned strands that control stresses at the end of beams by varying the height of the prestressing strand along the length of the beam to more closely follow the moment envelope from the applied loads.

Location of draped strands at beam ends shall start 3" down from the top for I-Beams and Michigan 1800 girders and 5" down from the top for Bulb-T beams downward. Draped strands at beam end shall correspond to the highest available strands at beam center.

If using draped strands limit the vertical force at the strand hold down point to 4,000 pounds per strand and the strand angle of inclination to less than 6 degrees. The calculation of the vertical force at the strand hold down point and the angle of inclination of the draped strands must take into account all applicable fabrication tolerances.

7.02.18 (continued)

Prestressed Concrete Design

A. General

3. Bond Breakers/Debonding (7-28-2025)

As defined in Section 5 of AASHTO LRFD, a debonded strand is a pretensioned strand that is bonded for a portion of its length and intentionally debonded elsewhere through the use of mechanical or chemical means.

Where debonding is used to control stresses at the end of prestressed concrete Bulb Tee and I (PCI) beams, strands should be debonded in pairs. A maximum of 44% (52% for continuous for live load structures) of the strands may be debonded. Limits and restrictions on the debonding of stands are outlined in AASHTO LRFD 5.9.4.3.3. Additional MDOT specific requirements are included in the paragraphs below.

The debonding should be staggered by placing the debonded strands into groups similar to the table below.

Number Debonded	Shortest	2nd	3rd	Longest
4	2			2
6	4			2
8	4	2		2
10	6	2		2
12	6	2	2	2
14	6	4	2	2

The above table has been developed to meet the requirements outlined in AASHTO LRFD 5.9.4.3.3. The shortest point refers to the closest point to the beam end that any debonding can be terminated without overstressing the beam. The longest point refers to the point that all debonding can be terminated. Consultant debonding schemes shall follow a similar rational method.

From the end of the debonding to the point where the strands are no longer required to control stresses or provide ultimate capacity, a double development length (minimum) of bonding shall be provided.

Spans less than 30'-0" need not be debonded. It is realized that the continuity moments of continuous for live load structures may reduce the effectiveness of debonding & increase the number of draped strands.

If placing strands in the bottom row, they should be placed on every third strand with the corner strands being bonded.

7.02.18 (continued)

Prestressed Concrete Design

A. General

4. To aid in stabilizing transverse reinforcement in the beam, a bar or strand shall be located in the bottom corners of the beam. Second row up for box beams and certain PCI beams. (8-20-2009)

5. PCI beams under open joints are susceptible to corrosion from brine intrusion into the strands and mild reinforcement. This is the most prevalent distress to PCI beams. This can be mitigated by sealing the beam ends with an elastomeric sealer as described in Section 7.03.11A.

PCI beams and spread box beams under expansion joints should be coated per the special provision for Warranty on Concrete Surface Coating. Apply the coating from the beam end a length the greater of twice the beam depth, or five feet. In addition, where the coating operation will have a minimal effect on the maintaining traffic schedule, and the cost of the project, the entire outside face of the fascia beam and its bottom flange, should be coated. On new construction or superstructure replacement the fascia beam can be coated prior to erection. (6-27-2022)

6. Continuous for live load prestressed concrete beams shall be designed as simple span beams for all positive dead load and live load moments. (9-2-2003)

7.02.18 (continued)

7. Slab Ties (6-27-2022)

Ensure slab ties sufficiently penetrate haunches and slab to facilitate composite action of beams and slab. See Bridge Design Guide 6.42.03A for details and section 7.02.20 G. In some instances, the number of slab ties can be minimized due to the shear resistance resulting from the contact area of the top flange of some beams. See AASHTO LRFD 5.7.4.2. Avoid use of EK04 slab ties in Bulb Tee beams unless this provision cannot be met.

8. Confinement Reinforcement (7-28-2025)

a. Primary Confinement Reinforcement. Provide reinforcement at the ends of I-Beams and Bulb Tee beams placed to confine the prestressing steel in the bottom flange for a distance equal to 1.0 times the beam depth. Provide #3 epoxy coated deformed bars with a spacing of 3" and shaped to enclose the strands.

b. Secondary Confinement Reinforcement. Beyond the primary confinement reinforcement, provide additional reinforcement placed to confine the prestressing steel in the bottom flange of I-Beams and Bulb Tee beams. Extend the secondary confinement reinforcement from the end of the primary confinement reinforcement to a minimum distance of

    1) 1.5 times the beam depth beyond the end of the longest debonded strand where debonded strands are required.

    2) 1.0 times the beam depth where debonded strands are not required.

    Provide #3 epoxy coated deformed bars with a spacing of 6" and shaped to enclose the strands.

7.02.18 (continued)

Prestressed Concrete Design

9. Stability During Lifting and Transportation (7-28-2025)

Investigate the buckling and stability of prestressed concrete beams during handling, transportation and erection on all projects. When deemed necessary by calculations, include any temporary measures needed to ensure the stability of the proposed beams in the plans for the project. Base the analysis on the stress limits specified in AASHTO LRFD.

Alternate means for ensuring the stability of prestressed concrete beams during handling, transportation and erection may be proposed by the Contractor or Fabricator. Any deviations from the details included in the plans must be submitted to the Engineer for review and approval as specified in Section 104 of the MDOT Standard Specifications for Construction.

7.02.18 (continued)

B. Prestressed Concrete Box Beam Design

1. Skew Bridges (6-27-2022)

The ends of the box beams shall be skewed to be parallel to the reference line. In extreme cases the ends of box beams can be set at a lesser skew, dependent upon substructure unit width.

2. Spacing (6-27-2022)

Spread box beams may be used and shall be treated similar to prestressed concrete I-beams (PCI). Space spread box beams such that the center to center distance is not less than 6’-0”.

3. Bridge Seats (12-17-2018)

For spread box beams the bridge seat shall be bolstered and level.

4. Bearings (6-27-2022)

Where the pressure is less than 100 psi, ½” joint filler may be used for a bearing pad. Where bearing pressures are greater than 100 psi, 4” minimum (generally 6”) by (beam width - 3”) elastomeric pads shall be used (¾” minimum thickness, increase in ¼” increments). Cast steel sole plate (¾” generally) in all beams. When the calculated bevel exceeds 1%, tilt sole plate as required. All position dowels for doweling beam to the substructure will be placed by drilling as described in the Standard Specifications.

5. Beam steel reinforcement, including stirrups, is Grade 60 (ksi) for all box beams except 17” & 21" box beams. For 17” & 21” box beams the design of transverse beam steel reinforcement, stirrups and slab ties (epoxy coated ED bars & stainless steel SD bars) is based on Grade 40 (ksi); the use of either Grade 40 or Grade 60 is allowed in fabrication of the beam. Longitudinal beam steel reinforcement (EA bars) is Grade 60 for 17” & 21” box beams. See note 8.07.03 P. For 21” and 27” beams shear requirements/values may dictate the use of a greater than 7600 psi concrete compressive strength (21”) or the use of stainless steel SD bars (27”). For additional information see Bridge Design Guides 6.65.10-12A. (12-27-2021)

7.02.18 (continued)

Prestressed Concrete Design

C. Prestressed Concrete Bulb Tee and I-Beam Design (6-27-2022)

1. Bearing Pads

For single-span structures 40'-0" or less in length, use dependent backwalls with 1" elastomeric pads under the beams and joint filler under the backwall.

For single- and multiple-span structures with spans over 40'-0", allowance for expansion is required in designing the bearing pads.

2. Sole Plates

Sole plates (3/4” generally) are to be cast in all beams and shall be tilted as required when the calculated bevel exceeds 1%. (11-24-2014)

3. Skew Bridges

On skewed structures, the ends of the beams shall be made square regardless of the angle of skew. The top corners may be clipped in order to accommodate a straight expansion joint across the structure.

One flange at each end of a Bulb-Tee beam can be shortened and/or clipped to accommodate a large skew and minimum clearances to a pavement seat. See Bridge Design Guide 6.60.03 Series and 6.60.13. (1-23-2023)

7.02.18 (continued)

4. Concrete Diaphragms(11-25-2024)

Only consider concrete diaphragms when replacing existing concrete diaphragms. For new construction , concrete diaphragms are not permitted.

Set back concrete end diaphragms 10" to 12" from the end of beam in order to permit the removal of the forms after the diaphragms are poured.

The bottoms of all diaphragms are to bear on the bottom of the lower beam fillet.

All diaphragms are to be cast separately from slab except with continuous for live load structures (optional construction joint). (5-6-1999)

5. Steel Diaphragms

Steel intermediate diaphragms and steel end diaphragms are required at independent backwalls for shorter construction duration. Locate end diaphragms no closer than 2'-0" from the centerline of bearing to provide access for painting and inspection.

Check edge of slab capacity. If deemed necessary, strengthen with edge beam - a strengthened strip of slab made to act as a vertical frame with the beam ends. Design edge beam in accordance with AASHTO LRFD. (3-24-2025)

Use details from Bridge Design Guide 6.60.12 A. - H. and include Special Provision in proposal. (11-26-2012)

6. Battered Beam Ends

Detail the ends of beams to be vertical when sitting on the substructure. When beam ends are battered to meet this requirement and the batter exceeds 3" (the "L" ("L’ ") dimension on the MDOT concrete beam typical detail sheet exceeds 3") additional reinforcement may be required to meet AASHTO requirements and to ensure the durability of the beam ends. Based on coordination with Michigan prestressed concrete beam Fabricators, a feasible solution has been included on the MDOT concrete beam typical detail sheets. (3-24-2025)

7.02.19

Slabs

For information on Ride Quality on new slabs see section 7.02.32.

A. Design (8-20-2009)

MDOT standard LRFD slab is designed using the following criteria:

1. The design loads for decks and deck systems should be specified depending on the method of analysis. When the approximate strip method is used, force effects should be determined on the following basis:

    a. Where primary strips are transverse and their span does not exceed 15.0 ft., the transverse strip shall be designed for the wheels of the 32.0-kip axle.

    b. Where primary strips are transverse and their span exceeds 15.0 ft., the transverse strip shall be designed for the wheels of the 32.0-kip axle and the lane load together.

    c. Where primary strips are longitudinal, the transverse strips shall be designed for all loads specified above, including the lane load.

2. The design truck shall be positioned transversally such that the center of any wheel load is not closer than:

    a. One foot (1.0 ft.) from the face of the curb or railing for the design of the deck overhang.

    b. Two Feet (2.0 ft.) from the edge of the design lane for the design of all other components.

3. Where the strip method is used, the extreme positive moment in any deck panel between girders shall be taken to apply to all positive moment regions. The extreme negative moment over any girder shall be taken to apply to all negative moment regions.

7.02.19 (continued)

4. For deck/slab design only, the top 1½” of slab is considered a wearing surface and is not included in the design depth, but is included in the dead load. See section 7.02.08 B. for composite action of deck slabs. (8-17-2015)

Design of deck slabs using the Empirical Design Method according to AASHTO LRFD 9.7.2 is an approved or allowed alternative. (6-27-2022)

When detailing empirical slabs on plans designate them as an “Empirical Slab”. (9-27-2021)

B. Overhang

Design slab overhangs for all applied loads and all applicable limit states on every project regardless of the width of the overhang. Horizontal loads on the slab overhang resulting from vehicle collision with the barrier shall be based on a TL-4 railing test level as specified in AASHTO LRFD Chapter 13. (3-27-2023)

Design overhang according to AASHTO LRFD 9.7.1.5. If the deck overhang with cantilever does not exceed 6.0 ft. from the centerline of the exterior girder to the face of a structurally continuous concrete railing, the outside row of wheel loads may be replaced with a uniformly distributed line load of 1.0 klf intensity, located 1.0 ft. from the face of the railing. (6-27-2022)

Limit overhang widths to 2'-6" if possible. Avoid overhangs greater than 6 feet. (3-27-2023)

7.02.19 (continued)

Slabs

C. Slab Haunches (9-22-2025)

Plans are to provide for the deck slab to be haunched at each beam to provide for variance in actual top of beams. The design should normally make allowance for a 1" minimum uniform haunch for steel beam bridges and a 2" minimum haunch for prestressed concrete beam bridges; however, the details should show the haunch as variable. A nominal 2" haunch should be used on structures with span lengths exceeding 100'-0". Minimum haunches must be maintained along the entire length of the beams. For steel beam bridges this should account for varying top flange plate thicknesses as well as any splice plates and filler plates at field splice locations. Accounting for these components of the superstructure may require deeper haunches to ensure that no part of the beam encroaches into the bottom of the bridge deck.

To aid in the construction of the haunched slab, the plans should include bottom of slab elevations over each beam and at equal intervals across the spans. These elevations should apply at the time that all structural steel has been erected, but no other loads applied; however, they should include allowance for additional deflection due to forms, steel reinforcement, deck concrete, and railing. For additional criteria when the haunch exceeds 6" see section 7.02.20 G. and Bridge Design Guide 6.42.03A.

D. Slab Thicknesses (6-27-2022)

Slab thicknesses are typically 9" and are to be uniform thickness with beams stepped to follow the crown of the roadway.

E. Slab Under Sidewalk

If the roadway slab extends underneath the sidewalk, it should be designed for full highway loading.

F. Nighttime Casting of Superstructure Concrete

All bridge deck pours are to be designated nighttime casting of superstructure concrete on all bridge decks. (5-6-99)

7.02.19 (continued)

G. Bridge Crown/Slope

Use 2% cross-slope on all projects with a deck replacement or greater scope except those that have compelling reasons to meet the existing cross-slope. Maintain constant slope across lanes of travel and shoulders, including bridges with full superelevation and ramp bridges. This will allow for ease of construction and deck screeding.

Bridge overlays and railroad and bridge approach projects may use 1.5 %. Local roads over may also use 1.5% unless the road approaches are or may become 2%.

Parabolic crowns being overlayed should be corrected to a minimum of 1.5%; otherwise a design exception or variance must be submitted. Deck replacement bridges with parabolic crowns shall be corrected to a 2% cross-slope. See Chapter 12 for criteria and procedure.
(12-5-2005) (11-28-2011) (2-21-2017)

The road approach shoulder slope shall be transitioned to meet the bridge shoulder slope. The transition shall be based on superelevation transition slope (Δ%) from Standard Plan R-107 Series. The procedure is outlined in section 6.05.05 of the Road Design Manual. (8-20-2009) (11-28-2011)

Finishing bridge decks with varying cross slopes present challenges in construction. As a result, it is desired to avoid cases where superelevation transitions or crown runouts fall on a bridge. Where having superelevation transitions or crown runouts on a bridge is unavoidable contact the BOBS Construction Section to ensure that appropriate mitigation is included in the plans and Proposal to minimize the risk of problems occurring in the field.
(9-22-2025)

7.02.19 (continued)

Slabs

H. Superelevation Using a Straight Line Friction Ratio

Standard Plan R-107-Series shall be used to incorporate superelevations on structures. When Standard Plan R-107-Series cannot be used, the straight line method on overlay projects and on a very limited basis for deck replacements can be considered. The straight line method allows for a lesser superelevation and thus decreases the HMA wedging on the high side of a bridge overlay. It also reduces the haunch depth on deck replacements. See Bridge Design Guide 6.11.02 for straight line chart. (12-5-2005)

I. Link Slabs (1-25-2021)

Design Requirements and Considerations

Link slabs may be used to eliminate deck joints at piers. A link slab is comprised of a reinforced concrete deck with a length that includes 7.5% of each adjacent span, not necessarily the same length for each span. Saw cut lines are located at centerline of pier and at 5% of link slab length in each individual span. Link slabs are not designed to transmit live load effects from one span to another. As a result, the bridge (beams) is analyzed as simply supported spans for all vertical loads. Thus, shear stud connectors shall be omitted within the limits of the link slab and a 0.31 LBS/SFT roofing paper bond breaker is applied between the top flange and the link slab to prevent composite action. The total number of shear stud connectors per span required to meet strength requirements shall still be provided. If required, increase deck removal limits to permit placement of additional shear developers beyond the link slab. The link slab reinforcing is replaced (transverse) and lapped with existing reinforcement (longitudinal) to minimize crack widths based on the anticipated strains due to live load rotations for a girder.

7.02.19 (continued)

While the changes to the live load and dead load effects from link slabs are usually not significant, changing the articulation of the superstructure may significantly change the thermal and braking loads on the bearings, piers, and abutments. Evaluate the bearings, substructures, or foundations to determine if they can accommodate the new force configurations. It may be necessary to convert fixed bearings to expansion, increase capacity of expansion bearings and replace fixed bearings to increase capacity for longitudinal forces, etc. of the spans being linked together. Evaluate existing substructure elements in accordance with the load requirements of the AASHTO bridge design specifications in effect when the bridge was built.

MDOT link slabs are designed using the following maximum bridge lengths and widths criteria:

1. Straight, no skew concrete bridge:
   Length ≤ 300 ft.
2. 45˚ max. skew concrete bridge:
   Length ≤ 200 ft., Width ≤ 100 ft.
3. Straight, no skew steel bridge:
   Length ≤ 275 ft.
4. 45˚ max. skew steel bridge:
   Length ≤ 175 ft., Width ≤ 100 ft.

See Bridge Design Guides 6.44.01, & 01A.

7.02.20

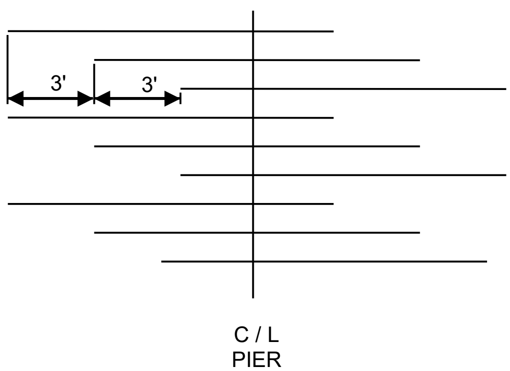
Slab Reinforcement

For general steel reinforcement information applying to both superstructure and substructures, see Steel Reinforcement (Section 7.04).

A. Negative Moment Reinforcement
(12-5-2005) (12-17-2012)

Where additional longitudinal reinforcement is required in regions of negative moment see AASHTO LRFD 6.10.1.7. If the longitudinal reinforcement is considered to be a part of the composite section, shear connectors shall be provided in negative flexure regions. Where shear connectors are used in negative flexure regions, the longitudinal reinforcement shall be extended into positive flexure region (AASHTO LRFD 5.10.8.1.2c). (6-27-2022)

Bar ends should have two 3’ staggers (see below) to minimize transverse cracking at bar terminations.



With continuous beam design, the bar length should be according to AASHTO LRFD 5.10.8.1.2c. (6-27-2022)

B. Bar Spacing (6-27-2022)

See AASHTO LRFD 5.10.3 and 9.7, section 7.02.19 and Bridge Design Guide 6.41.01.

C. Bar Laps

See Bridge Design Guide 7.14.02A.

Transverse slab reinforcement, if possible, is to be lapped as follows: top steel between the beams and bottom steel over the beams.

7.02.20 (continued)

D. Cover

All decks will provide 3" of clear concrete cover to the top of transverse reinforcement. See Bridge Design Guide 6.41.01. (5-6-99)

E. Placing of Transverse Bars

Transverse bars are generally placed perpendicular to the beams; however, where the angle of crossing is 70º or greater, transverse bars may be placed parallel to the reference lines if "S along the skew" falls in the same beam spacing range as "S normal to the beams" or the next larger range (see Bridge Design Guide 6.41.01).

Dimensioning is to be perpendicular to reference lines when the transverse bars are laid parallel to the reference line.

F. Epoxy-Coated Reinforcement

All bars in the superstructure are to be epoxy coated.

