Bridge Design Manual

NEBRASKA

Good Life. Great Journey.

DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

April 18, 2025

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Foreword

I am pleased to introduce you to the new Bridge Design Manual (BDM). If you are familiar with the Bridge Office Policy and Procedures (BOPP), you will first notice the name change. This updated title reflects the manual's true purpose—presenting policy and standards specifically related to bridge design within the department. While the BDM serves as a critical reference for bridge design policies, it is not intended to encompass all policies or procedures related to bridges in general. Our goal was to provide a clear, concise, and accessible resource for bridge design practices.

We greatly appreciate the patience of our community as we worked toward the publication of this comprehensive document. The development and maintenance of such a resource is no small feat, especially while meeting our ongoing obligations to the public. This delay does not reflect a lack of priority—it's quite the opposite. The manual is an essential tool, but its creation had to be balanced with the day-to-day demands of public service.

To ensure the most up-to-date information is always available, we are rolling out the manual chapter by chapter. This phased approach allows us to provide critical design guidance as it becomes ready while continuing to refine additional sections. Over the next year, more chapters will be released to keep the manual current with the evolving needs of our industry. In addition, interim revisions will be issued as NDOT Policy Letters, which will be posted on our website and take effect immediately upon transmittal. Please keep an eye out for these updates.

Chapter 7 is a particularly exciting addition, as it focuses on Repair and Preservation. It is entirely new to the manual and represents a compilation of preferences and practices within the NDOT Bridge Division. We are eager to share this chapter with bridge owners across the state, believing it will serve as a valuable resource for aligning bridge repair and preservation practices statewide.

Recognizing the need for a structured approach to policy development, we established an executive BDM committee, along with subcommittees composed of members across the Bridge Division. This framework formalizes the process for creating, reviewing, and publishing bridge design policies and ensures active engagement from all levels within the division. By adopting a collaborative and systematic approach, we are able to incorporate national and regional research, ensuring our policies reflect the latest advancements and maintain NDOT's position at the forefront of bridge design.

This approach also supports the development of the next generation of bridge engineers. With the challenges our industry faces, it is more important than ever to equip younger and less experienced staff with the knowledge and resources they need to grow. This manual serves not only today's professionals but also as a guide for those just entering the field, ensuring they are well-prepared for the demands of modern bridge design.

I encourage everyone who uses this document to provide feedback and suggestions for revisions or future policy development. We've outlined the process for submitting suggestions to the Bridge Division, and I ask that we all actively engage in shaping what future policy could and should be. All input will be carefully considered, and revisions will be incorporated into the manual as appropriate.

I would like to extend a special thanks to Kyle Zillig for his vision and leadership in keeping the team organized and focused throughout the manual's development. Additionally, I want to thank Mike Vigil and Ben Ptacek for their efforts in developing the brand-new chapter on Repair and Preservation, and Matt Wieseler for his significant contributions to the development of the various chapters, and his dedication in formatting the manual. Lastly, a big thank you to Emilie Hudon, our newly appointed Bridge Policy and Quality Assurance Engineer, for her energy and leadership in driving us toward a final document in a remarkably short time. Your collective hard work and commitment have been invaluable throughout this evolution of the Bridge Design Manual.

Ross Barron, PE
State Bridge Engineer

Acknowledgments

The Bridge Division recognizes the efforts of the BDM Executive committee who helped to develop with the development of the 2024 BDM:

Emilie Hudon, PhD, PE	Bridge Policy and Quality Assurance Engineer
Matt Wieseler, PE, SE	Bridge Policy and Quality Assurance Assistant Engineer
Kyle Zillig, PE	Bridge Design Engineer
Mike Vigil, PE	Bridge Management Engineer
Kirk Harvey, PE	Bridge Hydrology and Hydraulics Engineer
Ross Barron, PE	State Bridge Engineer

Additional assistance was provided by the consulting firm Benesch, and their representative, Aaron Buettner, PE.

Revisions

The NDOT Bridge Design Manual provides current policies and procedures for use in structural design projects within Nebraska. To ensure that the manual remains up to date and appropriately reflects changes in NDOT's needs and requirements, its contents will be updated on a periodic basis.

Users of this manual should monitor the Bridge Division website for Policy Letters and revisions to the manual.

The Bridge Division is responsible for evaluating changes in structural engineering literature (e.g., updates to the AASHTO LRFD Specifications, issuance of new relevant publications, revisions to federal regulations) and for ensuring that these changes are appropriately addressed through revisions to the Bridge Design Manual. It is important that users of the manual inform NDOT of any inconsistencies, errors, need for clarification, or new ideas to support the goal of providing the best and most up-to-date information possible.

Our standard details can be found on ProjectWise. Please refer to these documents when making suggestions for drawing revisions.

Revisions to the Bridge Design Manual between major releases (interim revisions) are issued through Policy Letters. Requirements detailed in these memoranda are effective and enforceable on the effective date listed in the Letter and in the Transmittal Log located on the NDOT website.

To propose a revision to the Bridge Design Manual contact the Bridge Policy and Quality Assurance Engineer at emilie.hudon@nebraska.gov, be sure to include "Bridge Design Manual" in the subject line and provide the following:

- Manual section number(s)
- Proposed revision
- Justification for revision

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Chapter 1 — Introduction and Definitions

1.1—INTRODUCTION

The author of the Bridge Design Manual (BDM) is the Bridge Policy and Quality Assurance Section of the Bridge Division of Nebraska Department of Transportation (NDOT).

Any mention of "Bridge Division" in the text refers to NDOT Bridge Division.

1.2— FORMAT

The BDM is written in a 2-column format, where the left column represents the policy, and the right column represents commentary.

The terms "shall", "must", or "will" denote requirements for compliance with the policy. The term "should" indicates a strong preference for the criterion. The term "may" indicates a criterion that could be modified or adapted if local condition warrant different conditions.

In the few locations where the BDM is written in a 1-column format, all the text represents policy and there is no commentary

The commentary may include background information or any supplemental information that may be useful to the designer..

1.3—PUBLICATION AND MAINTENANCE

Following the first publication, the BDM will be updated on a yearly basis, with only the changes from the previous release being highlighted with change bars in the margins.

Interim revisions or policy changes will be issued through Bridge Policy Letters.

All documents posted online can be found on the Bridge Division's website. A contact for the BDM is also provided for any questions or comments.

The appendices to this document are up-to-date at the time of publication. For the most up-to-date base sheets and special provisions, see "NDOT Production" ProjectWise Datasource. It is the designers' responsibility to incorporate the most up-to-date base sheets and special provisions in their design.

Bridge Division documents are available on the website at: https://dot.nebraska.gov/business-center/bridge/.

Base sheets are located on the "NDOT Production" ProjectWise Datasource in the current workspace configuration standards folder. Special Provisions are located on the "NDOT Production" ProjectWise Datasource in the Standard Plans/Bridge folder.

1.4—PURPOSE AND AUDIENCE

The BDM establishes the policies and defines preferred practices for the Bridge Division for the structural design, analysis, and detailing of new and existing bridges and bridge sized culverts.

The BDM is intended to provide preferred practices based on local expertise and experience, while allowing designers to adapt the practices to individual site and design conditions. Contents are provided to promote consistency in design analysis, contract documents, and constructibility considerations.

Policies for bridge inspection and load rating can be found in NDOT Bridge Inspection Program Manual (BIP). Policies for hydraulic analysis can be found in NDOT Hydraulic Analysis Guidelines.

All bridge designs are the responsibility of the Engineer of Record; Bridge Division does not accept responsibility for any errors or oversights in the use of this manual.

Deviations from the policies herein shall be discussed with the Bridge Division and approved in writing during the design phase of the project.

It is important for the Bridge Division to be made aware of deviations from the policy so the intent of the modification can be understood, any implications related to maintenance and construction can be evaluated, and for on-going policy evaluation.

1.5—COORDINATION WITH OTHER DOCUMENTS

NDOT bridge design shall be in conformance with the following publications, listed in descending order of precedence in the case of discrepancy:

A. NDOT Bridge Policy Letters
B. NDOT Bridge Design Manual
C. NDOT base sheets, standard details, and standard cells, as can be found on "NDOT Production" ProjectWise Datasource
D. NDOT Hydraulic Analysis Guidelines 2015, NDOT Roadway Design Manual 10/2023, NDOT Standard/Special Plans Manual for Designers & Consultants 10/2024
E. NDOT Supplemental Specifications current at the time of contract
F. NDOT Standard Specifications for Highway Construction 2017
G. NDOT Construction Manual 2023
H. AASHTO LRFD Bridge Design Specifications and subsequent interims and AASHTO LRFD Bridge Construction Specifications and subsequent interims.

All NDOT Manuals and Standard Specifications are available on the NDOT website.

See §1.6 for current AASHTO Specifications.

For construction methods not clearly defined in the documents listed above, the plans and project special provisions must show enough details (description and instructions) to ensure clarity for construction and for contract administration.

During construction, the hierarchy of the contract documents in case of discrepancy is listed in the Standard Specifications (§105.04.1.b). Designers should note that special provisions will govern over plans.

1.6—CURRENT AASHTO SPECIFICATIONS

At the time of publication, the current edition of AASHTO Load and Resistance Factor Design Bridge Design Specifications is the 9th edition (AASTHO, 2020), with subsequent interims.

At the time of publication, the current edition of AASHTO Load and Resistance Factor Design Bridge Construction Specifications is the 4th edition (AASTHO, 2017), with subsequent interims

1.7—DEFINITIONS

The definitions in this section are meant to clarify local practices. Refer to LRFD BDS for definitions of common bridge terms.

- Approach Section — Part of the Approach Slab. It spans from the bridge deck to the grade beam.
- Approach Slab — A reinforced concrete slab which spans the distance between the bridge deck and roadway pavement
- Bent — Bridge supports between abutments that are supported on a single row of piles without a footing. Open pile bents were used on many short span structures but are not common anymore. Concrete encased pile bents are still common.

- Berm Elevation — The horizontal portion of the graded channel profile immediately adjacent to the abutment. Given to the top of the unarmored soil (bottom of riprap) or top of concrete slope protection.
- Bridge — A structure, including supports, erected over a depression or any obstruction, with a passageway for carrying traffic or other moving loads and has a length measured along the centerline of roadway of more than 20 feet.
- Bridge Determination — Planning document listing the description of the work to be completed on the structure. Developed by the Bridge Management Section of Bridge Division.
- Critical Berm — The soil line in front of the abutment, after considering the Q_{100} scour and channel behavior.
- Crown — high point of the roadway when the cross-section slopes to both shoulders. A superelevated cross-section does not have a crown
- Design Standard — Minimum standard of design, construction, and maintenance for the project, generally determined by Roadway Division. See Nebraska Administrative Code Title 428 “Rules and Regulations of the Board of Public Roads Classifications and Standards” for additional information.
- End of Floor — End of the bridge deck at grade
- Freeboard — The clear distance between low superstructure elevation and design high water elevation.
- Grade Beam — The concrete element supported on piling which supports the approach slab away from the end of floor
- Haunch — any thickening of a concrete section at a support OR, when discussion girders, the distance between the bottom of the deck and the girder. For NU girders and rolled steel girders, it is the distance to the top of the top flange. For welded steel plate girder, it is the distance to the top of the web (includes the top flange). Haunch is the nomenclature for this dimension during the design phase, while shim is used in the field for the same dimension.
- Integral Abutment — Abutments type that is placed on vertical piles with sufficient flexibility so that the superstructure can contract and expand with changes in temperature. Girder ends are cast into the abutment during construction. Allows the expansion joint to be moved to the grade beam.
- Maintenance — A planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, slows future deterioration, and maintains or improves the functional condition of the system without increasing the structural capacity. (Nebraska Department of Transportation, 2022)
- Minimum Bridge Grade — The lowest point of grade line elevation at the PGL between CL abutments used by the Hydraulic Section for the hydraulic study. Actual grade line profiles for the Bridge Design should use the minimum grade or higher in order to avoid decreasing the freeboard.
- New Structure — A new bridge on a new alignment.
- NU girder — I-shaped precast/prestressed concrete girder with wide bottom and top flanges.
- Ordinary High Water — The point at which natural vegetation shifts from predominately water dependent species to terrestrial species.
- OnBase — The states enterprise content management system, used for document management, business process automation and records management.
- Paving Section — Part of the Approach Slab. It spans from the grade beam to the roadway pavement
- Pier — Bridge supports between abutments that do not qualify as Bents.
- Preservation — The application of treatments at the proper time to prevent or correct the deterioration of an asset in order to extend its service life. (Nebraska Department of Transportation, 2022)
- Reconstructed Structure — A new bridge on an existing alignment
- Redeck — Replacement of the existing deck to the same clear roadway width. Girders can be made composite, continuous, or converted from pin and hanger. May also include partial superstructure replacement.
- Rehabilitated Structure — Replacement of an existing deck and all of the superstructure. This can include repairs to the existing substructure.
- Semi-integral Abutment — Abutment type that uses a concrete turndown at the girder ends and allows longitudinal movement (for expansion ends). Allows the expansion joint to be moved to the grade beam.
- Shims — See Haunch definition
- Substructure — All portions of the bridge below the girder bearing devices or below the slab and its haunches in the case of slab bridges.
- Sufficiency Rating — A rating factor between 1 and 100 that indicates the overall condition of the bridge structure.
- Superstructure — All portions of the bridge above and including the girder bearing devices or above the bottom of the slab in the case of slab bridges.
- Turndown — Concrete diaphragms encasing girder ends at the abutments. Used for the semi-integral abutment type
- Widen — Addition of a section of bridge that increases the bridge clear roadway width. This may include widening the existing substructure, adding a line of girders, or both.

1.8—QUALITY ASSURANCE/QUALITY CONTROL (QA/QC) PROCESS

Reserved for future use

1.9—OPERATIONAL IMPORTANCE OF BRIDGES

Bridges that meet critical or essential bridges criteria shall be indicated on the BDS and shall use the operational importance factor for design listed in LRFDBDS Article 1.3.5.

Critical or essential structures shall include, at a minimum, Missouri River crossings and other structures as determined by Bridge Division.

1.10—ABBREVIATIONS

- AASHTO — American Association of State Highway and Transportation Officials
- AC — Asphaltic Concrete
- AC+M — Asphaltic Concrete with a Waterproof Membrane
- AADT — Annual Average Daily Traffic
- ADTT — Average Daily Truck Traffic
- APL — NDOT Approved Products List
- ASTM — American Society for Testing and Material
- AWS — American Welding Society
- BDS — Bridge Data Sheet
- BDM — Bridge Design Manual
- BIP — Bridge Inspection Program
- CADD — Computer-Aided Design and Drafting
- CDP — Cotton Duck Pad
- CL — Centerline
- CIP — Cast-in-Place
- CIT — Concrete Inverted Tee
- CSB — Concrete Slab Bridge
- CPG — Concrete Prestressed Girder
- CY — Cubic Yard
- EJ — Expansion Joint
- EPO — Epoxy Polymer Overlay
- EOF — End of floor (end of bridge deck at grade)
- FHWA — Federal Highway Administration
- FRP — Fiber Reinforced Polymer
- GPR — Ground-penetrating Radar
- HDPE — High-Density Polyethylene
- HLMR — High Load Multi-Rotational
- HMA — Hot Mix Asphalt
- HPS — High Performance Steel
- LB — Pound
- LF — Lineal Feet
- LRFDBDS — Load and Resistance Factor Design Bridge Design Specifications. This refers to the current edition of the AASHTO LRFD Bridge Design Specifications. References to specific AASHTO LRFD sections will include the article number (e.g. LRFDBDS 11.10.5)

Article numbers refer to the current specification at the time of publication.

- MASH — Manual for Assessing Safety Hardware
- MGS — Midwest Guardrail System

- MTV — Material Transport Vehicle
- NBI — National Bridge Inventory
- NCHRP — National Cooperative Highway Research Program
- NDOT — Nebraska Department of Transportation
- npUHPC — Non-proprietary Ultra-High Performance Concrete
- NSBA — National Steel Bridge Alliance
- OCR — Open Concrete Rail
- OHW — Ordinary High Water
- P&H — Pin and Hanger
- PCAN — Precaster Association of Nebraska
- PCC — Portland Cement Concrete
- PCI — Precast/Prestressed Concrete Institute
- PGL — Profile Grade Line
- PPC — Polyester Polymer Concrete
- PPF — Precompressed Polyurethane Foam
- QA/QC — Quality Assurance/Quality Control
- RDM — Roadway Design Manual
- SCC — Self-Consolidating Concrete
- SF — Square Feet
- SHPO — State Historic Preservation Office
- SN — Surface Friction Number
- SPG — Steel Plate Girder
- SRB — Steel Rolled Beam
- SSCR — Single Slope Closed Rail
- SWWR — Steel Welded Wire Reinforcement
- Standard Specifications — NDOT Standard Specifications for Highway Construction

All references to Standard Specifications in the text refer to the current NDOT Standard Specifications. The current Standard Specifications at the time of publications are from 2017. Any references to sections refer to that edition.

- SY — Square Yard
- TM — Temperature Movement
- TS&L — The preliminary plan showing Type, Size, and Location of the structure
- TYP — Typical
- UHPC — Ultra-High Performance Concrete
- UNO — Unless Noted Otherwise
- VPD — Vehicles Per Day
- WPG — Welded Plate Girder
- WPS — Welding Procedure Specification
- WWR — Welded Wire Reinforcement
- 3R — Resurfacing, Restoration and Rehabilitation

1.11—REFERENCES

AASHTO. (2017). LRFD Bridge Construction Specifications (4th ed.). American Association of State Highway and Transportation Officials. LRFDCONS-4

AASHTO. (2020). LRFD Bridge Design Specifications (9th ed.). American Association of State Highway and Transportation Officials. LRFDBDS-9

Nebraska Department of Transportation. (2022). *Transportation Asset Management Plan*. https://dot.nebraska.gov/projects/publications/tamp/

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Chapter 3 — Loads and Load Factors

3.1—INTRODUCTION

The current AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications and applicable AASHTO Guide Specifications shall be the minimum design criteria used for all bridges except as modified herein.

Loads in this chapter shall be utilized in the design of bridges and other non-bridge sized structures.

3.2—LOAD FACTORS AND COMBINATIONS

The limit state load combinations, and load factors (γ_i) used for structural design shall be in accordance with the AASHTO LRFD Bridge Design Specifications Table 3.4.1-1 except as modified herein.

3.2.1—Strength Limit States

Ductility (η_D) and Redundancy (η_R) factors shall be taken as 1.0. Operational Classification (η_I) factor shall taken as 1.05 for critical or essential structures and 1.0 for all other structures. Refer to §1.9 for definition of critical or essential structure.

For all State bridges the load factor for vehicular live load (LL) and vehicular dynamic load allowance (IM) for the Strength I load combination shall be increased from 1.75 to 2.0 for superstructure primary member design except for deck slabs. For substructure, deck slabs, and superstructure secondary member design, the Strength I live load factor shall remain 1.75.

At the time when Nebraska was converting from the Standard Specifications for Highway Bridges to the LRFD Bridge Design Specifications, structures were designed using a HS25 truck in lieu of the required HS20. In order to continue providing similar superstructure sizes and to ensure new structure designs attain similar rating factors the γ_{LL} factor was increased for the Strength I load combination.

3.2.2—Extreme Event Load Combinations

Per LRFDBDS Article C3.4.1 $\gamma_{EQ} = 0.5$ unless analytically determined otherwise with Bridge Division Approval.

3.3—PERMANENT LOADS

3.3.1—Noncomposite Dead Loads (DC1)

Bridge deck, haunch, stay-in-place forms (5.0 psf), and diaphragms are considered a non-composite dead loads acting on the girders before concrete deck has cured.

3.3.2—Composite Dead Loads (DC2)

Railings and signage are considered superimposed dead loads distributed equally to all girders or distributed uniformly across concrete slabs.

In general, concrete barrier or rail loads are distributed equally to all girders for normal cantilever conditions. See Table 3.1 for weights of standard railing sections.

Interior Median Curbs (DC2): Interior median curbs will be designed and specified in the Plans as a composite load. In other words, a construction joint is mandatory (see details in Section §5.4.2).

Table 3.1—Rail Dead Loads

Rail Shape	Rail Type	Weight
39 in. SSCR	Closed	0.365 klf
39 in. OCR	Open	0.438 klf
42 in. NU	Open	0.441 klf
	Closed	0.524 klf
	Median	0.873 klf
34 in. NU	Open	0.373 klf
	Closed	0.455 klf
29 in. Nebraska	Open	0.270 klf
	Closed	0.382 klf
32 in. New Jersey Barrier	Closed	0.345 klf
42 in. New Jersey Barrier	Closed	0.413 klf

3.3.3—Future Surfacing and Utilities (DW)

Future surfacing is assumed to be 35 psf and shall be applied to all new superstructure and substructure designs.

Future surfacing loads shall only be included in the deflection calculations for shims if the overlay is constructed with the new bridge.

Bridges shall be analyzed both with and without future surfacing to ensure load envelopes are captured.

For typical bridge design no additional future utility load is required to be considered during design. If it is known utilities will be attached during construction or stated on the BDS, utility loads shall be accounted for during design.

3.4—LIVE LOADS

3.4.1—Pairs of Design Tandems

The pairs of design tandems described in LRFDBDS Article C3.6.1.3.1 shall be applied on all bridges that carry roadways of State Functional Classification Interstate or any structures with an ADTT not less than 5,000 vpd. ADTT shall be calculated per structure, for typical bidirectional structures use bidirectional ADTT. For ramps and other one-way structures use unidirectional ADTT.

State Functional Classification for each project can be found in the NDOT73 Planning Document submitted for each project with the Project Initiation Request (typically found on OnBase).

3.4.2—Live Load Deflection Limits for Steel and Concrete Superstructures

The optional deflection criteria in LRFDBDS Article 2.5.2.6.2 shall be mandatory on all State structures.

3.4.3—New, Reconstructed, and Rehabilitated Structures

New, reconstructed, and rehabilitated structures shall meet the requirements of AASHTO LRFD Bridge Design Specifications of HL93 live load.

See Chapter 1 for definitions of these terms.

3.4.4—Existing Superstructures

Existing superstructures shall meet the live load requirements below based on the classification of work being performed.

- Redeck, with or without additional widening: if feasible the structure shall be strengthened to HL93. If strengthening is not feasible the original design load shall be used.
- Widening of existing decks: original design load

If the original design load is unknown the following shall be assumed:

- State structures: HS20
- County and Municipal Structures: HS15

Structures may not be load posted after completion of work, unless they are County or Municipal Structures, and have a National Functional Classification of Local with ADT fewer than 400 VPD.

3.4.5—Existing Substructures

Where the superstructure design load is HL93, if feasible, the substructure shall be strengthened to HL93. If strengthening is not feasible, a load rating of the existing substructure shall be performed. If the existing substructure has adequate capacity to prevent load posting it may be used in place, if not it shall be replaced.

For all other superstructure design loads, the substructure design load shall the be original design load. If the original design load is unknown the following shall be assumed:

- State structures: HS20
- County and Municipal Structures: HS15

3.4.6—Dynamic Load Allowance

Dynamic Load Allowance shall be applied per LRFDBDS Article 3.6.2 with the clarification that abutment caps in contact with the soil are considered buried elements.

Scour is generally considered a temporary condition therefore designers need not include dynamic load allowance on piles being checked under scoured conditions.

3.4.7—Vehicular Collision Force

Where bridge intermediate supports are located within the clear zone Bridge Division's preferred method to address structure protection is to use heavy construction with pier columns having a minimum thickness of 3 ft. and a minimum cross-sectional area of 30 ft.^{2}. The distance from the groundline up the beginning of rustication on hammerhead piers may be increased to 5 ft. to achieve this while maintaining a column section less than 3 ft. thick at the rusticated portion of the column. See §11.2.2.3 for hammerhead pier details.

3.5—WATER LOADS

Uplift and lateral loading on submerged girders shall be accounted for on structures where the design flood inundates part or all of the superstructure.

When considering buoyant force the elevation of any vent holes provided in girder webs shall be considered.

Designers may want to investigate lateral loading on intermediate supports taking into account any accidental misalignment between the axis of the intermediate support and the flow direction of the water.

For bridge components below the vent hole elevation, only the structure area contributes to the buoyant force (shaded area in Figure 3.1). For bridge components above the vent hole elevation the entire area cross section including trapped air contributes to the buoyant force (cross hatched area in Figure 3.1).

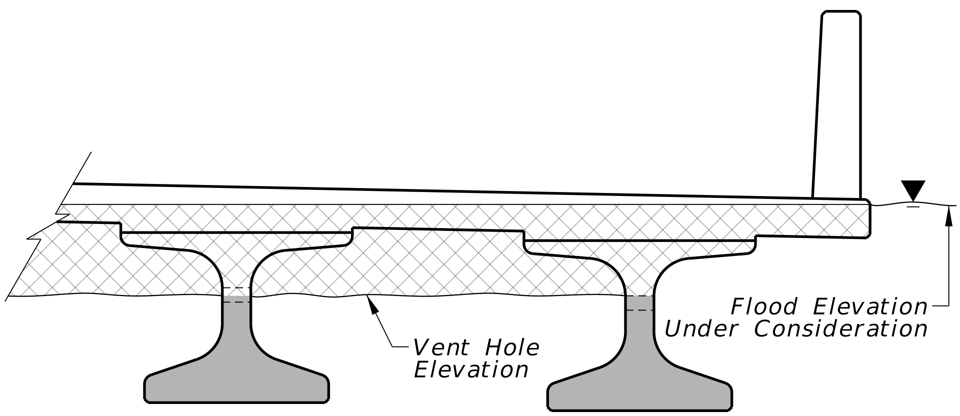

Figure 3.1—Buoyant Force Cross Section

3.6—WIND LOADS

Reserved for future use.

3.7—ICE LOADS

When ice loads are to be included in design (ice affected structure is checked on the Hydraulic Information Sheet), bridge hydraulics is to provide ice load parameters (thickness of ice and effective ice crushing strength).

Ice Load shall be evaluated as part of Extreme Event II Load combination with consideration of Q_{100} design scour in combination with ice load being applied at the midway point between OHW and Q_{100} water surface elevation.

For bridges not specified as Ice Affected this load need not be considered.

See Bridge Hydraulic Guidelines for more information regarding this topic.

3.8—EARTHQUAKE EFFECTS

Seismic analysis shall be carried out on every structure.

Most structures and sites in the State of Nebraska qualify for Seismic Zone 1 which entails only checking the connection between the superstructure and substructures per LRFDBDS Article 3.10.9.2.

Where reducing the lateral load specified in LRFDBDS Article 3.10.9.2 is desired, a Uniform Load Elastic Method analysis of the structure can typically be undertaken.

Typically the easiest method for determining Site Class is using the boring logs from the Geotechnical Section of Materials and Research Division. Using the logs the $\overline{N}$ Method detailed in LRFDBDS Table C3.10.3.1-1 can be performed.

3.9—EARTH PRESSURE

When local soil parameters are not given in the geotechnical report, a unit weight of γ = 125 pcf and an angle of internal friction of φ = 25° may be used.

For culverts with a rise of not less than 6 ft. a geotechnical report with local soil parameters is required.

3.10—SUPERIMPOSED DEFORMATIONS

3.10.1—Uniform Temperature

Force effects and movement resulting from uniform temperature change shall be included in the design of structures.

Other design policies may provide factors to modify thermal movement calculations.

Table 3.2—Uniform Temperature Parameters

Parameter	Superstructure Material	
	Concrete	Steel
α	6.0×10^{-6}/°F	6.5×10^{-6}/°F
$T_{MinDesign}$	−20°F	−20°F
$T_{MaxDesign}$	115°F	120°F
Design Temperature Rise	50°F	75°F
Design Temperature Fall	70°F	90°F

AASHTO Method "B" design temperatures can vary across the state but the values given may be used statewide.

3.10.1.1—Movement Calculation

Design parameters shall be as shown in Table 3.2 and be design thermal movement range shall be determined using the equation:

$$TM = \alpha \times L \times (T_{MaxDesign} - T_{MinDesign})$$

Where

TM = design thermal movement range

α = coefficient of thermal expansion

L = Length from point of no movement to the point at which the length of expansion is desired.

Design thermal movement range is principally used for sizing expansion devices.

See §4.5 for guidance on determining the point of no movement based on structure stiffness.

3.10.1.2—Force Effects Due to Temperature

Design force effects due to thermal movement shall be calculated separately for rise and fall using ranges from Table 3.2. Both shear and flexure deflections shall be used in determining force effects due to enforced deflections.

In lieu of the 65% of total TM allowed in LRFDBDS Article 14.7.5.3.2 Bridge Division has chosen to specify rise and fall temperature for use in determining forces induced by thermal movement and design displacement on bearing devices. This also prevents having to reset bearings if the bridge is erected at a temperature far from its neutral temperature.

3.10.2—Shrinkage and Creep

For routine structure design, in lieu of a more precise analysis, shrinkage and creep of precast/prestressed girders may be assumed as 0.0003 ft./ft. This assumption does not apply to post-tensioned or other complex structures which should be individually analyzed per LRFDBDS Articles 5.4.2.3.2 and 5.4.2.3.3.

3.11—FRICTION FORCES

For bearings that utilize a PTFE sliding surface, design coefficients of friction shall be per LRFDBDS Table 14.7.2.5-1. Standard Specifications specify the use of an unfilled flat PTFE surface.

At fixed supports the larger of the direct friction force and the unbalanced friction force between all supports shall be used for design.

3.12—VESSEL COLLISION

For major structures on the Missouri River, vessel collision loads shall be determined via a site specific study.

3.13—BLAST LOADING

Structures where blast resistance is required shall be denoted on the BDS.

Where blast loading analysis is required, consultation with a specialty engineer specializing in blast loading is mandatory to determine appropriate explosive characteristics.

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Chapter 5 — Concrete

5.1—OVERVIEW

This chapter applies to the design of both cast-in-place and precast concrete structures, both traditionally reinforced as well as prestressed.

5.2—MATERIALS

5.2.1—Concrete

Standard concrete classes and compressive strengths are summarized in Table 5.1.

Table 5.1—Standard Concrete Classes and Compressive Strengths

Structural Element	Concrete Class	Minimum 28 day Compressive Strength
Abutments, Grade Beams, Piers, Bents, and Footings	47B	3.0 ksi
Drilled Shafts	47B	4.0 ksi
Bridge Decks and Rails	47BD	4.0 ksi
CIP Concrete Box Culverts	47B	4.0 ksi
Precast Concrete Box Culverts	See Special Provisions	5.0 ksi
Prestressed Concrete Girders	See Standard Specifications	See §5.2.1.1.3

See following sections for additional details.

5.2.1.1—Strength of Concrete

Specification of f'_c shall be in 0.1 ksi increments for precast elements and 0.5 ksi increments for cast-in-place applications.

5.2.1.1.1—CIP Concrete Superstructures

Since conditions for placing and curing concrete for CIP components are not as controlled as they are for precast bridge components, 4.0 ksi concrete is typically used. Where significant economy can be gained or structural requirements dictate, 5.0 ksi concrete may be used with the approval of the Bridge Design Section.

5.2.1.1.2—CIP Concrete Substructures

3.0 ksi for abutments, grade beams, bents, piers, and footings

4.0 ksi for drilled shafts

Where significant economy can be gained or structural requirements dictate, up to 5.0 ksi concrete may be used with the approval of the Bridge Division.

5.2.1.1.3—Prestressed Concrete Girders

Nominal 28-day concrete strength, f'_c, for prestressed concrete girders is 8.0 ksi (with concrete strength at release , f'_{ci}, of 6.0 ksi). Where higher strengths are required, f'_c, of 10.0 ksi can be specified (with concrete strength at release , f'_{ci}, of 7.5 ksi). Variance from these two strengths requires Bridge Division approval. The specified concrete strengths for prestressed concrete girders shall be shown in the plans on the girder data sheet. Designers should specify the lowest concrete strengths (8.0 or 10.0 ksi) necessary to meet allowable stress limits, strength requirements, and these provisions.

The concrete strength limits have been chosen to minimize costs associated with high strength mixes and extended curing times. Designers should note that higher release strengths may require extended cure times in addition to providing less predictable camber and strength values. The $0.75f'_c$ release strength limit is based on industry recommendations from PCAN as well as information from PCI. The intent is to have a practical ratio between f'_{ci} and f'_c and to provide more a more reliable prediction of final compressive strength.

Where strand patterns vary significantly along the length of a structure, using differing concrete strengths span to span should be investigated.

Designers should also note that the strength design at release method, as described in Hanna et al. (2010), has been found to create cracking in the girders and is therefore not allowed anymore.