G. Additional Reinforcement When Haunch is Excessive (>4” concrete or >6” steel) (6-27-2022)

Add additional transverse and longitudinal reinforcement when haunch depths exceed 4” on prestressed concrete beams. Space additional transverse haunch reinforcement (EW05 or EK05 bars) between transverse bars, and ensure bars sufficiently penetrate haunch and slab. See Bridge Design Guide 6.42.03A for details.

Add additional longitudinal reinforcement when haunch exceeds 6” on steel beams. See Bridge Design Guide 6.42.03 and 8.07.01 for details.

7.02.21

Continuous Beam Design - Steel

A. Pour Sequence

Where temporary supports are not provided under continuous beams, a pour sequence is to be given to ensure that deflections occur as assumed in the design.

B. Preloading (Section Deleted.)(6-27-2022)

7.02.22

Screeding

A. Transverse Screeding (1-29-2024)

Transverse screeding shall be used for finishing all bridge decks.

When the skew angle is greater than or equal to 15º, the strike equipment is placed parallel to the reference lines.

For spans with different skew angles at each end, use the greater skew angle for equipment orientation.

B. Screed Elevations

In computing screed elevations, the specified camber should be used.

The following dead loads should be used in computing beam deflection for screed elevations:

10 LBS/SFT for formwork
10 LBS/SFT for reinforcing steel
145 LBS/CFT for plain concrete
150 LBS/CFT for reinforced concrete

Screed elevations for suspended spans are to be figured for the case of no deck concrete having been poured in any span.

Screed elevations for prestressed concrete beams are to account for long term effects by modifying the beam deflections using the following factors:

Factor applied to prestressing force at release = 1.9+0.6($I_{Girder} / I_{Composite}$)

Factor applied to beam self-weight at release = 2.1+0.7($I_{Girder} / I_{Composite}$)

Factor applied to slab when poured (including SIP forms, diaphragms and utility loads) = 1.0+1.1($I_{Girder} / I_{Composite}$)

Factor applied to barrier and sidewalk when poured = 2.3

I_{Girder} = moment of inertia of girder
$I_{Composite}$ = moment of inertia of composite section

(6-27-2022)

7.02.23

Stay-In-Place Forms

A. Use (9-2-2003)

Because of the design accommodations, any need for stay-in-place forms should be anticipated in the Contract Plans and Specifications.

The criteria for the use of metal stay-in-place forms are safety and economy in construction. Where practical, they should be included as a contractor option.

The use of concrete stay-in-place forms is not allowed.

B. Design (5-6-99) (9-21-2015)

The design of metal stay-in-place forms is the responsibility of the contractor. The corrugations for all stay-in-place forms should be filled with polystyrene. Use note 8.07.01 G. (6-27-2022)

When the use of stay-in-place forms cannot be economically justified the designer shall prohibit their use by including note 8.07.01 H. on the plans. (9-2-2003) (8-23-2021)

Detail steel beam tension zones on plans. Welding or mechanically fastening permanent metal deck forms or accessories to structural steel is prohibited. (6-16-2014) (3-26-2018)

7.02.24

Joints in Deck Slabs

A. Longitudinal Joints

Deck widths greater than 100'-0" require a longitudinal open/expansion joint. (5-6-1999)

1. Centerline (Median) Joint

   For bridges requiring a longitudinal open joint, which are also on roadways having a median barrier, the barrier is to be split, with the open joint extending up between the two halves.

2. Valley Gutter Joint

   To facilitate the construction of bridges with valley gutters, we will show an optional longitudinal construction joint 2'-0" inside or outside the gutter centerline (depending on beam placement), and the reinforcing steel will be detailed with a splice at the gutter centerline.

3. Construction Joints

   An optional longitudinal construction joint is to be shown on the plans when the bridge width exceeds 75'-0". For skews greater than or equal to 45°, this 75'-0" is measured parallel to the reference lines. This optional joint is to be placed at the edge of a pavement lane, regardless of location of the crown of the road.

   Longitudinal construction joints are not to be placed over the flange of a beam.

4. Part-Width Construction (6-27-2022)

   Where possible, longitudinal construction joints used to facilitate part-width construction should be placed at the edge of a pavement lane. This greatly improves ride quality and aesthetics. Do not place joints over a beam. Joints eventually leak, and subject beams to deterioration.

7.02.24 (continued)

Joints in Deck Slabs

B. Transverse Joints (9-27-2021)

1. Construction Joints and Reference Joints

At construction joints where movement is anticipated, an expansion joint device shall be used.

Construction joints over piers at fixed bearings are to be a sawed joint 1½” deep by ⅛” wide (minimum) in the top of slab. The joint is to be sawed within 24 hours of placing the curing and is to be filled to ¼” below top of concrete with polyurethane or polyurethane hybrid sealant Included in the bid item “Superstructure Conc, Form, Finish, and Cure, Night Casting.” Add note 8.07.01 K. to the plans.
(8-23-2021) (8-24-2020)

Typical construction joints (transverse, longitudinal and at fixed pin & hangers) are to be a sawed joint ½” deep by ⅛” wide (minimum) in the top of slab and is to be filled to ¼” below top of concrete with polyurethane or polyurethane hybrid sealant. Included in the bid item “Superstructure Conc, Form, Finish, and Cure, Night Casting.” Add details and note 8.07.01 M. to the plans. (8-24-2020) (8-23-2021)

For simple spans, transverse construction joints may be needed when the volume of concrete required to construct the bridge deck (including haunches) exceeds 300 cubic yards. The MDOT BOBS Bridge Construction Unit must be contacted to confirm the need for the construction joints based on the project specific conditions. When deemed necessary, locate the transverse construction joints at the 1/4 and 3/4 points of the span. (1-27-2025)

Reference joints located over integral and semi-integral backwalls are required joints and are not to be labeled as optional on plan details. Casting the approach slabs separate from the bridge deck and after the bridge deck has been cast is critical to proper movement and durability of the bridge. Reference joints are to be a sawed joint ½" deep by ⅛" wide (minimum) in the top of slab and filled ¼" below top of concrete with polyurethane or polyurethane hybrid sealant. Included in the bid item “Superstructure Conc, Form, Finish, and Cure, Night Casting.” Add note 8.07.01 M. to the plans. See Bridge Design Guides 6.20.03 A, B, & C and 6.20.04 for details.
(8-24-2020) (8-23-2021)(1-27-2025)

7.02.24 (continued)

Joints in Deck Slabs

2. Expansion Joints

   The maximum single opening in an expansion joint device shall be no more than 4", measured in the direction of traffic. When movement required is greater than 4" a modular expansion joint shall be used. (5-6-99)

   Expansion joint devices shall be installed ¼" to ⅜" below the adjacent deck elevation. This fact shall be taken into account during design. This recess is to prevent damage to the joint from snow plows. (5-6-99) (2-16-2015)

   The EJ3 Sheet included with the plans will designate the total travel that is required at each joint, measured along the centerline of bridge, and the angle of crossing rounded off to the nearest 10º. The length of the device required at each location will be shown, and these lengths totaled for one bid item, "Expansion Joint Device." The fact that the one item includes several minimum travel requirements should not affect the bid price since we currently find little or no difference when we list minimum travels separately. The EJ4 Sheet shall be used with replacement of existing neoprene expansion joint devices. Use of EJ4 Sheet (device) requires Form 0304 (Proprietary Item Certification (PIC) and Public Interest Finding (PIF)) be filled out and placed in the project file for Delcrete Elastomeric Concrete (D.S. Brown, 300 East Cherry Street, North Baltimore, OH 45872, Telephone: 419.257.3561). Delcrete is a PIC with "No Equally Suitable Alternative". See section 15.04 and section 11.08 of the Road Design Manual.
   (8-20-2009) (2-16-2015)

7.02.24 (continued)

   After contract award and before placing the order, the contractor shall inform the Construction Engineer which devices and models they intend to install. The standard shop drawings of the joint devices are located at Miscellaneous Bridge Details site. (6-27-2022)

   When an expansion joint device is used on a sidewalk it shall be fitted with a cover plate as described and detailed in Section 7.02.27 and EJ3 and EJ4 Sheets.(8-20-09)

7.02.25

Pavement Seats

Pavement seats are to be provided on all bridges except integral and semi-integral structures with continuous pour over reference lines (also see Section 7.03.01 C). (5-6-99)

7.02.26

Drain Castings

A. Location

Drain castings in bridge decks should be avoided where practicable. Where drain castings are necessary, they are to be spaced as required but located so as not to allow water to fall on slopes and/or roadways below. Design is to be based on Hydraulic Engineering Circular No. 21 (HEC 21), “Design of Bridge Deck Drainage”, or an equal. (5-6-99)

B. Special Reinforcement Steel

Where drain castings are called for in bridge decks, plans are to show that two epoxy coated reinforcing bars are to be placed diagonally at each corner of the drain casting (one top, one bottom). (5-6-99)

7.02.27

Sidewalks (9-2-2003) (11-25-2019)

In general, on a bridge where pedestrians must be accommodated and where maximum posted speed is 40 mph or less, a raised sidewalk should be provided if there is a raised sidewalk on the approach. Where posted speed is greater than 40 mph or there is no raised sidewalk on the approach, a walkway at roadway level should be provided and protected from traffic by an MDOT approved bridge railing.

Where sidewalks are required, they should be 5'-2" or greater in width. However, in circumstances where a 5'-2" width is not achievable a 4'-2" minimum width is permissible if crash tests allow. (8-20-2009)

7.02.27 (continued)

When the bridge railing length is greater than 200 feet, to adhere to Americans with Disabilities Act (ADA), the sidewalk must be 5’-0” wide (@ 2.1% maximum slope)* or a 5’ square passing space shall be located at intervals not exceeding 200 feet. The requirement is valid with a raised sidewalk as on Standard Plan B-25-Series, B-26-Series or B-27-Series and anywhere where the sidewalk is located behind a railing that separates pedestrians from traffic. For railing lengths less than 200 feet the sidewalk width may be 4’-2” if crash tests allow and does not require passing spaces. * Use a target cross slope (2.0%) less than the maximum to account for inconsistencies in concrete finishing. (2-26-2024)

Expansion joints located on sidewalks shall be fitted with cover plates to eliminate vertical depressions caused by the joint. See Expansion Joint sheets (EJ3 or EJ4). Detail cover plates that require a length greater than 11’ to be fabricated from two equal length pieces with a joint located at the centerline of the sidewalk or path. Provide a ¼” wide gap at the joint that is parallel to the centerline of the sidewalk or path. (1-23-2023)

For additional information refer to Bridge Design Guides 6.05.02, 6.29.10C, 6.29.17E and Road Design Manual Section 6.08.

Where a shared (multi) – use path or other mode of transportation is anticipated or proposed for the bridge, verify that all users have been accommodated and refer to appropriate specifications for design criteria. (12-16-2019)

A. Sidewalk Joints

Space sidewalk joints to match any joints in the slab. (9-25-2017)

B. Independent Sidewalk

If the sidewalk is independent of the roadway slab, the sidewalk is to be designed for maximum wheel loading for the bridge with overstressing as allowed by the current AASHTO Standard Specifications for Highway Bridges.

7.02.28

Railing (9-2-2003) (11-25-2019)

Where bridge railing is to be installed, and no sidewalks are present or the sidewalk is located behind the railing, install one of the bridge railing types currently approved for use by MDOT. The only MDOT-approved bridge railing types are Type 6 Barrier (see Standard Plan B-29-Series), Type 7 Barrier (see Standard Plan B-28-Series), 2 Tube railing (see Standard Plan B-21-Series), 4 Tube railing (see Standard Plan B-26-series), Aesthetic Parapet Tube railing (see Standard Plan B-25-Series) and 3 Tube With Pickets (see Standard Plan B-27-Series).

Where bridge railing is to be installed on raised sidewalks, use only 4 Tube railing (see Standard Plan B-26-series), Aesthetic Parapet Tube railing (see Standard Plan B-25-Series), or 3 Tube with Pickets (see Standard Plan B-27-Series).

A. Railing Types and Their Use (9-2-2003) (11-25-2019) (10-24-2022)

In general, use Bridge Barrier Railing, Type 6, on all new structures and reconstruction (major rehabilitation) bridge projects without sidewalks (see Standard Plan B-29-Series). Substitute Type 7 Barrier on structures where sight distance or clear roadway width is a problem (see Standard Plan B-28-Series). Bridge Barrier Railing, Type 6 and Type 7 are preferred on freeways and interstate routes. At stream crossings or scenic areas, use Bridge Railing, 2 Tube, Aesthetic Parapet Tube, 4 Tube or 3 Tube with Pickets (see Standard Plan B-21-Series, B-25-Series, B-26-Series or B-27-Series). Do not use Bridge Railing, 2 Tube on freeways and interstate routes or adjacent to pedestrian traffic because the height is insufficient. On bridges where pedestrian or bicycle traffic is separated from vehicular traffic by a standard barrier, it is not necessary to provide a vehicular railing at the fascias. In such cases, pedestrian fencing is desirable.

For structures without sidewalks, but where some pedestrian traffic is likely, install a Bridge Railing, 4 Tube, Aesthetic Parapet Tube or 3 Tube with Pickets.

7.02.28 (continued)

B. Joints (6-27-2022)

To avoid cracking, an open joint is required in concrete railings at all deck joints where reinforcing steel is not continued through the joint. False joints are not required in barrier railing.

Use a 1" joint in all concrete railings over the piers of continuous decks, at midspan on all structures with a span greater than 100'-0" and cantilever decks where the cantilever is more than 10'-0" long. The joint must be perpendicular to the centerline even on skewed bridges. Use a 1" joint filler to fill the joint to ½" from the bevels of the railing. Seal the remaining ½" with a polyurethane or polyurethane hybrid sealant. (5-1-2000) (2-21-2017)

C. Median Barrier vs. Bridge Barrier Railing (5-6-99)

Criteria for use:

1. Concrete barrier on a bridge must be reinforced and attached to the structure.
2. Barriers that function as railings must be at least 3'-6" in height.
3. Barriers that function as median barriers must be at least 2'-8" in height.
4. Concrete glare screens required on approaches must be continued across structures.
5. When structures are spaced 150'-0" apart or less along the traveled roadway, install a concrete barrier (Concrete Barrier, Single Face or approved alternate) between the two structures, in lieu of guardrail, to provide continuity. Approval by the agency having jurisdiction of the approaches is required.

7.02.29

Fencing

A. MDOT's Policy

For protective screening, follow AASHTO's A Guide for Protective Screening of Overpass Structures.

Consider protective screens under any of the following conditions:

1. Reported incidents of objects being dropped from an overpass.

2. On overpasses with walks where experience on nearby structures indicates a need for screens.

3. On overpasses in large urban areas used exclusively by pedestrians.

4. On overpasses near a school, a playground, or elsewhere where it would be expected that the overpass would be frequently used by children unaccompanied by adults.

B. Metro Region Criteria (9-2-2003)

Contact the Region Project Development or Bridge Engineer to determine if pedestrian screening/fencing must be added to projects. General criteria:

1. Include bridge screening when major bridge rehabilitation is scheduled for a structure.

2. Railroad structures must have bridge screening due to the presence of ballast and discarded rail spikes.

3. Screening is not required for structures without designated pedestrian access. This includes, but is not limited to, freeway to freeway connecting structures and all freeway ramp structures.

For additional information on pedestrian fencing, see Section **7.05** and Section **2.02.11**.

7.02.30

Bridge Mounted Noise Walls (3-23-2026)

While the guidance provided herein is specific to bridge mounted noise walls, it may be relevant to noise walls mounted on other types of structures. Follow the guidance herein where relevant and consider all other applicable design criteria and structure-specific issues.

A. General Considerations

In addition to the guidance provided in this section, refer to the design criteria outlined in the **MDOT Noise Barrier Wall Design Guidelines** as it is applicable to bridge mounted noise walls or when specified herein.

1. Naming Bridge Mounted Noise Walls

   Refer to the structure number naming convention in the **Michigan Ancillary Structures Inspection Manual (MiASIM)** when providing a name for bridge mounted noise walls on plans.

2. Wall Height

   Avoid steps in the noise wall height. Designers must follow the requirements outlined in the **MDOT Noise Barrier Wall Design Guidelines Section 2.03.03**.

3. Zone of Intrusion

   Place bridge mounted noise walls outside the zone of intrusion (ZOI) as defined by the **AASHTO Roadside Design Guide Section 5.5.2** when possible.

   When designers must locate noise walls inside the ZOI, it is preferred to anchor posts to the back face of bridge barriers to better accommodate ZOI requirements.

7.02.30 (continued)

4. Wall Endings

   Avoid blunt ends by providing a sloped panel at the end of the bridge. If the noise wall must continue off of the bridge, then extend the bridge mounted wall onto the bridge approach in order to provide an overlap with the adjacent ground-mounted wall satisfying the 4D rule as defined in the **MDOT Noise Barrier Wall Design Guidelines Section 2.01.02.** This may require use of a moment slab to provide the required overlap. Provide a sloped panel at the end of the structure-mounted noise wall. Adhere to the **MDOT Noise Barrier Wall Design Guidelines Section 2.03.02** for crash protection beyond the extents of the bridge.

5. Expansion Joints

   Bridge mounted noise walls and their connections must adequately accommodate the expansion, contraction, rotation, and increased stiffness of the bridge superstructure it is attached to. Determine locations of wall expansion joints according to **AASHTO LRFD Section 15.6**.

B. Wall Types

Coordinate the noise wall type selection with the project noise analysis and ensure the selected system provides the noise abatement used in the analysis. For example, the noise analysis may include use of absorptive noise barriers which must be considered in the selection of noise wall type.

Regardless of wall type, avoid gaps between the noise wall and bridge to prevent debris from falling onto traffic below. Include a joint in the noise wall wherever there is a joint in the barrier system, including expansion joints.

The most common bridge mounted noise walls include:

1. Metal post and concrete panel systems mounted to the back of concrete railings.

7.02.30 (continued)

2. Cast-In-Place concrete noise walls on top of concrete railings.

3. Metal post and composite or acrylic panel systems mounted to the back of concrete railings.

Various proprietary and non-proprietary systems of these noise wall types exist. If the designer uses a system with posts, they must detail the posts to be vertical.

Designers must only use systems that have successfully passed crash testing consistent with MASH criteria for both proprietary and non-proprietary systems. Crash testing documentation must consist of an FHWA letter or certified crash testing facility report. The MDOT Chief Structure Design Engineer must approve the crash testing documentation.

When roadway design speeds are 40 mph or less, design walls and their components for MASH TL-2 or higher. When design speeds are 45 mph or more, design walls and their components for MASH TL-4 or higher.

C. Non-Proprietary Wall Design Criteria

Design and detail non-proprietary noise walls and their connections per the requirements specified in **AASHTO LRFD Section 15.** Design and detail the associated bridge railing per the requirements specified in **AASHTO LRFD Section 13.**

See the **MDOT Noise Barrier Wall Design Guidelines Section 2.04.03** for guidance on handling, hauling, and lifting of precast wall elements.

The following information is supplemental to design guidance provided in **AASHTO LRFD**.

1. Wind Load

   Designers should determine wind loading using **AASHTO LRFD Section 15**.

7.02.30 (continued)

2. Snow and Ice Load

   Designers do not need to consider horizontal or vertical snow and ice loading on the noise wall as vehicle collision and live loads will govern.

3. Fatigue

   Avoid details with poor fatigue performance. Evaluate fatigue caused by wind loading using **Section 11** of **AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals**.

D. Proprietary Wall Design Criteria

If a proprietary system is used, it must meet the specifications provided in the previously approved special provision for Structure-Mounted Lightweight Noise Barrier Wall.

Designers must ensure that the bridge barrier is consistent with the barrier used during crash testing of the proprietary noise wall system.

E. Bridge Design Considerations

Designers must consider the effects bridge mounted noise walls have on the entire structure, both proposed and existing components, in particular but not limited to the deck and beams.

Designers may mount noise walls to existing bridges when the bridge is receiving a deck or superstructure replacement, or deck widening. Mounting noise walls to existing bridges outside of these circumstances is not permitted unless the designer replaces the existing bridge barrier. They must also remove and replace a portion of the existing bridge deck along the fascia to ensure applicable loads can be transferred accordingly. Determine the width of deck removal based on the beam spacing and the development length needed to provide adequate connection to the existing bridge deck reinforcement in accordance with **AASHTO LRFD 5.10.**

7.02.30 (continued)

Designers must consider the existing condition, original loading, and design criteria of the bridge’s existing components to remain when attaching noise walls to existing bridges. If adding a noise wall to an existing bridge requires strengthening of the existing beams, consider other alternatives including but not limited to a superstructure replacement or beam replacement. See the following sections for additional considerations.

The following guidance applies to both existing and new bridges.

1. Dead Loads

   When noise walls are mounted to bridges the increased dead load at the fascia requires the checking of various items:

   a. Potential Negative Camber

      Designers must account for the expected dead load of the noise wall when designing camber of steel members for new bridges. For prestressed members, negative camber must be checked in the final condition and avoided if possible.

      With existing bridges, designers must check for negative camber. If present, consider issues including, but not limited to, whether vertical clearance is reduced below required limits and possible public perception of “sagging” beams. Consider lightweight wall types or superstructure strengthening to avoid negative camber.

7.02.30 (continued)

b. Slab and Screed Adjustments

Designers must account for the weight of the noise wall system when calculating slab and screed elevations. Consider that contractors may set the screed machine to the cross-slope of the final slab surface when they construct the slab. When the wall and bridge barrier are placed after the deck is poured, the added weight may cause additional deflection or negative camber at the fascia beam. As a result, final deck cross slopes may vary producing potential ponding areas, and compromised ride quality. These issues may increase maintenance needs and shorten the service life of the bridge.

Evaluate the potential discrepancy in final cross-slopes and provide mitigation measures if the deviation from the proposed cross-slopes would exceed normal construction tolerances.

c. Dead Load Assumptions

Designers utilizing proprietary systems must estimate the noise wall dead load using the special provision requirements for the various noise wall components and include an allowance to address uncertainties in the actual provided weight of the system.

When proprietary noise wall system shop drawings are submitted, the designer must compare the actual dead load of the system with the assumed design load to confirm the actual load is less. Consider if any changes to slab and screed elevations are necessary given the actual weight of the provided system**.**

7.02.30 (continued)

d. Dead Load Distribution

Apply noise wall dead loads by distributing 75% of the load to the fascia beam and 25% to the first interior beam and apply this distribution to all bridge components impacted by the noise wall.

Use the least favorable loading cases, load factors, and combinations in all design checks.

2. Slab Overhang Design

Design slab overhangs for all loads induced by the noise wall as required by **AASHTO LRFD** and the provisions herein, including but not limited to dead load, wind load, and vehicular collision loads. Design slab overhangs per the requirements outlined in **Section 13** of **AASHTO LRFD.**

Additionally, consider the impacts on the first interior slab bay and the fascia rebar cut off lengths from all noise wall loadings. If the capacity is not adequate for an existing bridge, retrofit the interior slab bay to accommodate the increased load.

3. Uplift and Overturning

Bridge mounted noise walls can introduce unbalanced mass to the structure which can impact the load distribution across the girders and overall structural behavior.

Designers must consider overturning forces generated by horizontal wind loads on the wall in all applicable load combinations alongside vertical wind loads per **AASHTO LRFD Section 3.8.2**.

Ensure that all bridge components supporting either proprietary or non-proprietary noise wall systems resist the vehicular collision loads from **AASHTO LRFD Section 15.8.4**.

7.02.30 (continued)

Bridges with horizontally curved girders or complex geometries may require refined analysis to accurately capture the distribution of overturning forces. Ensure that uplift at bearings does not occur under any load combination. Consider using a lightweight noise wall system or selectively increasing the size of components to act as a counterweight in order to eliminate uplift.

4. Torsionally Stiff Fascia Beams

   Torsionally stiff fascia beams such as box beams are often assumed to resist minimal torsional loading and may not be explicitly checked for torsional effects. They may be required to resist significant torsional moments due to wind load effects when noise walls are mounted on bridges. Thus, designers must evaluate the torsional demand on the fascia beams and design them as specified in **AASHTO LRFD Sections 4.6, 5.7, and 6.11.** Consider a lighter noise wall system or provide additional load paths to distribute the torsion such as adding cross frames if beams are unable to be designed to resist the effects of torsion.

7.02.31 (8-23-2021)

Precast Box/Three Sided/Arch Culverts

Design criteria and considerations:

A. Verify with manufacturers the maximum span length available.

B. The number of manufacturers of the specified span length needs to be at least two.

C. When selecting culvert rise, consider all users of the waterway, along with normal water surface under clearance and freeboard at high water.

7.02.31 (continued)

D. For structure (culvert) lengths that can accommodate a clear span between guardrail posts of 25’-0” or less use “Guardrail Long Span, Detail MGS-1, MGS-2 or MGS-3” (Standard Plan R-72-Series) to span the culvert. Ensure that the requirements of Standard Plan R-72-Series (e.g., headwall location and size, guardrail post locations, etc.) are met prior to specifying the use of the standard. Otherwise, extend height of headwalls to 36” above plan grade elevation and attach guardrail to headwall as detailed on the plans.

E. Add a PVC (polyvinyl chloride) liner that covers entire top and sides of all buried culverts. For precast boxes, extend the liner to the top of the culvert bedding and turn out 6" horizontally across the top of the culvert bedding. For three sided and arch culverts, extend the liner down the leg of the culvert, horizontally across the top of the pedestal wall, and down the vertical face of the pedestal wall 18" or to the top of the footing, whichever is less. Extend the liner a minimum of 3 feet beyond the construction joint between culvert and wingwalls and turn up at back side of headwalls. At the ends of the culvert, adhere the perimeter of the liner with an adhesive as recommended by the PVC liner manufacturer. (1-29-2024)

   Where staged construction is used to install the culvert and a temporary MSE wall is required at the stage line, stop the PVC liner a horizontal distance equal to the temporary MSE wall height away from the temporary MSE wall face and adhere the perimeter of the liner with an adhesive as recommended by the PVC liner manufacturer. If a temporary MSE wall is not required at the stage line, the PVC liner may be installed across the stage line joint, or two pieces of PVC liner may be welded together as recommended by the PVC liner manufacturer. (1-29-2024)

   Include Special Provision for Polyvinyl Chloride Liner in proposal. (2-22-2022)

Additional information and criteria are included in the current Standard Specifications.

7.02.32 (11-28-2011)

Deck Replacements

With deck replacements or widening projects (or reconstruction projects), the structural adequacy of the entire structure shall be evaluated. In addition to the criteria listed below, deck replacements shall meet all requirements listed in this chapter (e.g. slopes, shoulder width, stay in place deck forms and approach items). Consider all modes of transportation and evaluate whether deck replacement or widening can accommodate all users. (12-16-2019)

A. Beams

1. On concrete T-Beam bridges the deck slab is an integral part of the support system and cannot be removed without dismantling the entire superstructure. The cost of deep chipping (or hydrodemolishing) combined with the installation of a cathodic protection system should be weighed against the cost of complete superstructure replacement.

2. On steel stringer bridges, the tops of beams shall be blast cleaned and coated with an organic zinc-rich primer. Shear connectors shall be placed to upgrade the capacity of existing non-composite decks. (12-5-2005)

3. On prestressed concrete side by side box beam decks, the existing wearing course is replaced with a 6" reinforced deck.

4. On older steel stringer bridges, lateral bracing was often added as a part of the original construction. If lateral bracing was removed subsequent to the original construction, consider whether the beams require temporary lateral bracing during the deck replacement. (6-27-2022)

B. Railings

Railings shall be upgraded when bridge deck replacements are planned. See section 7.02.28.

7.02.32 (continued)

C. Geometrics

Criteria for roadway widths and design loading have been established in ***A Policy on Design Standards - Interstate System***, 2016, and ***A Policy on Geometric Design of Highways and Streets***, ***2011, 6th Edition*** published by AASHTO. These criteria are based on the type of roadway carried by the structure and are summarized in this section. Non Interstate structures with deck replacements or widening projects (or reconstruction projects) shall adhere to ***A Policy on Geometric Design of Highways and Streets***, ***2011, 6th Edition*** design criteria (standards). Interstate structures shall adhere to ***A Policy on Design Standards - Interstate System***, 2016. MDOT policy has set bridge (shoulder) widths 2' (offset) greater than AASHTO widths for safety considerations of the traveling public. See Bridge Design Guides 6.05 Series & 6.06 Series. (6-27-2022)

7.02.32 Deck Replacements (Cont.)

CLEAR ROADWAY WIDTHS AND DESIGN LOADING FOR DECK REPLACEMENTS			
Type of Roadway		**Minimum Clear Roadway Width**	**Minimum Design Loading**
Non-Interstate Freeway		A, C	HL-93
Interstate Freeway		B, C	HL-93
Arterial (Non-Freeway Trunkline)	Rural	Table 7-3.	HL-93
	Urban	D, C	HL-93
Collector (Non-Trunkline)	Rural	Table 6-6.	HL-93
	Urban	Table 6-5., E	HL-93
Local (Non-Trunkline)	Rural	Table 5-6.	HL-93
	Urban	Table 5-5., E	HL-93

(A) The minimum clear roadway provided shall accommodate the pavement and full shoulders of the approach roadway or the minimum AASHTO requirements for lane and shoulder widths, whichever is greater.

(B) The minimum clear roadway provided shall accommodate the pavement and full shoulders of the approach roadway.

(C) For bridges in excess of 200'-0" in length, where the nearest offset from the edge of traveled way to either curb or barrier is greater than 4'-0" on the approaches, the nearest offset on the bridge shall be at least 4'-0" on each side. (12-5-2005)

(D) The minimum clear width on the bridge shall be the same as the curb-to-curb width of the street.

(E) The minimum clear roadway shall be the traveled way plus 1'-0" to each curb face. However, consideration should be given to providing the same width as the curb-to-curb approach width if it is cost effective to do so.

The tables shown below are derived from A Policy on Geometric Design of Highways and Streets, 2011, 6th Edition published by AASHTO and do not include clearances for bridge rail offset. See the Bridge Design Guides for MDOT offset criteria. (6-27-2022)

MINIMUM WIDTH OF TRAVELED WAY FOR RURAL ARTERIALS (FROM Table 7-3.)

Design Speed(mph)	Design Traffic Volume (veh/day): Under 400	400-1500	1500 -2000	over 2000
	Width of Traveled Way (ft)[a]			
40-45	22	22	22	24
50-55	22	22	24	24
60-75	24	24	24	24

[a] On roadways to be reconstructed, a 22 ft traveled way may be retained where the alignment is satisfactory and there is no crash pattern suggesting the need for widening.

MINIMUM CLEAR ROADWAY WIDTHS FOR RURAL ARTERIAL BRIDGES BEING RECONSTRUCTED (FROM Table 7-3.)

Design Traffic Volume(veh/day)	Min. Clear Roadway Width of Bridge
under 400	Traveled way + 4 ft (ea. side)
400-2000	Traveled way + 6 ft (ea. side)[b]
over 2000	Traveled way + 8 ft (ea. side)[b]

[b] For bridges in excess of 200 ft in length, a minimum width of traveled way + 4 ft on each side will be acceptable.

MINIMUM WIDTH OF TRAVELED WAY FOR COLLECTOR ROADS (From Table 6-5)

Design Speed(mph)	Design Traffic Volumes (veh/day): Under 400	400-1500	1500 -2000	over 2000
	Width of Traveled Way (ft)			
20-30	20[a]	20	22	24
35-40	20[a]	22	22	24
45-50	20	22	22	24
55-60	22	22	24	24

On roadways to be reconstructed, a 22 ft traveled way may be retained where the alignment is satisfactory and there is no crash pattern suggesting the need for widening.

[a] A 18 ft minimum width may be used for roadways with design volumes under 250 veh/day.

MINIMUM ROADWAY WIDTHS FOR NEW AND RECONSTRUCTED BRIDGES CARRYING RURAL COLLECTOR ROADS (From Table 6-6)		
Design Traffic Volume(veh/day)	Minimum Roadway Width of Bridge	Design Loading Structural Capacity
400 and Under	Traveled way + 2 ft (each side)	HL-93
400 to 1500	Traveled way + 3 ft (each side)	HL-93
1500 to 2000	Traveled way + 4 ft (each side)[a]	HL-93
over 2000	Traveled way + shoulders[a]	HL-93

Where the approach roadway width (traveled way plus shoulders) is surfaced, that surface width should be carried across the structures.

[a] For bridges in excess of 100 ft in length, the minimum width of traveled way plus 3 ft on each side is acceptable.

MINIMUM WIDTH OF TRAVELED WAY FOR LOCAL ROADS (from Table 5-5)				
	Design Traffic Volumes (veh/day)			
Design Speed(mph)	Under 400	400-1500	1500 -2000	over 2000
	Width of Traveled Way (ft)			
15	18	20	20	22
20-40	18	20	22	24
45-50	20	22	22	24
55-60	22	22	24	24

Where the width of traveled way is shown as 24 ft, the width may remain 22 ft m on reconstructed highways where there is no crash pattern suggesting the need for widening.

MINIMUM CLEAR ROADWAY WIDTHS AND DESIGN LOADINGS FOR NEW AND RECONSTRUCTED BRIDGES CARRYING RURAL LOCAL ROADS (From Table 5-6)		
Design Traffic Volume(veh/day)	Min. Clear Roadway Width of Bridge	Design Loading Structural Capacity
ADT 400 & under	Traveled way + 2 ft (ea. side)	HL -93
ADT 400-2000	Traveled way + 3 ft (ea. side)	HL -93
ADT over 2000	Traveled way + shoulders	HL -93

7.02.32 Deck Replacements (Cont.)

D. Salvaging Shear Developers
(10-24-2022)

For full or partial deck replacements on steel superstructures with stud type shear developers, shear developers should be left in place and reused whenever possible to minimize the risk of damage (and associated delays) to the steel beams/girders. For steel superstructures with spiral/coil type shear developers, remove the spirals/coils and install stud type shear developers using the appropriate pay items included in the MDOT Standard Specifications for Construction. If shear developer type cannot be confirmed with existing plans, contact the MDOT Bridge Construction Engineer to request a field investigation to confirm existing shear developer type.

The removal, furnishing, and installation of the additional shear developers is included in the special pay items listed in the Frequently Used Special Provision for Bridge Deck Removal and Salvaging Shear Developers on Steel Beams. Include a quantity equal to approximately 5% of the original shear stud total to account for existing damaged or deteriorated studs that must be removed and replaced. If additional studs are needed to meet strength requirements per AASHTO and Section 7.02.15, add quantity and detail proposed studs in relation to the existing studs on the plans. The location of the additional shear developers must account for the minimum spacing and edge distance requirements specified in AASHTO LRFD. If additional beam lines are being added to the superstructure as part of the deck replacement project the studs required on the new beam are included in the pay items listed in the Frequently Used Special Provision. Detail the transverse spacing and longitudinal pitch as part of the structural steel details.
(3-27-20223)

Where the existing shear developers are not tall enough to extend sufficiently into the new bridge deck per Bridge Design Guide 8.07.01, add EA04, EW05, and EK05 bars to haunch, similar to Bridge Design Guide 6.42.03A.

7.02.32 Deck Replacements (Cont.)

E. Pour Sequence for Superstructures with Pin and Hanger Assemblies (5-28-2024)

When replacing the bridge deck on superstructures with pin and hanger assemblies the pour sequence must be carefully considered. Generally, the sequence should call for the deck over the suspended spans to be poured first followed by the positive moment areas of the remaining spans. If the length of the cantilever (measured from the centerline of bearing at the pier to the centerline of the pin and hanger assembly) is relatively short, the negative moment areas over the piers and the cantilever can be poured with the positive moment area. If the length of the cantilever is relatively long, the negative moment area over the piers and the cantilever should be poured separate from the positive moment area of the cantilever span. Short cantilevers are considered 10 feet or less in length.

7.02.32 (continued)

The direction of the pour must also be considered. Generally, the following pour directions should be called for on the plans:

1. Suspended spans with pin and hanger assemblies at both ends – the pour can progress in either direction and should be left to the Contractor to decide based on their preferred means and methods.

2. Suspended spans supported on one end by an abutment and on the other end by a pin and hanger assembly - the pour should progress from the abutment toward the pin and hanger assembly.

3. Positive moment areas of cantilever spans with one end supported by an abutment – the pour should progress from the abutment toward the cantilever.

4. Positive moment areas of cantilever spans with both ends supported by piers and with cantilevers on both ends – the pour can progress in either direction and should be left to the Contractor to decide based on their preferred means and methods.

5. Negative moment areas of cantilever spans – the pour should progress from the point of dead load contraflexure toward the pin and hanger assembly.

Diagrams illustrating the pour sequence and pour direction for different superstructure configuration can be found in Figures 7.02.31 E.-A & -B.