5.2.1.1.4—CIP Concrete Box Culverts

4.0 ksi concrete will be used for the design of barrels, headwalls, and wingwalls.

5.2.1.1.5—Precast Concrete Box Culverts

5.0 ksi concrete will be used for the design of barrels

5.2.1.2—Classes of Concrete

See Standard Specifications for specific mix design.

Class 47B: Used in CIP bridge substructure and culverts.

Class 47BD: Used in CIP bridge superstructure and concrete box culverts when the top slab is used as the driving surface.

For prestressed concrete girders, the producers are responsible for the mix design, which means there is no concrete class associated with this material.

For precast concrete box culverts, the producers are responsible for the mix design, which means there is no concrete class associated with this material. See “PRECAST CONCRETE BOX CULVERT” Special Provision for more details.

5.2.1.3—Unit Weight

Unreinforced concrete unit weight shall be calculated based on the specified minimum compressive strength and the formula shown in Table 3.5.1-1 of the LRFDBDS.

For plant produced precast girders 0.01 kcf shall be added to the unreinforced concrete density to account for the weight of reinforcement.

For other mildly reinforced concrete 0.005 kcf shall be added to the unreinforced concrete density to account for the weight of reinforcement.

5.2.1.4—Modulus of Elasticity

The modulus of elasticity shall be determined as specified in Chapter 5 of the LRFDBDS. The correction factor K_1 shall be taken as 0.975.

In NCHRP Report 496 the authors calibrated K_1 based on aggregates from four different states. For Nebraska aggregates, K_1 was found to be 0.975 (Tadros et al., 2003a).

5.2.1.5—Shrinkage and Creep

Losses due to shrinkage and creep shall be calculated in accordance with the refined method provided in Chapter 5 of the LRFDBDS. The relative humidity, H may be taken as 70% for standard conditions. The age at time of deck placement, t_d, may be taken as 56 days. The maturity of concrete, t, may be taken as 27,000 days for standard conditions. In determining the maturity of concrete at initial loading, t_i, one day of accelerated curing by steam or radiant heat may be taken as equal to seven days of normal curing.

70% humidity was chosen as girders are produced on the east end of the state. 27,000 days is based on a 75 year design life for the structure.

5.2.1.6—Mass Concrete

Mass concrete is any volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat from hydration of the cement to minimize cracking. Temperature related cracking may be experienced in thick-section concrete structures, including spread footings, pile caps, bridge piers, thick walls, and other structures as applicable. Concrete placements with the least dimension greater than 5 feet shall be considered mass concrete. Drilled shafts need not be considered mass concrete. Designers should be aware of the cost of monitoring these temperature effects and try to keep the least concrete dimension below these limits. Bridges with components that meet these criteria shall include the Special Provision “MASS CONCRETE PLACEMENT” in the contract documents.

5.2.1.7—Self-Consolidating Concrete

Self-consolidating concrete may be used in structural members such as precast prestressed concrete girders as described in the Standard Specifications.

SCC may be specified for cast-in-place applications where the use of conventional concrete could be challenging and problematic. Examples are where new concrete is being cast up against an existing soffit, or in members with very dense/congested reinforcing steel. See “SELF-CONSOLIDATING CONCRETE” Special Provision for more details.

Since the early 2000s Nebraska's prestressed concrete girders have been produced almost exclusively with self-consolidating concrete (Lafferty, 2008). Any concrete mixtures for prestressed girders are qualified by NDOT Materials And Research Division. Refer to Section 705 of the Standard Specifications for Mix Design Approval and Changes.

5.2.1.8—Ultra-High Performance Concrete

Design shall be performed based on *Guide Specifications for Structural Design with Ultra-High Performance Concrete* (AASHTO, 2024).

Possible applications of UHPC on projects can be discussed with Bridge Division.

Some examples of work where UHPC is a good material candidate are:

- Joint headers (curing duration needs to be considered for joint headers)
- Link slabs
- Girder end repairs
- Precast connections
- Bearing pedestal retrofits
- Column Encasements

Designers shall specify NDOT npUHPC for precast, CIP, or overlay applications. Proprietary mix alternates will be allowed with Bridge Division approval.

Non-proprietary UHPC mixtures from local material sources have been developed by NDOT in conjunction with UNL. These mixes greatly reduce the cost of UHPC when compared to proprietary mixes, for more information, see Hu et al. (2023) and Morcous et al. (2023). Research is on-going for overlay application.

5.2.2—Mild Reinforcement

Reinforcing bars shall be deformed and conform to the Standard Specifications. Standard reinforcement for CIP concrete shall be mild steel with a yield strength of 60 ksi produced in accordance with ASTM A615. Higher yield strength material may be used with the approval of the Bridge Division, where justified.

High strength reinforcement requires special provisions added to the project.

All steel reinforcement used in drilled shafts, prestressed girders, and concrete pile shall be uncoated. Box culverts shall have epoxy coated reinforcing bars when the top surface is used as a driving surface, otherwise they shall use uncoated steel. All other steel reinforcing bars shall be epoxy coated.

Deformed welded wire reinforcement (WWR) used in prestressed concrete girders shall have a minimum yield strength of 70 ksi and be produced in accordance with ASTM A1064 or AASHTO M336. ASTM A615 Gr. 60 reinforcing substitution may be permitted when requested by the fabricator on a per case basis.

Preferred wire sizes of WWR are listed in the sections for each girder shape. This does not preclude the use of other sizes, but verification should be done to ensure fabricators can procure desired wire sizes in a timely and cost effective manner. Fabricators may desire to use a larger cross wire than specified in the base sheets in some cases due to ease of fabrication, this is acceptable.

Smooth welded wire reinforcement used in precast concrete culverts and concrete slope protection shall have a minimum yield strength of 65 ksi and be produced in accordance with ASTM A1064 or AASHTO M 336

5.2.3—Prestressing Strands

Prestressing strands shall be ASTM A416 uncoated, seven-wire low-relaxation steel with an ultimate strength of 270 ksi. Diameters of strand depend on the type of prestressed girder being designed. See §§5.5.2 and 5.5.4 for more information.

Stress relieved strands are not permitted.

5.3—CLEAR COVER

The clear cover for reinforcement in CIP concrete is mostly covered by standard note #046, cases not covered by that note are listed below and shall be detailed on the plans as such.

- CIP Slab Bridge Superstructures and Decks
  - Bottom — 1 in. (+1/4 in./-0 in.)
  - Top — 2 1/2 in. (±1/4 in.)
  - Sides — 3 in.
- CIP Approach Slabs
  - Top — 2 1/2 in. (+1/4 in./-0 in.)
  - Bottom and Sides — 3 in.
- Reinforcement projecting from slabs/decks into rails shall be sized for 3 in. clear cover from the top of the rail.

All plan sets containing CIP concrete work shall include standard note #046.

For precast concrete clear covers see the base sheets for each girder template.

5.4—CAST-IN-PLACE CONCRETE

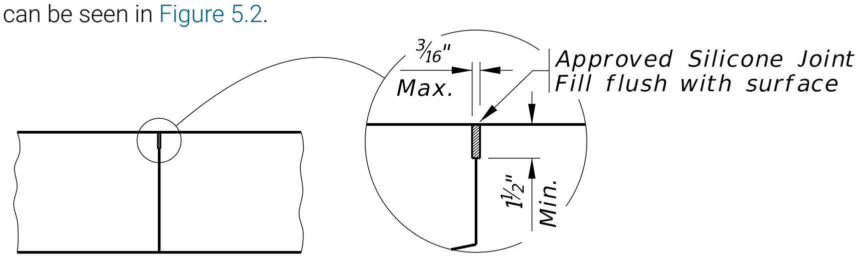
5.4.1—General Information

Cast-in-place concrete is currently the predominant material used in Nebraska for substructures and decks. This section provides information and guidance on detailing and design practices.

5.4.2—Construction Joints

Slab construction joints less than or equal to 8 ½ in. thick shall be indicated as a vertical line in the plans. A cell is available in the bridge deck library and can be seen in Figure 5.1.

Slab construction joints greater than 8 ½ in. thick shall provide a shear key. A cell is available in the bridge deck library and can be seen in Figure 5.2.

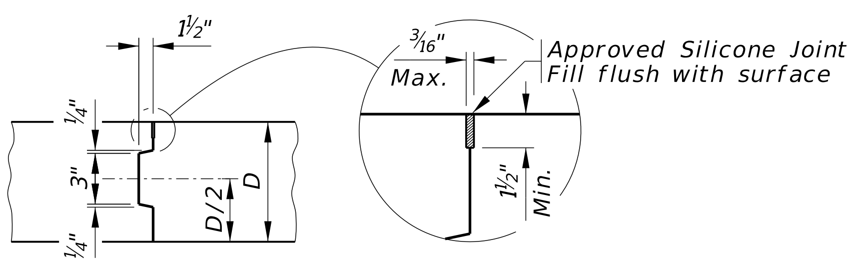


NOTES:
The Contractor shall prepare and seal the joint according to the manufacturer's recommendations.
Before sealing, the joint wall surfaces shall be sandblasted to remove any loose particles and concrete dust.
After sandblasting, the entire joint shall be cleaned with compressed air having a minimum pressure of 90 psi. The compressed air shall be free of any contaminants.
The joint shall be dry at the time of sealing.

SLAB CONSTRUCTION JOINT

Not to Scale

Figure 5.1—Slab Construction Joint for Slab Thickness ≤ 8.5 in.
Available as Cell "JOINT"



NOTES:
The Contractor shall prepare and seal the joint according to the manufacturer's recommendations.
Before sealing, the joint wall surfaces shall be sandblasted to remove any loose particles and concrete dust.
After sandblasting, the entire joint shall be cleaned with compressed air having a minimum pressure of 90 psi. The compressed air shall be free of any contaminants.
The joint shall be dry at the time of sealing.

SLAB CONSTRUCTION JOINT

Not to Scale

Figure 5.2—Slab Construction Joint for Slab Thickness > 8.5 in.
Available as Cell "JOINT2"

All joints listed in §§5.4.2.1 and 5.4.2.2 shall be detailed on the plans and listed as mandatory or optional as necessary.

All construction joints shown in the plans are mandatory unless explicitly stated as optional.

5.4.2.1—Mandatory Joints

The following is a list of locations where mandatory construction joints are required and shall be detailed in the plans:

- Horizontally:
  - Semi-integral abutments: In the deck turndown at the abutment, horizontally between the elevation of the approach slab seat and the bottom of the chamfer of the deck.
  - Concrete diaphragms at intermediate substructure elements: at ²⁄₃ of the girder height
  - Intermediate concrete diaphragms: Top of diaphragm (bottom of deck)
  - Interior median curbs: Top of deck (bottom of curb)
  - Permanent casing on drilled shaft: At the top
  - Box culvert walls: At the top of the slab (bottom of walls)
- Vertically:
  - End of floor to approach slab. May be replaced with optional non-corroding plate to force bond break if poured continuously
  - Longitudinally from End of Paving to End of Paving for phased construction.

The first two joints are to provide girders necessary lateral restraint prior to deck placement.

Joint at EOF is to allow some rotation of the approach slab seat

5.4.2.2—Optional Joints

The following list of locations where optional construction joints are to be detailed in the plans:

- Horizontally:
  - Box culvert walls: 2 in. below the bottom of the top slab
  - Box culvert turndowns (barrel and wing footings): at the top, or slightly below the top, of the turndown (bottom of bottom slab elevation)
- Vertically:
  - Transverse deck joints: per pouring diagram and parallel to the supports

This limits the allowable locations of the transverse joints in the slab if the Contractor needs to stop the pour for any reason, or if skip pouring is used. Refer to §9.2.5 for more details.

5.4.3—Concrete Slab Bridges

Concrete slab bridges are a very long lasting and robust structural system, they often last long enough that they are replaced due to functional obsolescence rather than structural deficiency. For the lengths of bridges shown in Table 5.2 it is common for a three-span slab bridge to be approximately 6 in. shallower than an equivalent three-span prestressed concrete girder bridge.

Designers should note that concrete slab spans over 50 ft. require the shoring to be designed by a Professional Engineer. Therefore, using spans of 50 ft. and shorter is preferred in order to reduce costs.

5.4.3.1—Slab Bridge Design Assumptions

The slab bridge design information contained in Table 5.2, Table 5.3, and Table 5.4 may be used to provide steel reinforcement for the bridge lengths and spans specified. The table can also be used for bridges of the same length and span with lesser width or loads. These tables were developed in accordance with the LRFDBDS, 9th Ed. The following criteria were assumed in developing the slab design table:

- Superimposed Dead Loads
  - 42 in. NU Open Concrete Rail = 441 plf (DC)
  - Future Surfacing = 35 psf (DW).
- HL93 Live Load without interstate or expressway traffic. See §3.2 for guidance on load combinations.
- Edge beam distribution factor is used across the entire width of the bridge.
- The effective design depth determined by assuming 1/2 in. for a sacrificial wearing surface.
- f'_c = 4 ksi concrete
- f_y = 60 ksi reinforcement, epoxy coated
- Design Clear Roadway Width = 44 ft.
- Concrete clear cover per §5.3.
- Skew Angle: 0°
- Haunch Depth: 6 in. @ abutments.
- J4 bars are continuous through Span 2 for bridge lengths 60 and 65 ft.

The pairs of design tandems required in §3.4.1 were not used in the slab bridge analysis.

Skews decrease the magnitude of the applied moment so the most conservative case was used.

5.4.3.2—Concrete Slab Base Sheet

The Concrete Slab Base Sheets provides typical reinforcement layouts for zero, ≤30° and >30° skewed bridges. The base sheets are to be coordinated with Table 5.2, Table 5.3, and Table 5.4. Designers must replace the reinforcement designations shown on the base sheet (J2 through J8) with standard bar marks and the haunch information from §5.4.3.3.

Transverse distribution steel shall be as listed here and as shown on the base sheet:

- Top reinforcement shall be #4 bars at 12 in. centers.
- Bottom reinforcement shall be #5 bars at 12 in. centers.
- For skews not greater than 30°, place transverse distribution steel parallel to the CL of the substructure elements.
- For skews greater than 30°, place transverse distribution steel perpendicular to the CL of roadway in the following manner:
  - Transverse top and bottom reinforcement will require bar sets only at the End of Floor.
  - Additional longitudinal steel in the bottom layer (#4 bars @ 24 in. centers) will extend through the haunch to allow the transverse steel in the bottom layer to be placed continuously over the haunch without bar sets at the intermediate supports.

Table 5.2 shall be used to determine individual span lengths, slab thickness at midspan, and the location of the optional construction joint each side of the intermediate supports based on overall bridge length.

Table 5.2—CSB Dimensions

Bridge Length	Span No. 1 & 3	Span No. 2	Slab Thickness	X
60'	18'-0"	24'-0"	11 1/2"	6'-3"
65'	19'-6"	26'-0"	11 1/2"	6'-8"
70'	21'-0"	28'-0"	12"	7'-1"
75'	22'-6"	30'-0"	12 1/2"	7'-6"
80'	24'-0"	32'-0"	13"	7'-11"
85'	25'-6"	34'-0"	13 1/2"	8'-3"
90'	27'-0"	36'-0"	14"	8'-8"
95'	28'-6"	38'-0"	14 1/2"	9'-6"
100'	30'-0"	40'-0"	15"	10'-0"
105'	31'-6"	42'-0"	15 1/2"	10'-4"
110'	33'-0"	44'-0"	16"	11'-0"
115'	34'-6"	46'-0"	16 1/2"	11'-4"
120'	36'-0"	48'-0"	17"	11'-9"
125'	37'-6"	50'-0"	17 1/2"	12'-2"
130'	39'-0"	52'-0"	18"	12'-6"
135'	40'-6"	54'-0"	19"	12'-10"
140'	42'-0"	56'-0"	19 1/2"	13'-2"

Table 5.3 shall be used to determine reactions for substructure design based on overall bridge length.

Table 5.3—CSB Reactions at Abutments and Intermediate Supports*

Table 5.3a—Service I Reactions (Unfactored)

Bridge Length	DC Reaction k/ft. of width		DW Reaction k/ft. of width		LL Reaction kips per lane No Dynamic Allowance	
	Abutments	Intermediate Supports	Abutments	Intermediate Supports	Abutments	Intermediate Supports
60'	1.08	4.44	0.24	0.87	47.47	74.21
65'	1.17	4.77	0.26	0.94	48.47	77.46
70'	1.30	5.27	0.28	1.01	49.42	80.35
75'	1.44	5.80	0.30	1.08	50.29	82.97
80'	1.59	6.34	0.32	1.15	51.11	85.44
85'	1.75	6.92	0.34	1.22	51.88	87.68
90'	1.84	8.02	0.34	1.30	52.38	91.22
95'	2.01	8.64	0.36	1.37	53.30	93.14
100'	2.19	9.29	0.38	1.44	54.92	94.98
105'	2.37	9.97	0.40	1.51	56.44	96.77
110'	2.55	10.66	0.42	1.58	57.88	98.49
115'	2.75	11.39	0.44	1.65	59.24	100.16
120'	2.95	12.13	0.46	1.72	60.56	101.82
125'	3.16	12.90	0.48	1.79	61.84	103.43
130'	3.37	13.70	0.50	1.86	63.06	105.00
135'	3.68	14.85	0.52	1.93	64.24	106.48
140'	3.91	15.71	0.54	2.00	65.37	107.99

Table 5.3b—Strength I Reactions (Factored)

Bridge Length	DC Reaction k/ft. of width		DW Reaction k/ft. of width		LL Reaction† kips per lane No Dynamic Allowance		LL Reaction† kips per lane With Dynamic Allowance	
	Abutments	Intermediate Supports	Abutments	Intermediate Supports	Abutments	Intermediate Supports	Abutments	Intermediate Supports
60'	1.35	5.55	0.36	1.30	83.08	129.86	107.52	163.19
65'	1.46	5.96	0.39	1.40	84.83	135.55	109.60	169.98
70'	1.63	6.59	0.41	1.51	86.48	140.61	111.55	175.95
75'	1.81	7.24	0.44	1.62	88.01	145.19	113.33	181.28
80'	1.99	7.93	0.47	1.72	89.44	149.52	114.98	186.29
85'	2.18	8.65	0.50	1.83	90.78	153.44	116.50	190.75
90'	2.30	10.02	0.52	1.95	91.67	159.64	117.46	197.98
95'	2.52	10.80	0.54	2.06	93.28	163.00	119.35	201.69
100'	2.73	11.62	0.57	2.16	96.12	166.22	122.87	205.21
105'	2.96	12.46	0.60	2.27	98.78	169.35	126.16	208.62
110'	3.19	13.33	0.63	2.37	101.28	172.35	129.24	211.86
115'	3.44	14.23	0.66	2.48	103.66	175.28	132.14	215.01
120'	3.69	15.17	0.69	2.58	105.98	178.18	134.96	218.10
125'	3.95	16.13	0.72	2.69	108.23	181.00	137.71	221.10
130'	4.21	17.12	0.75	2.79	110.36	183.75	140.30	224.00
135'	4.60	18.56	0.78	2.90	112.43	186.33	142.78	226.70
140'	4.89	19.63	0.81	3.00	114.40	188.97	145.15	229.44

* Reaction values given are for the bridge only and do not include any contribution from approach slabs.

† These reactions are intended for use in substructure design therefore they utilize 1.75 for γ_{LL} when factoring.

Table 5.4 shall be used to determine longitudinal reinforcement size and spacing based on overall bridge length.

Table 5.4—CSB Reinforcement

Table 5.4a—Top of Slab - Negative Steel Reinforcement

Bridge Length	At Intermediate Supports								Span 1 & 3	Span 2
	Longitudinal		J5 Bars			J4 Bars			S510	S405
	Size	Spacing	Length	L1	L3	Length	L2	L4	Length	Length
60'	#7	11"	17'-9"	9'-4"	8'-5"	51'-10"*	13'-11"		8'-7"	
65'	#7	11"	18'-8"	9'-4"	9'-4"	55'-4"*	14'-8"		9'-7"	
70'	#7	11"	19'-8"	9'-10"	9'-10"	31'-2"	15'-7"	15'-7"	9'-10"	8'-8"
75'	#7	11"	21'-0"	10'-9"	10'-3"	32'-2"	16'-2"	16'-0"	20'-9"	8'-8"
80'	#7	10"	21'-9"	11'-11"	9'-10"	32'-10"	16'-11"	15'-11"	11'-3"	8'-2"
85'	#7	10"	22'-7"	12'-4"	10'-3"	34'-0"	17'-8"	16'-4"	13'-0"	7'-9"
90'	#7	10"	24'-1"	11'-11"	12'-2"	37'-1"	18'-8"	18'-5"	12'-3"	6'-9"
95'	#7	10"	26'-7"	14'-0"	12'-7"	38'-6"	19'-11"	18'-7"	13'-2"	6'-1"
100'	#8	12"	26'-8"	13'-10"	12'-10"	41'-6"	21'-1"	20'-5"	12'-7"	6'-11"
105'	#8	11"	27'-9"	14'-8"	13'-1"	42'-9"	21'-11"	20'-10"	14'-0"	7'-8"
110'	#8	11"	29'-6"	15'-10"	13'-8"	44'-6"	23'-2"	21'-4"	15'-3"	7'-11"
115'	#8	10"	29'-8"	16'-4"	13'-4"	45'-2"	23'-9"	21'-5"	14'-9"	9'-1"
120'	#8	10"	31'-1"	16'-11"	14'-2"	46'-7"	24'-9"	21'-10"	16'-1"	12'-2"
125'	#9	11"	35'-7"	19'-8"	15'-11"	50'-8"	27'-0"	23'-8"	17'-4"	12'-5"
130'	#9	10"	28'-6"	14'-3"	14'-3"	48'-6"	24'-10"	23'-8"	18'-6"	13'-2"
135'	#9	10"	29'-2"	14'-7"	14'-7"	48'-11"	25'-1"	23'-10"	20'-11"	13'-5"
140'	#9	10"	30'-3"	15'-4"	14'-11"	49'-9"	25'-7"	24'-2"	22'-4"	14'-2"

Table 5.4b—Bottom of Slab - Positive Steel Reinforcement

Bridge Length			Span 1 & 3		Span 2		Haunches
	Longitudinal		J3 Bars	J2 Bars	J6 Bars	J7 Bars	J8 Bars
	Size	Spacing	Length	Length	Length	Length	Size
60'	#7	11"	18'-4"	13'-5"	23'-10"	14'-2"	#6
65'	#8	12"	20'-4"	14'-4"	26'-10"	14'-10"	#6
70'	#8	12"	21'-9"	15'-9"	28'-6"	16'-6"	#6
75'	#8	11"	22'-8"	16'-8"	29'-4"	17'-4"	#6
80'	#8	11"	24'-0"	18'-0"	31'-0"	19'-0"	#6
85'	#8	10"	24'-11"	18'-11"	31'-8"	19'-8"	#6
90'	#8	10"	25'-6"	19'-6"	31'-9"	19'-9"	#6
95'	#8	10"	26'-10"	20'-10"	33'-3"	21'-3"	#6
100'	#9	12"	29'-2"	21'-10"	36'-11"	22'-3"	#7
105'	#9	12"	30'-5"	23'-1"	38'-6"	23'-10"	#7
110'	#9	11"	31'-2"	23'-10"	39'-1"	24'-5"	#7
115'	#9	11"	32'-5"	25'-1"	40'-8"	25'-11"	#7
120'	#9	10"	33'-0"	25'-8"	41'-1"	26'-6"	#7
125'	#9	11"	34'-11"	27'-8"	43'-8"	29'-0"	#7
130'	#9	10"	35'-6"	28'-2"	44'-0"	29'-4"	#7
135'	#9	10"	36'-7"	29'-3"	45'-3"	30'-7"	#7
140'	#9	10"	37'-10"	30'-6"	46'-10"	32'-2"	#7

* Bar is continuous through Span 2

5.4.3.3—Haunch Detail Over Intermediate Supports

5.4.3.3.1—Bridge Lengths of 60 ft. to 85 ft.

- Haunch Depth = 6 in.
- Haunch Length = 4 ft.
- Place five (5) S501 bars at the face of each haunch.

Table 5.5—4 ft. Haunch Bill of Bars

Location	Mark	No.	Length	Type	A	B	C	D	E
J8		Varies	14'-0"	106	6'-0"	2'-0"	9"	9"	6'-0"
Haunch	S404	8	14'-0"	STR					

5.4.3.3.2—Bridge Lengths of 90 ft. to 140 ft.

- Haunch Depth = 9 in.
- Haunch Length = 6 ft.
- Place seven (7) S501 bars at the face of each haunch.

Table 5.6—6 ft. Haunch Bill of Bars

Location	Mark	No.	Length	Type	A	B	C	D	E
J8		Varies	18'-0"	106	8'-0"	2'-0"	1'-0"	1'-0"	8'-0"
Haunch	S404	8	18'-0"	STR					

5.5—PRECAST PRESTRESSED CONCRETE GIRDERS

5.5.1—General Information

5.5.1.1—Girder Templates

NU or IT girders shall be the preferred shapes. For widenings, designers shall check deflections when widening with a different shape. The use of AASHTO Sections requires approval from Bridge Division.

The precast concrete girder fabricators in the State of Nebraska have forms that match the dimensions of the NU and IT girders described later in this chapter. Due to increased fabrication cost, transportation cost, or both stemming from using different shapes, NU and IT shapes are the most cost-effective sections for Bridge Division.

5.5.1.2—Composite Section Properties

Haunch depth shall be ignored when determining composite section properties during design.

5.5.1.3—Service III Load Combination

Stress analysis based on transformed section properties is discouraged and should be avoided.

Using approximate prestress losses is permitted for preliminary design only. Refined analysis for prestress losses shall be used for final design.

The live load factor for the Service III Load Combination shall be:

- γ_{LL} = 1.0 when one or more of the following are true:
  - transformed section properties are utilized for stress analysis
  - elastic gains are taken into account for stress analysis
  - specified concrete compressive strength is greater than 10 ksi
- γ_{LL} = 0.8 when none of the above limitations apply

The 2016 Interim Revisions to the LRFDBDS and the AASHTO Manual for Bridge Evaluation have made the Service III load factor for live load dependent upon the method by which prestress losses are estimated. The purpose of this change was to maintain a uniform level of reliability against cracking of prestressed concrete components as explained in LRFDBDS C3.4.1.

The conservative nature of the design requirements defined in the BDM provide sufficient protection against cracking and adequate long-term performance of prestressed concrete superstructure components.

Designers should note that when changing from approximate to refined analysis of prestress losses the Precast/Prestressed Girder module of LEAP Bridge Concrete typically overwrites the γ_{LL} load factor for the Service III load combination to 1.0.

Tensile stresses in the bottom of the girder at the maximum positive moment location shall not exceed $0.095\sqrt{f'_c}$ nor 0.3 ksi without Bridge Division approval.

As Nebraska is a state with significant road salt application in the winter, tensile stresses in girders should be controlled to prevent corrosion and increase service life of prestressed structures. There is also concern of service limit states and their calibration to a uniform probability of exceedance (National Academies of Sciences, Engineering, and Medicine et al., 2014). Therefore Bridge Division has chosen to limit service limit state stresses until further research is completed.

5.5.1.4—Strands

Strands at the ends of girders that will not be encased in a concrete diaphragm or turndown shall be recessed, burnt back, and patched with cementitious grout. Notes to the fabricator shall be added to the girder data sheet detailing this condition.

Strands that are extended into the concrete diaphragm shall not be debonded.

The standardized position of the strands in the each girder template is given in their respective sections.

5.5.1.4.1—Extended Strands

Restraint moments (LRFDBDS-9, Article 5.12.3.3.2) do not need to be calculated if prestressing strands are extended and anchored into the turndown/diaphragm following the base sheet details.

Extended strands shall be projected 6 in. beyond the end of the girder and bent using two 45° bends. See base sheets for details.

The 6 in. strand extension is in violation of LRFDBDS Article 5.12.3.3.9c. This number is used based on real world observations, behavior, performance and surrounding state DOTs policies (Tadros & Jongpitaksseel, 2003).

5.5.1.4.2—Deflected Strands

The strands in Inverted Tee girders cannot be deflected. The strands in NU girders may be deflected see §5.5.2.2 for more information.

5.5.1.4.3—Partially Debonded strands

Where it is necessary to prevent a strand from actively supplying prestress force near the end of a girder, it shall be debonded. This can be accomplished by taping a close-fitting tube to the stressed strand from the end of the girder to some point where the strand can be allowed to develop its load. Debonding limits and strands shall be carefully indicated on the plans according to the design standard sheet. Permitted debonding locations can be seen in the section for each girder template.

It is important when this method is used in construction that the taping of the tube is done in such a manner that concrete cannot leak into the tube and provide an undesirable bond of the strand

5.5.1.4.4—Temporary Strands

Temporary strands in the top flanges of prestressed concrete girders may be required for shipping at the discretion of the precaster and the consent of the design engineer. These strands may be pretensioned and bonded only for the end 10 feet of the girder or may be post-tensioned prior to lifting the girder from the form. These strands can be considered in design to reduce the required transfer strength and to provide stability during shipping. These strands must be cut before the CIP end diaphragms are placed or a maximum of 10 days after stressing, whichever comes first. It is the responsibility of the precaster to state on the shop plans the method and sequence of post-tensioning, and the method and sequence of cutting the temporary strands.

5.5.1.5—Non-Prestressed Reinforcement

5.5.1.5.1—Transverse and End Zone Reinforcement

Vertical web shear reinforcement shall be uncoated steel welded wire reinforcement, deformed for concrete, as shown on the base sheets for each girder template. ASTM A615 Gr. 60 reinforcing substitution may be permitted when requested by the fabricator.

The area of the longitudinal wire shall be at least 40% of the area of the vertical wire.

The designer shall verify $A_v f_y/s$ of proposed steel is not less than the reinforcement necessary per design with the reduced yield strength of rebar compared to WWR.

5.5.1.5.2—Negative Moment Reinforcing Steel in Bridge Deck at Intermediate Supports

Negative moment reinforcement shall be provided in the bridge deck, taking into account deck longitudinal distribution reinforcement, by assuming 100% live load continuity at supports.

When designing for continuity designers may choose to only check negative moment capacity at the centerline of bearings on each side of the intermediate support. Satisfying negative moment capacity at the centerline of the intermediate support is not required.

When providing additional negative moment reinforcement precautions should be taken to prevent stress concentrations at the termination of the reinforcement. Typically this should be achieved by providing at least two different termination points for the supplemental reinforcement each side of the intermediate supports.

Additional information can be found in §9.2.2.2 and associated commentary.

5.5.1.6—Vent Holes

Where the bottom flange of the girder is designed as submerged during a flood event, vent holes shall be provided in the web to reduce buoyant forces on the structure. Vent hole locations shall meet both of the distances:

- 1 ft. from the face of the turndown/diaphragm
- H from the end of the girder

See Figure 5.3 for a graphical example.

Where vent holes are required see §3.5 for lateral and uplift loading requirements.

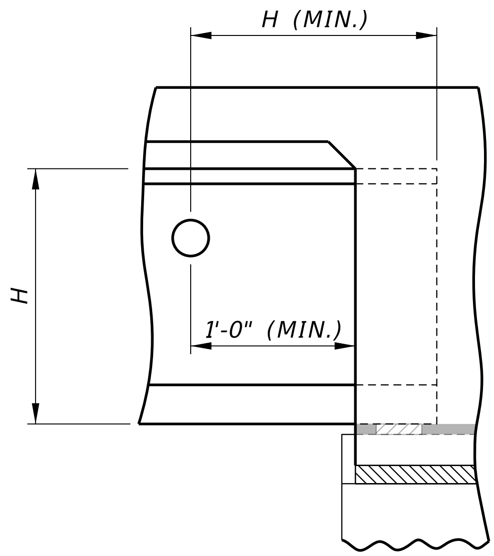


Figure 5.3—Precast Girder Vent Hole Location

5.5.1.7—Stay-in-Place Forms

Stay-in-Place metal forms are permitted. Designers shall apply the load over the width between girders. See §3.3 for load magnitude.

5.5.1.8—Camber

Camber is defined as the net result of upward deflection due to prestressing and downward deflection due to all dead loads. Camber and any correction for grade vertical curvature must be considered when determining girder seat elevations and concrete quantities. Bridge plans shall indicate typical vertical dimensions from the top of the girder flange to grade at all supports.

During design, calculated camber at the completion of construction shall be upward or straight. Time-dependent prestressing losses shall be calculated using t_f = 56 days during this check.

When haunch thickness at midspan is greater than 3 in., consideration should be given to also indicating the vertical dimensions from the top of the girder flange to grade at midspan as well.