Figure 7.02.31 E.-A

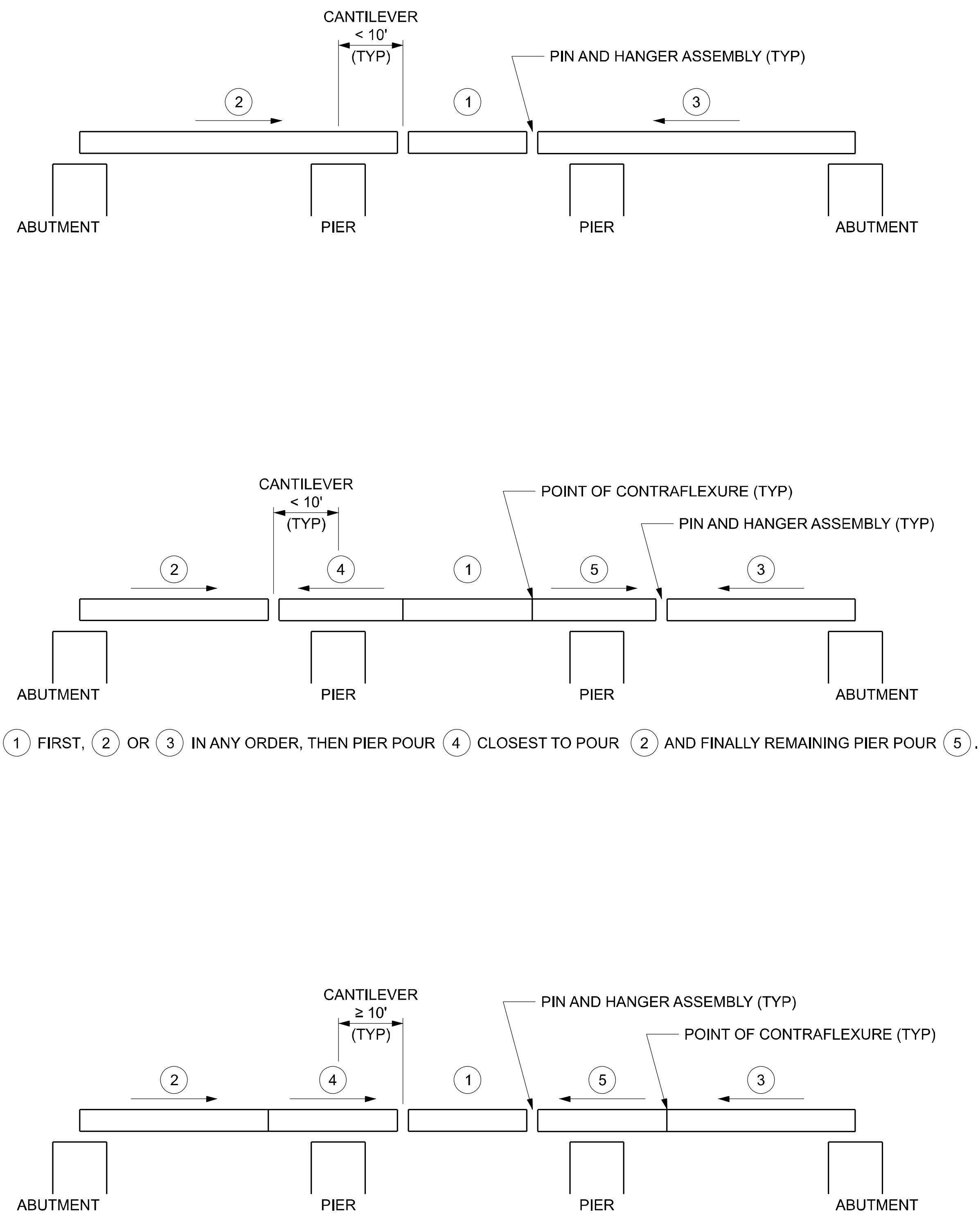
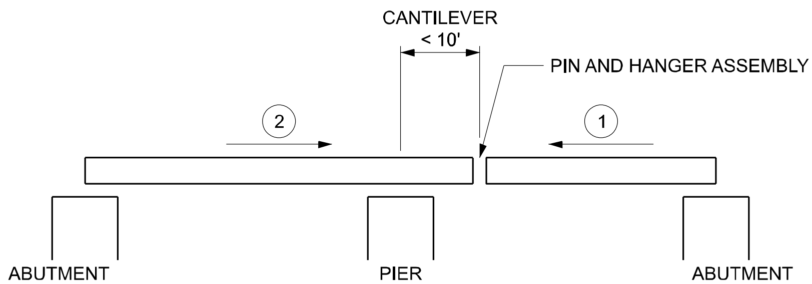
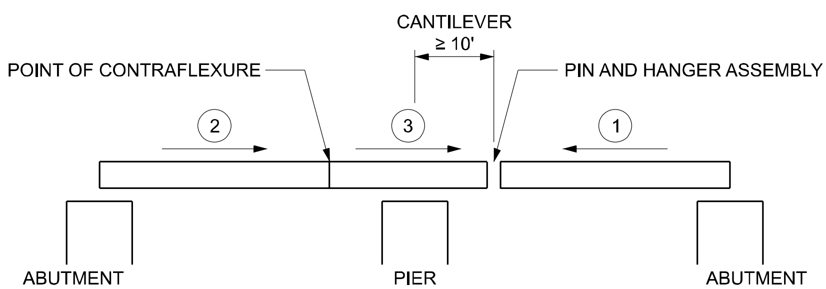


Figure 7.02.31 E.-B





NOTES:

CONCRETE POURS WITHOUT A POUR DIRECTION ARROW MAY PROGRESS IN EITHER DIRECTION AND SHOULD BE LEFT TO THE CONTRACTOR TO DECIDE BASED ON THEIR PREFERRED MEANS AND METHODS.

LEGEND

(2) POUR SEQUENCE NUMBER
→ POUR DIRECTION

PIN AND HANGER ASSEMBLY POUR SEQUENCES

7.02.33

Ride Quality

The purpose of a ride quality specification is to obtain a smoother riding pavement than is typically obtained with the traditional 10 foot straightedge smoothness requirements. Michigan first adopted a ride quality specification in 1979. The current specification prescribes classified levels of ride quality requirements described in subsequent paragraphs of this section.

Specific requirements for ride quality are identified by classification. Each classification (Class I, II, III & IV) specifies criteria for roughness, method of measurement, applicable incentives, disincentives, and corrective action. The matrix on the following page provides instructions for assigning ride quality classification based on scope of work, design speed, grade control and adaptability to production paving.

Ride quality requirements are not intended for application with stand-alone bridge projects. However, bridge deck replacements, and shallow or deep concrete bridge overlays included within the limits of a Class I ride quality section in a corridor project will be subject to ride quality requirements.

Using these criteria, the road designer will assign a ride quality classification to each applicable section of paving throughout the project. The locations and classifications are then tabulated for inclusion in the Notice to Bidders (generally done by the road designer).

The bridge designer will recommend if the bridge portions of a Class I section are to also be designated as Class I or are to be excluded by designation as Class II based on the type of work and adaptability to corrective deck grinding,

Within Class II, III, and IV areas, bridges are predetermined excluded areas from ride quality specifications between the two end reference lines or between the outermost limits of any structure expansion joint devices.

7.02.33 (continued)

The only pay item associated with ride quality is bump grinding. A small quantity should be included for each location where the contractor may be directed to grind *existing* pavement (i.e.: pavement not placed as part of the contract) in order to smooth the transition from old to new pavement. This includes the POB, the POE, and any *existing* bridge or railroad approaches within the project limits. 25 square yards for each lane at each of the above locations should suffice.

Bump grinding is normally not paid for in areas excluded from ride quality. Instead the pavement is accepted or rejected based on the 10 foot straightedge criteria. (***Standard Specifications for Construction***) If it does not meet the straightedge criteria, it is the contractor's responsibility to grind or replace at their cost.

For additional information on ride quality see the Road Design Manual section 6.04.05.

7.02.33 (continued)

Ride Quality Classification Selection Matrix

How To Use This Matrix		Contractor **has control** over grades		Contractor **has limited or no control** [2] over grades				
Divide the project into Sections [1] based on the amount of control the contractor will have over the final surface grades, the scope of work, and the design speed. Determine the recommended ride specification type for each section. Confer with Construction Field Services staff for exceptions or unique circumstances.		3R [3]	4R [3]	3R [3]	4R [3]	Single Course of Flexible Pavement (with/without milling)	Diamond Grinding Projects	Flexible Ultra Thin, Paver Placed Surface Seal
Section length **allows** for production paving [4]	Design Speed below 50 mph	Class II	Class II	Class II	Class II	Class III	Class III	Class III
	Design Speed 50 mph or above	Class I or II	Class I or II	Class II	Class II	Class III	Class III	Class III
Section length **does not** allow production paving [4]	Design Speed below 50 mph	Class III	Class II	Class IV	Class IV	Class IV	N/A	Class IV
	Design Speed 50 mph or above	Class II	Class I or II	Class IV	Class IV	Class IV	N/A	Class IV

Key:

Class I Ride Quality: Complete Projects (mainline only) where no excluded areas are allowed, a thresh hold IRI criteria must be met, and incentives and disincentives apply. Use Class I only on limited access roadway with design speeds 50 mph or greater and where most or all bridges include deck replacement, shallow concrete overlays, or deep concrete overlays. Investigate the feasibility of diamond grinding (at MDOT cost) any bridge decks not being replaced or overlaid. Where diamond grinding a bridge deck is not feasible, a limited section of the project can be designed as Class II Ride Quality such that the bridge would be a pre-determined excluded area within a project that would otherwise meet Class I ride Quality criteria.

Class II Ride Quality: Sections where threshold IRI criteria must be met, but incentives and disincentives do not apply. (Use Class II if all of the above requirements for Class I are not met.)

Class III Ride Quality: Sections where the pre-construction IRI must be maintained or improved by a certain percentage. Disincentives may apply.

Class IV Ride Quality: Sections where acceptance is based on a 10 foot straightedge criteria. Incentives and disincentives do not apply.

N/A = Not Applicable

Footnotes:

(1) A Section is defined as a length of paving which has the same characteristics (grade control, type of work, design speed).

(2) Locations where a contractor might not have control of grades include locations where they must pave adjacent to an existing lane with marginal ride quality, locations where there are existing curbs to match, and locations where there are frequent existing manholes or structures to meet.

(3) 3R means resurfacing, restoration, and rehabilitation. Primary examples include multiple course resurfacing, milling or profiling, concrete overlays and inlays (without removing subbase).4R means new construction or reconstruction. A primary example is complete removal and replacement of pavement (including subbase). See Chapter 3 for further definition and examples including projects with combined 3R and 4R work for classifications purposes on projects with multiple fixes.

(4) Production paving means a slipform paver can be used for concrete paving and that a HMA paver can be used without frequent stopping and starting and there is room for a haul truck to unload directly into the paver or a material transfer device while in motion. MDOT imposed construction staging requirements should be considered when making this determination

7.02.34

Bridge Mounted Sign Connections (2-24-2025)

On all projects that include the installation of a traffic sign supported by a bridge, the Bridge Engineer must confirm that the additional loading will not result in overstresses in the beam/girder based on AASHTO LRFD. If the analysis finds that the bridge beam/girder will be overstressed, design modifications to the bridge superstructure to keep the applied stresses below the allowable limits. This may include, but is not limited to installing stiffeners, installing additional diaphragms or cross frames, or installing lateral bracing.

The MDOT Sign Support Standard Plans for bridge sign connections with a plan date of 01/10/2024 or later have considered the capacity of the beam/girder and include any modifications required to keep the applied stresses below the allowable limits for the following conditions:

1. One sign is attached to the bridge beam/girder at any location along the span of the bridge.

2. For steel beams/girders the thickness of the web is equal to or greater than the minimum thickness specified in the MDOT Sign Support Standard Plans.

3. For concrete beams the concrete compressive strength of the deck and beam is greater than the minimum strength specified in the MDOT Sign Support Standard Plans.

4. The size of the sign falls within the maximum limits included in the applicable MDOT Sign Support Standard Plan.

7.02.34 (continued)

Any conditions outside of those listed above have not been considered in the development of the MDOT Sign Support Standard Plans and must be analyzed by the Bridge Engineer.

Generally, a traffic sign attached to a bridge will have a minor impact on the overall superstructure and on the substructure and it is typically not necessary to investigate the global effect of the bridge sign connection. The Bridge Engineer must determine whether project specific conditions outside of those listed above warrant a more detailed review that includes an analysis of the overall superstructure and the substructure.

For rehabilitation projects, see Section 12.07.11, Existing Bridge Sign Connections.

7.03

SUBSTRUCTURE

Design structures by placing all substructure units (piers & abutments) and slopes outside of the clear zone. For clear zone distances see Chapter 7 of the Road Design Manual. For substructure clearances also see Bridge Design Guide 6.06.01-.04. Provide guardrail protection for units or slope that cannot be placed outside of the clear zone. Place guardrail at a distance that will allow deflection as defined in Chapter 7 of the Road Design Manual. Design piers with base walls and guardrail approach terminals to maximize clear roadside distance in lieu of shielding piers with guardrail. Attach guardrail to base walls as detailed on Standard Plan R-67-Series. (7-24-2023)

Do not use steel sheet piling as support elements for substructures unless approved by the MDOT Geotechnical Section.
(3-28-2022)

7.03.01

Abutment Design

A. Design Cases

The following cases must be considered in the design of an abutment:

Case I
Construction state: abutment built and backfilled to grade.

Case II
Bridge open to traffic with traffic loading on the approach only.

Case III
Bridge with traffic on it and no load on approach.

Case IV
Contraction: Loading forces of Case II plus the effects of temperature contraction in the deck transmitted to the abutment. Tom.
Expansion: For integral abutments Case IV instead assumes the loading forces of Case III with the addition of an expansion force transmitted from the deck. (8-20-2009)

7.03.01 (continued)

B. Types

Fill material (lightweight fill or other low-density materials) can aid in the design of abutments. (6-27-2022)

1. Cantilever Abutment

   The maximum wall height for cantilever abutments is approximately 25'-0".

2. Counterfort Abutment

   Counterfort abutments should be considered when wall heights exceed 25'−0".

3. Curtain Wall Abutment

   Curtain wall abutments are to be considered where piles are required under the abutment and the abutment height does not exceed 9'-6" (see Bridge Design Guide 5.18.01).

   Curtain wall abutments of sufficient length to require expansion joints are to have the end piles battered outward parallel to the reference line. The purpose of this is to prevent the expansion joint from opening excessively.

7.03.01 (continued)

Abutment Design

B. Types

4. Integral and Semi - Integral Abutments

Integral and semi-integral abutments shall be used where practical to avoid deck joints. (5-1-2000)

Integral Abutment

Abutment walls (stub type) supported by one row of piles that allow movement through pile flexure (see Bridge Design Guide 6.20.04 series). Walls shall be a minimum of 5'-0" and 12'-0" maximum in height. The H-Pile webs shall be oriented parallel to the bridge reference lines and embedded a minimum of 30" into the abutment wall. Upon recommendation from Geotechnical Services Section pile holes shall be prebored. CIP piles may also be used if recommended by the geotechnical engineer. Embed CIP piles a minimum of 30” into the abutment wall. In general, integral abutments do not have return wingwalls that are attached to the abutment. (6-27-2022)

A separate design analysis needs to be performed on the abutment wall for active and passive pressures. Additional vertical dowels may be required at the abutment and backwall interface to resist the active surcharge and the passive resistance that have been introduced into the wall from bridge expansion. Additional vertical reinforcement may be required in the abutment wall and should also be designed. The pile spacing may need to be adjusted to prevent shear stress failure in the pile.

Due to scour considerations, the designer should usually avoid using Integral abutments at stream crossings unless spill through abutment criteria can be satisfied (Bridge Design Guide 5.47.01) or if abutments are placed outside of the scour limits of the stream crossing. (9-25-2023)

7.03.01 (continued)

Semi-Integral Abutment

Conventional abutment walls fixed in position with expansion and contraction movement of the bridge superstructure (see Bridge Design Guide 6.20.04 series). Abutments with a single row of piles should not be used.

The following design criteria are valid for both types of abutments.

a. Steel bridges are to be less than 300'-0" and concrete bridges are to be less than 400'-0" in length.

b. Use approach slab details on Standard Plan R-45-Series when the length of bridge contributing to expansion at an abutment is less than 50'-0" for concrete beam bridges and less than 25'-0" for steel beam bridges. (8-20-2009)

c. Angle of crossing shall be 60 degrees minimum and 120 degrees maximum. See Section 7.01.14 for MDOT skew policy. (12-5-2005)

d. Backfill shall be “Backfill, Structure, CIP” as per Standard Specifications.

   Place aggregate base or open graded drainage course (OGDC) over structure backfill to support approach slabs, sleeper slabs and approach curb and gutter.
   (10-22-2012) (12-28-2015)

e. Pavement seats are 9” wide for dependent backwalls, and approach slabs project to the bridge slab over independent backwalls. Avoid cantilevered pavement seats. (1-24-2022)

7.03.01 (continued)

Abutment Design

B. Types

4. Integral and Semi-Integral Abutments

f. (1-24-2022) Approach slabs are 20'-0" in length whenever possible.

Approach slabs 20'-0" in length are based on a longitudinal unsupported length of 10 feet measured along the centerline of the roadway, a slab thickness of 12" and a maximum concrete cover to the centerline of the bottom longitudinal reinforcement of 3". Deviation from these design parameters for specific projects requires a complete redesign of the approach slab.

Approach slabs with independent backwalls can be 6'-6" minimum length. For design speeds greater than 45 mph (posted > 40 mph) approach slabs may be up to 20' in length (measured along roadway centerline) as project and geometric limitations allow. Use shorter approach slab length (6'-6" min) if service road is in close proximity to the bridge abutment. (12-28-2015)

Abutments a with skew angle maintain the same skew angle at the end of the bridge approach slab and at the sleeper slab. Standard Plan R-45-Series reinforced approach pavements are cast perpendicular (90°) to the roadway centerline on the opposite end of the sleeper slab. See Standard Plan R-43-Series. (12-28-2015)

Cast 12" minimum thickness (9" for independent backwalls) bridge approach slab from sleeper slab towards reference lines at night with "Superstructure Conc, Night Casting (,High Performance)" and match the road approach thickness (9" minimum). (9-27-2021

Use a 20' concrete approach pavement as detailed on Standard Plan R-43 & R-45 - Series located on the road approach side of the sleeper slab.(10-22-2012)

7.03.01 B. 4. f. (continued)

Designate approach slabs as separate pours in the pour sequence of the superstructure. (9-21-2015)

See Bridge Design Guide 6.20.03A, .03B, 6.20.04 & .04B for approach slab details. (12-28-2015)

g. Continue bottom mat of deck reinforcement past reference joint into the approach slab with independent backwalls. See Bridge Design Guides 6.20.03A & .03B. For dependent backwalls lap or develop EA bars from deck slab to bridge approach slab. See Bridge Design Guides 6.20.04 & .04B. Provide adequate lap/development according to Bridge Guide 7.14.02 Series. (12-26-2023).

h. Add extra reinforcement over beams at the reference joint that extend into the approach slab and into the bridge deck slab. Lap or develop extra EA bars over beams according to Bridge Guide 7.14.02 Series. (12-26-2023)

i. Attach approach curb and gutter to the approach slab with bottom mat transverse reinforcement and to the bridge deck with bottom mat longitudinal reinforcement. Do not attach curb and gutter to the approach slab or the bridge deck on structures with return wingwalls. Using a bond breaker and sliding the approach slab over the return wingwalls is a design consideration. The extension of bridge railing to the sleeper slab will eliminate the need for curb and gutter in the bridge approach slab area. (1-24-2022)

j. A n inverted "T" sleeper slab shall be used with all approach slabs (except when Standard Plan R-45-Series approach is used by itself). Concrete to concrete slabs shall have an EJ3 (or EJ4) joint on the bridge side of the stub and an E3 joint on the road side. Place R-45-Series reinforced concrete slab on the road side of the inverted "T" sleeper slab. Provide elevations along stub of sleeper slab at construction centerline, lane lines and edge of metal. Provide elevations at toe of curb/barrier and top of curb if present. (1-24-2022)

7.03.01 (continued)

Abutment Design

B. Types

5. Spill – Through Abutment (9-24-2018)

   A spill-through abutment has fill-slope with a revetment on the streamward side.

a. Definitions

   1) Fill slope: side or end slope of an earth-fill embankment. Where a fill slope forms the streamward face of a spill-through abutment, it is regarded as part of the abutment.

   2) Revetment: rigid or flexible material designed and placed to inhibit scour or erosion.

7.03.01 (continued)

b. Design Considerations

   1) The dimensions and elevations of the revetment must be as defined by the Hydraulics Unit.

   2) If the Hydraulics Unit determines there is a high probability of the river laterally migrating over time, consideration shall be given to:

      a) Design the span lengths and substructure locations to accommodate the future path of the river.

      and/or

      b) Resist migration with stream armoring and/or design the abutment to remain stable at the 500-year flood event after stream migration has occurred.

   3) Additional ROW may be necessary to provide sufficient revetment, as defined by the Hydraulics Unit.

A multidisciplinary team consisting of the Hydraulics Engineer, Geotechnical Engineer and the Structural Engineer (Bridge Designer) should convene to determine the best design option when stream meandering is likely.

See Bridge Design Guide 5.47.01 for details and MDOT Drainage Manual section 6.4.5.6. for additional design criteria.

7.03.01 (continued)

Abutment Design

B. Types

6. Geosynthetic Reinforced Soil - Integral Bridge System (GRS-IBS) Abutment (10-28-2024)

GRS-IBS abutments may be considered on projects meeting the following criteria:

a. The anticipated foundation is a shallow foundation (spread footing).

b. Traffic over the bridge will be detoured during the construction of the project or the bridge is being constructed on a new alignment.

c. The proposed bridge is a single span or two span structure. Using GRS-IBS abutments on bridges with more than two spans is not desired currently due to a lack of national experience and history with these span configurations.

d. There are no existing or proposed utilities that will be attached to the proposed structure or located within the reinforced soil zone behind the proposed abutments. This includes storm sewers, traffic signals, and lighting.

e. The anticipated traffic growth is not expected to necessitate future widening of the bridge. If the need for future widening is likely the final full width substructure should be constructed as part of the project.

f. The skew angle of the substructure is 30 degrees (30°) or less. Zero-degree (0°) skew angles are preferred, if possible.