Research has shown that after the composite deck is poured dead load deflections do not increase, therefore long term deflections need not be checked.

Research has shown that the multipliers commonly used by in the precast industry for long-term deflection calculations are not applicable to todays large high strength precast bridge members with a composite deck rather than a thin topping (Binard & Patel, 2022; Martin, 1977).

At this point in time the erection stage multipliers from Martin's 1977 paper are the best we have. Bridge Division is currently researching adjustments to the erection multipliers for use on future projects.

Camber shall not be considered in the vertical clearance determination under a bridge. For the purposes of determining the vertical clearances, the bottom of the girder shall be considered a straight line between the bearings. LL+IM deflection shall be considered.

Girder camber at erection is a random variable that can vary by as much as ±50% (Tadros et al., 2011). Therefore, camber should not be relied upon to provide vertical clearance.

5.5.1.9—Dead Load Deflections

All girder bridge plans shall have deflections calculated at the span tenth points and labeled, “Deflections for Shims.”

Deflections for Shims Table in Prestressed Girder Information Sheet shall include instantaneous deflection due to dead loads only (including future wearing surface only if it is placed at time of initial construction).

5.5.2—Prestressed I-Girders

NU girders are a very common system on state and local structures. For girder dimensions see Figure 5.4.

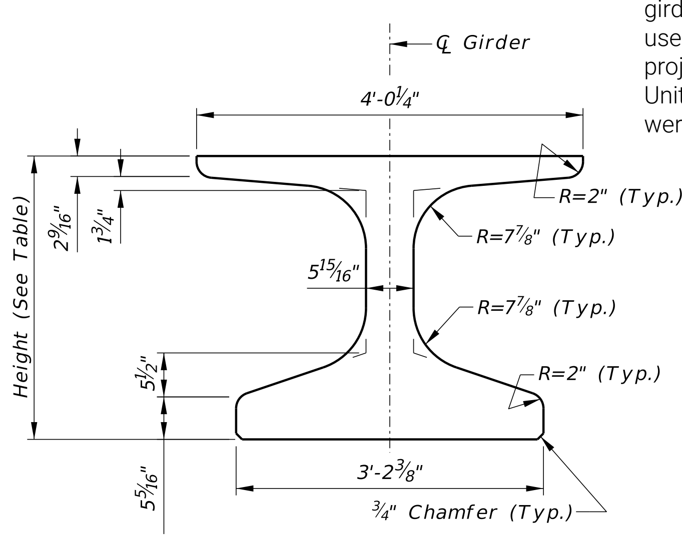


Figure 5.4—NU Girder Dimensions

Designers to note that the girder shape is not the exact height of the girders in inches. The NU series of girders were originally developed when FHWA required the use of metric units on any Federal-aid highway program project. When highway projects switched back to the United States customary units, the girder dimensions were not modified.

5.5.2.1—Section Properties and Layout

Girders shall be spaced at a maximum of 15 ft. center to center. For deck overhang limitations see §9.2.3.

After accounting for roadway grade, prestressing, and instantaneous dead load deflections; NU girders shall be detailed with minimum 1 in. haunch at the edges of the girder flanges. During construction and redecking, concrete girders (including AASHTO shapes) may be embedded up to 1 in. into the decks at a minimum number of locations.

Table 5.7—NU Girder Section Properties

Shape	Height in.	Area in.^{2}	Y_{nc} in.	I_g in.^{4}	Weight (8 ksi concrete) klf	Weight (10 ksi concrete) klf
NU35	35.44	648.3	16.13	110,218	0.711	0.720
NU43	43.31	695.1	19.57	182,262	0.763	0.772
NU53	53.13	753.3	23.95	301,949	0.827	0.837
NU63	63.00	812.0	28.43	458,653	0.891	0.902
NU70	70.88	858.7	32.05	611,646	0.942	0.954
NU78	78.75	905.5	35.70	791,107	0.994	1.006

5.5.2.2—Strand Pattern

NU Girder bottom flanges can accommodate up to 60 strands. An additional four strands tensioned to 5 kips are permitted in the top flange for the precaster's use. Permitted locations for strands are shown in Figure 5.5.

The 5 kip strands are typically neglected during final design calculations.

Standard strand extension into diaphragms and abutments is eight per girder, the extended strand locations are shown in Figure 5.5 and in the NU Girder base sheets.

Figure 5.5 is a visual representation of a fully populated strand pattern for an NU Girder. Precasters are given leeway on which exact slots to populate in the form for partially full rows based on what they feel will be the most efficient for their production. Deflected strands, extended strands, and strands used to tie reinforcement cages to are filled first and after that there is not a set sequence.

C3/R4 and C6/R3 should typically remain bonded when populated because they may have bottom flange confinement reinforcement tied to them. Per LRFDBDS Article C5.9.4.3.3 tying of mild reinforcement to debonding material is not desirable and bonded strands will help minimize cracking due to the Hoyer effect.

Standard strand diameter for use on NU Girders is 0.6 in. Other diameters require Bridge Division approval for their use.

0.7 in. strand has been successfully used on experimental bridges in the past, but has not been adopted by the industry or designers.

Span charts for use in preliminary design are available on NDOT's website (Hanna et al., 2010).

Engineering judgment should be taken when using these preliminary design table as they have been found to be slightly unconservative. Designers should note to use the preliminary design tables for Working Stress Method at release, not for Strength Design Method at release. Bridge Division is currently revising these preliminary design tables for future use.

For all NU girders, additional U-shaped bars (G501) shall be added to girder ends to resist the stresses due to lifting and handling of girders (see base sheets). Designers shall hold strands low enough to avoid conflict with G501 bars. When no other solution is possible the G501 bars may be removed and top deflected strand placed 2 in. below the top of the girder.

Allowable tensile stresses in the girder at release may be increased by providing additional mild reinforcement to resist tensile load, this may necessitate increasing the length or bar size of the G501 bar. See LRFDBDS Article C5.9.2.3.1b and Table 5.9.2.3.1b-1.

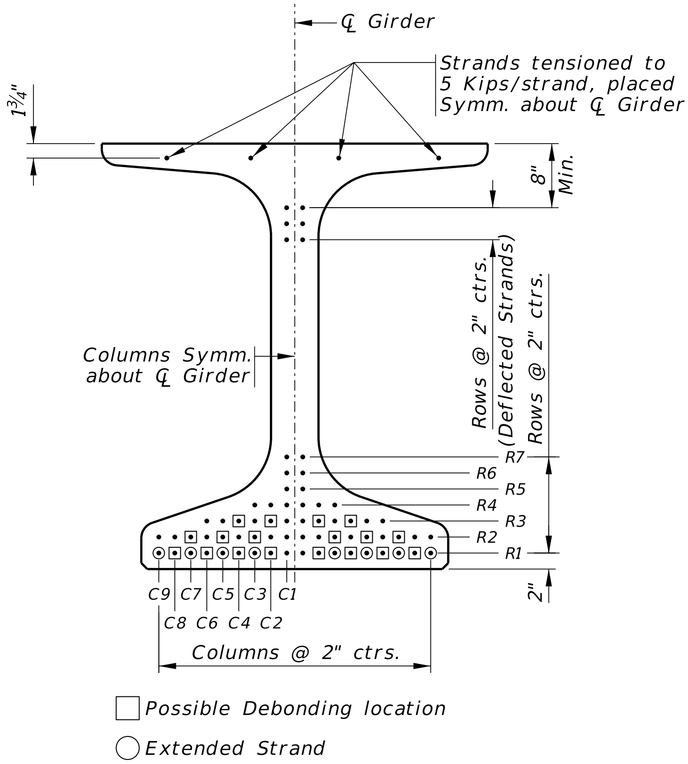


Figure 5.5—NU Girder Strand Pattern

5.5.2.2.1—Cost-Optimized Strand Pattern

For fabrication, the most cost-effective girder design will have up to 8 strands harped within a single hold-down device (limited to 25 kips) at each of the hold down points. Debonding can then be used to meet the permitted stress limits.

For girders that fall outside the cost-optimized strand patterns, all limits in §§5.5.2.2.2 and 5.5.2.2.3 must be met.

5.5.2.2.2—Limitations to Debonding

Strands may be debonded to help limit stresses in the girders up to the limits listed in LRFDBDS Article 5.9.4.3.3.

5.5.2.2.3—Limitations to Harping

Up to 14 strands in Column 1 may be deflected. At the girder ends, the strands are deflected to a normal pattern (meaning 2 in. vertical spacing between strand rows). The centroid of strands at both the girder end and the deflection point may be varied to suit girder stress requirements. When the centroid is varied, the placement of the deflected strands must still be as a group.

The strand hold-down points are normally located at the 0.4 and 0.6 tenth points of the prestressed girder. The hold-down location may be adjusted if it improves the stresses in the girder or at the request of the girder fabricator. If the fabricator desires a change from what is shown in the plans, they shall submit for approval a strand deflection configuration that meets plan centroid requirements.

The harping of strands is beneficial to the stability of girders during shipping. This has a more pronounced effect on longer spans.

The maximum hold-down force that can be accommodated by Nebraska producers is 25 kips per device and a maximum of two devices can be used at each harp point. Each hold-down device can accommodate up to 8 strands but does not need to contain all 8 strands before a second one is used. Deflected strand patterns must be designed to accommodate this limitation. When hold-down forces are a concern, the deflected strand exit location at the girder ends needs to be held as low as possible while maintaining the concrete stresses within allowable limits.

Typically girders less than 50 ft. long are fabricated using only straight strands due to high hold down forces required.

When calculating forces per hold-down device, strands in the same row must be in the same hold-down device.

Due to the conservative nature of the hold-down force limitation, no additional friction factor is necessary when calculating hold-down forces.

5.5.2.3—Transverse Reinforcement

The preferred reinforcement in the bursting zone is two mats of D25 wires at 2 in. spacing. Reinforcement may be increased to

- two mats of D31 wires at 2 in. spacing or
- two mats of D25 wires at 2 in. spacing with a third mat of D18 wires at 2 in. spacing in the center of the web.

D25 and D31 wires shall only be used in the end zones for containing bursting forces, D18 wires shall be used elsewhere. D18 wires at 12 in. spacing is the minimum permitted reinforcement in I-shaped girders.

Where strand debonding is utilized, designers may choose to only count strands bonded at the end of the girder when determining the prestress force at transfer for calculating pretensioned anchorage zone splitting resistance at girder ends in LRFDBDS Art. 5.9.4.4.1. Reference design example 9.6 in the PCI Bridge Design Manual (Precast/Prestressed Concrete Institute, 2023).

D31 wires are the largest size that can be used while still maintaining minimum cover on the transverse reinforcement.

5.5.2.4—Intermediate Diaphragms

Spans greater than 160 ft. shall have an intermediate diaphragm at midspan. Standard bent plate diaphragm details are provided in the sheet cell library for NU63 and deeper templates.

Designers should consider placing additional intermediate diaphragms when designing submerged girders.

5.5.2.5—Vent Holes

When vent holes are required by design, one 3 in. diameter hole shall be provided on the high end of each girder. The centerline of the hole shall be placed 1 ft. 1 in. from the top of the girder. Designer shall ensure deflected prestressing strands are held low enough or are above the vent hole to provide a minimum 3/4 in. clear cover.

5.5.3—Threaded Rod Connected I-Girders

Reserved for future use.

For more information see Sun et al. (2016).

5.5.4—Prestressed Inverted Tee Girders

Inverted tee girders are a commonly used system for shorter span lengths. For girder dimensions see Figure 5.6.

Designers to note that the girder shape is not the exact height of the girders in inches. The IT series of girders were originally developed when FHWA required the use of metric units on any Federal-aid highway program project. When highway projects switched back to the United States customary units, the girder dimensions were not modified.

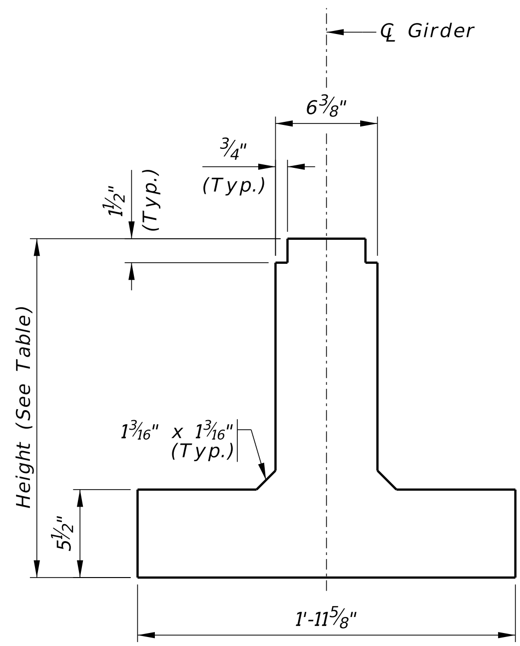


Figure 5.6—IT Girder Dimensions

5.5.4.1—Section Properties and Layout

Girders shall be spaced at a maximum of 37 in. center to center. For maximum deck overhang see §9.2.4.

The use of 8 in. thick decks on shallow inverted tee sections should be approached with caution. Construction issues with excessive deflection due to deck weight are not uncommon.

Table 5.8—IT Girder Section Properties*

Shape	Height in.	Area in.^{2}	Y_{nc} in.	I_g in.^{4}	Weight (8 ksi concrete) klf	Weight (10 ksi concrete) klf
IT13	13.31	178.9	4.50	2,034	0.196	0.199
IT17	17.25	204.0	5.79	4,472	0.224	0.227
IT21	21.19	229.1	7.22	8,334	0.251	0.255
IT25	25.13	254.2	8.76	13,871	0.279	0.282
IT29	29.06	279.3	10.37	21,300	0.306	0.310
IT33	33.00	304.4	12.05	30,837	0.334	0.338
IT36†	36.94	329.5	13.77	42,688	0.362	0.366

* These section properties are for the gross section as specified on the base sheets. Designers should consider that the girders are embedded ¾ in. into the deck in addition to the mandatory sacrificial wearing thickness specified in §9.2.1 when calculating composite section properties.

† In most cases a NU35 girder would be preferable to a IT36.

5.5.4.2—Strand Pattern

IT Girder bottom flanges can accommodate up to 22 strands. An additional two strands located at the top of the web are tensioned to 5 kips/strand and may be increased to help control stresses if required. Permitted locations for strands are shown in Figure 5.7. Top strands are not permitted to be cut in midspan. Draping strands is not permitted on IT girders, the only options to control end stresses is to debond strands or adjust the tension in the top strands.

Allowable tensile stresses in the girder at release may be increased by providing additional mild reinforcement to resist tensile load. See Article C5.9.2.3.1b and Table 5.9.2.3.1b-1 in the LRFDBDS.

The 5 kip strands are typically neglected during final design calculations.

Standard strand extension into diaphragms and abutments is four per girder, the extended strand locations are shown in Figure 5.7 and in the IT Girder base sheets.

Standard strand diameter for use on IT Girders is 0.5 in. Other diameters require Bridge Division approval for their use.

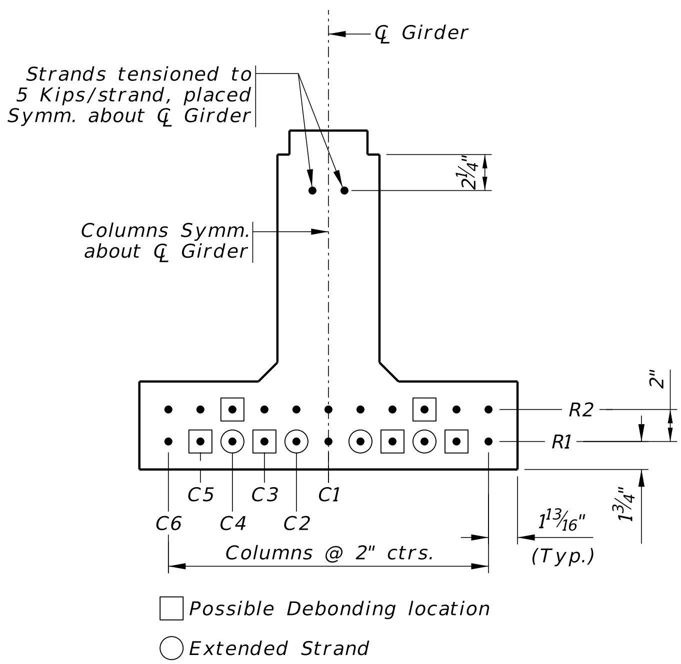


Figure 5.7—IT Girder Strand Pattern

5.5.4.3—Transverse Reinforcement

The preferred reinforcement in the bursting zone shall two mats of D20 wires at 2 in. spacing. Beyond the anchorage zone D14 wires shall be used. D14 wires at 8 in. spacing is the minimum permitted reinforcement in inverted tee girders.

Where strand debonding is utilized, designers may choose to only count strands bonded at the end of the girder when determining the prestress force at transfer for calculating pretensioned anchorage zone splitting resistance at girder ends in LRFDBDS Art. 5.9.4.4.1. Reference design example 9.6 in the PCI Bridge Design Manual (Precast/Prestressed Concrete Institute, 2023).

5.5.4.4—Intermediate Diaphragms

Intermediate diaphragms shall be required for all Inverted Tee girder bridges. A minimum of one diaphragm at midspan between the three outside girders is required. Where structures are phased during construction diaphragms shall be provided at the edges of the structure as well as at the edge of Phase I, see Figure 5.8 for an example.

Designers should consider placing additional intermediate diaphragms when designing submerged girders.

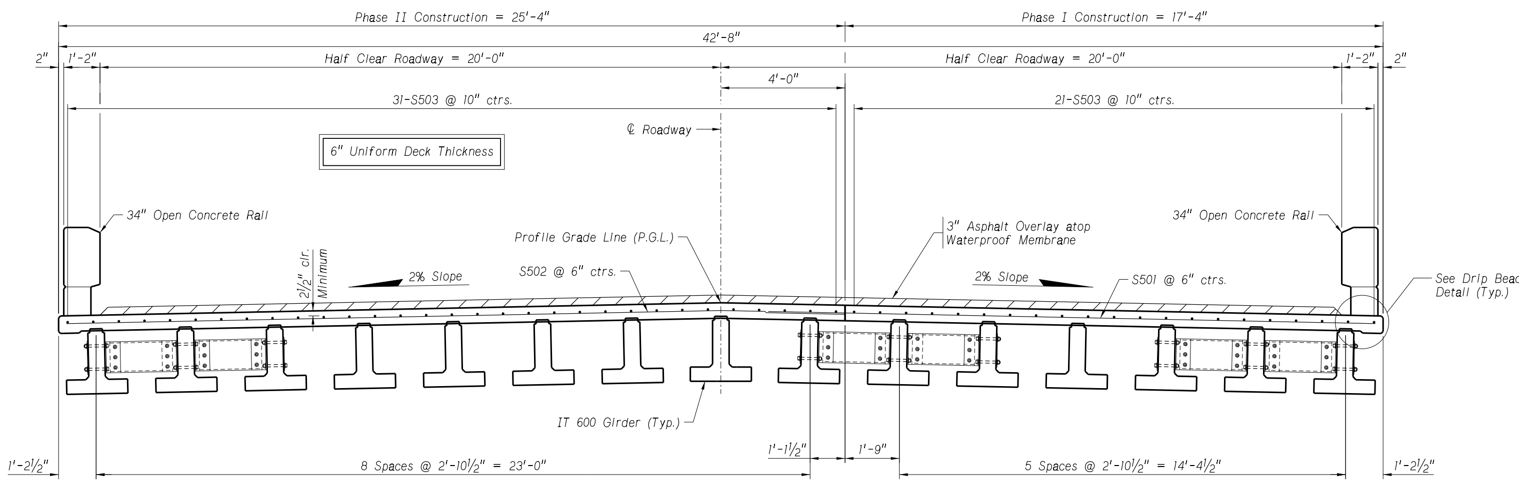


Figure 5.8—Location of intermediate diaphragm for phased construction of IT girder bridge (example taken from S081 08543, CN 42785, 2020)

Standard channel section diaphragm details are provided in the sheet cell library. A sample cross section for concrete intermediate diaphragms is provided in Figure 5.9.

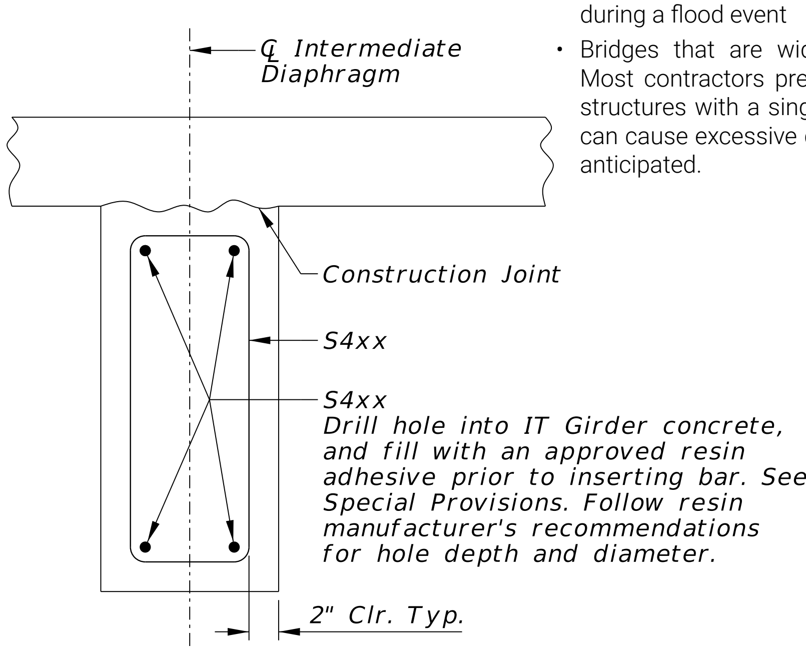


Figure 5.9—IT Girder Intermediate Concrete Diaphragm

Concrete or full width intermediate diaphragms may be of use when the diaphragm is expected to see more load than average such as:

- Bridges expected to be at least partially submerged during a flood event
- Bridges that are wide and built in single phase. Most contractors prefer to construct these type of structures with a single wide finishing machine that can cause excessive deflection in edge girders if not anticipated.

5.5.4.5—Vent Holes

When vent holes are required by design, one 2 in. diameter hole shall be provided on the high end of each girder. The hole shall be placed a minimum ¾ in. clear below the top strands and any additional top reinforcement added for release stresses.

5.5.4.6—Live Load Distribution

The distribution factors of live load per lane for Inverted Tee girders shall be S/11 for moment and S/5.5 for shear, where S is the beam spacing in feet. These factors are based on research done by the University of Nebraska (Kamel & Tadros, 1996; Martindale et al., 2019).

5.6—OTHER PRECAST ELEMENTS

5.6.1—Non-Prestressed Elements

Reserved for future use.

Precast concrete elements (non-prestressed) have been used by Bridge Division for:

- Abutment Caps
- Wings
- Grade Beams
- Approach and Paving Sections with rails

See Belden - Laurel (S020 38968) and Murray - US-34 and 75 (S001 02613) for details. Designers are encouraged to use precast concrete elements for ABC projects.

5.6.2—Prestressed Deck Panels

Reserved for future use. See §9.3 for details.

Full depth precast deck panels (NU Deck) have been used on Belden - Laurel (S020 38968), Kearney East Bypass (S010 05463R), and 198th - Skyline Drive (SL28B00216).

5.6.3—Post Tensioned Elements

Reserved for future use.

5.6.3.1—Post Tensioned I-Girders

Reserved for future use.

For more information see Jaber et al. (2006) and Tadros et al., (2003b) as well as the 2016 NDOR Bridge Office Policies and Procedures Manual.

5.6.4—UHPC Decked I-Girders

Reserved for future use.

For preliminary information see Morcous & Tadros (2023).

At the time of publication, the Belvidere North Bridge (S081 01847L) has been awarded but not constructed This is the first bridge in North America with UHPC superstructure that was awarded in a competitive bid, and the first bridge with non-proprietary UHPC mix that was developed by UNL to create a cost effective UHPC mix with local materials.

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Last Updated: April 18, 2025

Chapter 7 — Repair and Preservation

7.1—OVERVIEW

The primary objective of the Bridge Preservation Program is to maximize the useful life of bridges in a cost-effective manner, applying appropriate treatments and maintenance activities at the opportune time.

The FHWA Bridge Preservation Guide is very informative and comprehensive. This guide outlines many of the repair and preservation strategies adopted by NDOT (U.S. Department of Transportation Federal Highway Administration, 2018).

A general list of work items for Preventative Maintenance Activities of Bridges is included in the Roadway Design Manual, Chapter 1, §6.C.2.a. The activities are divided into Cyclical or Condition-based groups; the work is eligible for Federal funding (Nebraska Department of Transportation, 2023).

For information on preservation friendly design for new construction see §15.11.

7.1.1—Scoping of Repair and Preservation Projects

The needs for Nebraska Bridges are assessed by the Bridge Management Section of the Bridge Division. Bridges that are in Good or Fair condition and that are not recommended for replacement or rehabilitation are considered candidates for preservation or maintenance.

Engineering judgment is necessary when developing a scope of work for preservation and repairs because of unique site conditions and maintenance history for each existing bridge.

The main communication tool between the designer and Bridge Division for repair and preservation projects is the Bridge Determination. The approved BD can be found on OnBase and may be updated during the life of a project.

7.1.2—Organization of this Chapter

The following subsections of this chapter represent the current preferences of the Bridge Division for bridge repair and preservation. This chapter will evolve rapidly and represents a snapshot of the current practices.

To illustrate these practices, examples from recent plan sets are included. These examples are not intended to serve as standard cells but rather to provide general guidance on the level of details required. It is the designer's responsibility to adapt the detail to the specific structure.

Where applicable, references to the current Special Provisions and Pay Items are also provided.

7.2—SUPERSTRUCTURE PRESERVATION WITH OVERLAYS

7.2.1—Thin Overlays

7.2.1.1—Epoxy Polymer Overlay

Multi-Layer Epoxy Polymer Overlays are a deck treatment method, that use a high build epoxy resin and broadcast aggregate, resulting in a thin overlay. This system can be used when asphalt is not available for an asphalt overlay and waterproofing membrane system or when a thicker overlay would cause additional undue expense e.g., if asphalt placement would lead to the replacement of a finger expansion joint.

EPO is not recommended for use on approach slabs due to vapor drive upwards from soil through the approach slab.

7.2.1.2—Deck Surface Treatment

This is a flood coat system used as a "healer/sealer". The material is a low viscosity resin capable of deep penetration into the deck to attempt to fill/bond small cracks and voids while acting as an overall surface sealer to prevent chloride penetration.

Use 90 sf/gal for estimating purposes for the Pay Item quantity. This gives a 10% buffer to allow for wastage during construction over the amount required to be applied by the Special Provisions.

For existing bridges with open rails and sidewalks deck surface treatment is the preferred preservation method on the sidewalk.

This Pay Item sees more frequent use in District 5 due to the limited availability of asphalt in that part of the State.

Consider applying Surface Treatment to raised medians, but the extensive preparation required by the Special Provisions may not be justified.

7.2.2—Rigid Overlays

7.2.2.1—Polyester Polymer Concrete Overlay

This rigid overlay consists of batching polyester concrete on-site and placing the overlay material with use of a screed or finishing machine. PPC overlays can be placed at a nominal thickness to preserve bridge decks as well as placed in variable thickness as a grade correction strategy. Full depth headers, block-outs, or repairs may be performed with this material. Minimum specified thickness for this material is $^3/_4$ in.

7.2.2.2—47B-OL Concrete Overlay

This rigid concrete overlay replaced the Silica Fume deck overlay system, which is not currently used by Bridge Division. This material has been used to protect entire bridge decks as well as used as a localized repair on decks that experienced shallow cover during the deck placement. Minimum specified thickness of overlay for this material is 2 in.

The first use of this overlay type was in 2012 on S080 36014. As of 2023, 7 total bridges have received this overlay type.

7.2.3—Asphalt Overlay and Waterproof Membranes

Asphalt Overlay and Waterproof Membranes are currently the most used superstructure preservation method for Bridge Division.

Between 1973 and 1977 approximately 27 bridges were built, and prior to opening to traffic, received an experimental deck preservation treatment using waterproofing membrane and an asphalt overlay. Most of these bridges are still in service as of 2022. The first known bridge is westbound I-680 over the Missouri River (S680 01343L) built in 1973. Of these bridges that are no longer in service, several were replaced for reasons such as realignment of the roadway, poor timber piles, or damaged due to floods.

Repairs of bridge decks and approach slabs are typically performed in conjunction with AC+M, see §7.2.3.6.

7.2.3.1—Asphalt Overlay Details

7.2.3.1.1—Mix and Placement of Overlay

The asphaltic concrete used on bridge decks as an overlay will almost always be the same mix provided for the resurfacing or maintenance strategy on the roadway. The best practice is to place an asphalt overlay in 2 lifts, for the purpose of achieving the best smoothness. Every effort shall be made to place the overlay in conjunction with the lifts placed on the roadway and to avoid use of headers. When feasible, asphalt overlays shall be placed with the lane lines in mind to avoid joints from landing in wheel paths.

Some situations may require an asphalt mix for the bridge overlay that is different than the mix specified on the roadway project.

When asphalt overlays and waterproofing membranes are part of Bridge-Only projects, the source of asphalt needs to be carefully considered. Not all districts have easy access to asphalt for smaller projects.

The asphalt types below are listed in order of preference based on optimal density and durability properties.

- SLX, by far the optimal mix for bridge overlays
- SPR
- SPH, only effective if the oil volume is increased, otherwise it is difficult to get density.

Most, if not all, of the overlays placed in the 1970's were placed in two lifts. The special provisions typically read "asphaltic concrete on waterproofing membrane shall be placed in at least two approximately equal layers. The first layer shall be at least ³⁄₄ in. thick after compaction".

MTVs for the placement of asphalt on roadway resurfacing projects are commonly used and are recommended for the asphalt placement on bridges.

Some of the benefits of using an MTV are:

- Helps maintain a consistent temperature of mix going through the paver.
- Prevents aggregate/material segregation by remixing the mix.
- Allows non-stop paving

7.2.3.1.2—Thickness of Overlay

To prevent delamination, cracking, and rutting of asphalt, the overlay thickness shall be 2 in. minimum.

A uniform 3 in. overlay thickness shall be used for all new bridges and for existing bridges that have bridge rails that are 30 in. or taller.

For existing bridges with 29 in. tall bridge rails, either provide a uniform thickness of 2 in., or provide 3 in. asphalt thickness across the driving lanes, and within the shoulder, taper down to 2 in. minimum thickness at the face of the bridge rail as shown in Figure 7.1.

Designers shall coordinate grade change tie-ins or inlay considerations beyond the ends of the bridge with Roadway Design as needed.

For bridges on a cross-slope, ensure that the taper on the high side does not create a flat spot. A flat spot is defined as a slope of 0.5% or less

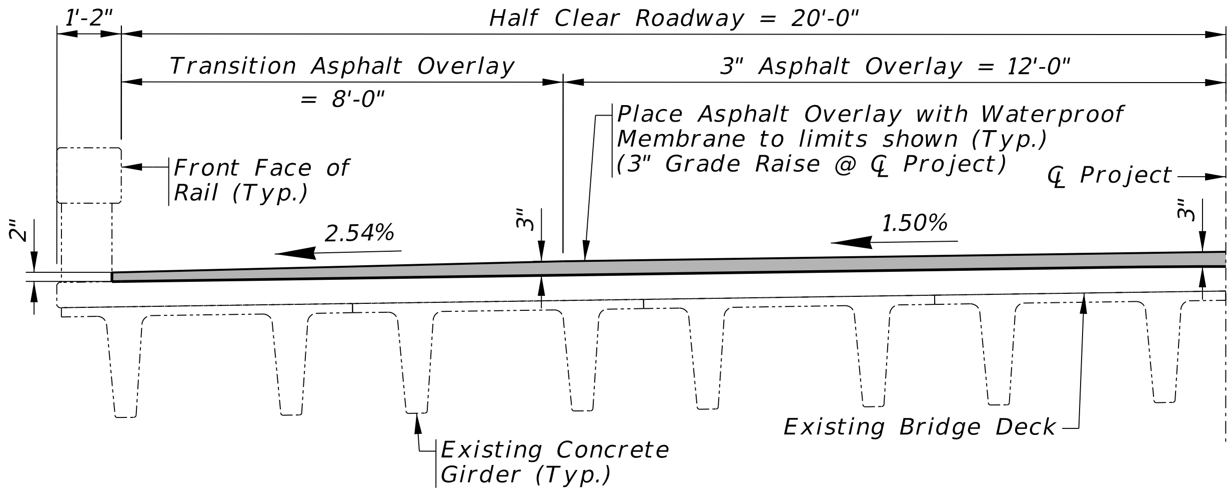


Figure 7.1—Tapered Asphalt Overlay
(example taken from S006 14316, CN 71208, 2022)

7.2.3.1.3— Wick Drains

When wick drains are used, designers should ensure that wick drains have proper termination points to daylight the water. Wick drain details should be provided and shown the plans in the following situations:

- Closed rail bridges with two or more floor drains per gutter line shall have the wick drains run between and terminated at the floor drains. If the bridge is superelevated, the wick drain would only be placed on the low side of the deck. It is not necessary to place wick drains in the area between the end of the bridge and the first floor drain.
- Bridges with raised medians with two or more floor drains per gutter line shall have wick drains provided like paragraph 1, above.