7.03.01 (continued)

GRS-IBS abutments at bridges over water must be designed to account for the estimated scour in accordance with the MDOT Drainage Manual.

Back to back GRS-IBS abutments may be considered in the median between two bounds of traffic and may be beneficial to minimize differential settlement between the structures over each bound. When considered, wingwalls should generally be used in lieu of returnwalls due to constructability and to provide Contractor access to build the abutments. Back to back GRS-IBS abutments should not be considered in waterways.

When the criteria listed above are met a more detailed discussion with the Geotechnical Engineer should take place to confirm that GRS-IBS abutments are a feasible solution. Generally, this discussion should take place during the development of the Structure Study.

GRS-IBS piers will not be considered at this time.

7.03.01 (continued)

Abutment Design

C. Wall Design

1. Wall Thickness

   The minimum wall thickness for abutments is 2'-0". This is to be increased in 2" increments when required to provide 4½" minimum clearance between edge of masonry plate (or elastomeric pad) and front face of the abutment. (8-6-92)

2. Cantilever Wall Design

   Cantilever walls 16'-0" or higher are to be designed for both bending and direct stress.

3. Steel Reinforcement

   All wall reinforcement shall be epoxy coated. Horizontal bars in the front faces of abutment walls should be continued around the corners at the wingwalls. EC#6 bars are to be placed diagonally across the inside corners. (5-6-99)

4. Vertical Construction Joints

   a. There is to be vertical continuity of all construction joints from the footing upward; however, a wall joint does not require a footing joint below.

   b. Spacing

      (1) Curtain wall abutments - 35'-0" maximum spacing.

      (2) Cantilever abutments - 25'-0" maximum spacing.

5. Horizontal Construction Joints

   For walls over 30'-0" high, there should be a horizontal construction joint approximately at mid-height. (9-2-2003)

7.03.01 (continued)

6. Vertical Expansion Joints

   Vertical expansion joints shall be spaced approximately 90'-0" apart. There should be a construction joint in the footing directly below each expansion joint in the wall.

7. Bridge Seat Steps

   Where the bridge seat is stepped, the ends of the steps shall be at 45° to the bridge seat and parallel to the centerline of the bridge to accommodate any movement due to temperature changes. (5-6-99)

8. Pavement Seats

   Pavement seats are to be provided on all bridges. They should be cantilevered from the rear face of independent backwalls. (5-6-99)

7.03.02

Footing Design

A. Footing Thickness

The minimum thickness of footings is normally 2'-6"; however, this may be reduced to 2'-0" for short walls. When the wall thickness at its base becomes 3'-0" or greater, the footing thickness is to be increased to 3'-0. Footing thicknesses are to be increased in 6" increments.

B. Footing Width (12-5-2005)

Spread footings should be sized so that the safety factor for overturning about the toe of footing is at least 2.0. The minimum footing width is 6'-0" for cantilever abutments and 4'-0" for curtainwall abutments. Footings with piles should be sized so that the resultant force is located between rows of piles.

C. Footing Joints

Construction joints should be placed in footings to limit concrete pours to 90 CYD. These joints are provided for construction convenience and should be labeled "optional." Where a footing joint is used, it should be located directly under a wall joint.

D. Footing Elevations

Bottoms of footings are normally set 4'-0" below existing or proposed ground line to avoid frost heave. For substructure units in or adjacent to a waterway, bottoms of footings are normally to be set 4'-0" below bottom of channel; where tremie seals are used, the bottoms of footings may be set higher. For substructure units in or adjacent to a waterway, The Hydraulics/Hydrology Unit shall be consulted for an estimate of the total potential scour depths at the foundations. The tops of footings shall be set at or below the estimated elevation of contraction scour (scour resulting from the constriction of the waterway at the crossing). Structure stability shall be analyzed based on the estimate of total scour at each substructure unit with the advice of geotechnical engineers. If necessary, countermeasures to prevent scour will be incorporated according to FHWA and AASHTO standards. (5-6-99)

7.03.02 (continued)

E. Steel Reinforcement

Where a tremie seal is used and there are no piles, the bottom footing reinforcement shall be 9" above bottom of footing. Where a tremie seal is used and there are piles, the reinforcement shall be 1'-3" above the bottom of footing.

F. Passive Soil Pressure

1. Passive soil pressure may be used in the footing design for retaining, wing and return walls, but not for abutments, to resist sliding and overturning forces. Generally, these resisting forces shall be relied upon only when the footing is in a cut and the soil is not disturbed. The location of utility trenches and edge drains should be considered in making a determination of undisturbed soil. In a river environment, passive soil pressure shall not be used.

   Use resistance factors for sliding of spread footings as defined in AASHTO LRFD Table 10.5.5.2.2.-1. Use resistance factor of 0.80 for cast in place concrete on sand and use (where allowed) resistance factor of 0.50 for passive earth pressure component of sliding resistance.
   (8-20-2009)

2. When the passive soil is on a slope, the soil height shall be reduced as follows:

   a. Berm with 1V:2H slope - reduced 1'-0"

   b. 1V:2H slope - reduced 2'-0"

7.03.02 (continued)

Footing Design

G. Bearing Resistance – Spread Footings (8-20-2009)

1. Geotechnical Engineer shall provide:

   a. Nominal Bearing Resistance (q_n)

      1) For foundations on rock, a single value of nominal bearing resistance (q_n) will be provided for all footing widths.

      2) For foundations on soil, nominal bearing resistance (q_n) will be provided graphically, by plotting nominal bearing resistance (q_n) versus effective footing width (B').

   b. Strength limit state resistance factor for bearing resistance (φ_b) and sliding resistance (φ_τ). Refer to AASHTO LRFD Table 10.5.5.2.2-1.

   c. Service limit state resistance factors shall be taken as 1.0, except as provided for overall stability.

2. Foundation recommendation memo/report investigates nominal bearing resistance (q_n) based on:

   a. Bearing failure – Strength Limit State

   b. Tolerable settlement criteria – Service Limit State (1.5" max settlement recommended by MDOT)

7.03.03

Pier Design

A. Future Widening

On bridges where we are to provide for future widening, a vertical construction joint, as shown in Bridge Design Guide 5.27.03, is to be provided in the pier cap.

B. Column

1. Size

   In general, 3'-0" diameter columns should be used. Columns with a diameter of less than 3’-0” may be used, when necessary, but the height of the base wall must be increased in accordance with MDOT Bridge Design Guide 5.22.01 to provide additional protection in the event the pier is struck by a heavy vehicle. Column diameters less than 2’-6” are not permitted. (7-24-2023)

2. Reinforcement

   Care should be used in spacing vertical column bars in order to avoid excessive interference with the pier cap reinforcement. Double rows of column bars or larger diameter columns should be considered to alleviate this problem.

3. Construction Joint

   If pier columns are over 30'-0" high, a construction joint should be placed at approximately mid-height.

4. Spacing

   Columns should be spaced far enough apart so as to be appealing to the eye; if beam spacing is far enough apart, a column may be placed under each bearing. (7-24-2023)

7.03.03 (continued)

Pier Design

C. Pier Caps

Pier caps meeting the requirements outlined below shall be included in the design of all multi column piers. (7-24-2023)

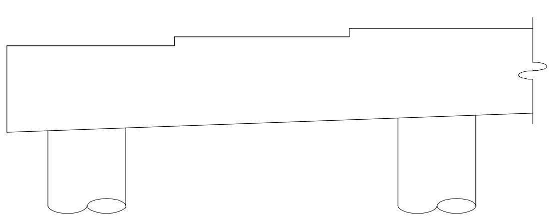
1. Size

   The pier cap is to be approximately 3" wider than the diameter of the column and should provide 4½" minimum clearance between the edge of masonry plate (or elastomeric pad) and the face of the cap.

   Hammer head pier caps are occasionally used on MDOT projects. These piers have a greater tendency for cracking in the tension zone than standard pier caps. Design procedures to prevent cracking (especially in tension zone), including post tensioning the caps, must be investigated. (9-2-2003)

2. Bolsters

   When one end of the pier cap is on a considerably different elevation than the other, the difference shall be provided for by increasing the column heights as shown below.



   Ends of bolsters are perpendicular to the faces of the cap and rise at 90° from the top of the pier.

3. Joints

   Construction joints should be provided at 25’-0" maximum spacing. A 1" open joint may be required to control temperature moments in long piers with short columns.

7.03.03 (continued)

4. Reinforcement Steel Spacing

   In order to permit the vibrator to adequately penetrate and vibrate the concrete in pier caps, the clear distance between the top bars should not be less than 3½”. This may, in some cases require the use of special size bars or double rows of bars.

5. Part Width Construction of Cantilevered Pier Caps (12-5-2005)

   To reduce potential problems with large pier cap cantilevers during construction base design on the following criteria:

   a. Avoid splicing reinforcement at points of maximum stress. Where this is not practical, stagger the splices.

   b. Calculate the clear distance between contact lap splices assuming the bars are placed in a horizontal plane unless otherwise noted on the plans.

   c. Use temporary supports during staged construction to shore cantilevered pier caps exceeding five feet in length.

   d. Design structural elements using a dead load factor of 1.5 if live loads (unanticipated construction loads) are not applied to elements.

D. Pier Base Walls

Account for the AASHTO LRFD vehicle collision force in the design of all new bridges, bridge replacements, and pier replacements. Design piers that are within the clear zone to account for the AASHTO LRFD vehicle collision force as outlined in Section 7.01.04.K. The base wall is to be 3" wider than the column to prevent vehicle snagging and should extend 5’-0” (min.) above the ground line. Any approach guardrail is to be anchored to the base wall according to Standard Plan R-67-Series. (7-24-2023)

7.03.04

Cofferdams (8-6-92)

Cofferdams shall be used on all substructure units where tremie concrete is required for water control. When shallow water is present; i.e., less than 2'-0", other methods of water control that allow the contractor maximum flexibility may be appropriate. The Geotechnical Services Section should be contacted in this case to determine if a cofferdam is required. (2-26-2018)

The driving line for cofferdam sheet piling shall be 1'-6" outside the footing outline or at the edge of the tremie concrete. Deep excavations may use driving line greater than 1'-6" outside the footing outline to allow for more efficient bracing schemes. Consult with Geotechnical Services Section.
(8-20-2009)

Since a cofferdam is generally a sheeted enclosure, the plans should show and note the limits of the enclosure. The contractor must know if he will be required to completely enclose the excavation or whether sheeting on three sides will suffice.

Often, a portion of a sheet pile cofferdam is to remain in place. On these projects, there will be two bid items. "Steel Sheet Piling, Temp, Left in Place" will be measured and paid for in the specified manner. The remainder of the enclosure along with dewatering, etc., will be paid for as "Cofferdams." This division of pay items should be clarified by a plan note.
(9-27-2021)When cofferdams are not used on structures crossing streams or encroaching on water courses, Plan Note 8.05 Q shall be used. (8-23-2021)

Where a sheet piling enclosure is required for lateral soil support but not for the exclusion of water, "Steel Sheet Piling, Temporary" should be called for.

For additional information see Subsection 7.01.10.

7.03.05

Subfootings

Use subfootings under footings for all substructure units regardless of their location unless a differing means to support forming, reinforcement and concrete during placement is specified by the geotechnical engineer. Extend subfootings 1'-3" outside of footing lines and in general provide a thickness of 3½"; subfootings may be 5½" thick where water and/or soil conditions are such that unsuitable conditions might arise. Maintain foundation excavation limits of 1'-6" outside of footings in all cases. Concrete for subfootings is to be bid separately as "Conc, Grade 3500, Subfooting" and has the material properties of Concrete, Grade 3500.
(6-24-2024)

7.03.06

Tremie Seal Design

Generally, tremie seals should be called for on all structures where it is expected that difficulty will be encountered in pumping the water down below the bottom of footing. Do not include weight of tremie when computing pile loads except when the estimated scour depth is below the bottom of tremie. (5-6-99)

A. Design

The tremie seal shall be designed to resist the hydrostatic pressure at the bottom of the tremie by a combination of its weight, plus the bond on the cofferdam and piles. The allowable bond stress is 10 psi on the piles and 5 psi on the cofferdam, providing the piles and the sheeting have sufficient resistance from dead weight and soil friction to resist the load thereby induced. Where shells are used or permitted as an option, the total resistance available will be the weight of the shell plus soil friction less any buoyancy force exerted on the shell. Allowable tension in bending on the tremie seal is 30 psi.

B. Hydrostatic Head

Hydrostatic head should be figured from bottom of tremie seal to ordinary water surface elevation. Include note 8.05 R. on plans. (8-23-2021) (5-6-1999)

7.03.07

Excavation

All foundation excavation is to be "Excavation, Fdn" unless there is a considerable amount of rock excavation involved; in this case, excavation is to be divided into two bid items: "Excavation, Rock Fdn" and "Excavation, Fdn." (9-27-2021)

Unbraced excavations adjacent to in-service spread footings shall not be permitted. Earth retention designs shall be sealed by a licensed engineer. (11-28-2011)

7.03.08

Steel Sheet Piling

For additional information see Subsection 7.01.10.

Evaluate the potential for vibration induced damage to existing structures and utilities.
(11-28-2011)

A. Driving Line

1. Temporary Steel Sheet Piling

   The driving line for temporary steel sheet piling is 1'-6" outside the footing outline or at the edge of the tremie seal.

2. Permanent Steel Sheet Piling

   The inside face of permanent steel sheet piling is to be along the footing outline. Allowance for additional concrete and excavation is to be made due to the structural shape of the sheet piling.

B. Lateral Limits

Lateral limits of open-ended permanent sheeting must be extended beyond the limits of the required excavation. For estimating this extension, use a 1V:1H slope from bottom of excavation to existing ground.

C. Temporary Steel Sheet Piling Left in Place

On some projects requiring temporary sheeting, it is specified that the sheeting be left in place. The sheeting is not required for permanent support, but disturbance caused by its removal could be damaging. The bid item “Steel Sheet Piling, Temp, Left in Place” is used in these instances.
(5-6-99) (9-27-2021)

In general, sheeting at stage lines that is adjacent to permanent backfills should be specified as left-in-place and cut off to approximately 3' below the final pavement grade. If sheeting must be removed, contact the Geotechnical Services Section to determine feasibility. (2-26-2018)

7.03.08 (continued)

D. Permanent Steel Sheet Piling (10-24-2001)

1. Design

A required section modulus is calculated based upon a piling design. (MDOT Geotechnical Services Section may recommend a section modulus.) A section is chosen from Appendix 7.03.08 D. (Sheet Piling Section Moduli) using the tabulated "effective" modulus in place of the calculated section modulus.
(2-26-2018)

Cold rolled sections have an additional reduction factor, thus it is possible to have a cold rolled section with a higher nominal section modulus, but a lower effective section modulus. To avoid field substitutions resulting in less than designed "effective" section modulus, the plans shall indicate the minimum acceptable nominal section modulus for both hot and cold rolled sections based on values given in Appendix 7.03.08 D. (see note 8.06.06 C.). (8-23-2021)

In addition to Appendix 7.03.08 D., which is to be used for all permanent installations, sheet piling sections subject to severe environments should also be hot dipped galvanized.

Designers are responsible to determine the domestic production and availability of the sheet piling sections they specify.
(2-26-2018)

7.03.08 (continued)

2. Background/Commentary

Appendix 7.03.08 D. was developed by the Illinois DOT. It contains sheet pile sections and their effective section modus. This effective modulus was calculated by reducing the nominal value for the effects of corrosion, and in some cases for a Hartman reduction factor.

Hartman Reduction Factor - tests by Hartman Engineering indicate that cold rolled sections failed at 83% of the expected value based on conventional bending theory. The Hartman study concluded that these failures were because the cold rolled sections have larger widths, depths, and width to depth ratios which promote failure prior to yielding the tension flanges. Cold rolled sections shown on the table have their section modulus reduced by 17% to account for the lower yield values. Illinois DOT took the report's conclusion a further step and applied the Hartman reduction factor to "light duty" hot rolled sections also.

Corrosion - all tabulated sections were reduced to mitigate the effects of corrosion. Illinois DOT assumed a 50 year service life and a corrosion of about 0.00059 inches per year. This translates to about 1/17" of total corrosion (two sides) for the service life.

MDOT requires a 75 year service life and a slightly higher corrosion rate, thus the requirement for hot dipped galvanized sections in severe environments.

7.03.09

Piles

A. General

1. River and Stream Crossing

   Both vertical and battered piles should be used under abutments where footings are higher than the river stream bed. The abutments must be protected from scour. The appropriate riprap design as specified by the Design Engineer - Hydraulics/Hydrology will be provided by special provision. A well graded riprap will be provided for foundations subject to pressure flow or velocities greater than 7 feet per second. Piers must have the top of the footing or pile cap below the stream bed a depth equal to the estimated contraction scour depth. (5-6-99)

2. Biaxial Bending and Compression of Piles (8-20-2009)

   The combined biaxial bending and compression of piles shall be checked and analyzed for integral abutments and all river structures according to current AASHTO LRFD Bridge Design Specifications 2.6.4, 6.9 & 10.7.

7.03.09 (continued)

3. Economic Analysis to Determine Nominal Pile Driving Resistance (R_{ndr})(8-20-2009)

   For driven pile, an economic analysis of the foundation support system shall be completed optimizing pile type, pile section and construction quality control method pertinent to the particular project in question. The Resistance Factor for Driven Piles (φ_{dyn}) used in design determines the construction quality control method that must be used to certify the Nominal Pile Driving Resistance (R_{ndr}). Do not specify dynamic testing with signal testing (P.D.A. testing) for H-piles driven in non-cohesive soil profiles where the driven pile length is expected to exceed 80 feet. Use AASHTO LRFD Tables 10.5.5.2.3 - 1, 2 & 3 in analysis and resistance factor determination and coordinate findings with Geotechnical Services Section. For additional information on pile resistance see section 7.03.09 B. (6-27-2022)

   General rules for Resistance Factor (φ_{dyn}) (detailed analysis shall be performed):

Project Driven Pile Cost	Pile Certification Method	Resistance Factor(φ_{dyn})
<$300,000	FHWA-Modified Gates Formula	0.50
≥$300,000	Dynamic Testing/ Signal Matching (PDA Testing)	0.65 *
>$500,000	Static Load Test with Dynamic Testing/ Signal Matching (PDA Testing)	0.80

* This resistance factor applies to the Beginning of Redrive (BOR) case. Do not specify PDA testing for End of Drive (EOD). (11-28-2011) (9-21-2015)

7.03.09 (continued)

Piles

A. General

4. Test Piles

   Test piles are to be provided for all projects using piles unless the Geotechnical Services Section determine that they are not necessary. (2-26-2018)

   At least two test piles shall be provided for at each substructure unit placed on piles. Test piles are to be vertical piles unless otherwise approved by the Geotechnical Services Section. (6-27-2022)

   Timber test piles shall be located in a manner that will best serve as a basis for ordering the balance of the piles.

5. Pile Embedment

   Piles are to be extended into the footing a distance of 6". When a tremie seal is used, the piles are to be extended into the footing a distance of 1'-0".

6. Concrete Displaced by Piles

   No deductions in concrete quantities will be made for steel pile embedments or for pipe pile embedments of 1'-0" or less.

7. Edge Distance

   The usual minimum edge distance for piles is 1'-6". This may be reduced to 1'-3" where special conditions require.

8. Abutment Piling

   When piling is required for abutments, a careful study should be made to ensure that the piling will clear previously placed or proposed culvert pipe.

7.03.09 (continued)

9. Pile Batter

   Generally, piles are to be battered no flatter than 3V:1H. Where soil conditions are not good enough to provide sufficient lateral pile resistance, we may increase the angle of batter to 2.5V:1H or even 2V:1H. This measure, however, should be a last resort since it is difficult to maintain driving accuracy when the batter is flatter than 3V:1H

   For CIP piles, do not batter flatter than 3V:1H. (1-27-2025)

10. Pile Numbering

    A pile numbering scheme shall be shown on the plans for those units having piles. Each pile shall be assigned a number in a particular row or on an individual basis.