To help ensure long-term asphalt overlay durability on bridge decks with waterproof membranes, it is recommended that adequate membrane-level drainage be provided to ensure water is quickly removed from the bridge deck. Poor drainage results an overlay that is saturated with water, leading to accelerated deterioration of the asphalt including potholes, cracking, stripping, and debonding.

Wick drains installed above the membrane, should help provide the necessary drainage on closed rail bridges. For bridges with closed rails without floor drains, designers shall determine whether or not to incorporate wick drains based on bridge geometry (vertical curve, width of bridge, potential for daylighting).

Open rail bridges allow for water to drain off the structure before any seepage into the asphaltic concrete can occur. Therefore, wick drains are not necessary for open rail bridges.

7.2.3.1.4—Payment

Asphalt overlay (by others) is typically noted in the Bridge plans, but the Pay Items and quantities for the asphaltic concrete are computed by M&R or Roadway Division and are not shown on the Bridge Plans.

7.2.3.2—Waterproof Membrane Details

There are two different types of membrane systems used to preserve existing and new bridge decks, including slab bridges:

- Preformed Fabric
- Liquid-Applied

Base Sheets for each membrane type are available for use in the Appendices.

Waterproofing membrane pay items will be listed as Group 9 Items on the front sheet of the bridge plans.

Membranes are paid for by the plan area of covered deck and approach. The limits of the waterproofing membrane will be specified in the bridge determination and will be shown in the plans.

For bridges with no approach slabs, the membrane is commonly extended 5 ft. beyond end of floor.

Figure 7.2 shall be used for membrane selection. On bridges that are selected for Hot Liquid or Fabric at the end of the flow chart, current practice is to indicate Hot Liquid Membrane on the plans and include the Special Provision that allows substitution of Fabric Membrane post-letting.

Consider using the fewest membrane types on a single project, i.e., consider using a single membrane type that satisfies all design factors.

Membrane type specified for a repair project is not always provided in the Bridge Determination.

- Fabric membranes are usually more cost effective and are the most common type used on new bridges. Fabric membranes can be substituted for Hot Liquid membranes post-letting except when existing concrete overlays are to be removed. Special Provisions may apply.
- Due to difficulty of fabric installation on horizontally curved and superelevated bridge deck, liquid membrane types are typically selected.
- Hot liquid membranes do not require a separate tack coat placement, unlike fabric or cold liquid types which do require tack coat.
- Cold liquid membranes possess superior crack bridging capabilities.

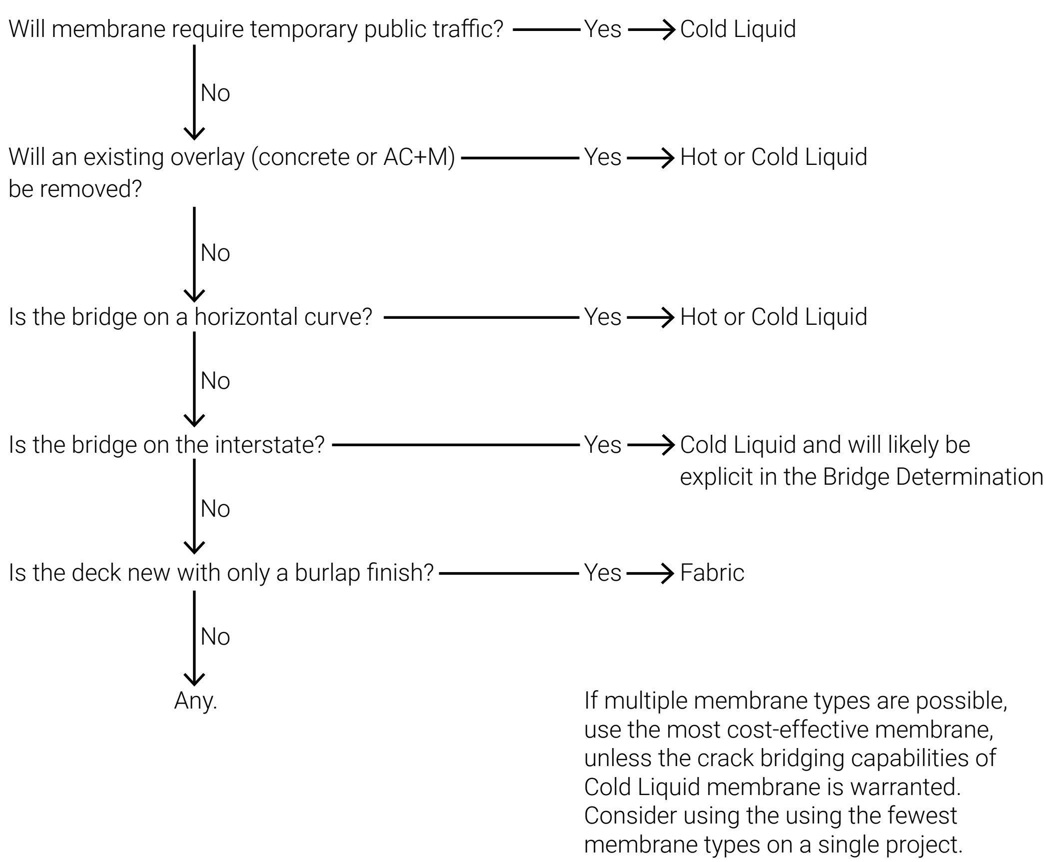


Figure 7.2—Membrane Selection Flowchart

7.2.3.2.1—Preformed Fabric Membranes

Specify Preformed Waterproofing Membrane, Type 2 for existing bridges, the Pay Item Bridge Deck Preparation must also be specified.

Specify Preformed Waterproofing Membrane, Type 3 for new bridge decks and new slab bridges. Newly placed concrete shall be drag finished with wet burlap. No tining, grooving, brooming, or other texturing shall be used.

Preparation of Bridge deck is not required for Type 3 membranes as all work to correct the deck surface to meet a smoothness requirement is covered in the special provision.

Fabric Membranes are measured by the square yard.

Preformed Fabric Membrane system consists of the membrane and a compatible primer coat. Tack coat is applied, and the membrane is paved with a hot asphaltic concrete wearing course.

Preformed Waterproofing Membrane, Type 1 was meant to be used on existing structures with adequately smooth decks that minimal preparation was necessary. Due to coordination issues with pay items and roughness determination this approach has been discontinued and the amount of field preparation is determined by the NDOT Field Engineer during construction.

7.2.3.2.2—Liquid-Applied Membranes

Liquid Membranes are measured by the square foot.

7.2.3.2.2a—Cold Liquid-Applied Membrane

Cold liquid-applied membranes are typically used to preserve higher value assets.

When closure time in high traffic areas is a concern, traffic can be placed temporarily on the completed membrane for up to 7 days.

Newly placed concrete shall be broom finished. No tining, grooving, or other texturing shall be used.

Cold Liquid-Applied Membrane system consists of a liquid primer, base coat membrane, top coat membrane, and broadcast aggregate layer. Tack coat is applied, and the membrane is paved with a hot asphaltic concrete wearing course.

The membrane chemistry has several allowable variations: polyurea, methyl methacrylate, or a blended polyurethane/methyl methacrylate. One polyurea product (Polyflex) is not to be placed on non-cementitious patches that consist of magnesium phosphate. No reactivity concerns are known between the allowable variations and deck repair materials listed in the special provision, “Polymer Bridge Deck and Approach Repair”.

7.2.3.2.2b—Hot Liquid-Applied Membrane

Hot liquid-applied membranes are mainly applicable to existing bridge decks or existing slab bridges. A few test cases of Hot liquid-applied membranes were applied to new bridges. However, the best historical performance has been with preformed fabric, due in part to no tining or texturing on new bridges.

Hot liquid-applied membranes have experienced some difficulty in asphalt placement in hot ambient conditions. Hot liquid-applied membranes are especially effective when an existing concrete overlay is to be removed by milling.

Hot Liquid-Applied Membrane consists of a non-modified, heated performance grade bituminous binder, that is spray applied to the prepared bridge deck. The membrane is then completely covered with a non-woven polypropylene paving fabric. The membrane is then paved with a hot asphaltic concrete wearing course. No additional tack coat is needed.

There is no requirement for the finishing of newly placed concrete.

7.2.3.2.3—Payment

Pay items for the waterproof membrane are to be listed as Group 9 Items on the front sheet of the bridge plans. In addition, standard note #111 shall be placed on the bridge plans when Group 9 Items are shown.

7.2.3.3—Curbs on Open Rails

Bridges with Open Rails shall be considered for the placement of a Curb Angle or Concrete Curb along the gutter line located at piers, bents, or ends of floor to deflect chlorides away from these areas.

Before detailing curb angles or concrete curbs, check for unacceptable water spread that might extend into the lane.

7.2.3.4—Expansion Joint Selection with Asphalt Overlay Projects

See Table 7.1, Table 7.2, and Table 7.3 for guidance.

Total thermal movement shall be decomposed into longitudinal and transverse movements (from the perspective of the joint) when designing joints on skewed structures.

Joint Selection Guidance for projects without an Asphalt Overlay is provided in §14.1.1.

For joints located on the bridge deck, the value of a watertight expansion joint that can protect the underlying structural elements cannot be overstated. For this reason, strip seals, modular joints, and finger joints with a drainage membrane are recommended for these locations. Other factors, such as maintenance level repairs, may preclude use of these joint types on a project.

Details for PPF, Strip Seal, and Modular Joints can be found in §14.1.

Table 7.1—Selection Guide for Joints Located Over Grade Beams on Existing Approaches for Placement of Asphalt Overlay

<table>
<tr><th>TM Range</th><th>Typical Existing Joint Types</th><th>Typical Existing Nominal Joint Width At 50°F</th><th>Joint Recommendation</th></tr>
<tr><td rowspan="3">TM < 1 in.</td><td rowspan="2">Simple Gap*
Armored Joints</td><td>≤ 2 in.</td><td>Saw and Seal Joint</td></tr>
<tr><td>> 2 in.</td><td rowspan="5">Asphalt Plug Joint</td></tr>
<tr><td>Strip Seals</td><td>2 in. minimum</td></tr>
<tr><td rowspan="2">1 in. ≤ TM < 1 ½ in.</td><td>Simple Gap*
Armored Joints</td><td>1 in. – 4 in.</td></tr>
<tr><td>Strip Seals</td><td>2 in. minimum</td></tr>
<tr><td rowspan="2">1 ½ in. ≤ TM < 3 in.</td><td>Simple Gap*
Armored Joints†</td><td>1 ½ in. – 3 ½ in.</td><td rowspan="2">PPF Joint with Bridge Joint Nosing‡</td></tr>
<tr><td>Strip Seals†</td><td>2 in. minimum</td></tr>
<tr><td rowspan="2">3 in. ≤ TM < 4 in.</td><td>Armored Joints†
Strip Seals†</td><td>< 3 in.</td><td rowspan="2">Strip Seal</td></tr>
<tr><td>Modular Joints†
Steel Finger Joints†</td><td>≥ 3 in.</td></tr>
<tr><td>TM ≥ 4 in.</td><td>Modular Joints†
Steel Finger Joints†
Segmental Joints†</td><td>> 4 in</td><td>Modular Joint
Finger Joint</td></tr>
</table>

* Simple Gap Joint types: Joint Filler, PPF, Preformed Silicone, Compression Seals

† Existing joints utilizing a steel extrusion or other steel hardware at the top of deck shall not have new steel hardware installed directly atop them

‡ Remodel concrete to adjust gap to be appropriate for the expected thermal movement

Table 7.2—Selection Guide for Joints Located Over Grade Beams on New Approach Slabs with Asphalt Overlay

TM Range	Nominal Joint Width At 50°F	Joint Recommendation
TM < 1 in.	1 in.	Saw and Seal Joint
1 in. ≤ TM < 1 ½ in.	2 in. minimum 3 in. maximum	Asphalt Plug Joint
1 ½ in. ≤ TM < 3 in.	2 ½ in.	PPF Joint with Bridge Joint Nosing
3 in. ≤ TM < 4 in.	2 in.	Strip Seal
TM ≥ 4 in.	5 in. minimum	Modular Joint Finger Joint

Table 7.3—Selection Guide for Joints on Deck or at End of Floor on Asphalt Overlay Projects

TM Range	Typical Locations	Typical Existing Joint Types	Typical Existing Nominal Joint Width At 50°F	Joint Recommendation
TM < ½ in.	EOF btwn. Approach* Fixed Pin (P&H Bridges)	Simple Gap† Armored Joints	1 in.	1 in. Partial Depth Saw and Seal Joint Asphalt Plug Joint
½ in. ≤ TM < 1 ½ in.	EOF btwn. Approach*	Simple Gap† Armored Joints	1 in. – 3 in.	Asphalt Plug Joint
1 ½ in. ≤ TM < 4 in.	EOF btwn. Approach* Exp. Joints (P&H Bridges) Abutment Backwall Deck Joints between units	Simple Gap† Armored Joints Strip Seals Steel Sliding Plates	1 in. – 4 in.	Strip Seal Shallow Strip Seal with Bridge Joint Nosing‡
TM > 4 in.	Abutment Backwall Deck Joints between units	Modular Joints Steel Finger Joints Segmental Joints	> 4 in	Modular Joint Finger Joint

* Existing Slab Bridges, or Bridges with turndowns but older approaches

† Simple Gap Joint types: Joint Filler, PPF, Preformed Silicone, Compression Seals

‡ Bridge Division approval required

7.2.3.4.1—Bridge Joint Nosing with Asphalt Overlays

Bridge Joint Nosing can be used on Asphalt Overlay projects only as listed in Table 7.1, Table 7.2, and Table 7.3.

Bridge Division approval is required to use Bridge Joint Nosing in conjunction with Shallow Strip Seal and in any other application.

When joints are required to be raised to the height of the Asphalt Overlay, Bridge Joint Nosing is used as a rigid concrete header on both sides of certain expansion joint types. The material is either a polyester or elastomeric concrete.

Previous NDOT policy was to install Bridge Joint Nosing at joints on new concrete decks and/or approaches without an asphalt overlay. This practice has been discontinued, as the detail has been shown to delaminate from the underlying concrete in many cases.

While Bridge Joint Nosing can be used in non-overlay applications to repair minor concrete spalling at expansion joints, it is preferable to repair these areas in accordance with §7.4.1.

7.2.3.4.2—Asphalt Plug Joints

Asphalt Plug Joints consist of a flexible asphaltic binder, mixed with aggregate (Figure 7.3). The material is installed over an expansion gap to provide a smooth riding surface. They have proven to be a good option for small movement joints on Asphalt Overlay Projects, as well as a maintenance retrofit to repair existing joints.

While Asphalt Plug Joints can be used as a maintenance level repair for joints on bridges with a concrete riding surface, use of these joints in new construction without overlay is discouraged.

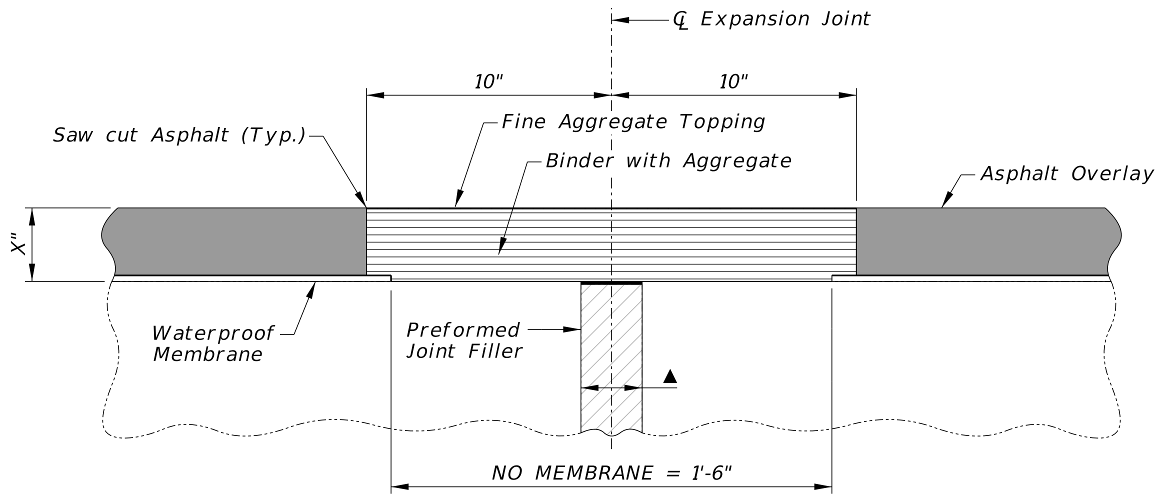


Figure 7.3—Asphalt Plug Joint Standard Detail

Standard notes and details for the Asphalt Plug Joint are provided in the Expansion Joint Base Sheet. The Base Sheet notes define when Backer Rod, Bridging Plate, or both are needed to be included as part of the system. When installing an Asphalt Plug Joint over an existing Strip Seal, the Bridging Plate is mandatory.

Asphalt Plug Joint pay item will be listed as Group 9 on the front sheet of the bridge plans and standard note #111 shall be included.

7.2.3.4.3—Asphalt Overlay Saw & Seal Joints

Saw & Seal Joint pay item will be listed as Group 9 on the front sheet of the bridge plans and standard note #111 shall be included.

7.2.3.4.3a—End of Floor—Non-Movement

For bridges receiving an asphalt overlay, a Saw and Seal joint at the End of Floor should be included on all projects, unless there is a movement joint at the End of Floor. This joint will only be cut 1 in. deep to control cracking of asphalt overlay. Fill with hot pour sealer (Figure 7.4).

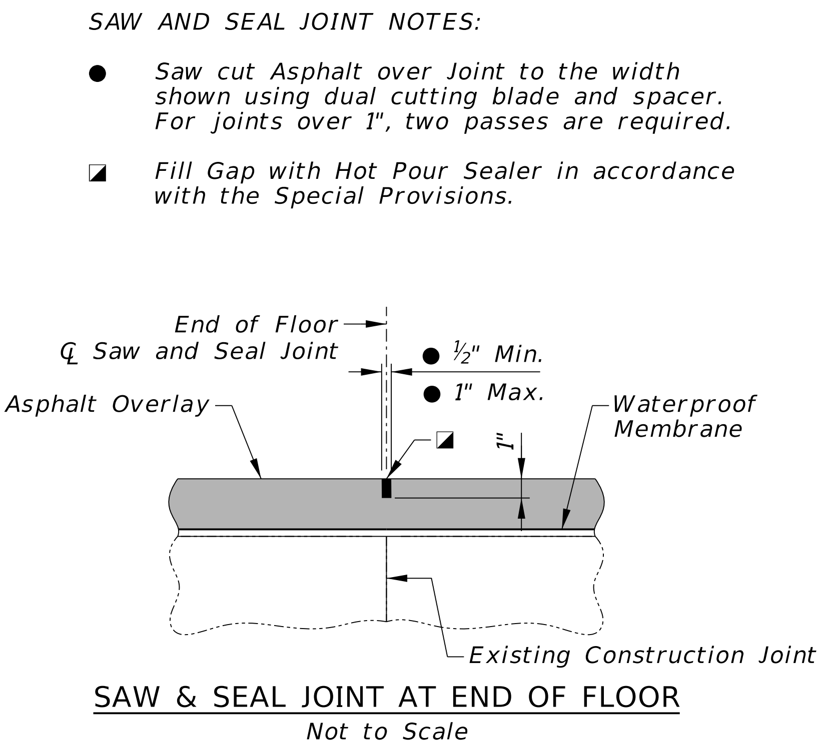


Figure 7.4— Saw and Seal Joint at EOF (Non-Movement)

7.2.3.4.3b—Grade Beams or Sleeper Slabs

For TM ≤ 1 in., install Saw & Seal joint to match existing joint width and fill with hot pour sealer only. If existing Grade Beam Joint is greater than 2 in. wide, or a Strip Seal type joint, a plug joint will be required (Figure 7.5).

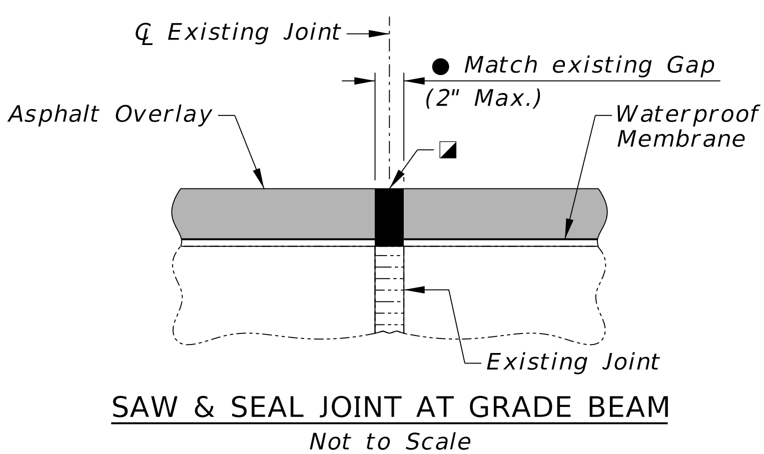


Figure 7.5—Saw and Seal Joint at Grade Beam (Expansion Joint)
See Figure 7.4 for symbols definitions and notes.

7.2.3.4.3c—End of Paving Pressure Relief Joint

Where there is no underlying joint (tie-bar joints and approaches against AC roadway pavement) install a 1 in. deep saw and seal joint above the EOP as shown in Figure 7.6.

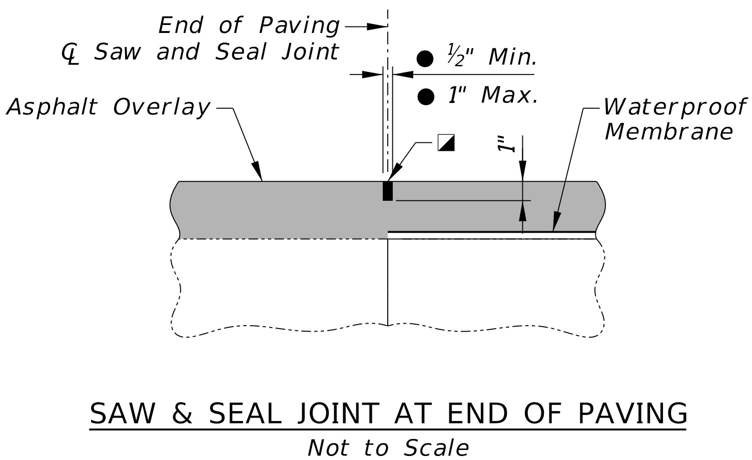


Figure 7.6—Saw and Seal Joint at EOP (Non-Movement)

See Figure 7.4 for symbols definitions and notes.

Where there is a 1 in. existing joint at the EOP mirror that joint as a 1 in. wide saw and seal joint in the overlay as shown in Figure 7.7. This style of joint was typical for approach construction from circa 1990 to 2005 and 2023 to current.

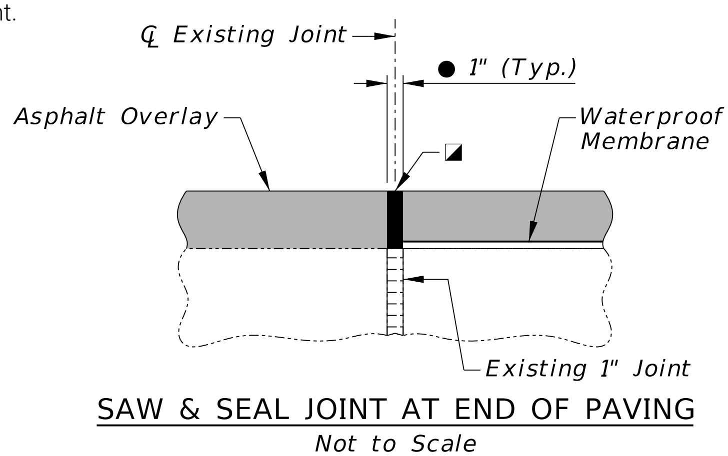


Figure 7.7—Saw and Seal Joint at EOP (Existing 1 in. Joint)

See Figure 7.4 for symbols definitions and notes.

Where there is a 3 in. existing joint at the EOP mirror that joint as a 3 in. wide saw and seal joint with support for the hot pour sealant in Figure 7.8. This style of joint was typical for approach construction from circa 2005 to 2022.

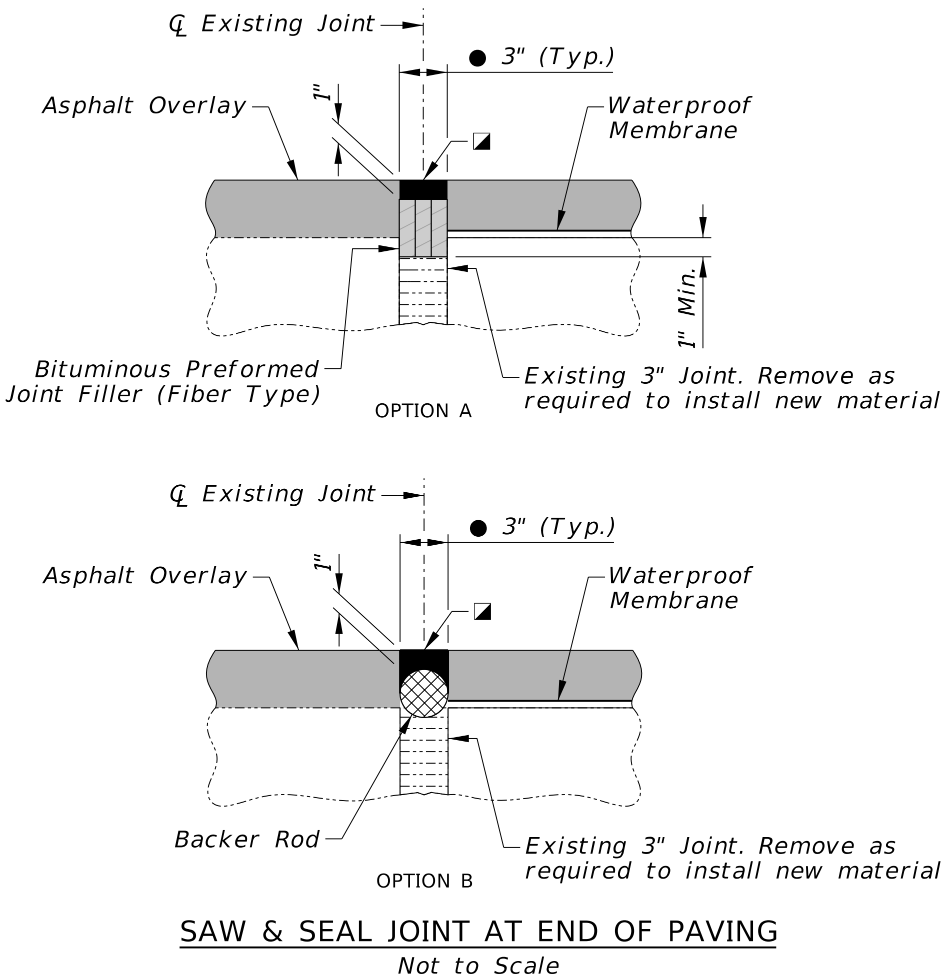


Figure 7.8—Saw and Seal Joint at EOP (Existing 3 in. Joint)

See Figure 7.4 for symbols definitions and notes.

Prior to circa 1990 the standard was a 4 in. pressure relief joint, joints that wide require an asphalt plug joint, see §7.2.3.4.2.

7.2.3.5—Delayed preservation of New Decks with AC+M

The most efficient construction results from placing asphalt overlays in conjunction with mainline asphalt paving projects for numerous reasons, which are beyond the scope of this document. When new bridges are built or bridges are re-decked on bridge-only projects, the decision can be made to delay the placement of the AC+M until a future resurfacing project that includes the bridge in the project limit.

Some design considerations for delayed deck preservation:

- For an AC+M project that is programmed and planned within a 2-year time frame with a high level of confidence, the new deck shall not receive grooving, but shall receive a burlap finish only. Otherwise, grooving shall be applied to the bridge deck.
- Consider joint selection and detailing that will avoid the placement of a second set of joints at the level of the asphalt overlay on the future project.

An example of delayed preservation is as follows:

- New construction: Three new bridges were built on CN 13204, Adams West Bridges in 2018. The bridges received a burlap finish only, per addendum No. 3, Call № 100, letting 18 Jan. 2018. The Midwest Guardrail System (thrie beam and w-beam) was installed at 34 in. height when the bridges were built.
- Delayed Preservation: The bridges received fabric waterproofing membranes and asphalt overlays in 2022 on CN 13186, Adams West.

NDOT Material and Research Division (M&R) has provided guidance about the safety of new bridge decks relative to burlap surface texture. M&R performed skid testing of burlap finished, non-tined PCC in November 2015; the bridge deck of S080 39562 (NW 48th Street, Lincoln, NE) and a concrete shoulder just west of the same bridge. Average friction numbers were 42 and 49, respectively. These numbers are in the range of a new HMA pavement and slightly higher than 1 year old HMA pavement (see typical values in Table 7.4). M&R has stated that bridges should be evaluated on a case-by-case basis, but in general are comfortable allowing burlap finished decks open to traffic as long as they have membrane/overlays included within the 5-year program following the new deck opening to traffic.

Table 7.4—Typical Friction Numbers

Comparison of Surface Treatments (Li et al., 2012)		
Chip Seals	Immediately after Application	SN 50-70
	At 12 months	SN 44-52
	Failure Segments at 12 months	SN 20-30
Microsurfacing	Immediately after Application	SN 28-57
	At 12 months	SN 40-60
	Continuous Decline after 12 months	
Ultra Thin Bonded Wearing Course	Immediately after Application	SN 48-59
	At 33 months	SN 32-39
Dense Graded HMA Overlays	Immediately after Application	SN 32-52
	At 12 months	SN 30-40
Fog Seals reduce friction by 20% to 33% until the return to the original SN after about 12 months		

7.2.3.5.1—Joint Considerations

Joint concepts to be considered when the grade raise associated with AC+M is scheduled for a separate project within 5-10 years of initial construction.

7.2.3.5.1a—Small Movement Joints

Use Preformed Joint Filler in the gap in the initial condition. At the time of overlay, install a Saw and Seal Joint or Asphalt Plug Joint above it, see Figure 7.9 for an example.