11. Lateral Pile Resistance (8-20-2009)

    Lateral pile resistance as determined by a Geotechnical Engineer may be used to resist horizontal forces on substructure. See AASHTO LRFD Bridge Design Specification 10.7.

    Scour potential for the structure shall be accounted for when the Geotechnical Engineer determines nominal horizontal pile resistance.

12. Pile Driving Vibration Evaluation (11-28-2011)

    Driven piles located within a distance of 100 ft of historic or vibration sensitive structures shall be evaluated for damage potential from vibration and/or vibration induced settlement.

    Driven piles shall not be located within a 25 ft radius of existing spread footings, critical utilities, or in-service pavements without mitigation and/or vibration and settlement monitoring specifications.

13. Loads Applied to Piles

    Piles are to be designed for compression, shear, and/or moment loads only. Piles are not to be designed for tension loads unless approved by the MDOT Geotechnical Services Section. (10-28-2019)

7.03.09 (continued)

Piles

B. Nominal Pile Resistance (R_n)
(8-20-2009)

Design substructures with an initial nominal pile resistance of 500 kips. In some cases, the soil profile will indicate that a higher or lower nominal pile resistance would be more economical. The recommendation from the Geotechnical Services Section will indicate what nominal pile resistance to use. (6-27-2022)

1. Pile Designation/Maximum Nominal Pile Driving Resistance (R_{ndr})

   a. Steel H Piles (11-28-2011)

Pile	**(R_{ndr})**
HP 10X42	275 kips
HP 10X57	350 kips
HP 12X53	350 kips
HP 12X74	500 kips
HP 12X84	600 kips
HP 14X73	500 kips
HP 14X89	600 kips

   b. Metal Shell Piles (5-24-2021)

Pile	**(R_{ndr})**
Metal Shell 12" O.D. w/0.312" Walls	250 kips
Metal Shell 14" O.D. w/0.312" Walls	350 kips
Metal Shell 16" O.D. w/0.375" Walls	400 kips
Metal Shell 16" O.D. w/0.500" Walls	500 kips

   c. Timber Piles

Pile	**(R_{ndr})**
Timber Pile	150 kips

7.03.09 B. (continued)

A wave equation analysis, which uses typical pile types and driving equipment known to be locally available, shall be performed by the Geotechnical Engineer to verify drivability. (11-28-2011)

Use steel H-Piles meeting the requirements of AASHTO M270 Grade 50. Use metal shell piles for CIP piles meeting the requirements of ASTM A252 Grade 3 (45 ksi) or Grade 3 Modified (50 ksi). (5-24-2021)

2. In general, the Resistance Factor for Driven Piles (φ_{dyn}) = 0.50 assuming that the Nominal Pile Driving Resistance (R_{ndr}) is verified using the FHWA-modified Gates Dynamic Formula. The Resistance Factor (φ_{dyn}) = 0.65 when dynamic testing with signal matching (P.D.A. testing) is used and (φ_{dyn}) = 0.80 with static load tests. (See AASHTO LRFD Table 10.5.5.2.3-1 Resistance Factors for Driven Piles) (11-28-2011) (11-23-2015)

   For piles with a R_{ndr} between 150 kips and less than 250 kips verified using the FHWA-modified Gates Dynamic Formula, use a resistance factor (φ_{dyn}) = 0.40. (6-27-2022)

   For piles with a R_{ndr} less than 150 kips verified using the FHWA-modified Gates Dynamic Formula, use a resistance factor (φ_{dyn}) = 0.25. (6-27-2022)

3. In general, Resistance Factor (φ_{dyn}) times the Nominal Pile Resistance (R_n) = Factored Nominal Resistance (R_R).

   $(\varphi_{dyn}) \times (R_n) = (R_R)$

   The above equation does not hold true in the case of possible downdrag, and/or scour.

4. The nominal pile resistance to be shown on the plans should be equal to the actual demand, based on the final pile layout, divided by the appropriate Resistance Factor for Driven Piles (φ_{dyn}), rounded up to the nearest 10 kips. Do not simply use the Maximum Nominal Pile Driving Resistance (R_{ndr}) for the pile type. (2-26-2018)

7.03.09 (continued)

Piles

C. Pile Quantities

1. Cast-in-Place Concrete Piles

The following items shall be shown on the plans:

a. Length of each pile - Furnished and Driven.*
b. Total length of piles - Furnished and Driven.
c. Test piles - Each (Furnished and Driven length plus 10').*
d. Number of pile points - Each. (Use when a special pile point is required.)
e. Furnishing equipment for driving piles - Lump Sum.

*Length to the nearest 5'. (5-6-1999)

If a maximum pile penetration elevation is shown on the plans do not call for pile lengths extending beyond the maximum pile penetration elevation. (6-27-2022)

2. Steel H Piles

Use the same items as cast-in-place concrete piles except exclude pile points.

3. Piles of Designated Nominal Pile Resistance

Use the same items as cast-in-place concrete piles except exclude pile points and pile splices.

7.03.10

Slope Treatment Under End Spans

A. Type

1. New Bridges

On all new grade separations, "Slope Paving, Conc" is to be placed under the end spans on the berm and backslope to the bottom of ditch.
(5-6-1999) (9-27-2021)

2. Widening Projects

On widening projects, match existing slope protection if the material is reasonably available.

If pier widening is located within the clear zone, follow the requirements outlined in Section 7.01.04 K. (7-24-2023)

3. Stream or River Bridges (5-6-1999)

The Hydraulics/Hydrology Unit will specify riprap to be used as a scour countermeasure. A special provision for well-graded riprap for foundations shall be included in the proposals of projects where there is either pressure flow or velocities exceeding 7 feet per second. See Subsection 8.05 for hydraulic analysis and design guides for approved methods of stream diversion.

B. Dual Structures

For dual structures on a common abutment, call for slope protection on the slope and berm between the structures.

C. Limits

The slope protection is to be extended 1'-6" beyond the slab fascias or for structures with turnback wingwalls, it should extend to outside face of the wingwalls.

Generally, riprap is to be placed on all disturbed slopes to an elevation of 2'-0" above extreme high water. Under the deck riprap shall extend to the face of the abutment.

7.03.11

Concrete Sealers (5-1-2000)

When substructure units are new or patched; the entire surface of the substructure unit shall be coated (sealed) to prevent deterioration.

The following materials are used as sealers or waterproofing agents:

A. Elastomeric Surface Coating(6-27-2022)

These materials are a rubberized coating. They create a uniform color and texture making them a good aesthetic treatment. Use Elastomeric coatings on all substructure surfaces where aesthetics are important. (Where aesthetics are an issue, consult the Roadside Development area for coloring considerations.) Use elastomeric coatings on patching projects to mask the mottled look of the patching.

B. Penetrating Waterproofing Sealers

Clear sealers with the consistency of water. Provide sufficient protection for vertical surfaces of substructure units but offer no aesthetic value. Use to seal substructure units where aesthetics are not important. Use on top surfaces only where the substructure unit is not under an expansion joint.

C. Epoxy Sealers

Opaque sealers offer a (nearly) impenetrable barrier. Use epoxy sealers to coat the top horizontal surface of pier caps and abutment bridge seats under expansion joints. (All top surfaces should be considered, even those not under joints.) This material should not be used to encapsulate the entire substructure unit as it does not "breathe" and can cause concrete degradation in such instances.

7.03.11 (continued)

D. Silane Coating (6-27-2022)

A sprayed-on film/coating for the inhibition of water ingress into the pores of concrete. Silane coating is an effective surface treatment method to extend the durability of a concrete elements. Generally, barriers, deck slab fascias, piers, abutments, retaining walls or beams. Do not specify its use on traffic surfaces or with other coatings/sealers. Use when requested by Region Bridge Engineer. Include Special Provision for Silane Treatment for Bridge Concrete.

It is advisable to erect beams prior to coating horizontal surfaces. Areas underneath bridge bearings shall not be coated with elastomeric or epoxy sealers. Coating under bridge bearings with penetrating waterproofing sealers is allowed. (9-2-2003)

7.03.12

Mechanically Stabilized Earth (MSE) Wall Requirements (8-20-2009)

Design, construction and other considerations related to permanent and temporary MSE walls shall be according to Load and Resistance Factor Design (LRFD) method as defined by AASHTO and MDOT. (2-26-2018)

A. Wall Design Criteria:

1. The bridge designer and geotechnical engineer are responsible for providing the MSE fabricator with the following information:

    a. Factored bearing resistance at the base of the reinforced soil mass.

    b. Vertical dead and live loads, horizontal loads, and factored bearing pressure applied to the reinforced soil mass from the bridge or other structures/appurtenances. (6-27-2022)

2. The geotechnical engineer is responsible for performing a global stability analysis, estimating the factored bearing pressure, calculating factored bearing resistance, settlement analysis, checking sliding stability and overturning. Global stability must be checked for all stages of construction, including for temporary MSE walls that are utilized to permit part-width construction operations. (2-26-2018)

3. In addition, the engineer shall incorporate all design aspects of the special provision for MSE Retaining Wall System in the design for the MSE walls.

7.03.12 (continued)

Mechanically Stabilized Earth (MSE) Wall Requirements

B. Wall Configuration:

1. The preference of wall geometry at bridges is as follows:

    a. Straight walls, in line with the abutment wall.
    b. Walls turned back at 45 degrees, or turned back with a large radius.
    c. Walls turned back 46 to 90 degrees.
    d. Acute angles should not be used.

2. The use of complex geometries such as tiered walls or back-to-back walls must be approved by MDOT's Geotechnical Services Section. For back to back MSE walls the base width, (W_b) distance between walls, divided by the height of the taller wall (H_1) shall be greater than or equal to 1.1 ($W_b/H_1 \geq 1.1$). (11-28-2011)

7.03.12 (continued)

Mechanically Stabilized Earth (MSE) Wall Requirements

C. Bridge Abutments at MSE Walls

1. Pile supported abutments are required in most cases.

   a. Maximize pile spacing to reduce interference with soil reinforcement.

   b. Incorporate/consider pile bending in design (loose soil vs. stiff soil).

   c. Use pile liner/pile protection to eliminate downdrag between MSE wall backfill and abutment pile as well as to allow slightly more lateral movement in the pile. (6-27-2022)

2. Spread footings may be allowed if either of the following conditions are met:

   a. The MSE wall is on bedrock.

   b. The bridge is single-span, not constructed part-width, and spread footings are recommended by the Geotechnical Section.

3. Embed footings 1’-6” below the top of coping to allow a minimum of 6” clearance above the top of soil reinforcement. Four foot (4’) minimum embedment is decreased due to free draining ability of Backfill, Select material required behind MSE walls.

4. The use of sliding slab abutments (BDG 6.20.03A) and integral/semi-integral abutments with a sleeper slab closer than 20’ to the abutment (BDG 6.20.04 series) is allowed with a 20’ concrete approach pavement as detailed on Standard Plan R - 43&45 - Series located on the road approach side of the sleeper slab.
(11-28-2011) (2-26-2018)

7.03.12 (continued)

D. Abutment Footing Clearances and Setbacks

1. The edge of pile supported footings shall be located with a minimum clearance of 2 feet from the back face of the MSE facing panels.

2. The edge of spread footings shall be located with a minimum clearance of 5 feet from the back face of the MSE facing panels.

3. The centerline of the front row of piles shall be setback a minimum of 4.5 feet from the back face of the MSE facing panels. (6-27-2022)

E. Soil reinforcement length requirements

1. Soil reinforcement length is determined by design, but shall not be less than 0.7 times the wall height (H), or 8 feet whichever is greater.

2. The wall height (H) is to be measured from the proposed finished grade where it intersects the back of the wall face, to the top of the leveling pad.

3. For walls supporting a sloping surcharge, the value H1 shall be substituted for H in the above minimum requirements, where
$H1 = H + (\tan \beta \times 0.3H)/(1-0.3\tan \beta)$
β = angle of backslope

4. For walls with abutments within 0.5 times the wall height, the height (H’) of wall shall be measured from finished roadway surface to the top of the leveling pad. The value H’ shall be substituted for H in the above minimum requirements.
(12-22-2011)

5. For any section of MSE wall, the soil reinforcement will be the same length from top to bottom.

6. Attaching soil reinforcement to substructure as a means to provide horizontal resistance/anchorage is not allowed.

7.03.12 (continued)

Mechanically Stabilized Earth (MSE) Wall Requirements

F. Drainage around MSE walls

Consideration shall be given to drainage at design phase to decrease the possibility of issues at MSE wall construction.

1. A 30 mil thick PVC Liner (impervious membrane) is required between the roadbed and the soil reinforcement. It should be located a minimum of 8 inches above the soil reinforcement. The liner should extend 6 feet beyond the ends of the soil reinforcement. Extend liner transversely to 8 inches from slope line or return wall (if present). Place an underdrain 1 foot from the end of the PVC liner running transverse to the road and 1 foot from each end of liner running longitudinally along the roadway. Connect the underdrains and dispense drain 3 feet minimum from any soil reinforcement. (12-19-2016)

2. Foundation underdrains should be used, and located as low as possible to provide positive flow.

3. Curb and gutter at the edge of the roadway with a catch basin should be used to collect the drainage. Locate the catch basin a minimum of 25 feet past the end of the MSE wall reinforcement, if possible. The curb and gutter should continue 10 feet past the catch basin.

   Other means to collect and dispense water may need to be used if the 4" maximum curb height on Standard Plan R-32 Series is insufficient. (11-25-2019)

   See Section 7.07.02 for additional information

4. Use a minimum 20' concrete approach slabs (to reduce voids under approaches) for structures with MSE walls at the abutments. This includes sliding slab approaches (Bridge Design Guide 6.20.03A). (11-23-2015)

7.03.12 (continued)

G. Utilities and MSE Walls

1. Avoid utilities through or underneath MSE walls.
2. If utilities cannot be avoided, encase the utility in a protective conduit that extends 10 feet beyond the limits of the Backfill, Select.

3. Pipe culverts through MSE walls should be avoided.

4. Water and sewer lines within 10 feet of an MSE wall should be encased.

5. Do not place foundations for other structures/appurtenances in the reinforced soil zone unless approved by the Geotechnical Services Section. (6-27-2022)

6. Avoid utilities, pipe culverts, water and sewer lines parallel with the MSE wall and within the 1:1 load bearing area of the reinforced soil zone. If this cannot be avoided, then provide separation between the utility and the MSE wall with permanent steel sheet piling. Design the permanent steel sheet piling to support the MSE wall for the scenario when the utility needs to be installed with an open excavation. (6-27-2022)

H. Leveling Pad Dimensions:

1. Minimum length is 10 ft

2. Maximum height change for each step is 3 ft or ½ panel height

7.03.12 (continued)

Mechanically Stabilized Earth (MSE) Wall Requirements

I. Miscellaneous Requirements

1. Obstructions, such as footing piles, utilities, catch basins, etc. need to be shown on the plan, elevation, and section drawings for the MSE walls.

2. The limits of the Backfill, Select should extend 1 foot beyond the end of the straps at the bottom of the wall, and slope upward at a 45 degree angle.

3. The Plans should clearly identify the MSE wall horizontal alignment, top of coping elevations, proposed ground line in front of wall, limits of concrete surface coating, texturing notes, design height (H), PVC liner, foundation underdrains, areas where cast-in-place coping is required, moment slab/barrier details, utilities, appurtenances, obstructions to the soil reinforcement and notes from BDM Chapter 8.

4. On return walls, keep the barrier inside of the MSE wall, not on top.

5. The water table must be considered by the geotechnical engineer during his/her investigation. Fluctuations in the water table must be accounted for in the investigation and must also be specified on the Plans (i.e. 100 year flood even should be labeled on the plans).

6. Terminate woven wire fence (Standard Plan R-102-Series) and high tensile eight wire fence (Standard Plan R-97-Series) against the side of the wall opposite the stabilized earth, PVC liner and the soil reinforcement. Do not drive fence posts in mechanically stabilized earth. Detail fence termination on plans. (1-27-2025)

7.03.13

Drilled Shafts (3-26-2018)

Due to relative ease of construction and economy, driven piles are generally preferred for most deep bridge foundations. For unique structures with high vertical or lateral loads, limited footprint, or at sites with deep scour, shallow rock or hardpan, or where foundations are to be built adjacent to vibration sensitive structures, drilled shaft foundations may be appropriate. Drilled shafts are most ideal for sites where short, permanently cased shafts can be socketed into rock or hard pan.

Feasibility of drilled shafts for bridge foundations is subject to the approval of the MDOT Foundation Analysis Engineer. Guidelines used for feasibility evaluation follow:

A. Avoid use of drilled shafts if soil boring logs indicate the presence of gas pockets, artesian/confined aquifers, or nested cobbles/boulders.

B. Shafts up to 50 feet in length, bearing in hard pan or rock, are acceptable. Longer shafts are difficult to case and should be avoided with one possible exception; that being sites where deep lacustrine clay overlies hardpan or rock.

C. Due to increased construction risk, avoid uncased shafts in the drift. Permanent casing, sealed into the competent strata below the drift, is the preferred construction scenario. Temporary cased or uncased designs will be evaluated based on the merits of the site and typical contractor tooling.

7.03.13 (continued)

D. In the absence of a site specific load test program, shafts must be sized such that the full factored geotechnical resistance, is derived from either shaft resistance or end resistance.

1. Friction shafts must develop the full factored vertical side resistance in hardpan and/or rock.

2. End-bearing shafts must be sized such that the full factored vertical resistance is derived from end- bearing on rock or hard pan.

E. Belled drilled shafts are prohibited.

F. Drilled shafts in gas bearing formations are prohibited.

G. Drilled shafts are to be designed for compression, shear, torsion, and/or moment loads only. Drilled shafts are not to be designed for tension loads unless approved by the MDOT Geotechnical Services Section. (10-28-2019)

Contact MDOT's Geotechnical Services Section with questions.

7.04

REINFORCEMENT

7.04.01

Steel Reinforcement (11-28-2011)

A. Epoxy-Coated Reinforcement
(6-27-2022)

All reinforcement is epoxy coated. This includes, but is not limited to, abutments, return walls, curtainwalls, pier columns, pier crash walls and backwalls.

B. Steel Reinforcement at Joints

Steel reinforcement is to extend through construction joints and stop at expansion joints.

C. Allowable Length

Generally, bar lengths should be limited to 50'-0" but may be increased to 60'-0" to avoid excessive lapping. These lengths are based on transportation charges.

Normally, #3 reinforcement is not available in lengths greater than 40'-0". Therefore, unless unusual conditions warrant an exception, the maximum length of #3 bars shown on the plans should be 40'-0". (8-6-92)

D. Fabricating Tolerance

The permissible tolerance for cutting reinforcing bars to length is 1". The bars should be made long enough to ensure that the minimum lap and proper edge distance is provided in case the bars are cut 1" short of the plan dimension.

The permissible tolerance for fabricating the "B" bars in pier columns is 1". The bars should be detailed with a gap between the bottom of bars and the top of footing in case the bars are fabricated 1" longer than shown on the plans.

7.04.01 (continued)

E. Wall or Column Vertical Steel Reinforcement (5-6-99)

In order to facilitate placing and supporting long reinforcing bars that are anchored in footings, splices in vertical reinforcement should be provided. Short dowels can be used for wall front reinforcement, with laps just above the footings.

Laps for reinforcement in back of walls or in columns should be at least 4'-0" or 5'-0" above top of footing so as not to be in the area of maximum stress. Laps should not normally be provided for bars that do not extend to full height of wall or pour.

Where walls or columns are of such height as to require horizontal construction joints, bar laps should be provided above these joints.

F. Bar Size Substitutions

When using Grade 60 reinforcing, the AASHTO specification for distribution of flexural reinforcement may require using small bars at close spacing. Therefore, it may not always be permissible to make a total area substitution with fewer larger bars.

G. Minimum Bar Size

To avoid handling damage, the minimum bar size shall be #4. An exception to this is the temperature steel in decks. These bars are to be #3.

7.04.02

Stainless Steel Reinforcement (11-28-2011)

A. Criteria For Use

As an alternative to epoxy coated reinforcement, stainless-clad and solid stainless steel reinforcement should be selectively used in bridge deck construction. Designers will need to examine whether the additional expenditure is warranted for enhanced durability of the structure. The designer should consider use of stainless-clad and solid stainless reinforcement under one or more of the following circumstances.

1. The additional expenditure for stainless-clad and solid stainless reinforcement, including cost savings from reduced cover requirements, should be no more than eight percent of the programmed structure cost.

2. For structures on trunkline roads where future repair and maintenance would be very disruptive to traffic and where mobility analysis defines the project as significant and mitigation measures to minimize travel delay are needed (See Work Zone Safety and Mobility Policy).

3. Over navigable waterways or protected wetlands sensitive to environmental impact from construction activity.

4. Where the deck cross section is less than 9 inches, due to local geometric restrictions or in widening projects where the dead load is limited to the capacity of the existing substructure.

5. Bridges located over high volume railway lines where access and right of way restrictions exist.

7.04.02 (continued)

When using stainless-clad or solid stainless steel reinforcement for new bridge deck construction, the designer should consider using empirical deck design when that type of design reduces the amount of steel reinforcement.

Combine stainless-clad reinforcement with solid stainless reinforcement to optimize the material costs.

B. Cost

In estimating the cost of stainless-clad and solid stainless steel reinforcement, current prices should be obtained from suppliers. Stainless-clad and solid stainless steel reinforcement costs are more volatile and variable than for carbon steel and are sensitive to bar length, diameter and the waste when cutting from relatively short stock bars. Prices may vary significantly between suppliers.

C. Detailing and Availability

Stainless-clad and solid stainless steel reinforcement is similar to normal carbon steel reinforcement in the design, detailing and construction process. Use stainless-clad and solid stainless steel reinforcement in both reinforcement mats in the bridge deck, and in other locations as warranted. Dissimilar metals contact, whether with epoxy coated reinforcement, uncoated reinforcement, or galvanized steel, is not considered detrimental when embedded in concrete. The standard cover requirement of three inches can be reduced to two inches.

Stainless-clad reinforcement is available in standard U.S. customary sizes of #5 or greater, with maximum lengths of 40’-0”, and available in Grade 60. Solid stainless steel reinforcement is available in all standard sizes and lengths, and available in both Grade 60 and Grade 75.

7.05

PEDESTRIAN, ORNAMENTAL AND RAILROAD FENCING (8-6-92)

7.05.01

Electrical Grounding System

Pedestrian bridges, pedestrian screening and ornamental fencing (8’-0” or greater in overall height) are to be grounded as specified in the standard specifications. Details described in the specifications need not be shown on the plans.

7.05.02

Pedestrian Fence Fabric

Use 2” mesh size opening unless 1" opening is recommended by the Region. Mesh size opening of 1" is preferred on pedestrian fencing for structures in the Detroit metropolitan area and it should be noted on the plans. When 1” mesh size opening is proposed in close proximity to an intersection, consult with the Geometric Design Unit, Design Division, Bureau of Development, to evaluate potential conflict with intersection sight distance. For limits of the metropolitan area see Appendix 12.01.01. (3-28-2022)

Six, eight or ten foot fence fabric is generally used to design pedestrian fencing for structures. Ten foot fabric is used for metal and post and tube railings. Six and eight foot fabric are used in combination with concrete railing to attain desired height. (3-28-2022)

7.05.03

Pedestrian Fence Posts

Posts for bridge fencing should be 2½” (2.875" O.D.) steel pipe. The steel type and maximum post spacing should be as shown below.

Maximum Unsupported Post Height	Mesh Size	Steel Type (ASTM)	Maximum Post Spacing
9'-0"	1"	F1043	8'-6"
9'-0"	2"	F1043	10'-0"
7'-0"	1"	F1083	8'-6"
7'-0"	2"	F1083	10'-0"

(10-24-2001)

7.05.04 (3-28-2022)

Pedestrian Fence Height

Fence is straight and 10 feet total height minimum when no pedestrian traffic is expected on a structure or when an existing sidewalk or brush block is less than 3’-0” in width. Type 6 & 7, 2 Tube, Aesthetic Parapet Tube flush mount (without sidewalk), 4 Tube with brush block, 3 Tube With Pickets with brush block, Concrete Block Retrofit and existing Type 4 & 5 fall into this category.

Fence is curved and 9 feet (+/-) total height when pedestrian traffic is expected on a structure or when an existing sidewalk or brush block is 3’-0” or greater in width. Aesthetic Parapet Tube with sidewalk, 4 Tube with sidewalk and 3 Tube With Pickets with sidewalk fall into this category.

Existing open parapet, existing solid parapet, existing R4, R5 and R9, existing 3 Tube or 5 Tube can use either straight or curved fence depending on sidewalk or brush block width (less than 3’-0” or 3’-0” or greater).

7.05.05 (3-28-2022)

Anti-Climb Shield

Place anti-climb shields with all pedestrian fencing to prevent climbing on the outside of the fence (over traffic or precipitous drop). Shields are generally located at the second or third vertical fence post from each end of the pedestrian fencing.

7.05.06 (3-28-2022)

Ornamental Fencing Guidelines

Approval for structural adequacy for all proposed ornamental fence installations on MDOT bridges and bridge railings is required by MDOT's Bureau of Bridges and Structures (BOBS). Ornamental fences installed on new bridges and bridge railings must be designed according to the current edition of the AASHTO LRFD Bridge Design Specifications. Contact BOBS Chief Structure Design Engineer for questions concerning the design requirements for ornamental fences on MDOT bridge railings. Contact MDOT BOBS Bridge Construction Unit and Structural Fabrication Unit for questions related to the materials and construction of ornamental fences on MDOT bridges and bridge railings.

Contact MDOT's Geometric Design Unit, Design Division, Bureau of Development (BOD), for questions regarding the crashworthiness of proposed ornamental fence installations on MDOT bridge railings.

Do not attach ornamental fences to steel tube bridge railings (e.g., 2 Tube railing, 4 Tube railing, and 3 Tube With Pickets railing).

The use of ornamental fencing does not alleviate the need to protect the motoring and pedestrian traffic. Use pedestrian fence with fabric in addition to ornamental fences as described in this section and section 7.02.29. If pedestrian fence is not required, ornamental fence can be used on its own.

Include anti-climb shields with ornamental fencing, regardless of pedestrian fence with fabric use. Anti-climb shields can simulate ornamental fence or pedestrian fence.

When an entity other than MDOT requests an integration of ornamental fencing or other highway aesthetic elements within the MDOT right-of-way (ROW) they shall also follow the Highway Aesthetic Element Guidelines. Review of any structures integrating ornamental fencing should be routed through the BOBS Chief Structure Design Engineer.

7.05.06 (continued)

Unless proven crashworthy by full-scale crash testing, as determined by MDOT, under NCHRP 350 or MASH criteria and under the appropriate test level, the proposed ornamental fence and/or combined bridge railing and ornamental fence must meet the following requirements:

A. Regardless of design speed, ornamental fences may be placed on bridge railings or bridge decks without the installation of additional barrier protection when located beyond the clear zone based on the design speed and average daily traffic at the proposed installation site.

See Road Design Manual Section 7.01.11 for Clear Zone chart. (7-24-2023)

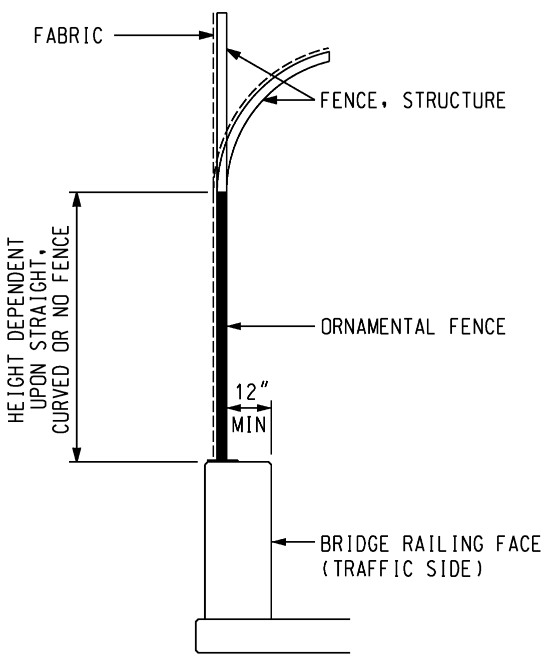
7.05.06 (continued)

B. On roadways with design speeds of 40 mph or less, ornamental fences on bridge railings and bridge decks located within the clear zone may be installed when meeting all of the following conditions:

1. Fence is installed on top of or behind the bridge railing.

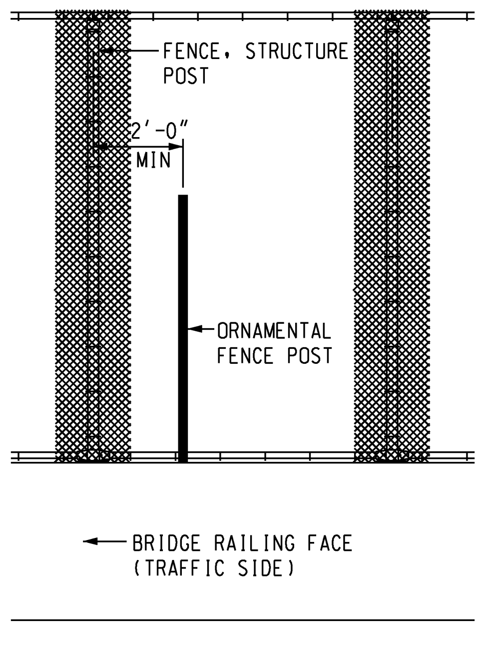
2. Fence is placed at least 12 inches from the top of the bridge railing face (i.e., the portion facing traffic).

   Bridge rail widening and/or ornamental fence placement behind the bridge railing may be necessary to provide the 12 inch minimum offset from the ornamental fence to the bridge railing face.



7.05.06 (continued)

3. Locate fence posts between (2'-0" minimum laterally from) railing structural posts (Aesthetic Parapet Tube railing), not directly behind. When ornamental and pedestrian fence posts are both used on a railing, locate posts 2'-0" minimum laterally from one another.



4. Fence components that could become an occupant compartment intrusion threat, as determined by MDOT, are not allowed. Contact MDOT's Geometric Design Unit, Design Division, BOD, for questions.

   Tapering the ends of ornamental fence components to minimize snagging potential is one way to alleviate occupant compartment intrusion.

C. On roadways with design speeds of 45 mph or greater, ornamental fences on bridge railing and bridge decks located within the clear zone may only be installed if an additional bridge railing or roadside barrier is installed between the traveled way and the ornamental fence.

7.05.07 (3-28-2022)

Railroad Fencing and Splashboard

A. For railroad bridges over roadways or rivers, Region Project Development or Bridge Engineer will determine whether to provide bridge screening due to the presence of ballast and discarded rail spikes.

B For bridges over railroad grades, splashboards are placed on top of railings to prevent snow, ice, or other debris from being thrown onto the tracks and passing trains. The final height of the railing and splashboard must satisfy the railroad's requirements. The railroad may request the use of splashboards possibly in combination with fencing.

Contact the Railroad Grade Separations Engineer of MDOT Office of Rail to coordinate these and other design parameters with the railroad.

7.06

RECONSTRUCTION PROJECTS

Include saw cut depth dimensions when removing portions of abutments, piers and columns on the plans. (8-20-2009)

7.06.01

Placement of Temporary Barrier
(9-21-2015)

A. 26" or More Laterally Available

For widening jobs or part-width construction of a new bridge, when 26" or more laterally is available between the toe of a temporary barrier on the construction side and a precipitous drop-off, place standard temporary concrete barrier or temporary steel barrier meeting MDOT specifications near the drop-off. No special hardware or procedures are necessary. See Standard Plan R-126-Series.

B. Less Than 26" Laterally Available

When there is less than 26" laterally between the toe of the barrier on the construction side and the precipitous drop-off, place an appropriate limited deflection temporary barrier detail meeting the requirements of Standard Plan R-53-Series, or an approved alternative. Refer to Standard Plan R-126-Series for placement and to Standard Plan R-53-Series for additional information regarding limited deflection temporary barrier details.

For more definitive write-up and discussion of detailed placement options see Section 7.01.70 of Road Design Manual.

7.06.02

Concrete Anchors (5-6-99)

A. Expansion Anchored Bolts

In addition to field testing, we will ensure sound anchorage by reducing the design loads. The values to be used will vary with the application as shown below:

PULLOUT VALUES OF EXPANSION ANCHORED BOLTS IN POUNDS						
Application	Approx. Safety Factor	⅜"	½"	⅝"	¾"	⅞"
Noncritical Design Loads (Including noncritical, static or shock loads)	4	875	1,620	2,565	3,775	5,240
Vibratory Loads (e.g., Sign Supports)	12	290	540	855	1,260	1,755

Design details should always call for two or more anchors for redundancy.

B. Bonded Anchors (Adhesive and Grout Anchors)

All bonded anchors shall be **detailed** for embedment depth on the contract plans. For A307 bolts the embedment depth shall be taken as "9d" (9 times the nominal bolt diameter). For Grade 60(ksi) reinforcing steel the embedment depth shall be taken as "12d" (12 times the bar diameter). (5-1-2000)

In addition to field testing, we will ensure sound anchorage by reducing the design loads. For all applications a safety factor of 4 should be applied to 125% of the threaded rod/reinforcements yield strength to obtain the allowable design tensile load on the anchor. The allowable tensile load shall be computed per:

Allowable tensile load = (125% f_y A_T)/4
A_T = tensile stress area
= net section through threads
(for reinforcing steel use nominal area)

For a list of qualified products (bonded anchors) see Materials Source Guide.

(6-27-2022)

C. Allowable Shear for Post Installed Concrete Anchors (Expansion and Bonded Anchors) (per AASHTO Table 10.32.3A)

Allowable Shear = 0.30 f_y A_T
A_T = tensile stress area
= net section through threads
(for reinforcing steel use nominal area)

7.06.03

Substructure Protection (6-27-2022)

Where we are reconstructing an existing substructure unit; i.e., capping or extending it, and there is a transverse joint in the superstructure directly above, the entire top existing and proposed, and all other existing faces of the unit shall be coated with penetrating water repellent treatment or another concrete surface coating. See Section 7.03.11.

7.06.04

Hanger Assembly Replacement and Temporary Support Guidelines for Redundant Bridges

For additional information on temporary supports see Subsection 7.01.10.

A. Construction Methods

The choice of method can best be made during an on-site inspection, preferably the Scope Verification or Plan Review, where Region/TSC personnel can offer opinions. (5-6-99)

1. Temporary Support From Below Using Column and Footing Arrangement

    a. Does not require lane closure above; i.e., traffic over work area.

    b. May require lane closure below depending on location of suspender.

2. Temporary Support From Above Using Multiple or Single Beam Suspension Arrangement

    a. Requires lane closure above.

    b. May require lane closure below because of underclearance restrictions.

    c. Joint replacement at expansion end and removal of portions of deck at fixed end will probably be required for multiple beam suspension.

7.06.04 (continued)

B. Preliminary Investigation for Temporary Support From Below

1. Request Borings and Factored Nominal Soil Pressures from Geotechnical Services Section if pressures are unknown or pressures greater than 2500 psf is required. (6-27-2022)

    a. Consideration should be given to possible differential settlement below temporary support footing.

    b. Borings are not required if footing is placed on paved surface. Assume a bearing pressure of 17 psi. (5-6-99)

2. Determine Utility Locations

    Underground utilities may be damaged by settlement of temporary support footing pressure.

3. Determine Obstructions of Temporary Support Footing

    Consider pier location and skew.

4. Read Current Specifications for This Type of Work Prior to Starting Design.

7.06.04 (continued)

Hanger Assembly Replacement and Temporary Support Guidelines for Redundant Bridges

C. Design of Temporary Support From Below (see Appendix 7.06.04 for nomenclature):

1. Loading (9-26-2022)

a. When traffic is detoured or is being maintained on a portion of the structure that does not cause load on the temporary supports and construction equipment will be used on the span being temporarily supported, base the design on the following:

1) Use the appropriate Allowable Stress Design (ASD) load factors and load combinations specified in the AASHTO Guide Design Specifications for Bridge Temporary Works to obtain the maximum design load effects.

2) Use a uniformly distributed personnel and equipment load (CP) of 25 pounds per square foot applied to the tributary width of the beam being temporarily supported unless a project specific load has been determined to be appropriate.

3) Unless the actual rated reactions of the equipment to be used by the Contractor in the construction of the project are known, an equipment reaction (CR) should be selected from Appendix 7.06.04.C based on the span length of the beam being temporarily supported. If the actual rated reactions of the equipment to be used by the Contractor are known, they should be used in the design and distributed in accordance with the AASHTO Standard Specifications for Highway Bridges.

7.06.04 C. (continued)

4) For the design of the column, column base plate, jack base plate, hydraulic jack capacity, channel shims, and jack bearing plate increase the reactions of the equipment by 30 percent to account for impact on the structure being supported. Impact does not need to be included in the design of the footing (timber and concrete).

5) If the equipment to be used by the Contractor in the construction of the project is known, the load factor applied to the reaction from the construction equipment may be reduced to 1.00 for all load combinations.

b. For all other cases base the design on the following:

1) Use 1.25 (DL+LL+I) for column, column base plate, and jack base plate design.

2) Use 1.0 (DL+LL+I) for channel shim and jack bearing plate design.

3) Use 1.25 (DL+LL) for footing design (timber and concrete).

4) Use 1.25 (DL+LL+I) for hydraulic jack capacity.

5) If traffic is detoured or is being maintained on a portion of the structure that does not cause load on the temporary supports live load can be omitted from the design of the temporary support.

7.06.04 (continued)

Hanger Assembly Replacement and Temporary Support Guidelines for Redundant Bridges

2. Materials (5-6-99)

   a. Use AASHTO M270 steel, f_y = 36,000 psi. (Do not mix steel types used for temporary support.)

   b. Use Concrete Grade 3500, f'_c= 3000 psi. (6-28-2021)

   c. Use Structural Grade Timber, F_b = 1200 psi. F_v (horiz.) = 100 psi.

3. Column Design

   a. Size for axial load plus bending in both perpendicular directions.

   b. Use 0.1 x flange width and 0.1 x beam depth rounded up to nearest ½" for assumed eccentricity.

   c. Use pinned-pinned end condition (restraint at base plate small). Effective length factor, K = 1.0.

   d. Check lateral loading on column from thermal movement of bridge. Use 75°F temperature variation. Combine thermal load with (DL+LL+I).

4. Column Base Plate Design

   a. Avoid use of stiffeners (high welding cost).

   b. Size for axial load plus bending. Use eccentricity assumed in column design.

   c. Do not attach base plate to footing.

   d. Use F_b = 0.75 F_y.

7.06.04 (continued)

5. Jack Base Plate Design

   a. Design as plate fixed on three sides, free on one side. (See Young, W.C., Roark's Formulas for Stress and Strain, pg. 469. Available in MDOT Library. See Appendix 7.06.04 for excerpt.) (5-6-99)

   b. Use equivalent rectangular uniform load from jack bearing area.

   c. For uniform load 2/3 of plate width, use $f_b = 60\ q/t^2$ (see item a above, q = load per unit area in psi and t = plate thickness in inches).

   d. For uniform load 1/3 of plate width, use $f_b = 17\ q/t^2$ (see item a above, q = load per unit area).

   e. Linear interpolate for uniform load between 1/3 and 2/3 of plate width.

   f. Use F_b = 0.75 F_y in psi.

   g. Weld to column.