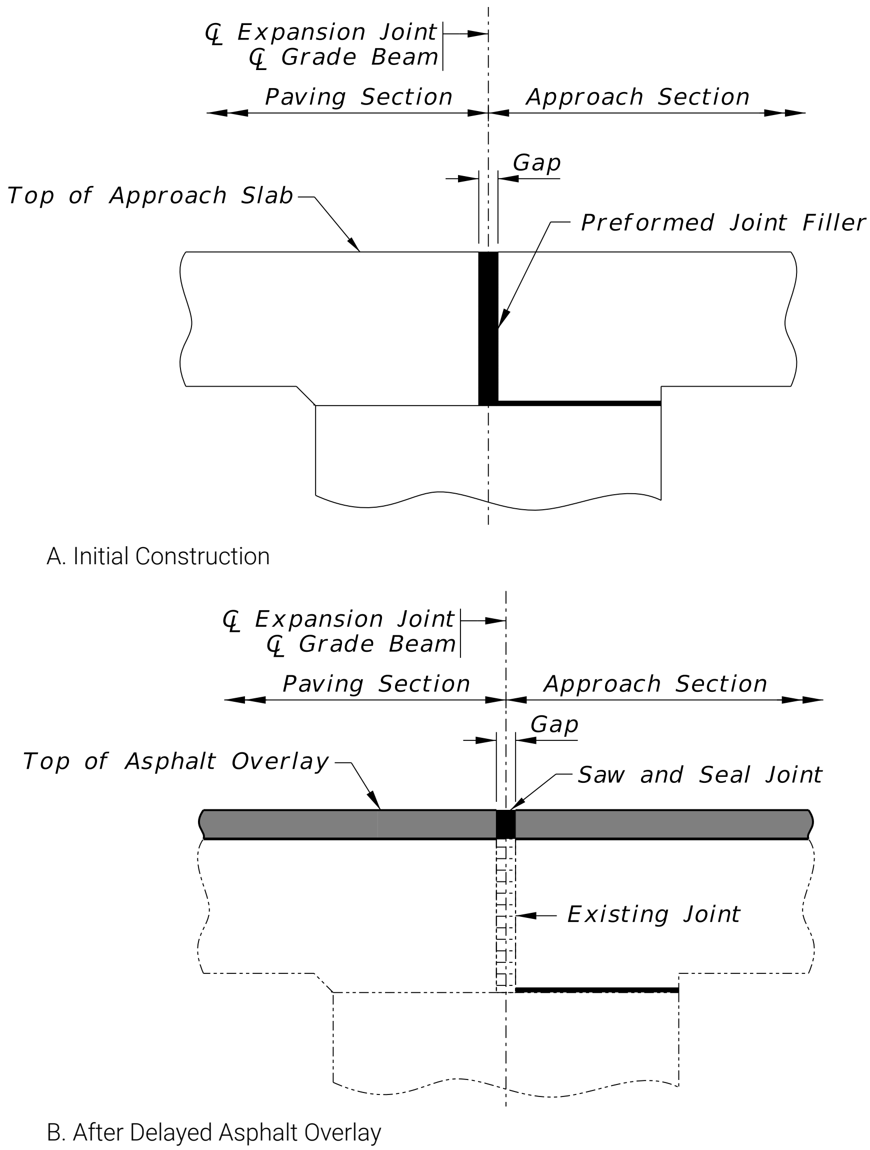
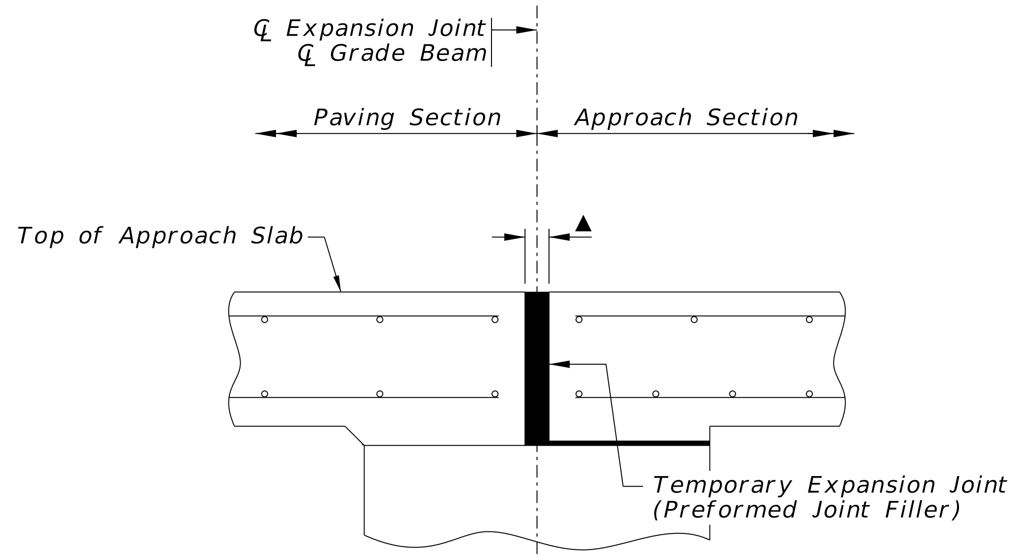


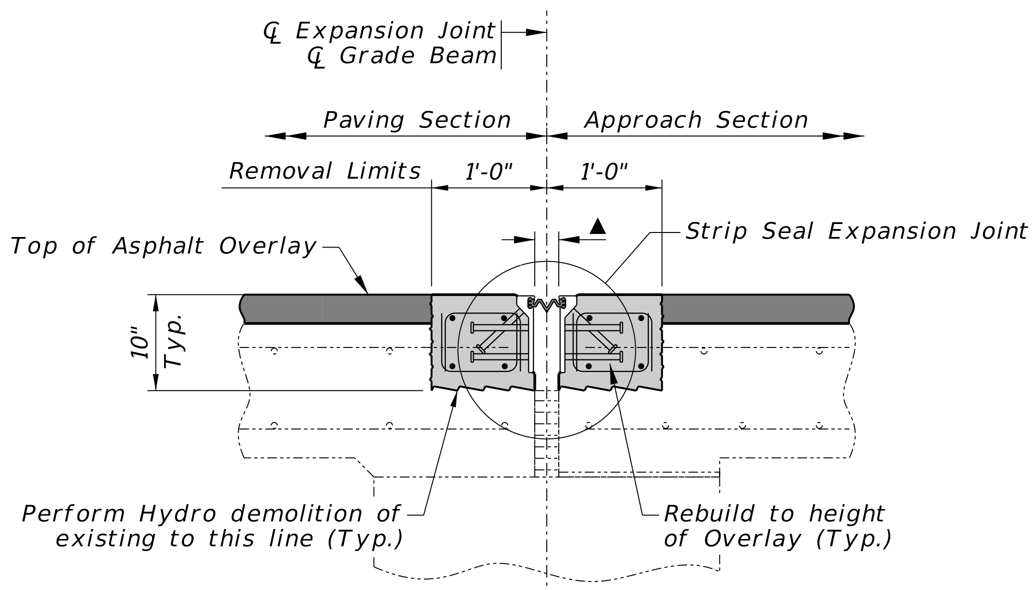
Figure 7.9—Small Movement Joints at Initial Construction and After Delayed Asphalt Application

7.2.3.5.1b—Medium Movement Joints

The example shown in Figure 7.10 uses a simple gap in the initial condition. At the time of overlay, hydro demolition is used to remove existing approach slab concrete to the depth needed for installation of a strip seal anchorage.



A. Initial Construction

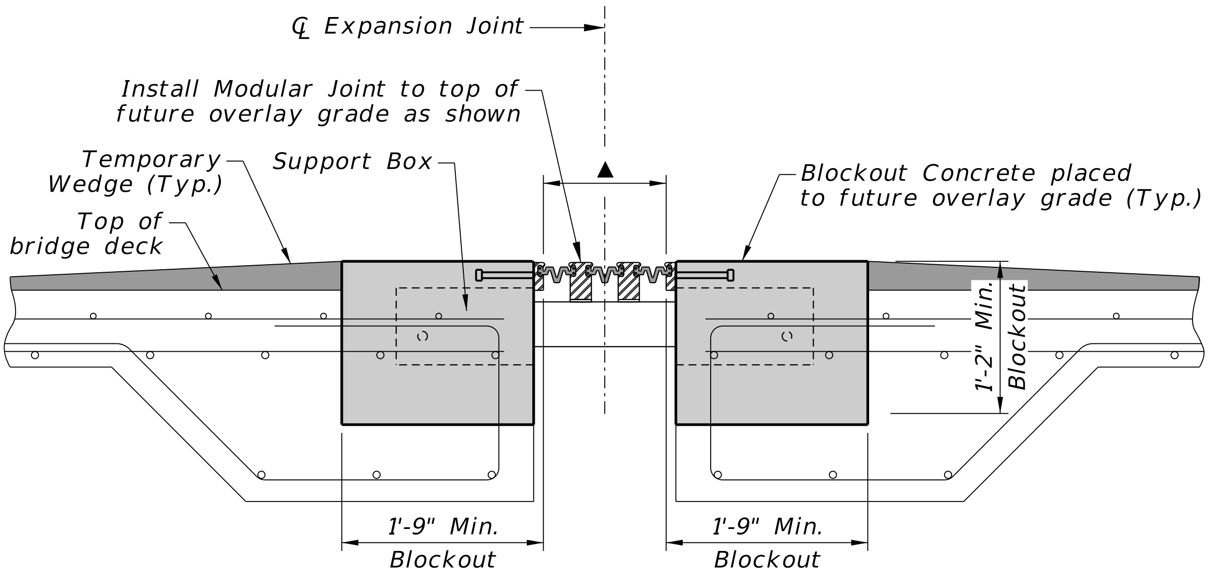


B. After Delayed Asphalt Overlay

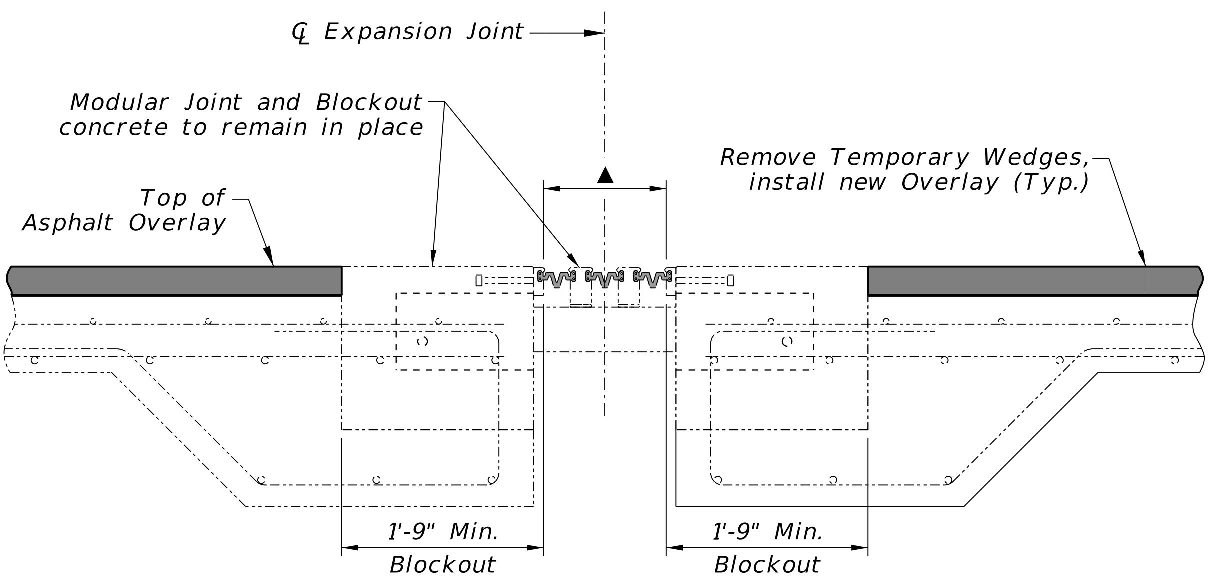
Figure 7.10—Medium Movement Joints at Initial Construction and After Delayed Asphalt Application

7.2.3.5.1c—Large Movement Joints

The example shown in Figure 7.11 utilizes temporary wedges on each side of a large modular expansion joint device to install the joint to overlay grade at the time of initial construction. Coordination with Roadway Division will be required.



A. Initial Construction`



B. After Delayed Asphalt Overlay

Figure 7.11—Large Movement Joints at Initial Construction and After Delayed Asphalt Application

7.2.3.6—Removal of Existing Overlays

7.2.3.6.1—Concrete Overlay to be removed

When placing a waterproof membrane on a bridge with an existing concrete overlay, the existing overlay shall be removed to the top of the concrete bridge deck. The pay item "CONCRETE SURFACE MILLING" shall be included in the plans. In exceptional cases, AC+M may be placed on top of existing concrete overlays.

Common practice prior to 2010 was to place concrete overlays on decks at the time bridges were widened. It was typical for bridges to also have new bridge approaches built to the height of the concrete overlay. To meet minimum overlay thickness on the approach slabs, the top of the approaches are milled for 1 in., while the concrete overlay on the bridge is removed. This lead to a 1 in. grade raise, as shown in Figure 7.12.

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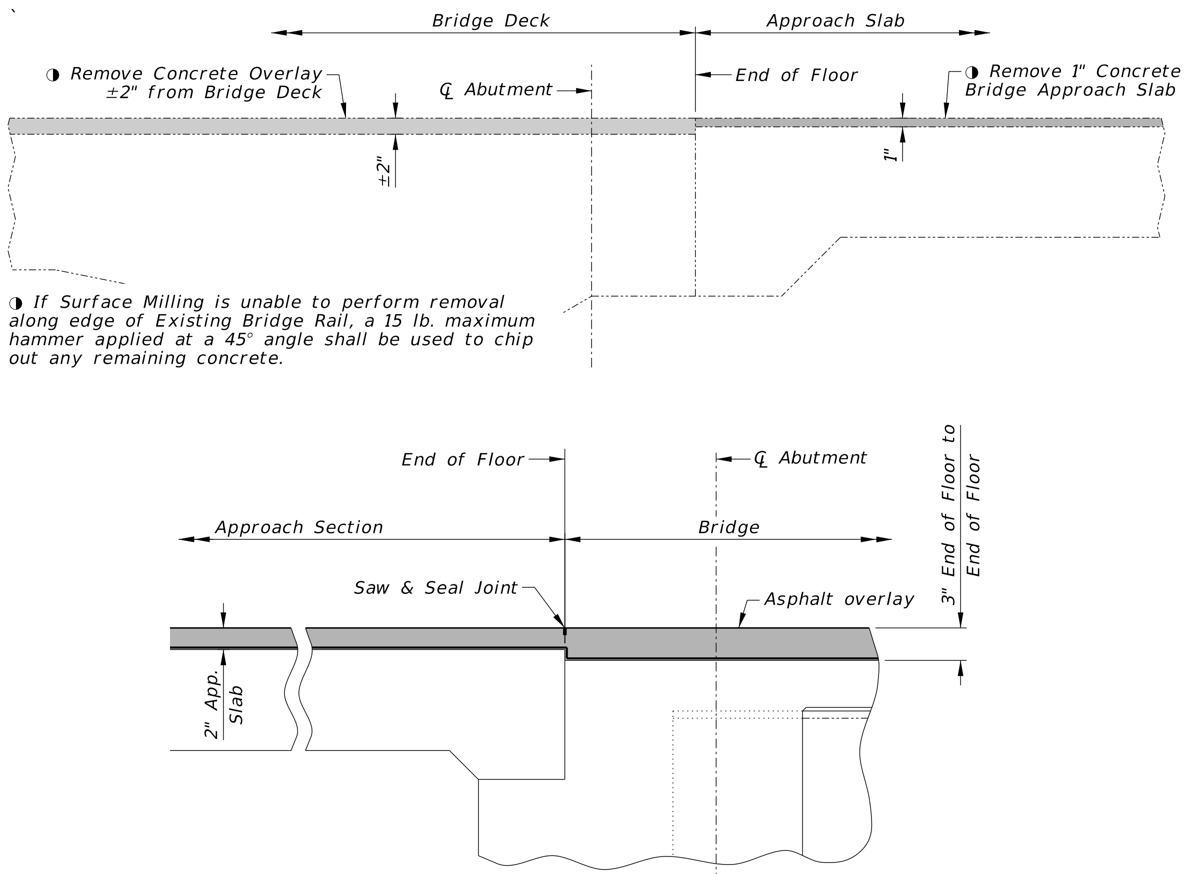


Figure 7.12— Removal and Asphalt Overlay Detail for Bridges with Existing Concrete Overlays (example taken from S025 00256, CN 71227, 2023)

7.2.3.6.1a—Parabolic deck cross-section

For existing bridges that have a parabolic deck cross section, a detail similar to Figure 7.13 should be shown on the plans.Asphalt Overlay to be removed

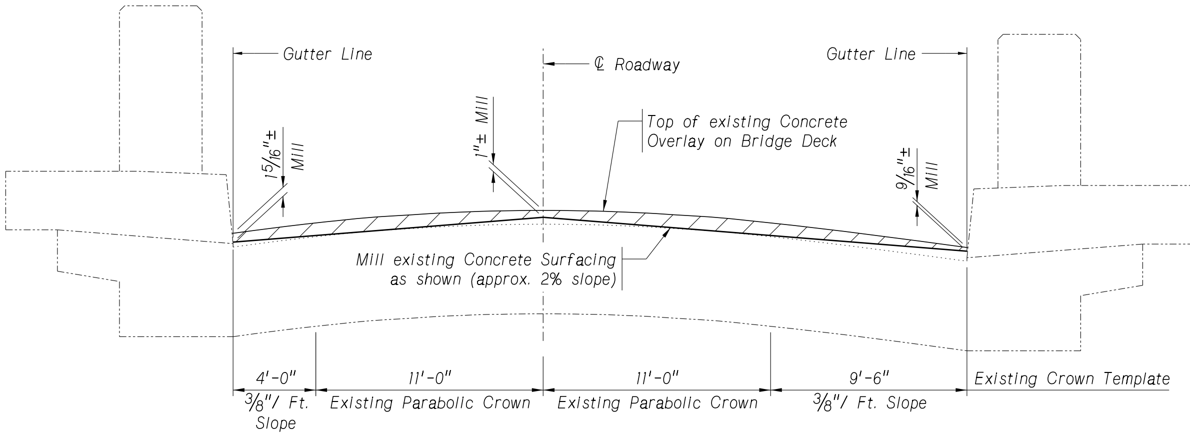


Figure 7.13—Milling of Existing Concrete Overlay for Parabolic Bridge Decks (example taken from S080 32744R, CN 42888, 2020)

7.2.3.6.2—Asphalt Overlay to be removed

When placing a waterproof membrane on a bridge with an existing asphalt overlay, the existing asphalt shall be removed to the top of the concrete bridge deck. The pay item "REMOVE ASPHALT SURFACE FROM BRIDGE" shall be included in the plans.

7.2.3.6.3—Waterproof Membrane to be removed

In cases when there is an existing waterproof membrane on the bridge that is intended to be removed, it shall be called out in the plans. Typically, the removal of existing membranes is laborious, and contractors need to be made aware of their existence.

7.3—COMMON REMODELING GUIDELINES AND EXAMPLES

7.3.1—Approach Slab Addition or Replacement

The following guidelines apply to adding and replacing Approach Slabs.

This policy attempts to encapsulate the most common Approach Slab Addition/Replacement scenarios and may not apply to all situations.

7.3.1.1—Approach Slab

7.3.1.1.1—General

Approach addition/replacement shall follow the same criteria as new approach slabs, see §15.1.

As part of an Approach addition/replacement, designers shall, as much as possible, remodel EOF to move any existing joint to the new grade beam location.

7.3.1.1.2—Preliminary Investigation

Review of the As-Built Roadway Plans is needed in many cases to determine whether there are approach slabs on the existing bridge. This determines whether Approach slabs are being added or replaced.

- Approach Slab Replacement - Most existing Bridges were built with Reinforced Concrete Approach Slabs, which for many years were included in the Roadway Standard Plans.
- Approach Slab Addition - In less common circumstances, the existing bridge will not have Approach Slabs. This is usually in cases where the adjacent roadway is purely asphalt pavement. For these cases, Approach Slabs will have to be added to the bridge.

Do not assume the presence of an approach seat on the turndown or abutment indicates the presence an approach slab, this was not uncommon in the past.

Satellite imagery has become high enough resolution it is often possible to tell an approach slab exists just by looking for reflective cracking at the end of floor and grade beam/sleeper slab.

7.3.1.1.3—Abutment Pile Evaluation

When adding Approach Slabs to an existing Bridge, the designer shall evaluate the capacity of the existing Abutment Piles to determine whether they can handle the additional load of the new approach slabs and turndowns (if necessary). Live loading for Abutment Pile Evaluation shall be per §3.4.4.

For Approach Slab Replacements, no analysis of the Abutment Pile is needed. The Abutment Pile are assumed to have accounted for this load during the original design.

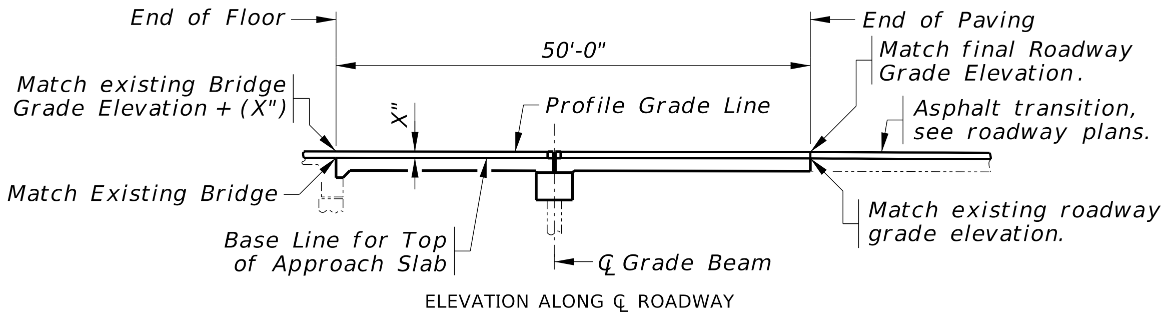
Until Chapter 3 is published, use the following for live loading:

Evaluate Abutment Piles for the live loading that was used when the bridge was constructed.

7.3.1.1.4—Use of Relative Elevations

For Approach Slab addition/replacement, unless the bridge is undergoing more extensive remodeling work (Redeck or Rehab) on the project, it is typically not necessary for the designer to calculate and report Profile Grade Elevations in the plans.

In these cases, the profile grade can be established in the field during construction. Figure 7.14 shall be put in the plans for the field personnel.



NOTES:
Profile Grade elevations for approach slabs shall be established in the field during construction.

Base Line Elevations for top of Approach Slab shall be built X" below Profile Grade Line to account for a X" Asphalt overlay depth.

Grade elevations for grade beams and other desired locations shall be determined by straight line interpolation along field established Profile Grade Line.

Details shown in the plans for the approach slab, grade beam and other components shall be used to set elevations below grade.

<u>BASE LAYOUT OF APPROACH SLABS</u>

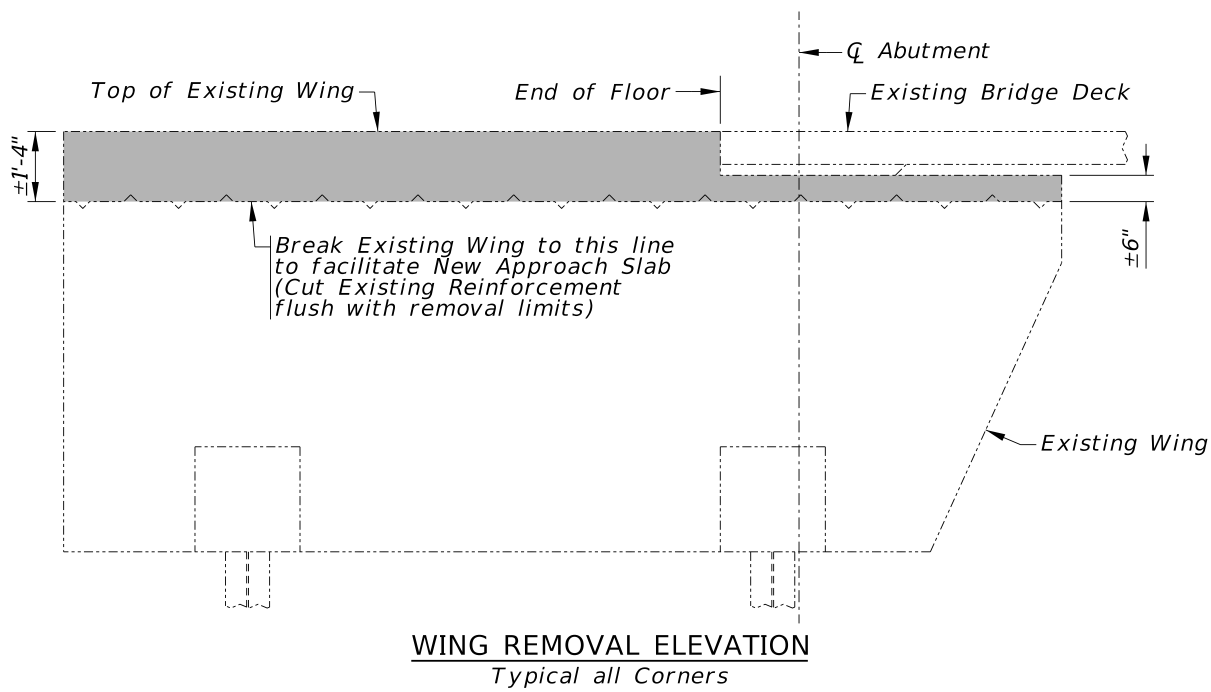
Approach Slab No. 2 shown, Approach Slab No. 1 similar

Figure 7.14—Base Layout for Use of Relative Elevations in Approach Slab Remodel (available as Cell “013-Appr. Slab Base Layout”)

7.3.1.2—Existing Wingwall Remodel

In most approach slab replacement projects, the existing Abutment wings will be incorporated into the new work. When existing wingwalls interfere with placement of new approach slabs, the top of the wing wall shall be broken down to accommodate the thickness of the new approach and 2 in. of polystyrene, see Figure 7.15.

Existing Abutment wing walls that interfere with new Approach Slab placement will typically be broken down 1 ft. 4 in.± from the top of the new Approach Slab elevation. This depth will accommodate the placement of a 1 ft. 2 in. thick Approach Slab and a layer of 2 in. polystyrene to be placed atop the Wing Wall.



<u>WING REMOVAL ELEVATION</u>

Typical all Corners

Figure 7.15—Partial Wingwall Removal to Accommodate New Approach Slab (example taken from S006 36293)

7.3.1.3—Adding or Replacing Sheet Pile at Abutments

Installation of new Steel Sheet Pile to convert existing flared wings to U-Type and/or behind an existing abutment cap need only be considered if there is a hydraulic concern regarding scour, which will require coordination with the Hydraulics Section, or a structural concern regarding the condition of the existing wings, see Sheet Pile Policy in §11.1.5.

7.3.1.4—Turndown Modifications

7.3.1.4.1—Preliminary Investigation

Before initiating final plans, designers should investigate, and sketch out as necessary, the as-built details of the abutment, expansion joint, and bridge deck to determine the most appropriate backwall modification for the bridge.

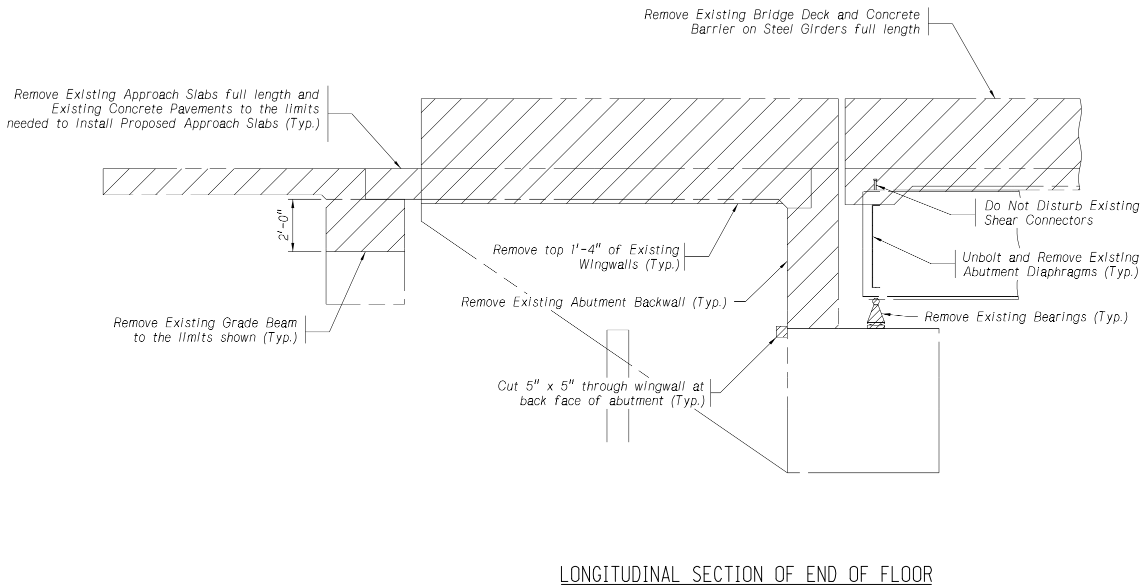
When eliminating an existing expansion joint with a turndown, it's important to keep the remaining abutment substructure elements isolated from the expansion and contraction of the superstructure. This is particularly true on long span bridges subject to large thermal movements.

7.3.1.4.2—Existing Abutment/Backwall Considerations

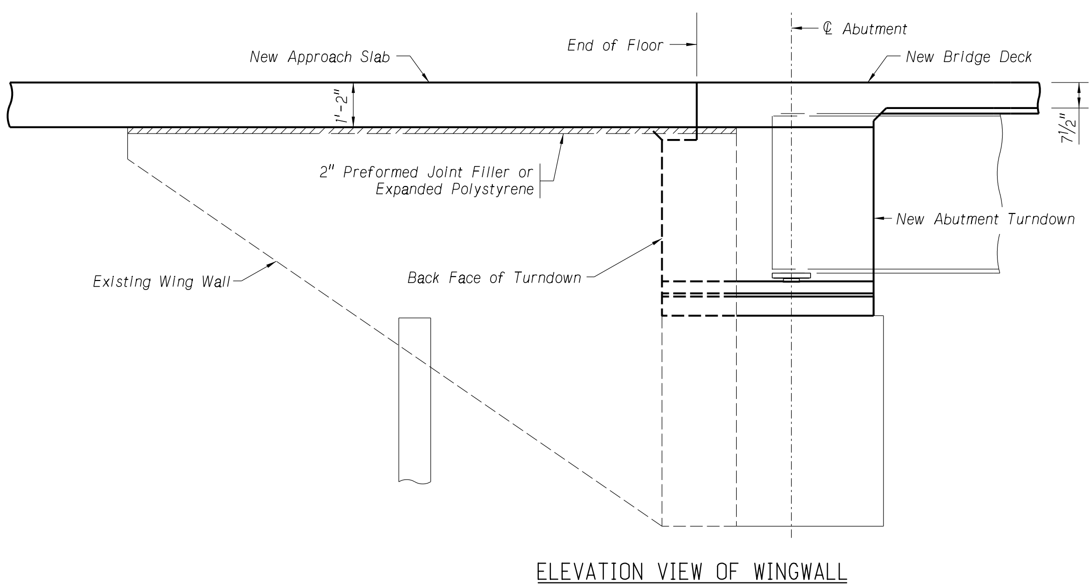
Typical Existing Bridge Abutment scenarios are listed below.

7.3.1.4.2a—Backwall Abutment

Remodel End of Floor with turndown, move expansion joint to grade beam, see Figure 7.16 for example details.



A. Removals



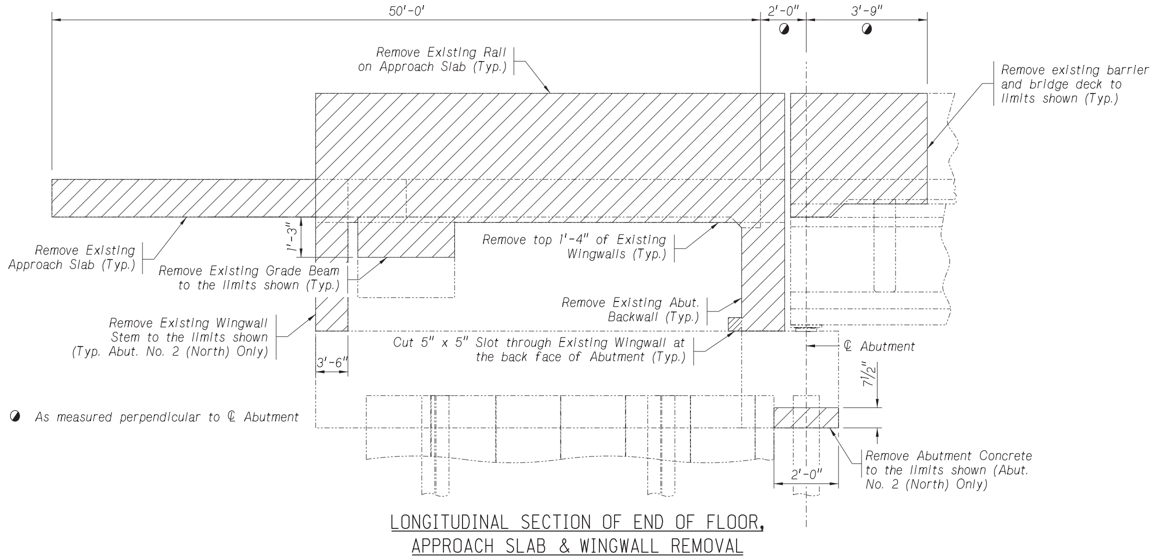
B. Final Condition

Figure 7.16— Remodeling of Abutment with Backwall to Semi-Integral Abutment with Expansion Joint at Grade Beam. Bearings without Anchor Rods

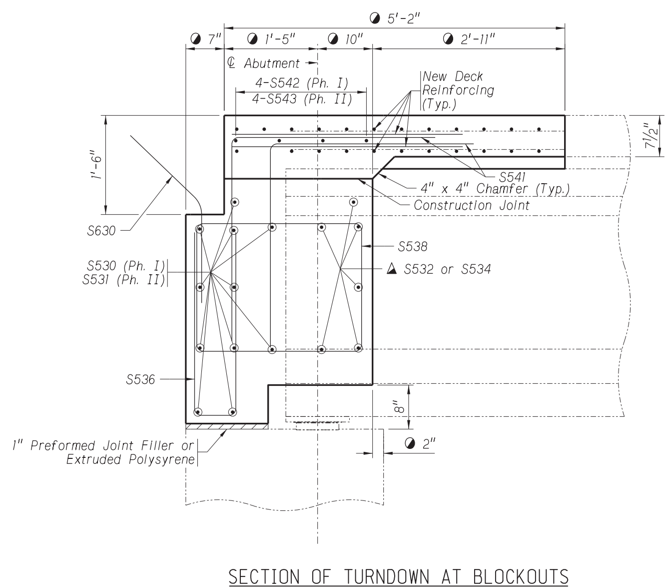
(example taken from S064 07011R CN 22689, 2018)

7.3.1.4.2b—Bearings with Anchor Rods

Block out turndown around existing bearings as needed to accommodate proper functionality of the components (Figure 7.17).



A. Removals



B. Turndown Final Condition

Figure 7.17—Remodeling of Abutment with Backwall to Semi-Integral Abutment with Expansion Joint at Grade Beam. Bearings with Anchor Rods

(example taken from SL56G00048, CN 61590, 2020)

7.3.1.4.2c—Existing Abutment Cap without Pedestals & Heavy Skew

Consider guiding expansion by installing a stepped grade beam as seen in Figure 7.18.

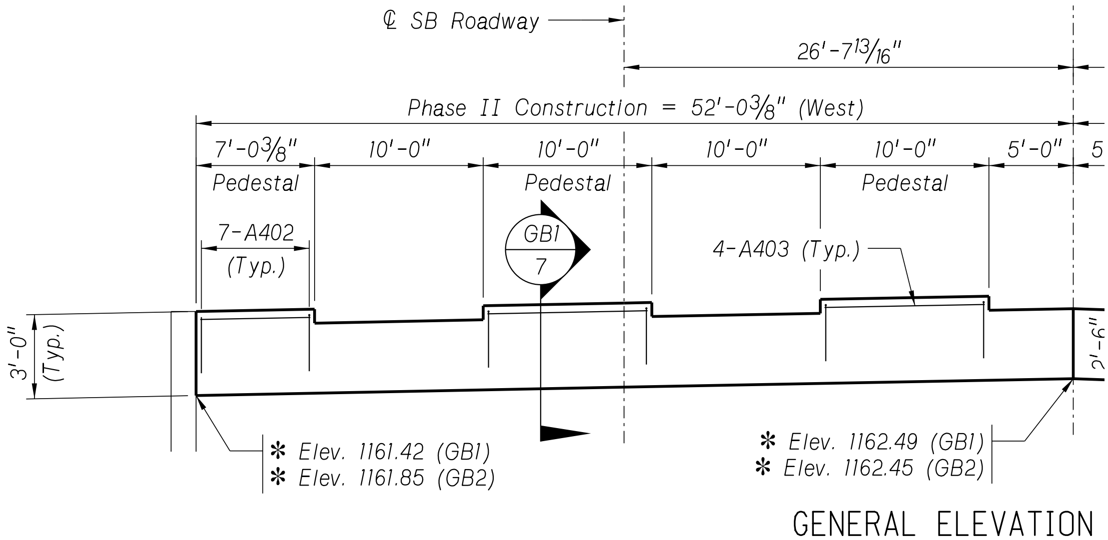


Figure 7.18— Stepped Grade Beam to Guide Expansion for High Skews without Pedestals at Abutments (example taken from S080 40350, CN 13111, 2016)

7.3.1.5—Adding Turndowns

7.3.1.5.1—Full Depth Turndown

Turndown details should closely follow the examples shown in §9.5.2.

Blockouts shall be detailed around girder bearings with anchor rods. The depth of the blockout shall be sufficient to accommodate the full range on anticipated movement

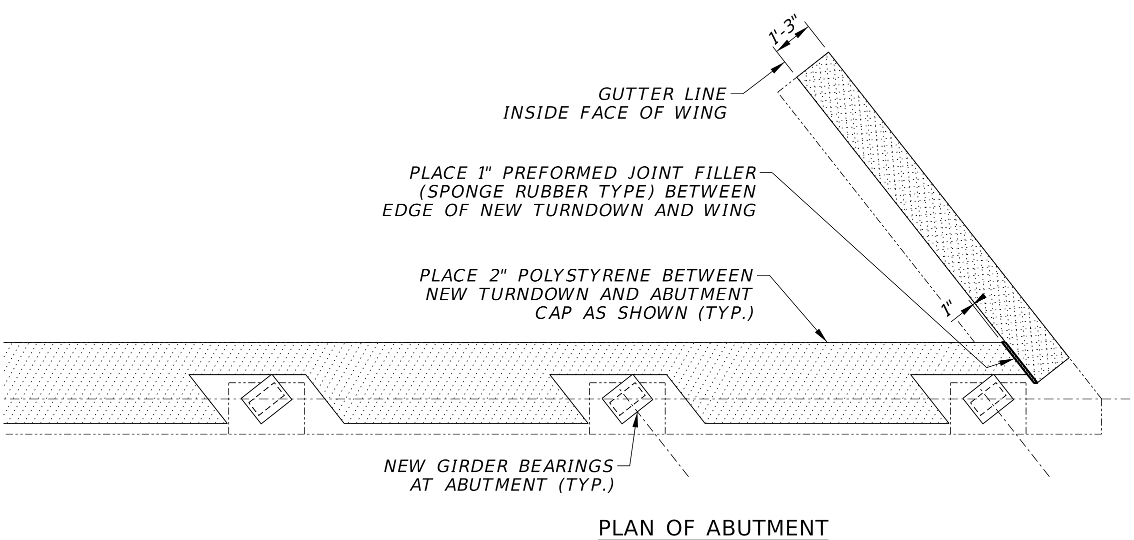


Figure 7.19—Joint Filler for Turndown Remodel

(example taken from S030 42623R, CN 22688, 2019)

at the support. The skew of the existing pedestals should also be considered see §9.5.1 for blockout dimensions.

Joint filler or Polystyrene (2 in. minimum on horizontal faces, 1 in. minimum on vertical faces) shall be installed between the turndown and abutment. Wings may also need to be remodeled/removed at the intersection of the abutment to accommodate the new turndown, see Figure 7.19.

7.3.1.5.2—Partial Turndown

A partial turndown provides the benefits of eliminating the joint at the end of floor without removing the back wall completely. It also avoids forming around/blocking out the existing girder pedestals and bearings.

Use of Partial turndown details are encouraged in cases of deep superstructures (depth greater than 3 ft.) with existing backwalls in good condition. Heavy skew angles with challenging abutment geometry are also good candidates for utilizing these details.

When using partial turndown details, High-Density Polyethylene sheets are used to provide a sliding surface is installed between the remaining backwall and new turndown, see Figure 7.20.

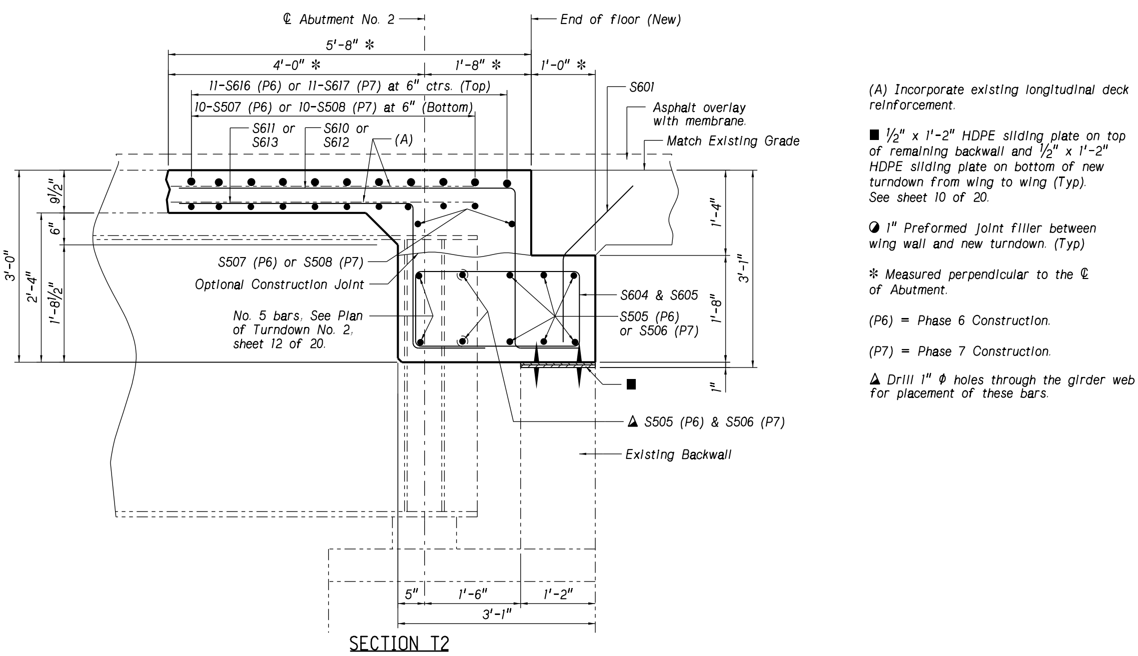


Figure 7.20—Partial Turndown with HDPE Sliding Plates
(example taken from S080 45314R, CN 22646, 2017)

7.3.1.6—Approach Seat Modifications

Typical bridge/abutment types that can be used in place for Approach Slab work are shown below.

- Existing Slab Bridge — May be used in place without need to remodel the deck side of the bridge. The minimum approach slab seat width is 7 in.
- Existing Turndown Bridge — May be used in place. Typically, only requires replacement if the existing turndown is in extremely poor condition. Most issues with existing turndowns are localized in nature and can be repaired with Concrete Patching.
- Existing Integral Abutment — May be used in place. Minor remodeling of the deck side may be required for situations where the existing joint is on the back face of the abutment at the end of floor. These joints should be moved to the Grade Beam.

For these cases dowel tie bars into the existing Approach Slab seat to tie in the new approach slab. Typically #6 @ 12 in. centers are used for the dowels.

7.3.1.7— Grade Beam

Grade Beams for Approach work on existing bridges shall follow the same criteria as Grade Beams on new Bridges, see §11.1.6. Live loading for new Grade Beam Piles shall be per §3.4.5.

In special replacement circumstances (MSE straps, utilities, etc.) Approaches may be replaced in kind, on top of an existing sleeper beam with Bridge Division approval.

7.3.1.7.1—Geotechnical Coordination

Pile recommendation from Geotech is necessary for approach slab projects. In many instances, Geotech will recommend the existing borings from the As-Built Plans be used on the geology sheet for approach work instead of taking new borings.

When this is the case, the boring information can be shown in the plans without developing a full geological profile. A line to approximate the bottom of the grade beam should be shown on the boring details as seen in Figure 7.21.

Until Chapter 3 is published, use the following for live loading:

Evaluate Grade Beam Piles for the live loading that was used when the bridge was constructed. For HS-25 and less, report the design pile bearing in tons. For HL-93, report the design pile bearing in kips.

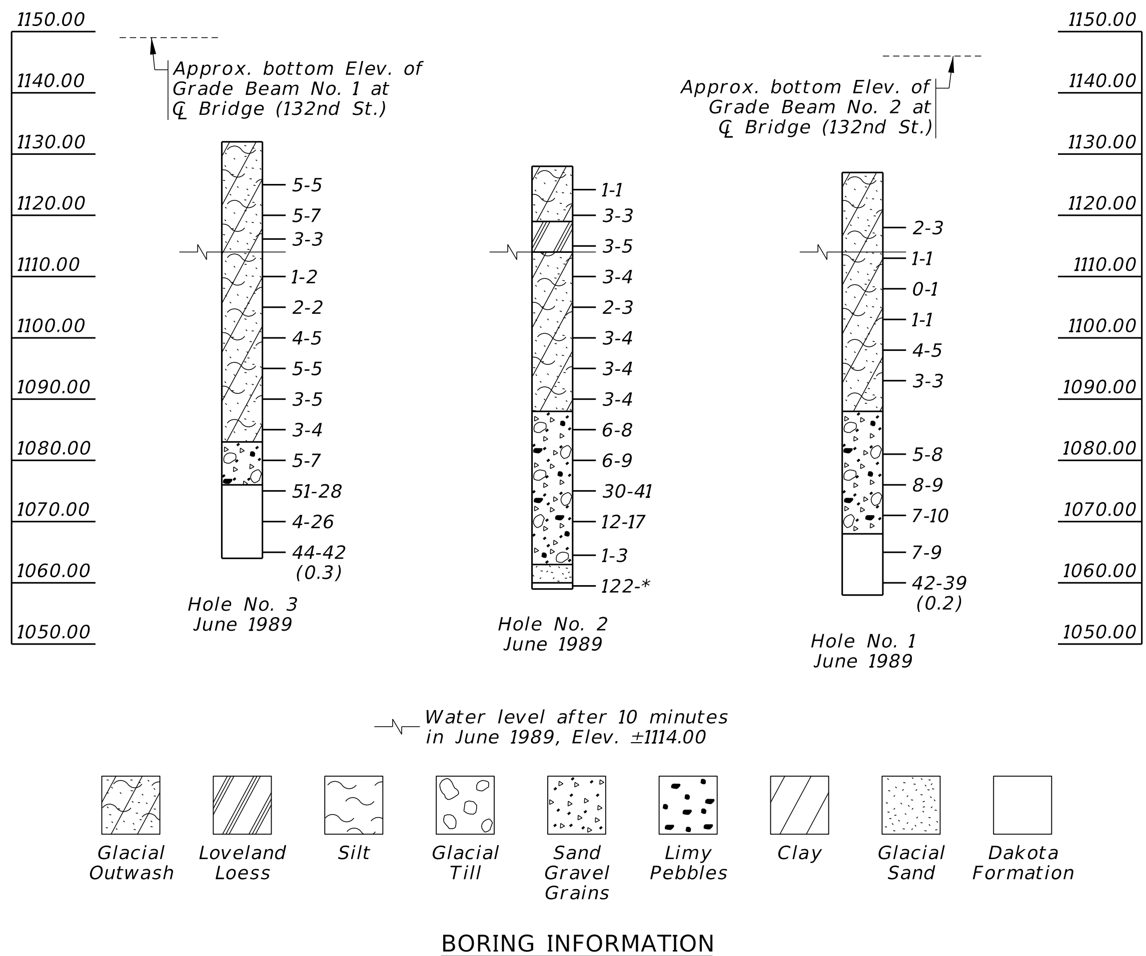


Figure 7.21— Reuse of Existing Borings for New Approach Slab

(example taken from S006 36293 CN 22812, 2023)

7.3.1.7.2—Helical Pile

Coordination with M&R Division Geotechnical Section will be required to check for location suitability. Geotechnical Section will not typically suggest helical pile, but will provide the information if requested, §10.3.

7.3.1.7.3—Field Measured Cut-off Elevations

When relative elevations are used and grade will be established in the field, report "Field Measured" for the cut-off elevations in the pile data table.

7.3.1.8—Granular Backfill

1 ft. 0 in. minimum of Granular backfill shall be placed under the Approach Section and Paving Section for Approach slab replacements and additions.

When remodeling the existing abutment for turndown, granular backfill should be called out to at least 6 in. below the lowest extent to the remodeling work. Use the shallow or deep detail as appropriate. See Granular Backfill Policy in §11.1.8.

7.3.1.8.1—Existing Granular Backfill

Some existing bridges have been constructed with granular backfill behind the pavement under the approach section. When replacing approaches on these bridges, backfill details acknowledging the existing backfill shall be provided, see Figure 7.22.

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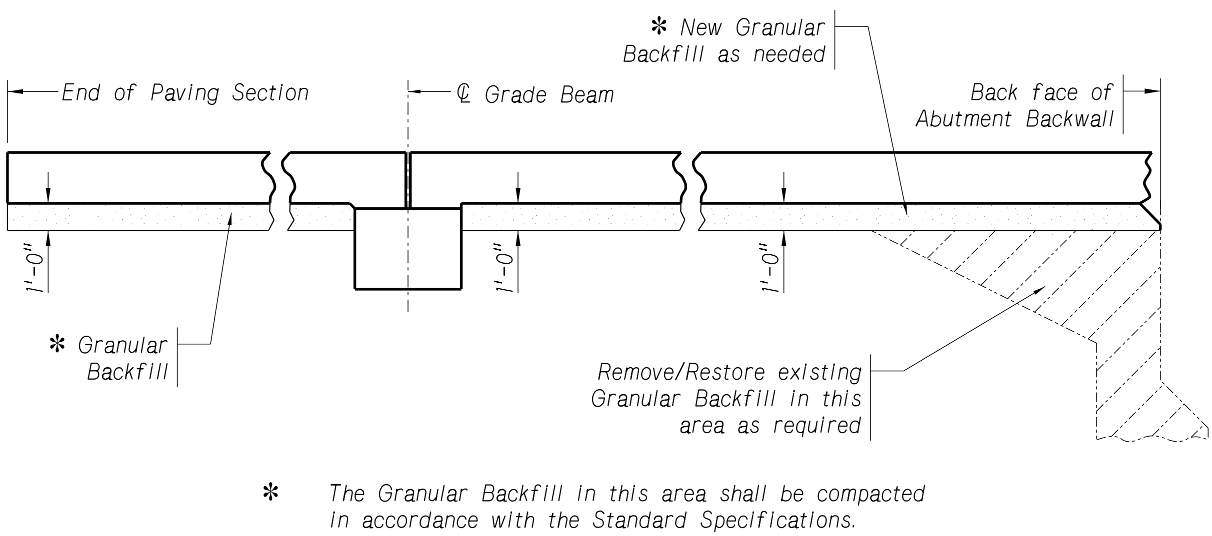

Figure 7.22—Section of Granular Backfill showing Extents of Existing Backfill (example taken from S077 12752, CN 22762, 2022)

7.3.1.8.2—Drainage of Granular Backfill

Vertical drainage matting can be effective at preventing water from passing through the cold joint between the turndown and abutment. When remodeling the existing abutment for turndown or new approach slab seat, drainage matting shall be installed behind the turndown. Standard details may need to be modified to fit the particulars of the situation, see Figure 7.23 for an example.

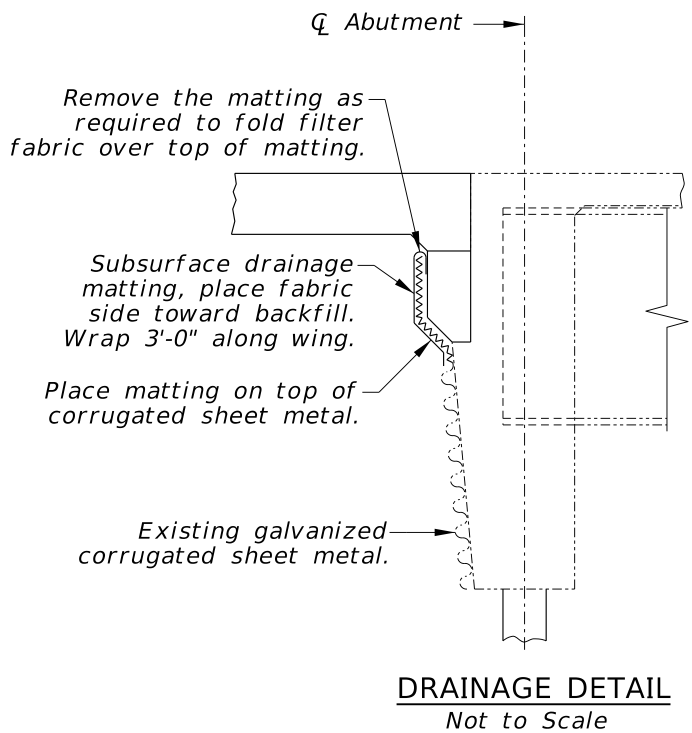

Figure 7.23—Abutment Drainage Detail with New Approach Slab Seat

(example taken from S075 09202, CN 22647, 2022)

If the designer wishes to install a transverse drainage system, it must be daylighted out of the granular backfill, see the example shown in Figure 7.24.

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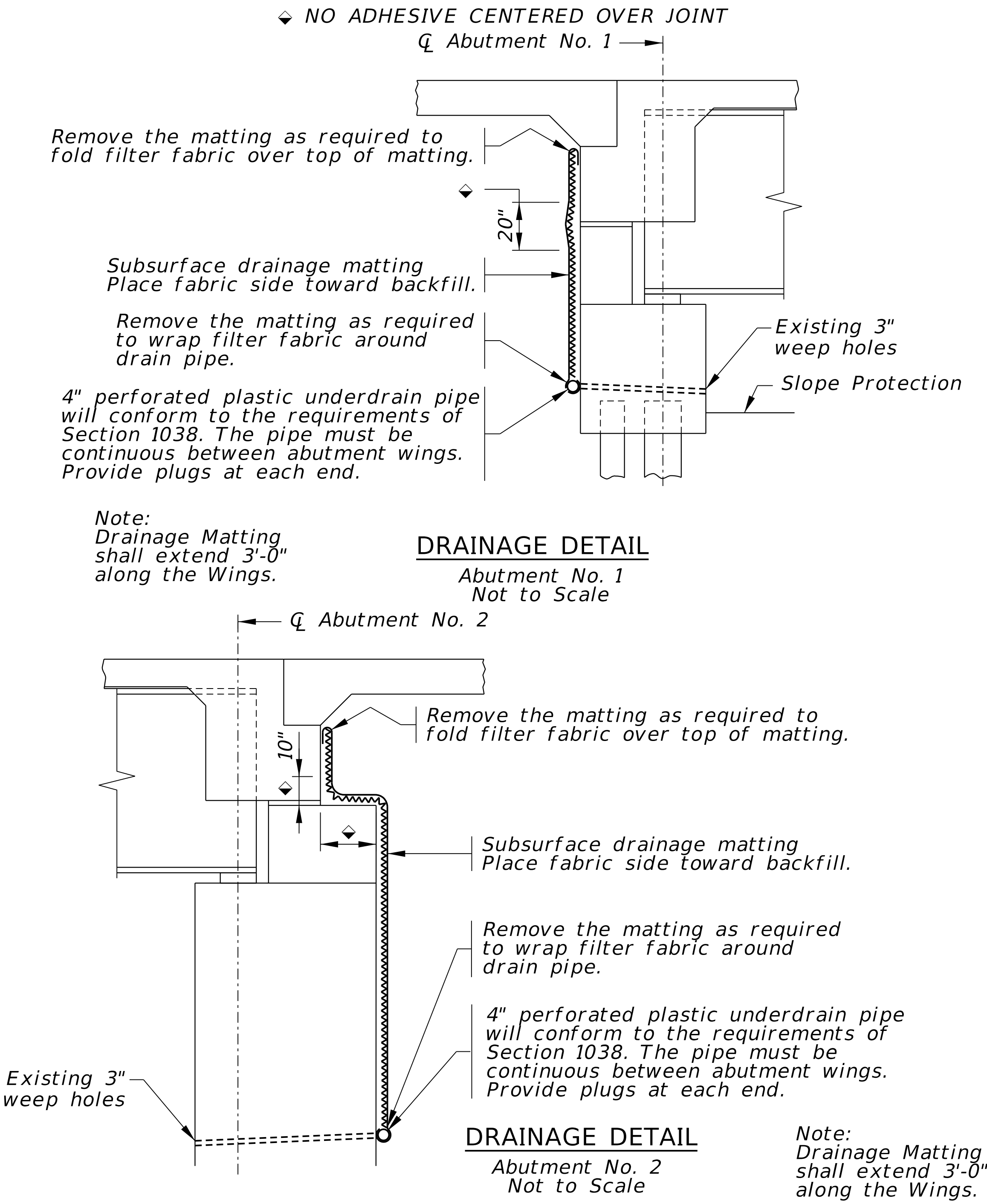


Figure 7.24—Abutment Drainage Details with Drainage Daylighting

(example taken from S075 09169, CN 22647, 2022)

7.3.2—Rail or Buttress Remodel

When scoping documents call for rail updates, the following guidelines should be followed.

Scoping documents use the following terminology to describe rail improvements:

- UPDATE RAIL – Work involving the removal of part or all the existing rail configuration and constructing a new concrete rail tied directly to the bridge deck. May include the term PARTIAL REDECK to include removal and replacement of the deck edge to facilitate the new rail. See §13.1 for more information.
- UPDATE BUTTRESS FOR MGS – Work involving the removal and replacement of the ends of the existing concrete rail to accommodate a new guardrail connection.

On 3R resurfacing projects, the guardrail will commonly be updated to MGS. The MGS will typically be installed with 34 in. guardrail height which requires the control bolt be located 24 in. above the top of deck. Most existing buttresses will need to be updated to satisfy these criteria. See NDOT Standard Plan 740-R1.

Responsibility for determining whether the existing Thrie Beam Connections need replaced rests with Roadway Design. See RDM Chapter 1 §6.C.2.a (Nebraska Department of Transportation, 2023). This will be determined during preliminary design or sometimes during the scoping phase of a project.

Due to the vast variety of existing rail configurations, it is not possible to explicitly address every possible configuration that may be encountered within the context of this policy. In the event an existing rail configuration is encountered that is outside the scope of this policy the appropriate rail remodeling strategy may be discussed with Bridge Division.

For many bridges built prior to 1980, the existing rail configuration consisted of a steel or aluminum handrail mounted to a concrete curb. When rail updates are scoped for bridges with this rail type, consideration should be given to replace the rail and deck edge to the same standards used for new construction.

Previous Bridge Division practice was to retrofit these rail types by construction of a concrete block type rail on top of the existing curb (referred to as a curb remodel), or by replacing the curb entirely with a new concrete block type rail connected to the deck is made using adhesive anchor bars. Few of the bridges of this type are still in service unaltered.

There is recently published research with Midwest Roadside Safety Facility and UNL to investigating options to retrofit existing buttresses for attachment of the MGS. More information can be found in Rosenbaugh et al., (2024). Bridge Division is currently investigating how to incorporate these devices into future plans.

7.3.2.1—Buttress Updates

See §13.3.2 for Standard End Section requirements for MGS Connections.

When the control bolt location is moved from its existing location, the new locations (Station & Offset) shall be coordinated with Roadway Design.

When using doweled in reinforcement as part of a buttress update plan, Special Provision 7-16 "Doweling into Concrete – Post Installed Adhesive Anchors" shall be used. In addition, a note regarding installation of these bars and the assumed hole depth shall be shown in the plans. These notes are available in the Repair cell library.

These scenarios are listed in order of preference. Flexibility for the exact location of the end of the Rail is required, depending on the configuration of each existing bridge project.

7.3.2.1.1—New Approach Slab

Place guardrail connection with the Standard End Section 6 ft. 6 in. from the centerline of the grade beam (Table 7.5).

Table 7.5—Examples of Buttress Update with New Approaches

Structure №	Existing Rail Type	Control №	Sheet №	Plan Year
S002 30589	29 in. Nebraska Rail	61596	4, 10	2017
S014 02929	32 in. NJ Barrier	42566	3, 11	2018

7.3.2.1.2—Existing Approaches with an Existing End Section on the Paving Section

Remove and remodel End Section on Paving Section Side of the Grade Beam (Table 7.6).

Table 7.6—Examples of Buttress Updates with Existing Approach Slabs and without At-Grade EJ Blockout

Structure №	Existing Rail Type	Control №	Sheet №	Plan Year
S080 42831L&R	34 in. NU Rail	13279	3, 5	2018
S080 11725	29 in. Nebraska Rail	61565	4	2018
S080 44292	42 in. NJ Barrier	22524	3, 5	2016
S080 45180R	42 in. NU Rail	22594	3, 5	2019

When encountering existing bridge rails ends with a 4 in. tall blockout above the deck on each side for the expansion joint, it is good practice to remodel the rail on both sides of the joint to eliminate the gap (Table 7.7).

Table 7.7—Examples of Buttress Updates with Existing Approach Slab and At-Grade EJ Blockout

Structure №	Existing Rail Type	Control №	Sheet №	Plan Year
S084 00825	29 in. Nebraska Rail	31923	3, 5	2017
S066 10571	29 in. Nebraska Rail	13100	4, 5	2017
S084 04418	29 in. Nebraska Rail	32064	5	2017

7.3.2.1.3—Buttress Mounted on Wings or on End of Bridge at End of Floor

No approach work scoped, existing bridge rail is mounted on U-type wing or ends on the Bridge at End of Floor

Widen the approach slab as required, and place guardrail connection with Standard End Section on Approach Side of the End of Floor Joint, on top of widened approach slab (Table 7.8). This will require breaking down the existing wingwalls (see §7.3.1.2).

Many existing bridges were constructed with the Guardrail connections mounted directly onto the Abutment wings. This is known by the Bridge Division to be an undesirable detail, as it creates a full depth joint between the wing and approach slab. This joint allows water to seep in behind the abutment, which can lead to damage of the substructure elements and loss of backfill.

Table 7.8—Examples of Buttress Updates With Existing Approach Slabs and with Existing Buttress Mounted to Deck or Wings

Structure №	Existing Rail Type	Control №	Sheet №	Plan Year
S034 36557	29 in. Nebraska Rail	12988	3, 4	2016
S030 42628R	32 in. NJ Barrier	22688	3, 6	2019
S283 04514	29 in. Nebraska Rail	71184	3, 5	2017

The designer can also choose to extend the rail along the full length of the Approach Section and construct a new Standard End Section on Paving Section Side of the existing Grade Beam in these situations.

7.3.2.1.3a—Collision Damage

In the case of collision damage for rails mounted directly on wings, It is acceptable to replace in kind the wing mounted rail and any damaged portions of the wing.

7.3.2.1.4—Flared Wings

No approach work scoped, bridge has flared wings, no existing approach slabs (or approach slabs unknown)

Place guardrail connection with either the Standard or Retrofit End Sections at End of Floor, on the Bridge itself (Table 7.9).

Table 7.9—Examples for Buttress Updates without Approach Slabs and with Flared Wings

Structure №	Existing Rail Type	Control №	Sheet №	Plan Year
S183 02243	29" Nebraska Rail	71189	2, 3	2019
S059 04555	29" Nebraska Rail	32126	3	2018

7.3.3—Bridge Joints Remodel—Repair or Replacement

The following guidelines apply for removing, repairing, and replacing joints on Bridge Preservation projects.

Expansion Joints for existing bridges are sized based on expected Temperature Movement at the joint location. See §3.10 for guidance on calculating the Temperature Movement (TM) for expansion joints.

7.3.3.1—Elimination of Deck Joints

Many existing bridges are configured with Expansion Joints placed at the end of bridge deck adjacent to an abutment backwall, or an integral abutment with the joint oriented between the bridge and approach at the end of floor. If scoping documents call for addition or replacement of Approach Slabs, these joints should be moved to the gap between the Approach Section and Paving Section. See §15.1 for more information.

This mainly applies to joints near the end of floor. Existing Bridges with interior deck joints are more complicated to eliminate, as they are typically only on long span bridges with high TM, on bridges with simply supported superstructures, or on Pin & Hanger type bridges that would require replacement of the existing bearings to reconfigure the expansion movement. For these reasons interior deck joints are typically maintained in their existing location.

Some DOTs have had good results using UHPC link slabs to eliminate deck joints, but these generally call for bearing replacements (Haber et al., 2022). Discuss the use of UHPC link slab with the Bridge Division.

7.3.3.2—Expansion Joint Remodeling

The following guidance applies for joints that are scoped to remain in their existing location.

If an existing joint, the concrete surrounding the joint, or both are in poor or severe condition due to advanced deterioration, the expansion joint location should be remodeled.

The designer will make this determination during design of the repair plans by performing a visual assessment of the joint. Table 7.10 can be used as a guide to aid the decision-making process.

The decision whether to remodel the surrounding expansion joint concrete is subjective, and can be influenced by several factors, including joint age, overlay type, and location on the structure.