6. Channel Shims Design (two per support)

   a. Size for axial load plus bending perpendicular to web.

   b. Use 0.1 x flange width rounded up to nearest ⅛" for assumed eccentricity.

   c. Weld to jack base plate.

7.06.04 (continued)

Hanger Assembly Replacement and Temporary Support Guidelines for Redundant Bridges

7. Jack Bearing Plate Design

    a. Size for bending about centerline existing girder or beam web.

    b. Use load on channel shims for bending calculations.

    c. Use $F_b = 0.75\ F_y$.

8. Hydraulic Jack Capacity

    a. Specify minimum jack capacity required (based on axial load only).

9. Timber Footing Design

    a. Use double mat (minimum) with square or rectangular timbers.

    b. Size for axial load plus bending. Use eccentricity assumed in column design. Use allowable soil pressure from Geotechnical Services Section. On a paved surface assume a bearing pressure of 17 psi.
    (5-6-99) (2-26-2018)

    c. Check flexure and horizontal shear. Allow 25 percent overstress to account for short duration of loading.

    d. Column base plate full width across top mat.

    e. Top mat full width across bottom mat.

    f. Specify channels lag-bolted to timbers across top of both mats, each end (lag-bolt to each timber).

7.06.04 (continued)

10. Concrete Footing Design

    a. Use bottom mat steel reinforcement only, both directions.

    b. Size for axial load plus bending. Use eccentricity assumed in column design. Use allowable soil pressure from Geotechnical Services Section. (2-26-2018)

    c. Check flexure, beam (one way) shear and slab (punching) shear.

    d. Specify concrete to be stenciled with "top" on side opposite steel reinforcement. Stencil “bottom” as required.

11. Footing Placed on Soil

    a. Specify compaction of original ground to not less than 95 percent of its maximum unit weight to a depth of 9" and to 1'-6" outside footing outline.

    b. Specify Structure Embankment (CIP), if required, to 1'-6" outside footing outline.

    c. Specify level under footing.

    d. Specify Granular Material Class III, compacted to not less than 95 percent of its maximum unit weight, to 1'-6" outside footing outline for leveling.

    e. Specify 1V:1H slope down to natural ground for all required fill material.

7.06.04 (continued)

Hanger Assembly Replacement and Temporary Support Guidelines for Redundant Bridges

12. Footing Placed on Pavement or Paved Shoulder

    a. Specify level under footing.

    b. Specify 21AA aggregate, HMA cold patch material, or approved equal to 1'-6" outside footing outline for leveling.

    c. Specify 1V:1H slope down to pavement for required fill material.

13. Placement of Temporary Support

    a. Centerline temporary support under area where pin plate exists.

    b. Centerline temporary support under stiffener, if possible.

    c. Show location on plans.

14. Bracing for Temporary Support (10-27-2025)

    a. Bracing may be required to ensure the stability of the temporary support or temporary support system while in use. Reasons why bracing may be required include, but are not limited to, providing resistance to lateral movements or limiting the unbraced length for tall temporary support columns.

    b. Design bracing for the temporary support or temporary support system to resist all loads required by the applicable AASHTO design specifications.

    c. Design and detail bracing and bracing connections to account for and accommodate any settlement that may be experienced while the temporary support system is installed and in use.

7.06.04 (continued)

15. Shop Drawing Review (3-26-2018)

    a. Send temporary support shop drawings to MDOT Structural Fabrication Unit for review.

    b. Check all weld sizes and member sizes against plan requirements.

16. Maintaining Traffic

    a. Temporary supports must be completely shielded from traffic.

    b. Place temporary concrete barrier in area of temporary support in all cases (both sides if narrow median).

17. Hanger Assembly Removal Sequence

    a. Minimize risks in case of support failure or excessive settlement.

    b. Adjacent beam suspender operational.

    c. Opposite end suspender operational.

D. Checks on Existing Girder or Beam, Temporary Support From Below

1. Loads: Use design axial load of column for checks.

2. Web Checks

    a. Web buckling - distribute load on 45° from edge of jack bearing plate (effective length factor, K = 1.0).

    b. Web crippling.

    c. Specify bolted stiffener (not included in payment for temporary supports), if required, bearing against bottom flange.(6-27-2022)

7.06.04 (continued)

Hanger Assembly Replacement and Temporary Support Guidelines for Redundant Bridges

3. Diaphragm Clearance

   a. On sharply skewed bridges, determine if diaphragm needs to be cut to allow placement of new pin (note on plans if cutting is required).

   c. Determine if repair of cut diaphragm is required. Use field bolted cover plate if repair is necessary.

E. Design of Temporary Support From Above

1. Consideration should be given to providing redundancy in temporary support. Avoid nonredundant schemes if possible.

2. Multiple Beam Support Loading

   Use 1.25 DL + 2.0 (LL+I) maximum.

3. Single Beam Support Loading

   Use 1.25 (DL+LL+I).

4. Materials (5-6-99)

   Use AASHTO M270 steel,
   F_y = 36,000 psi.
   (Do not mix steel types used for temporary support if the pieces are to be joined by welding.)

5. Hanger Assembly Removal Sequence

   a. Minimize risks in case of temporary support failure.

   b. Adjacent beam suspender operational.

   c. Opposite end suspender operational.

   d. Maintaining traffic may demand deviation from items b and c.

F. Checks on Existing Beam or Girder, Temporary Support From Above

1. See Article D for required checks to be made.

7.06.04 (continued)

G. Hanger Assembly Plan Dimensions and Field Measurements

1. Dimensions on Plans

   a. Pins - give diameter and length.

   b. Link plate - give length, width, thickness and C-C pins.

   c. Other details and dimensions shown on Bridge Design Guides 8.14.02, 8.15.01 and 8.15.01A. Specify stainless steel washers and cotterpins.

   d. If existing suspender must be shown on the detail sheets, this detail shall be shown accurately.

2. Field Measurements

   a. If field measurements differ from plan dimensions, correct shop drawings to reflect actual dimensions.

   b. Use average C-C pin distance for specific hanger locations where one side is different from the other.

   c. Increase pin length to account for girder or beam offset, if required. Select longest length required and use for all pins.

3. Shop Drawing Review
   (3-26-2018)

   a. Send suspender assembly shop drawings to MDOT Structural Fabrication Unit, for review.

7.07

APPROACH ITEMS

7.07.01

Guardrail

All new guardrail anchorages to bridges will utilize thrie beam guardrail according to Standard Plan R-67-Series and will be anchored directly to the bridge railing or pier filler walls. (5-6-99)

Where there are independent backwalls, that is, where there will be thermal deck movement at the abutments, the movement will be accommodated by the slots in the expansion section of the guardrail anchorage.

For additional information see Road Design Manual Section 7.01.16.

7.07.02

Curb and Gutter for Rural Bridges (6-27-2022)

The types and lengths of bridge approach curb and gutters (including valley gutter, where required) shall be determined by the road/bridge designer and shown on the General Plan of Structure Sheet.

For additional information see Road Design Manual Section 6.06.08 and MDOT Drainage Manual.

7.07.03 (5-6-99)

Bridge Approach Pavement

To eliminate approach pavement settlement, a concrete approach section will be used for all new bridges and bridge replacements, deck and superstructure replacement projects and concrete overlays. For hot mix asphalt (HMA) deck overlays, a concrete approach section is not necessary. The details of the approach slab shall be as specified on Standard Plan R-45-Series except on existing structures, where the grade will not be raised; the length of the approach slab shall match the existing slab joint. (9-2-2003)

Use approach pavements for integral and expansion bearing semi-integral abutment designs according to Bridge Design Guide 6.20.04 Series. At semi-integral abutments with fixed bearings use approach pavement as specified on Standard Plan R-45-Series. (3-27-2023)

Use approach pavements for sliding slab over backwall designs according to Bridge Design Guides 6.20.03 Series. (1-24-2022)

7.08 (6-27-2022)

UTILITY ITEMS

7.08.01

General

For additional information regarding utilities see:

Chapter 9 and Sections 14.16, 14.26, 14.39 and various other Chapter 14 sections of the Road Design Manual

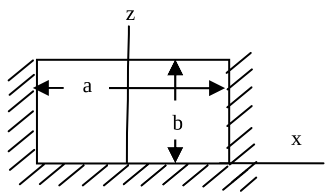
MDOT Development Services Division's Utility Coordination Procedure Manual.

HOT ROLLED SHEET PILING SECTION MODULI		
Sheet Pile Designation	Section Modulus in^3/ft.	
	Nominal	Effective
PZ-22	18.1	15.3
AZ 12	22.3	19.2
PZC-12	22.4	19.2
AZ 12-770	23.2	19.9
PLZ-23	30.2	20.7
AZ 13-770	24.2	21.0
AZ 13	24.2	21.2
PZC-13	24.2	21.3
AZ 14-770	25.2	22.0
AZ 14	26.0	23.0
PLZ-25	32.8	23.0
PZC-14	26.0	23.1
PZ-27	30.2	25.5
PZC-17	31.0	26.6
AZ 17	31.0	26.6
AZ 17-700	32.2	27.7
AZ 18-700	33.5	29.0
AZ 18	33.5	29.3
PZC-18	33.5	29.5
AZ 19-700	34.8	30.4
AZ 19	36.1	32.0
PZC-19	36.1	32.1
AZ 24-700	45.2	40.4
AZ 25	45.7	40.7
PZC-25	45.7	41.1
AZ 26	48.4	43.5
AZ 26-700	48.4	43.6
PZ-35	48.5	43.6
PZC-26	48.4	44.0
AZ 28	51.2	46.4
AZ 28-700	51.3	46.6
PZC-28	51.2	46.9
PZ-40	60.7	54.6
PZC-34	63.8	58.3
PZC-36	67.0	61.6
AZ 37-700	68.9	61.7
PZC 37-CP	68.8	64.1
PZC-38	70.6	65.3
AZ 39-700	75.5	68.2
PZC 39-CP	73.0	68.3
AZ 41-700	76.2	69.4
PZC 41-CP	75.8	71.0
AZ 46	85.5	77.7
AZ 48	89.3	81.7
AZ 50	93.3	85.9

COLD ROLLED SHEET PILING SECTION MODULI		
Sheet Pile Designation	Section Modulus in^3/ft.	
	Nominal	Effective
SZ-12	8.6	5.2
SZ-14	9.8	6.2
CZ-67	10.7	6.5
SZ-15	10.4	6.6
CZ-72	11.7	7.3
SPZ-16	13.2	8.4
SPZ-84	13.6	8.9
CZ-95RD	15.2	10.2
SZ-18	16.2	10.2
CZ-95	15.5	10.5
SPZ-19.5	16.6	11.2
CZ-101	16.5	11.3
SZ-20	17.8	11.4
CZ-107	17.5	12.2
SPZ-22	18.3	12.7
SZ-22	19.6	12.7
CZ-113	18.4	12.9
SCZ 19	18.8	13.0
SPZ-23.5	19.3	13.6
SCZ 21	19.9	14.1
SZ-222	26.8	16.6
SZ-24	29.5	18.6
SCZ 22	29.8	19.4
CZ-114RD	31.6	20.1
SKZ 20	31.7	20.7
SZ-27	32.4	20.7
SCZ 23	31.6	21.3
SPZ-23	31.3	21.5
CZ-114	31.6	21.7
SKZ 22	33.4	22.5
SCZ 25	33.5	23.2
SPZ-26	34.8	24.4
SKZ 23	35.6	24.6
CZ-128	35.5	24.8
SCZ 26	35.3	25.1
CZ-134	37.2	26.3
SKZ 24	37.7	26.7
CZ-141	39.1	27.9
SCZ 29	39.1	28.7
SKZ 25	40.1	29.2
CZ-148	40.9	29.4
SCZ 30	40.9	30.6

Bridge Design Manual Appendix 7.03.08 D.

10. Rectangular plate; three edges fixed, one edge (a) free



COEFFICIENTS FOR DESIGNING BASE PLATES

Roark's Formulas for Stress and Strain, (Sixth Edition), page 469
Warren C. Young

10a. Uniform over entire plate

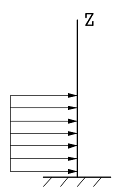
(At x = 0, z = 0) Max $\sigma_b = \dfrac{-\beta_1 q b^2}{t^2}$ and $R = \gamma_1 q b$

(At x = 0, z = b) $\sigma_a = \dfrac{\beta_2 q b^2}{t^2}$

(At $x = \pm \dfrac{a}{2}$, z = b) $\sigma_a = \dfrac{-\beta_3 q b^2}{t^2}$ and $R = \gamma_2 q b$

a/b	0.25	0.50	0.75	1.0	1.5	2.0	3.0
β_1	0.020	0.081	0.173	0.321	0.727	1.226	2.105
β_2	0.016	0.066	0.148	0.259	0.484	0.605	0.519
β_3	0.031	0.126	0.286	0.511	1.073	1.568	1.982
γ_1	0.114	0.230	0.341	0.457	0.673	0.845	1.012
γ_2	0.125	0.248	0.371	0.510	0.859	1.212	1.627

10aa. Uniform over 2/3 of plate from fixed edge

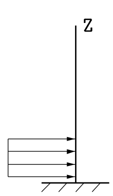


(At x = 0, z = 0) Max $\sigma_b = \dfrac{-\beta_1 q b^2}{t^2}$ and $R = \gamma_1 q b$

(At $x = \pm \dfrac{a}{2}$, $z = 0.6b$ for $a > b$ or $z = 0.4b$ for $a \le b$) $\sigma_a = \dfrac{-\beta_2 q b^2}{t^2}$ and $R = \gamma_2 q b$

a/b	0.25	0.50	0.75	1.0	1.5	2.0	3.0
β_1	0.020	0.080	0.164	0.277	0.501	0.710	1.031
β_2	0.031	0.110	0.198	0.260	0.370	0.433	0.455
γ_1	0.115	0.230	0.334	0.424	0.544	0.615	0.674
γ_2	0.125	0.250	0.344	0.394	0.399	0.409	0.393

10aaa. Uniform over 1/3 of plate from fixed edge



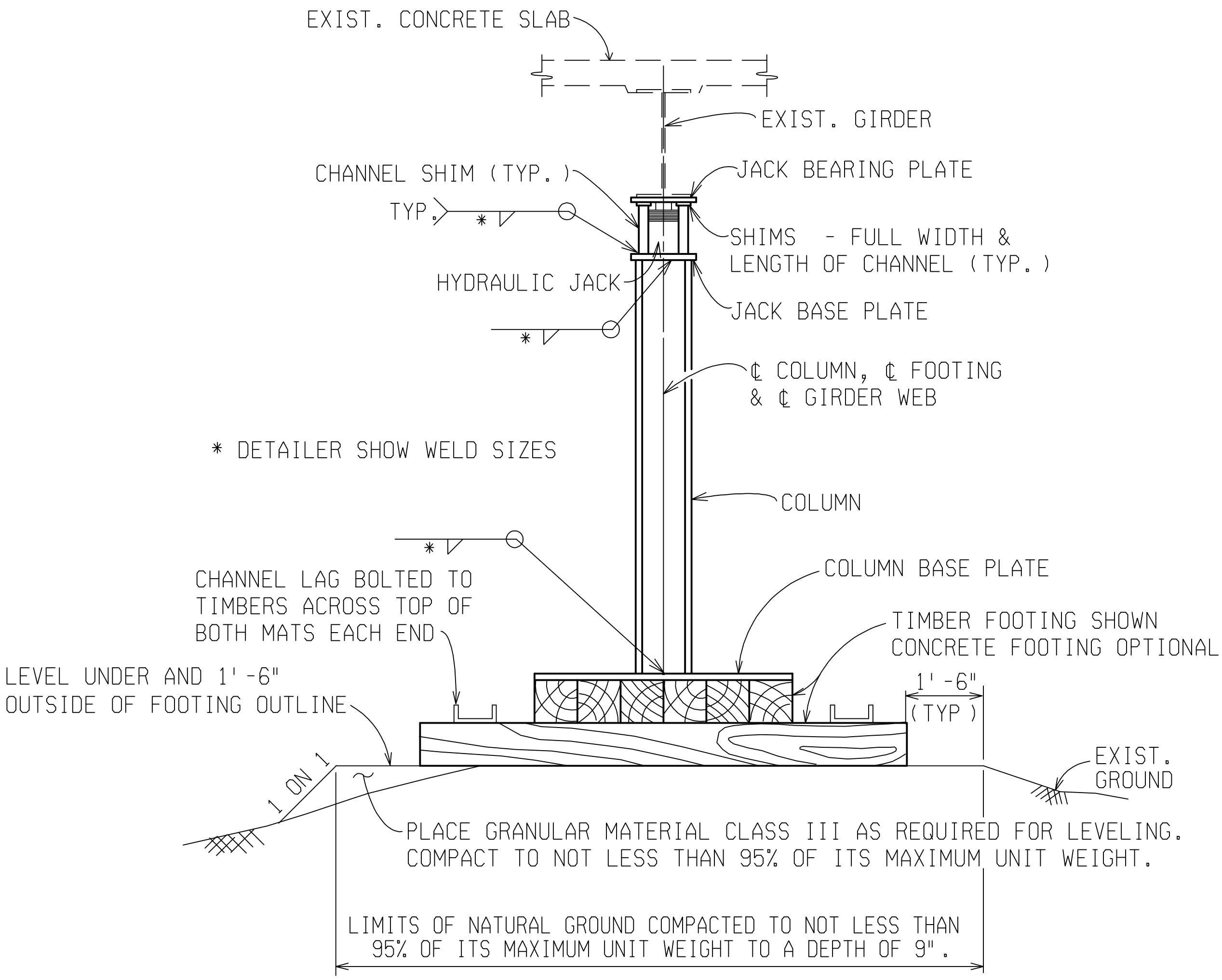
(At x = 0, z = 0) Max $\sigma_b = \dfrac{-\beta_1 q b^2}{t^2}$ and $R = \gamma_1 q b$

(At $x = \pm \dfrac{a}{2}$, $z = 0.2b$) $\sigma_a = \dfrac{-\beta_2 q b^2}{t^2}$ and $R = \gamma_2 q b$

a/b	0.25	0.50	0.75	1.0	1.5	2.0	3.0
β_1	0.020	0.068	0.110	0.148	0.202	0.240	0.290
β_2	0.026	0.063	0.084	0.079	0.068	0.057	0.040
γ_1	0.115	0.210	0.257	0.291	0.316	0.327	0.335
γ_2	0.111	0.170	0.194	0.185	0.174	0.170	0.180

Notation:
"a" and "b" refer to plate dimensions, and when used as subscripts for stress, they refer to the stresses in directions parallel to the sides "a" and "b", respectively. "Φ" is a bending stress in pounds/square inch which is positive when tensile on the bottom and compressive on the top if loadings are considered vertically downward. "R" is the reaction force, in pounds/inch, normal to the plate surface exerted by the boundary support on the edge of the plate. "q" is the load per unit area in pounds/square inch.

MICHIGAN DESIGN MANUAL - BRIDGE DESIGN	APPENDIX 7.06.04
DETAIL OF TEMPORARY SUPPORT FROM BELOW	ISSUED: 11/27/01



ELEVATION

(FOOTING PLACED ON SOIL SHOWN)

Construction Equipment

Table of Lane Reactions on Simple Spans. Impact not included.

Span (ft)	Reaction (pounds)		Span (ft)	Reaction (pounds)
20	15,000		90	17,600
25	15,600		100	17,600
30	16,100		120	17,700
35	16,400		140	17,800
40	16,600		160	17,900
45	16,800		180	17,900
50	17,000		200	18,000
60	17,200		300	18,100
70	17,300		400	18,100
80	17,500			

09-26-2022