The terms "Good", "Fair", "Poor", and "Severe" are terminology found in NBI and the NDOT BIP Manual.

See §14.1 for guidance when remodeling existing Large Movement Bridge deck joints.

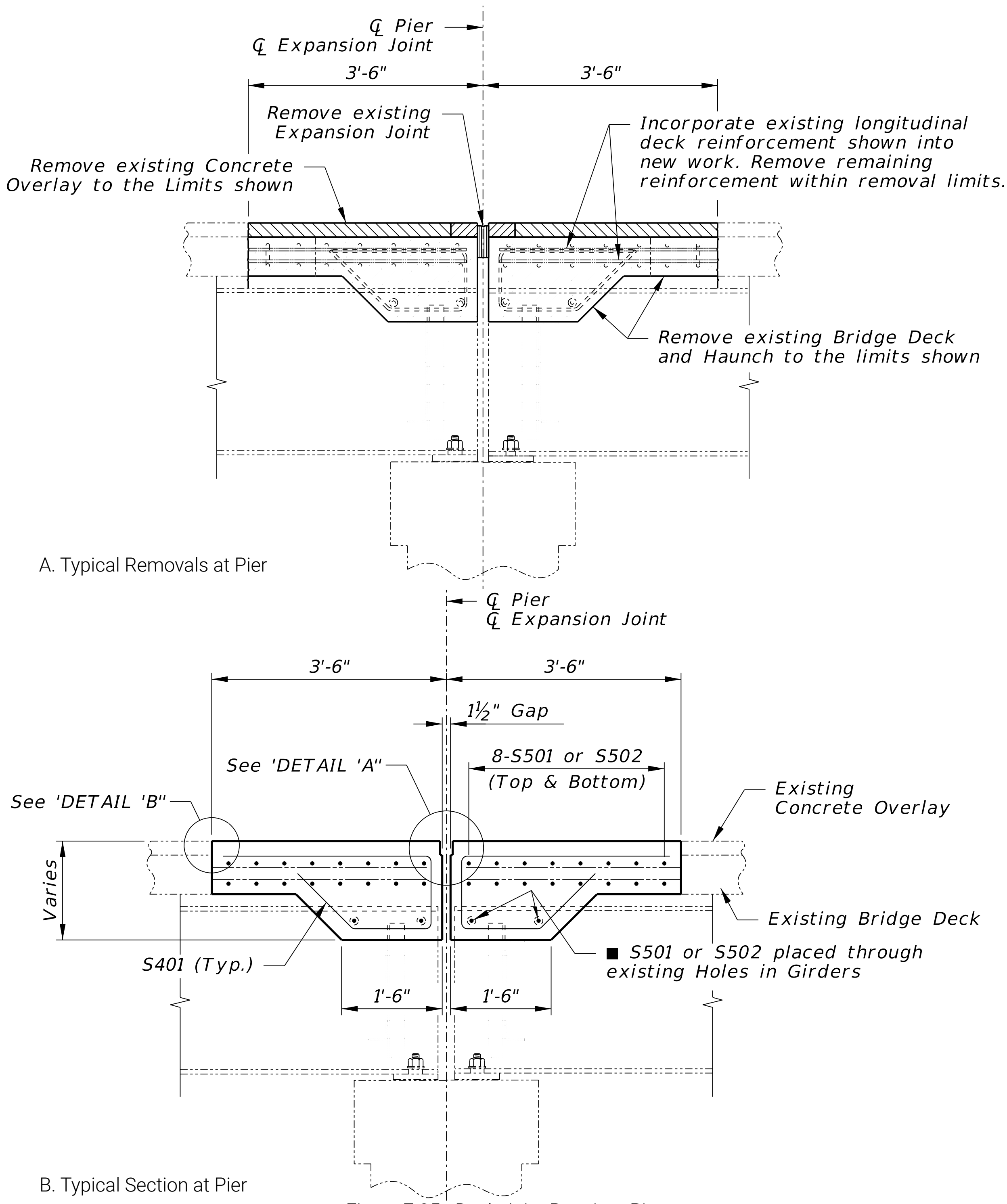
Table 7.10—Guidance For Expansion Joint Remodeling

Condition of Existing Joint and Surrounding Concrete	Recommendation
Good - Fair	Perform Minor Repairs*
Poor - Severe	Remodel Concrete

* Bridge Deck or Approach Slab Repair Pay Items can be used to repair the concrete surrounding the existing joint. It can also be used to replace damaged portions of existing steel armoring.

7.3.3.2.1—Expansion Joint at Intermediate Supports

Figure 7.25 shows a remodel at a Pier Deck Joint. The deck concrete shall be broken back the distance needed to achieve a full development length, l_d, on the existing longitudinal bars into the new concrete. Edge Beams (Thickened haunches on each side of the joint) should be used.



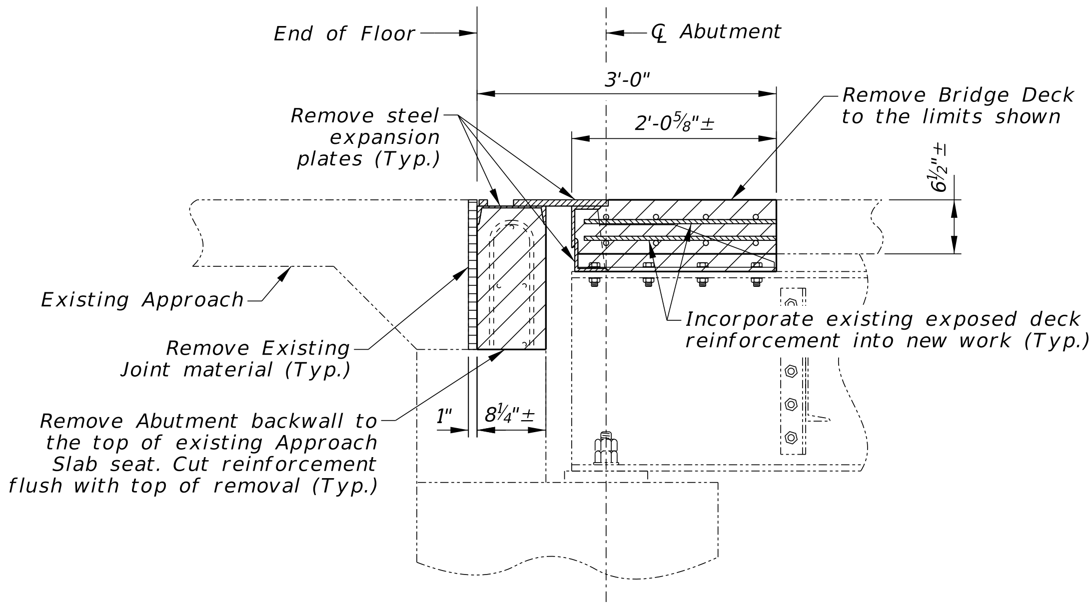
A. Typical Removals at Pier

B. Typical Section at Pier

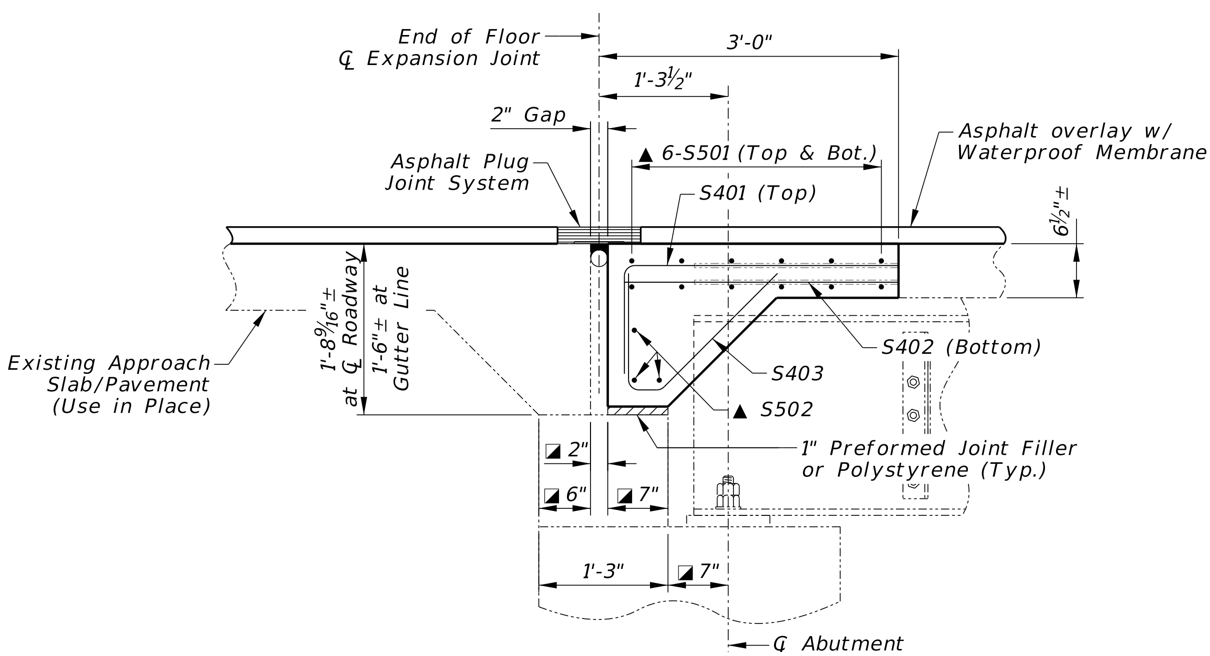
Figure 7.25—Deck Joint Repair at Pier
(example taken from S275 03900, Contract ID M3003, 2016)

7.3.3.2.2—Expansion Joint at EOF

When remodeling existing Expansion Joints at the End of Floor, two joints can be combined into one in certain situations by removing the top portion of the existing backwall and extending the deck, as shown in Figure 7.26.



A. Typical Removal at EOF



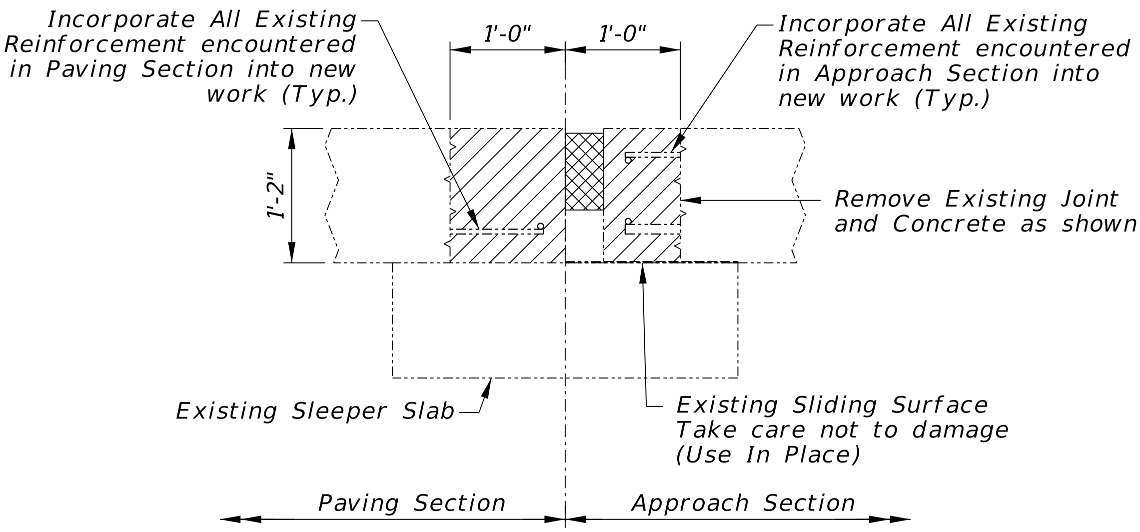
B. Typical Section at EOF

Figure 7.26—Joint Remodel at EOF with Backwall
(example taken from S025 03360, CN 70633, 2013)

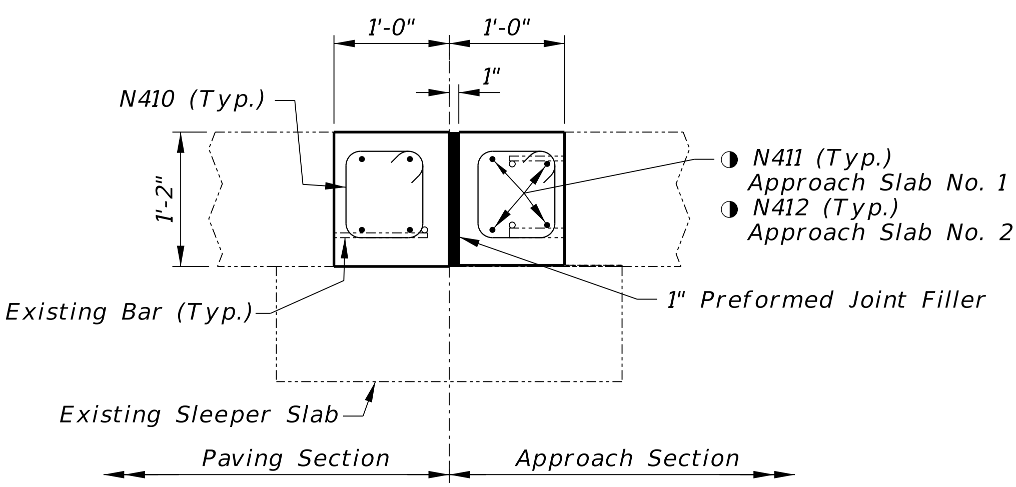
7.3.3.2.3—Expansion Joint at Grade Beam or Sleeper Slab

Expansion Joint Remodeling can be utilized at existing Approach Slab joints over the Grade Beam or Sleeper Slab if conditions warrant. To avoid loss of support for the approach slab, existing concrete shall be broken 1 ft. each direction of the existing joint. Figure 7.27 shows the remodeling of an existing oversized joint in poor condition to a new gap width more appropriate for the anticipated movement at the location.

A similar detail can be used to remodel existing strip seals to PPFs, when TM allows.



A. Removal at Existing Pressure Relied Joint



B. Final Condition with New Gap

Figure 7.27—Expansion Joint Remodel, Pressure Relief Joint over Grade Beam or Sleeper Slab (example taken from S080 45252, CN 22594, 2019)

7.3.4—Bearing Replacement

Existing bearings shall be replaced with the same type of Bearings used for new Bridges, see §14.2 for more information regarding new bearings. Refer to §3.4.5 for the live loading to be used for bearing replacement.

Rocker Bearings can be replaced in kind on an individual basis. This approach should be reserved for cases where only some of the existing bearings need to be addressed.

Until publication of Chapter 3, use the live loading for bearing replacement:

The live loading that was used when the bridge was constructed should be used for determining Bearing loads. For existing Bridges with less than HS20 design loading it is recommended to use HS20 loads.

7.3.4.1—Pedestal Retrofit

When replacing rocker bearings, the depth difference between the existing and new bearing types allows for standard concrete pedestals to be doweled on to the top of the existing cap, see §14.2.9 for bearing pedestal policy.

The height of the new pedestals shall be determined by maintaining the depth between the bottom of the existing girder and top of existing cap, see Figure 7.28 for an example.

This is an ideal use case for UHPC for the pedestal material, particularly for emergency repairs.

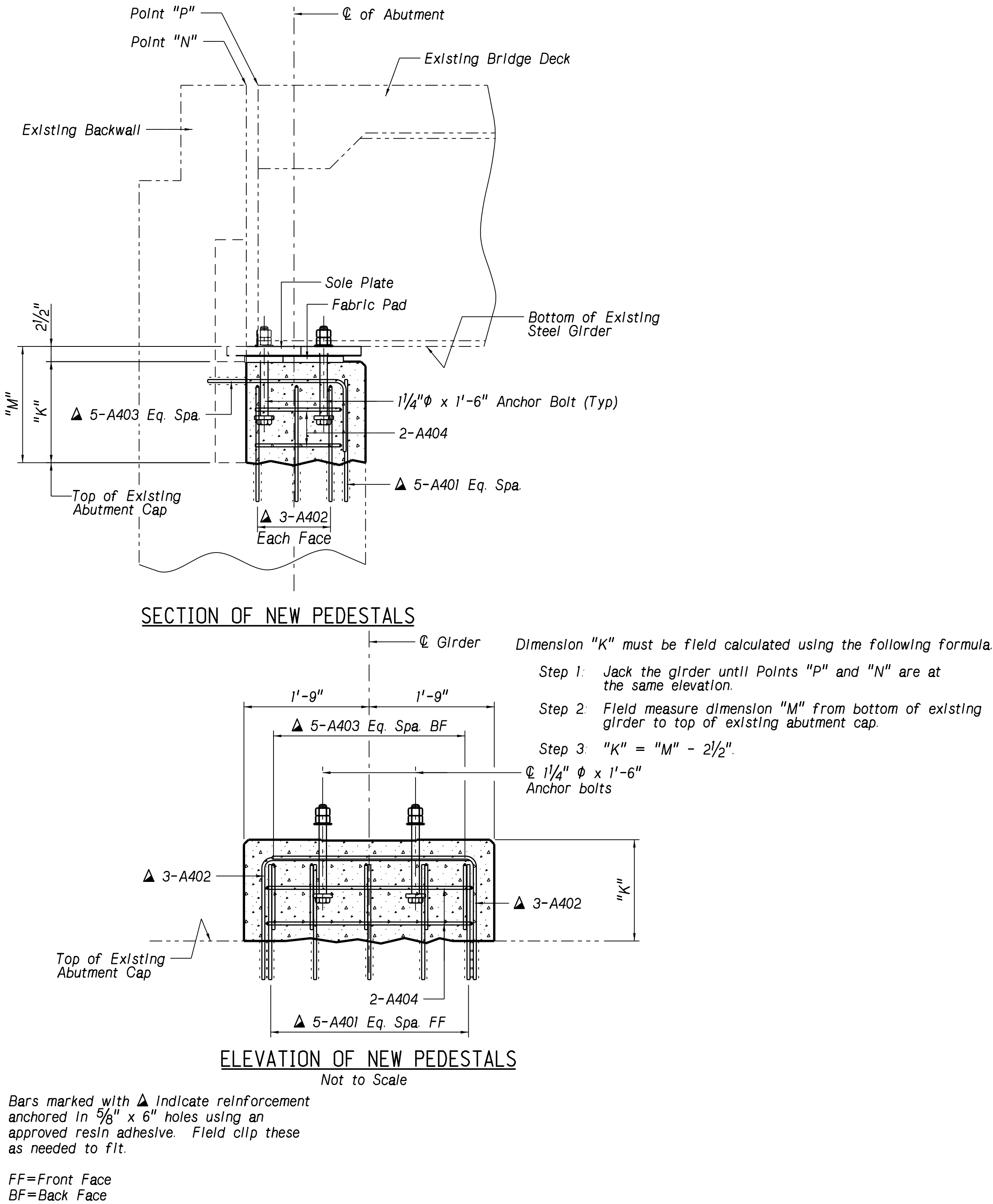


Figure 7.28—Pedestal Retrofit for Bearing Replacement

(example taken from SL55W00049L, CN 13224A, 2016)

7.3.4.2—Anchorage Considerations

Replacement Bearings shall follow the same criteria as Anchor Rods on new Bridges, see §14.2.10. One of the following two options should be used when installing anchor rods into existing concrete:

- Swedged anchor rods
- All-thread rod anchored with epoxy resin adhesive

Anchor Rods are not required when bearing replacement is performed in conjunction with remodeling the Abutment for full depth turndown.

7.3.4.3—Temporary Support for Bearing Replacement

When replacing Bearings on repair projects, a pay item for temporary support shall be included in the plans. Temporary Support will need to be furnished by the Contractor during the replacement operation. The contractor is responsible for designing the temporary support or sub-contracting the design. The contractor will be liable/assume responsibility for the accuracy and reliability of the design of temporary supports.

7.4—COMMON REPAIR GUIDELINES AND EXAMPLES

7.4.1—Bridge Deck and Approach Slab Repair

7.4.1.1—Concrete Repairs

Use the percentages in Table 7.11 for estimating repair quantities on the Bridge Plans. These quantities are often revised during PS&E when more accurate measurements are determined from field inspection.

In some instances, the district will perform pre-letting estimates of the amount of deck and approach repairs on a given project. In such an instance, the values in the table shall be superseded by actual field estimates.

For structures over 700 SY a more precise estimate should be produced rather than using these percentages.

Table 7.11—Deck Repair Area based on Condition

Deck Condition Rating (NBI)	Percentage of Deck Area
5	15%
6	10%
7	5%
8 or 9	2% (5 SY minimum to establish a bid quantity)

Use 3% (5 SY minimum) of approach slab area for preliminary estimates.

Class 47BD-4000 concrete is the default material used for deck and approach repairs.

Polymer Concrete should be used instead of 47BD-4000 where appropriate to reduce lane closure duration.

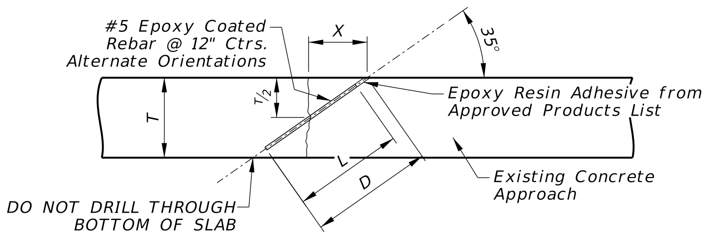
7.4.1.2—Crack Repairs with Cross Stitching

This procedure is used to stabilize and seal large cracks in approach slabs or paving slabs. For locations with a top mat of reinforcement, the bars shall be located using GPR or equivalent and existing reinforcement shall be avoided.

The Special Provision "DOWELING INTO CONCRETE STRUCTURES - POST INSTALLED ADHESIVE ANCHORS" shall be included when this repair is specified on a project.

Figure 7.29 shows the preferred detail and Figure 7.30 shows example of the appropriate and inappropriate level of cracking.

Cross Stitching is a detail that is typically used on concrete pavement. Consult Bridge Division for guidance on application to projects. At the time of publication, Cross Stitching Details have been shown on plans for S075 04868L (CN 13452) but have not been let.



CROSS STITCHING
EXISTING CONCRETE APPROACHES

Note: Deformed bar shall be 1" below the surface

"T"	"X"	"D"	"L"
8"	5¾"	12"	9¾"
9"	6½"	13½"	11½"
10"	7"	14"	12½"
11"	8"	16"	13"
12"	8½"	17½"	14"
13"	9½"	20"	18"
14"	10"	21"	18"

Figure 7.29—Cross Stitching Approach Repair Detail

A. Completed cross-stich repair (photo taken near SL28B00163L)

B. Completed cross-stich repair (photo taken near SL28B00163L)

C. Good candidate for cross-stitch repair (photo taken from S080 42697L)

D. **NOT** a good candidate for the cross stitch repair. The damage is much too extensive (photo taken from S081 04546R Approach)

Figure 7.30—Level of Cracking Appropriate for Cross-Stitch Repair as shown in Figure 7.29.

Red arrows on sub-figures A and B indicate locations where cross stitching repair has been completed.

7.4.2—Concrete Patching

7.4.2.1—Overview

When scoping documents call for repair of concrete bridge components, the following guidelines should be followed.

Concrete Patching is most suitable for repair of shallow deterioration caused by exposure to moisture and de-icing chemicals over time. This type of damage is considered nonstructural in nature. The core of the concrete element in most cases is sound.

Typically, concrete damage that is structural in nature will manifest itself by severe cracking or distortion. The Concrete Patching Special Provision is not the preferred method for repairing this type of damage.

In cases where the damage is considered structural, the damaged concrete should be removed and remodeled as appropriate, including detailed placement of new reinforcing steel.

7.4.2.1.1—Typical Locations

Repair work will typically be called out in the bridge determination as follows:

- Repair Abutment, Repair Wingwall, Repair Bent Cap, Repair Pier Column — In general, this is work to repair faces of deteriorated substructure components.
- Repair Bridge Rail — In general, this is work to repair faces of deteriorated curbs, barriers, or rails.
- Repair Deck Edge, Repair Deck Overhang – In general, this is work to repair deteriorated concrete along the deck edge or deck underside.

See §7.4.1 for Concrete Bridge Deck and Approach Repairs.

The need for repairs is determined by review of the Bridge Inspection Photos and a site visit when necessary. The construction procedures for performing these repairs are outlined in the standard Special Provision, “7.9 CONCRETE PATCHING OF STRUCTURES.”

Figure 7.31 to Figure 7.33 show examples of Concrete Patching with Shotcrete. Repair mortars or concrete may also be used. See Special Provisions and §7.4.2.3.1 for details.

Figure 7.31—Concrete Patching at Abutment with Shotcrete

(example taken from S089 00411)

Figure 7.32—Concrete Patching at Deck Edges, with Shotcrete
(example taken from S006 33065)

Figure 7.33—Concrete Patching at Bridge Rail, with Shotcrete
(example taken from S064 07011)

7.4.2.2—Plan Considerations

7.4.2.2.1—General

Concrete Patching is typically called out in the plans by inserting inspection photos of the damaged areas, along with an annotated or cross hatched callout of the damaged concrete to be repaired (Figure 7.34).

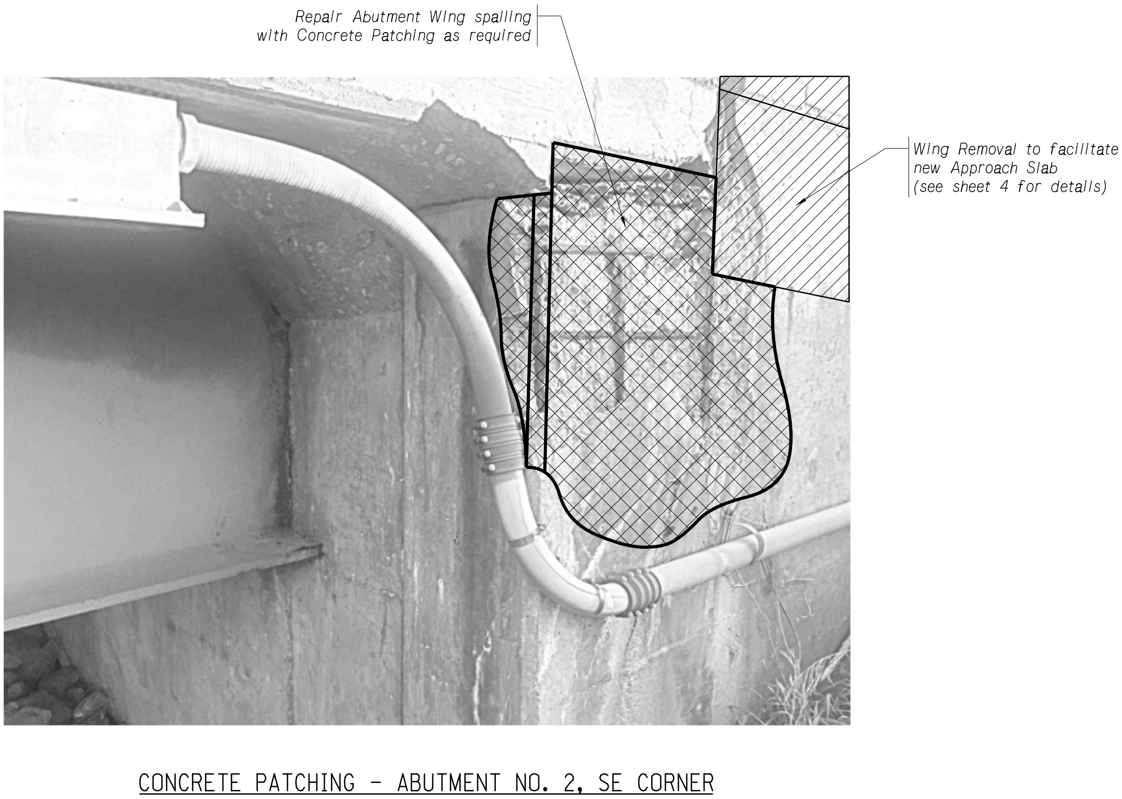


Figure 7.34—Concrete Patching as Shown on Plans with Hatching
(example taken from S275 17757, CN 22578A 2020)l

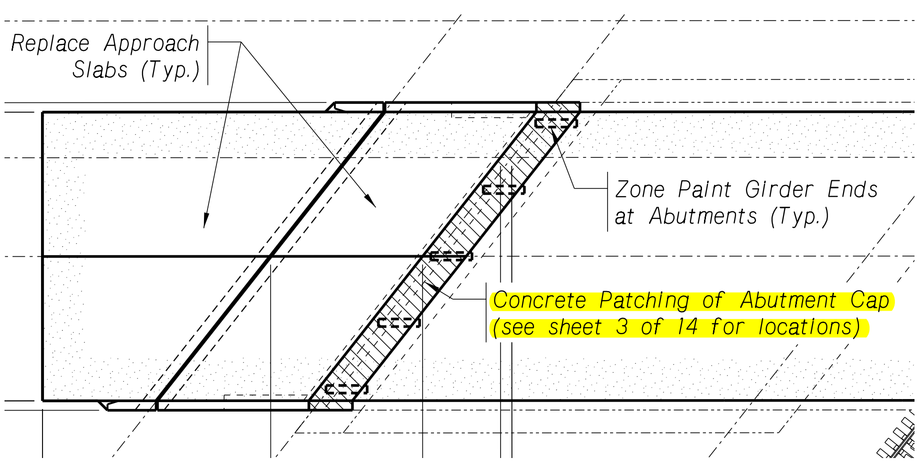


Figure 7.35—Concrete Patching Callout on General Plan

(example taken from S030 42623R, CN 22688, 2019)

Generally, each location that has been identified as needing repair with Concrete Patching is shown in the General Plan (Figure 7.35). If there are multiple areas on any given bridge of the same type of damage, a single photo illustrating the typical condition may be used.

Alternatively, if a single concrete component has multiple areas that need patching, a conventionally drawn elevation view outlining all the repair locations may be shown. Note that this method will still require the designer to approximate the damaged areas.

7.4.2.2.2—Quantity Estimates

A quantity for Concrete Patching, based on estimates of all patching work identified, shall be shown in the Bridge plans. This quantity is measured in SF, with a minimum of 5 SF.

Due to the approximate nature of such estimates, exacting quantities are not warranted. The actual quantity repair will be field determined during construction.

7.4.2.3—Material Considerations

The Special Provision allows the Contractor the option of three methods to perform the Concrete Patching. The three types are:

- Structural Patching Materials on the NDOT Approved Products List (APL). These materials are generally placed by hand with a trowel, without the use of forms.
- Shotcrete
- Portland Cement Concrete: Classes 47B, 47BD, and 47B-OL. Repairs with concrete generally require the use of forms.

Each method has its own benefits and drawbacks for a given application.

7.4.2.3.1—Option to Require Shotcrete for Patching

Shotcrete is well suited for vertical and overhead placements, particularly along deck edges. Shotcrete requirements are called out in Part B of the Concrete Patching Special Provision.

The designer has the option to specifically require that Shotcrete be used to perform a repair. This is done by adding a note to the plans (Figure 7.36).

Figure 7.36—Note on Plan for Requiring Use of Shotcrete
(example taken from S275 03181, CN 32267, 2016)

7.4.3—Crack Epoxy Injection

Cracks in existing concrete that are not of structural concern may be repaired with injection of Epoxy Compounds (Figure 7.37 and Figure 7.38). The construction procedures for performing these repairs are outlined in the standard Special Provision, “7-11 CRACK EPOXY INJECTION.”

A quantity for Crack Epoxy Injection shall be shown in the Bridge plans. This quantity is measured in LF, with a minimum of 5 LF, estimated in a similar manner to Concrete Patching.

When appropriate, designers should consider specifying this repair method in addition to Concrete Patching.

The minimum crack width in order to feasibly inject epoxy is approximately 0.02 in., cracks narrower than this need not be addressed.

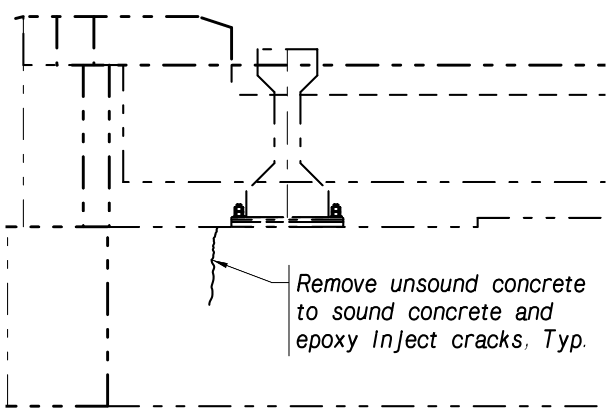


Figure 7.37—Epoxy Crack Injection Plan Note
(example taken from S283 05534 CN 71185, 2018)

Figure 7.38—Epoxy Crack Injection (Before on the Left, and After on the Right)

(example taken from S080 19045)

7.4.4—Girder Repairs

The following information applies to girder repairs. The examples have been assembled to aid designers in developing plans when conditions call for repairing steel or prestressed concrete girders.

7.4.4.1—Steel Girder Repairs

7.4.4.1.1—Pin and Hanger Repair

This repair may be specified when bridge inspection reports indicate cracked or broken pins. The typical repair requires the use of a temporary beam placed on the bridge deck, using tie rods through the deck, to provide girder support while pins are replaced.

Special Provision: Not created, see plan examples for notes (Table 7.12).

Table 7.12—Pin and Hanger Repair Examples

Structure №	Repair Type	Plan №	Repair Year
S077 11185	Replace Hanger Pins	77-3(1011)	1990
S180 00323L	Replace Hanger Pins	S180-3.23	1990

7.4.4.1.2—Fatigue Retrofit/Crack Repair

This repair is applied to existing steel girder webs where a crack is present. A crack arresting hole with a bushing should be installed at the end of the crack. This method appears to have been effectively used on floor beams.

Special Provision: "7.24 STEEL CRACK REPAIR" (Table 7.13).

Another fatigue retrofit method is Needle Peening. By needle peening, the material is plastically deformed at the weld toe in order to introduce beneficial compressive residual stresses. Peening has been shown useful at the base plate weld of sign structures to extend the fatigue life of the connection.

A similar method of repair used previously was holes drilled at the end of cracks without installing a bushing.

Table 7.13—Steel Crack Repair Example

Structure №	Repair Type	Control №	Repair Year
S012 00398	Floor Beam Crack Arrest	80799	2013

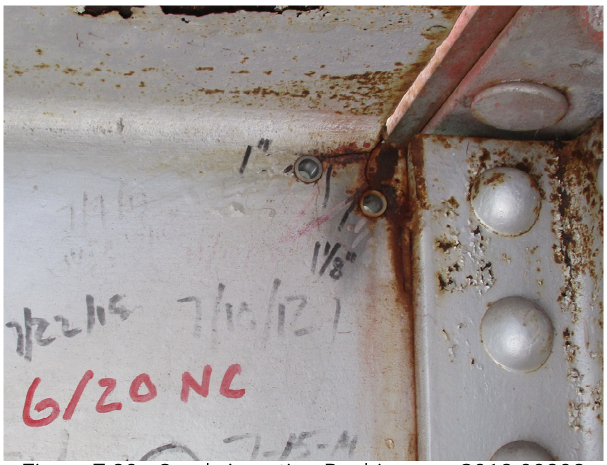


Figure 7.39—Crack Arresting Bushings on S012 00398

7.4.4.1.3—Heat Straightening of Steel Girders (Impact)

Heat straightening is the method used for repairing damage to steel elements caused by traffic impacts. Additional repairs such as removal of cross frames and welding backing plates to repairing gouges in the web may be necessary depending on the nature of the damage. If the flange, web, or both have been sheared, field splices will be needed in addition to heat straightening.

NOTE: When any girder has been impacted, a full review of the deck above the girder should be completed. Repairs to damaged decks, caused by the impact, must be performed.

In-service bridges section shall be made aware of damage and review the bridge prior to repairs being made.

Special Provision: Not created, see plan examples for notes (Table 7.14).

Heat straightening of steel I-beam and plate girder bridge members provides a viable repair method if impact distortion is not too severe (plastic strains less than $100\varepsilon_y$) and if primary members are not cracked or fractured (Geoghegan et al., 2023).

Heat straightening techniques may be used to repair any combination of the following distortions.

- weak axis distortion
- strong axis distortion
- torsional distortion
- local distortions such as
  - local flange buckling
  - web buckling
  - plate member bends or crimps

Table 7.14—Heat Straightening Girder Repair Examples

Structure №	RepairType	Plan №	Repair Year
S076 00176	Heat Straightening	Emergency Repair	2011
S076 00176	Heat Straightening	M6001	2022
S080 44827R	Heat Str. & Web Patch	M2080K	2018
S129 00039	Heat Str. & Web Splice	Emergency Repair	2002
S180 00079	Heat Str. & Web Patch	Emergency Repair	2020

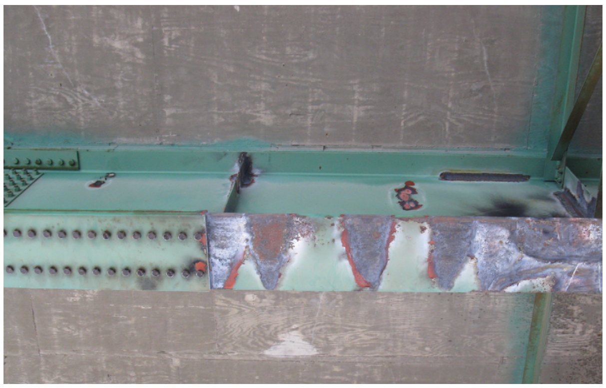


Figure 7.40—Girder after Heat Straightening on S129 00039

The following is taken from *Emergency Response Manual for Over Height Collisions to Bridges* (Iowa DOT Office of Bridges and Structures & HDR, 2016).

“Heat straightening is a repair procedure in which controlled heat is applied in specific patterns to the plastically deformed regions of damaged steel in repetitive heating and cooling cycles to gradually straighten the material. The process relies on internal and external restraints that produce thickening (or upsetting) in the heating phase and in-plane contraction in the cooling phase. When heat straightening is done properly, the temperature of the steel should not exceed what is referred to as the phase transition temperature, at which material properties of the steel can change significantly. Heat straightening generally requires multiple cycles of heat and restraint to incrementally return the member back to its original shape. Steel should be allowed to cool to 120°C (250°F) before reheating steel.

Research on material properties of steel exposed to strain ratios of 100 or less has indicated minimal change in material properties for steel members heat straightened two or less times.

Typically, two heat straightening repairs to the same section of beam will lead to a modest decrease in modulus of elasticity and ductility, but an increase in both yield stress and tensile strength. However, guidelines provided by the FHWA recommend that the same areas of steel members should not be heat straightened more than twice due to concerns over an increased loss of ductility and a substantial decrease in fatigue life (Geoghegan et al., 2023)

Prior to initiating heat straightening procedures, the damaged area of the beam should be thoroughly inspected to verify that it is free from cracks; the affected area of the beam should be blast cleaned to a bright metal finish and any gouges or nicks in the steel shall be removed by grinding to remove stress risers. For Grade 36 or 50 carbon steels, the maximum temperature used for heat straightening shall be 649°C (1200°F); for Grade 70W steel, the maximum temperature used for heat straightening shall be 565°C (1050°F); for Grade 100 and 100W steels, the maximum temperature used for heat straightening shall be 593°C (1100°F). Temperatures shall be monitored using temperature indicating crayons, a contact pyrometer (thermocouple with digital readout), or a bimetal thermometer. The use of heat straightening techniques typically includes the use of internal or external retraining forces. These forces shall be computed in advance and a constraint plan shall be established before applying heat, which defines the location of external jacks and the required bracing of undamaged members at jacking locations. Jacks, come-alongs, or other force application devices shall be gauged and calibrated so that the force applied can be controlled and measured. The load shall not be adjusted during heating or before the member has cooled to below 315°C (600°F)

Contractors who perform heat straightening services should have a minimum of 10 years of experience consisting of a minimum of three successful heat straightening projects. Heating shall be with an oxygen-fuel gas mixture using a #8 or smaller torch tip typically sized based on the thickness of metal being heated. Heating patterns may be triangular (vee-shaped), strip, or rectangular. Heating patterns that should be spaced and marked out along the length of the damaged area before starting. Quenching the heated area with water, mist, or an air-water mix to accelerate cooling is not permitted. However, after the steel naturally cools to a temperature below 315°C (600°F), cooling with dry compressed air is permitted.

Inspection of heat straightening work should include verifying that the straightened steel members meet tolerance requirements listed in the contract documents. Following completion of heat straightening, non-destructive testing, such as magnetic particle testing, may be needed to confirm that no cracks formed as a result of the straightening procedures."

7.4.4.2—Prestressed Concrete Girder Repairs

NDOT and UNL have published a synthesis report on the repair of precast/prestressed girders (Kodsy et al., 2020).

7.4.4.2.1—Crack Epoxy Injection

Epoxy crack injection may be used for repairing cracks in prestressed girders (Table 7.15, Figure 7.41, and Figure 7.42).

Special Provision: "7.11 CRACK EPOXY INJECTION".

Table 7.15—Girder Crack Epoxy Injection Examples

Structure №	Repair Type	Control №	Repair Year
S050 07686	Girder Crack Epoxy Injection	22456	2013
S070 10755	Girder Crack Epoxy Injection	42693	2018
S002 50816	Girder Crack Epoxy Injection	13371	2020


Figure 7.41—Crack Epoxy Injection on S070 10755


Figure 7.42—Crack Epoxy Injection on S002 50816

7.4.4.2.2—Concrete Repair (Deterioration)

When prestressed concrete girders have damage, due to expansion joint leaks or other exposure, concrete patching can be performed with a high strength concrete repair material compatible with the existing prestressed concrete strength (Table 7.16). See concrete patching guidelines outlined in §7.4.2.

Special Provision: "7-14 PRESTRESSED CONCRETE GIRDER REPAIR"

Table 7.16—Prestressed Concrete Girder Repair (Near Supports) Examples

Structure №	Existing Rail Type	Control №	Repair Year
S030 25933	Girder Patching	42571	2011
S070 10755	Girder Patching	42693	2018

Shotcrete repair has been found to be an effective repair when strands have not been damaged, see §7.4.2 for more details on Concrete Patching.

MnDOT has successfully used shotcrete at girder ends and experimentally validated the restoration of shear strength (Shield & Bergson, 2018).

7.4.4.2.3—Impact Repair

7.4.4.2.3a—Concrete Repair (Impact)

When prestressed concrete girders have minor delaminations, caused by traffic impacts, that do not affect the strands, concrete patching can be performed with a high strength concrete repair material compatible with the existing prestressed concrete strength (Table 7.17). See concrete patching guidelines outlined in §7.4.2.

NOTE: When any girder has been impacted, a full review of the deck above the girder should be completed. Repairs to damaged decks, caused by the impact, must be performed.

Special Provision: "7-14 PRESTRESSED CONCRETE GIRDER REPAIR".

Table 7.17—Girder Impact Repair Examples

Structure №	Existing Rail Type	Control №	Repair Year
S030 42866	Girder Patching	22582	2020
S080 06462	Girder Patching	51431	2010
S080 34413	Girder Patching	District Plan	2021

7.4.4.2.3b—Strand Repair (Impact)

When traffic impacts cause the prestressing strands to be severed, the stands may be repaired using strand couplers to re-tension the prestressing strands prior to restoring the concrete. Figure 7.43 shows the process of splicing strands in the field after an impact (Enchayan, 2010).

NOTE: When any girder has been impacted, a full review of the deck above the girder should be completed. Repairs to damaged decks, caused by the impact, must be performed.

Special Provision: Not created, see plan examples for notes (Table 7.18).

. Table 7.18—Girder Strand Splice Repair Examples

Structure №	Existing Rail Type	Plan №	Repair Year
S006 35981	NU - Strand Repair	AFE-B028	2015
S071 05850L	NU - Strand Repair	51409	2009
S680 00713	NU - Strand Repair	M2004	2019

A. Strand Impact Damage

B. Concrete Removal

C, Cutting Damaged Strands

D. Strand Splicing

E. Splices Installed

F. Finished Grout

Figure 7.43—Strand Repair Method after Impact
(example taken from S680 00713, 2019)

7.4.4.2.4—UHPC Repairs

Ultra-High Performance Concrete (UHPC) may be used to replace damaged concrete in precast/prestressed concrete girders.

UHPC allows for both increased shear resistance of the girder as well as increased bearing resistance at the support. Experimental studies showed that girder ends repaired using this concept could meet or exceed their intended capacity at the ultimate limit state.

7.4.5—Steel Pile Repair and Preservation

The use of field painting is an option when FRP jackets or Encasement options are not feasible. Open steel bents with welded bracing are not good candidates for FRP due to the bracing. Painting can be the most affordable option when concrete for a large encasement is not readily available due to the remoteness of the location. Refer to §7.5.3 for more information about painting of existing piling.

7.4.5.1—Concrete Encasement

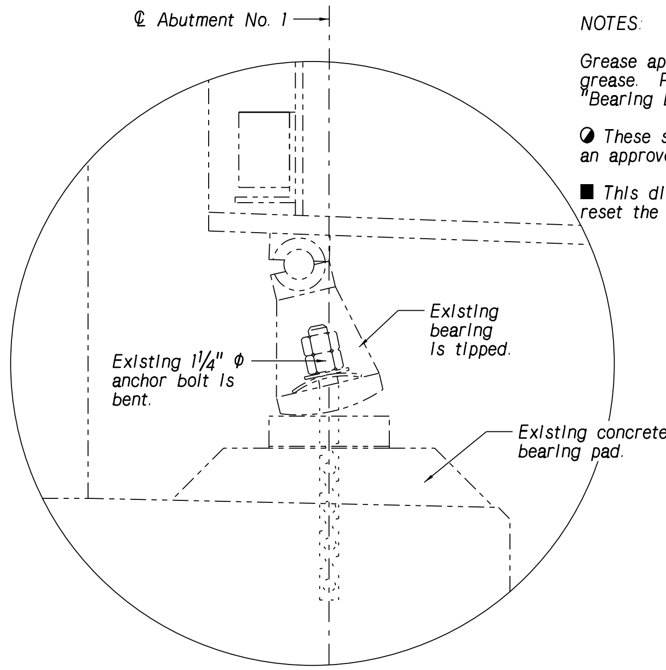
The use of reinforced concrete is a preferred treatment of steel piles that are moderately to severely deteriorated or damaged. Encasement can also be used for preservation, however requires similar work space as needed for FRP jacket. Encasement is an option to fully or partially encase open pile bents without bracing. Large encasements require the dead load to be considered.

7.4.5.2—FRP Encasement

The use of FRP jackets or sleeves is a preferred method of preservation. However, other methods may be more suitable when access to the pile is limited, especially at abutments with low clearance. Ample space is necessary to prepare the pile surface and handle the FRP segments. The surface preparation needs to be clean, but not to the extent required for painting

7.4.6—Resetting Rocker Bearings

When Rocker Bearings have become tipped but are otherwise in good condition, resetting of the existing bearings can be performed instead of full bearing replacement, an example detail can be seen in Figure 7.44.



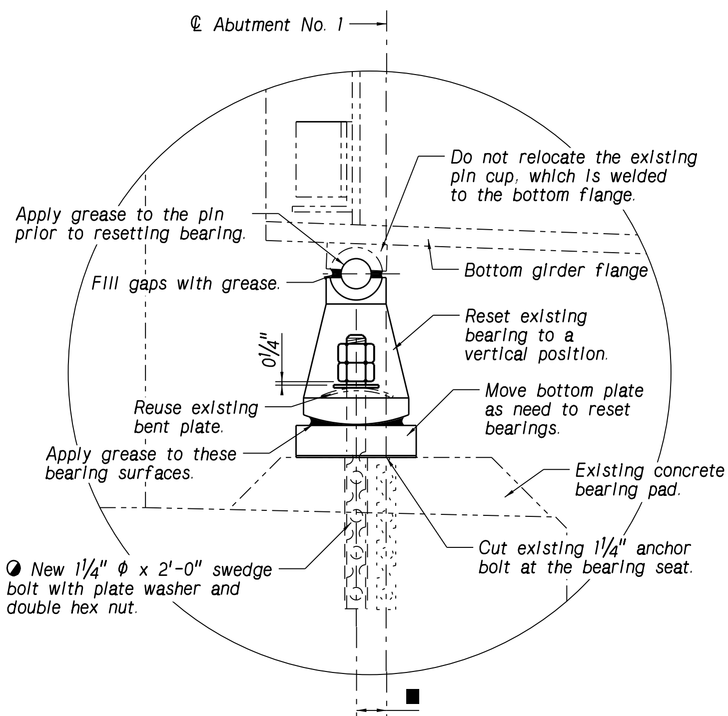
NOTES:

Grease applied to bearings shall be a suitable automotive bearing grease. Payment for labor and materials shall be subsidiary to "Bearing Device Repair"

◑ These swedge bolts shall be anchored in 1⅜" x 10" holes using an approved epoxy resin adhesive.

■ This dimension will be field determined as needed to reset the bearing.

DETAIL OF EXISTING BEARING CONDITION AT ABUTMENT NO. 1



DETAIL OF BEARING RESET AT ABUTMENT NO. 1

Figure 7.44—Rocker Bearing Reset and Lubrication
(example taken from S137 01328, 81001, 2017)

7.4.7—Floor Drain Retrofits

The construction procedures for performing the repair are outlined in the standard Special Provision, "7-20 REPAIR FLOOR DRAIN".

7.4.7.1—Drains in Closed Rails

Standard structural tubing of $3/16$ in. thickness shall be used for the drainpipe. The tube shall be sized to fit into the existing drain opening (Figure 7.45).

The existing deck and/or curb may need to be broken out to install the drain retrofit. Repair of the underside of the deck around the existing drain outlet is also typically required.

It is acceptable to slope asphalt overlay to drains contained in rails, see overlay base sheet. Where possible do not slope overlay beyond the edge of the shoulder into the driving lanes.

Many existing bridges in the State of Nebraska were constructed with open holes in the deck at the gutter line to facilitate drainage. Over time, the concrete around the open hole becomes significantly deteriorated due to constant exposure to moisture. The most common technique to repair these types of floor drains is to install a drainpipe to keep the moisture off the concrete.

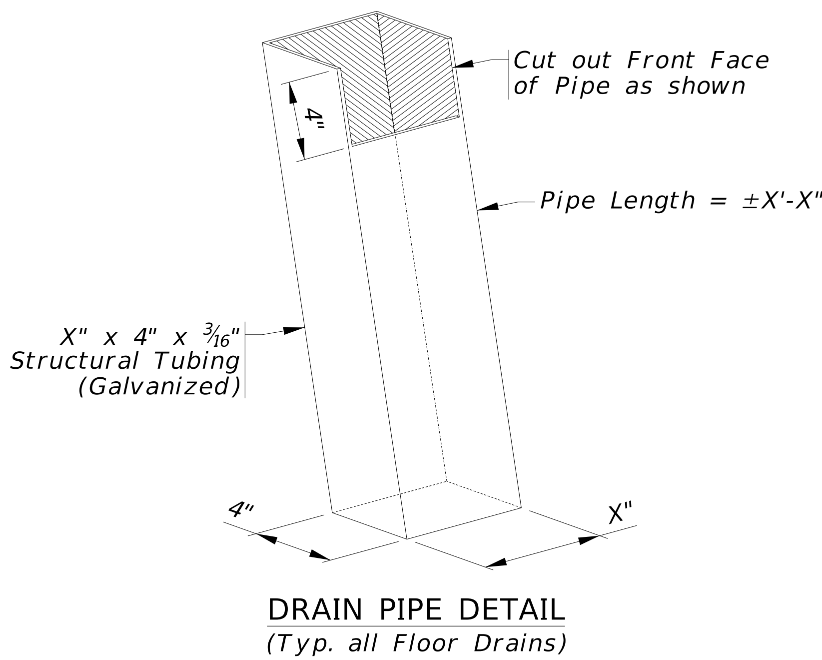


Figure 7.45—Retrofit for Drain in Closed Rails
(Available as Cell "017A-Drain Pipe Detail")

7.4.7.2—Drains in Shoulders

When doing a repair in conjunction with an Asphalt Overlay w/Membrane, the use of new frame and grates shall be considered to eliminate large dips encountered when the asphalt is simply sloped to the drains (Figure 7.46).

A. Shoulder Drain With Frame Prior to Overlay

B. Shoulder Drain With Frame After Overlay

Figure 7.46—Shoulder Drain with Frame on S480 00310B

It is preferred not to slope overlay to drains in shoulders unless the shoulder width is adequate to keep entire depressed area out of the driving lane. Details similar to Figure 7.47 should be avoided where ever possible.

Figure 7.47—Drain in Driving Lane

(example from S180 00323LR)

A typical Floor Drain Repair detail is shown in Figure 7.48. This detail shall be modified as needed for use in the plans when doing floor drain repairs.

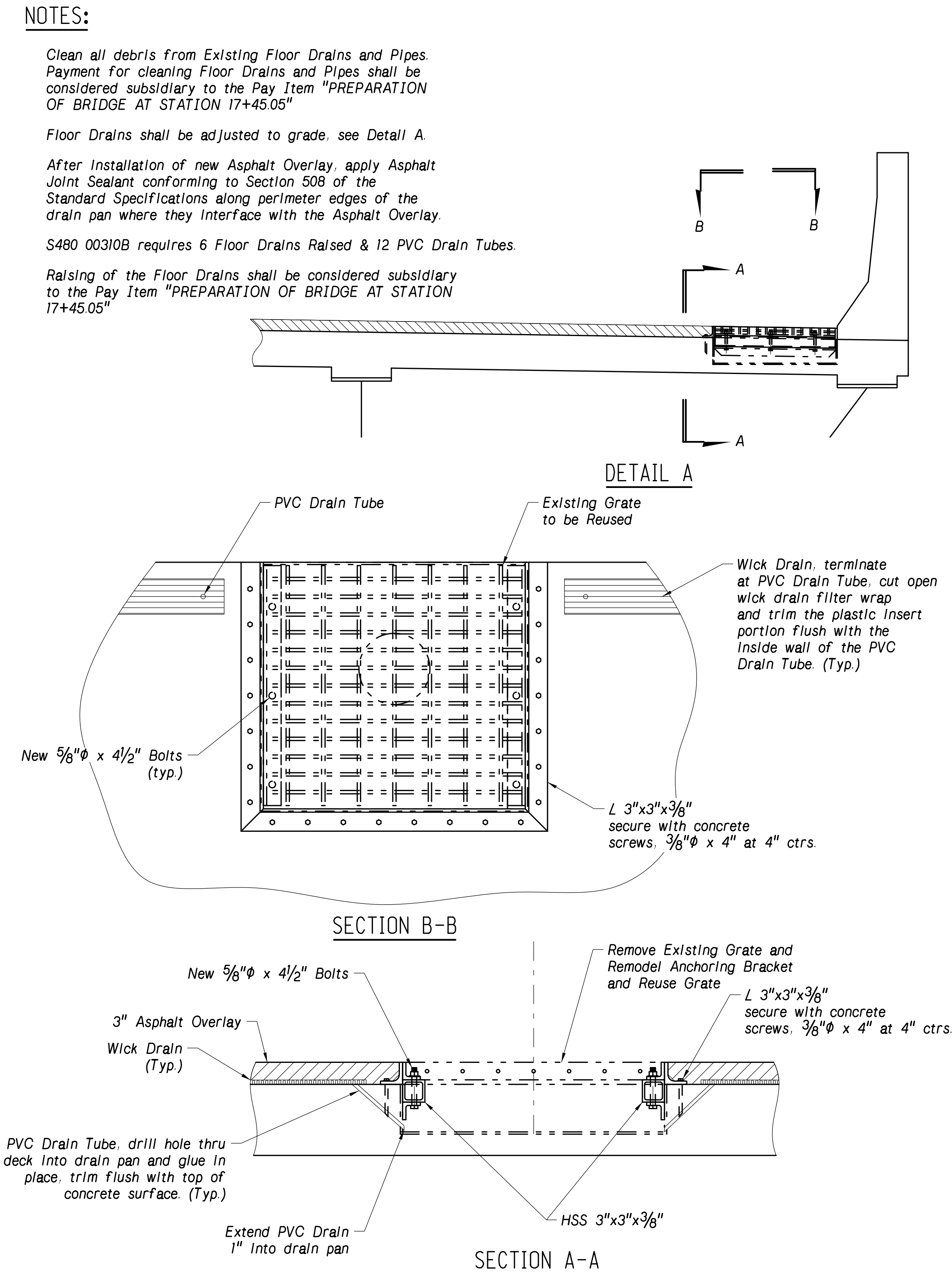


Figure 7.48—Frame Detail Around Shoulder Drain for Overlays

(example from S480 00310B, 22611, 2023)

7.5—STEEL PAINTING GUIDELINES AND EXAMPLES

When scoping documents call for repainting of the entire existing steel superstructure of a bridge, this is considered major work. This section focuses on the three main types of minor painting work typically called out in the plans for Bridge Preservation projects.

The construction procedures for performing painting of this nature are outlined in the standard Special Provision, "7.19 PAINTING STEEL".

7.5.1—Zone Painting of Girder Ends

Existing Steel Girders (weathering steel or with existing coatings) that are discontinuous at intermediate supports, pin & hangers or abutment joint locations should be considered for zone painting.

Zone painting limits should be shown on the plans but typical paint zone is 6 ft. to 10 ft. along the girder.

Bearing devices if not specifically described in the work description should be included in the paint quantity.

When existing steel girders are painted, the new zone painting shall be detailed to show existing coatings feathered surrounding the end of the new zone painting.

All painting shall be in accordance with section 709 of the standard specification.

7.5.2—Painting Existing Bearings

Painting existing bearings is typically called out in the plans by inserting an inspection photo of a typical Bearing to be painted, along with an annotated or cross hatched callout to clean and paint the bearings, see Figure 7.49.

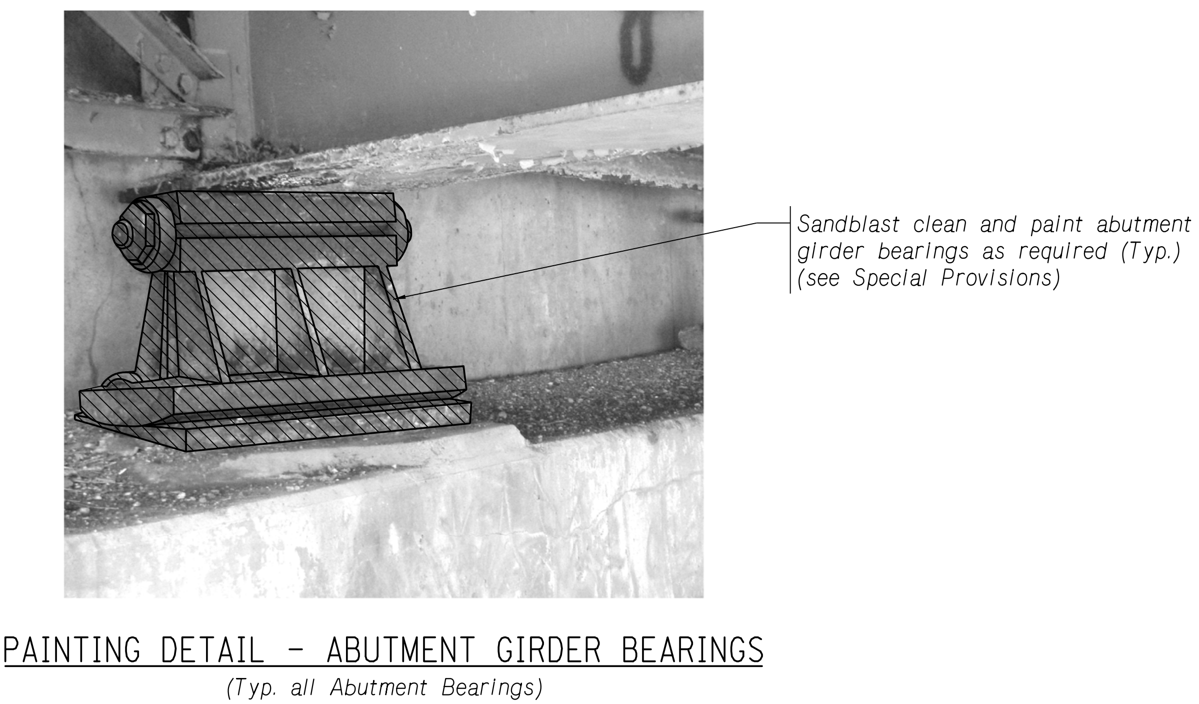


Figure 7.49—Painting Existing Bearing

(example taken from S050 07712, CN 22456, 2014)

7.5.3—Painting Piles

Painting of existing steel pile is not believed to be a cost-effective strategy. Surface preparation can be very challenging due to limited access and the need to excavate piles below the current ground line to expose susceptible areas. Following surface preparation, the Contractor shall test for the presence of soluble salts. If salts are detected, the substrate shall be pressure washed with a solution in accordance with manufacturer's recommendations until the salt is removed.

7.6—SPECIAL CONSIDERATIONS FOR HISTORIC BRIDGES

In compliance with the Section 106 of the National Historic Preservation Act which declares that any project involving Federal funds must take into consideration the impact that the project might have on properties eligible for or listed on the National Register of Historic Places. Since Federal funds are often involved in aid to highway and road improvements in Nebraska, the provisions of Section 106 apply to most highways and many county bridges in the state.

Responsibility for administering Section 106 rests with Nebraska SHPO, and when Federal funds are involved, negotiation is conducted between the FHWA and the SHPO to verify resource consideration and protection. Even if Federal funds are not involved in a project, it is good stewardship of these important historic resources to preserve them for future generations.

In addition, Section 4(f) of the U.S. Department of Transportation Act protects historic sites from highway project effects unless there is "no feasible and prudent alternative". If the project cannot avoid affecting a historic property, the project must be planned to minimize the damage. The Surface Transportation and Uniform Relocation Assistance Act of 1987 goes further. Asserting that it is "in the national interest to encourage the rehabilitation, reuse, and preservation of bridges significant in American history, architecture, engineering, and culture". The Act permits the Federal Government to reimburse costs associated with preserving historic bridges or mitigating unavoidable damage.

7.6.1—Priorities for Treatment of Historic Bridges

The preferred treatment for a historic bridge is to have it continue to carry vehicular traffic at its original site with minimal modification.

If it is not feasible to keep the bridge at its original site, every effort should be made to find an appropriate site to which it could be relocated for vehicle use. There is a marketing and advertising requirement in the agreement between the FHWA, NDOT, and the Nebraska SHPO to notify the public and other government entities of the availability of the bridge for reuse.

If the bridge can no longer carry vehicular traffic or could do so only at the expense of its historic integrity, the next best solution to evaluate is non-vehicular use at its original site with minimal modification (e.g., pedestrian or bike bridge).

If the bridge can no longer carry vehicular traffic, no "as is" use is feasible, or it cannot be left in place, adaptive uses should be evaluated, with preference given to reuse that retains the bridge at its original location. If no suitable in-situ adaptive use can be found, the bridge can be relocated to a less demanding vehicular crossing or adapted for non-vehicular use at the new location (preferably in the public domain).

If the bridge cannot remain at its original site and cannot be moved, it shall be documented to the standards of the Historic American Engineering Record before demolition, disassembly, or modifications that will destroy its historic integrity. If possible, the structure should be disassembled carefully and stored until a new location for it can be found or significant components should be incorporated into any new bridge at the site or salvaged for educational purposes.

7.6.2—Alternative Evaluation and Documentation – A Requirement

When a project will affect a historic bridge, the FHWA and the Nebraska SHPO will judge whether the project will adversely affect the structure, and whether adequate alternative evaluations have been conducted to avoid or minimize the effect. It is the responsibility of the NDOT and/or the county to provide a report containing a full description of all alternatives considered, to avoid, minimize or mitigate affects to the structure. (Example of such analysis is available upon request from NDOT). When documenting the need for replacing or preserving a bridge, technical, legal, financial, and safety considerations must be weighed in reaching the final decision. The problem with the structure must be clearly stated, be it structural or functional.

The following range of alternatives (listed by priority) must be considered carefully before plans to alter or remove a historic bridge are finalized.

- Continued use of the bridge for vehicle traffic at its original location, with restoration and rehabilitation.

    - Passive, non-structural actions to lower the live load on a bridge should be considered as a first alternative when load is of concern. Lowering the posted load limit and restricting traffic to one direction are examples of ways to retain a bridge in service without structural modification.
- Use of the bridge for non-vehicular traffic at the site.
    - Issues involved with this option include what to do with vehicular traffic. This may be a considerable problem at an important crossing when there are no alternate bridges convenient or capable of handling greater traffic loads.
    - It may also be problematic if physical or economic considerations require use of the existing bridge site for a new bridge.
    - One alternative that has been used is to build a new bridge alongside the old one, altering alignment to properly accommodate the new location.
    - The existing bridge may be closed to vehicular traffic but is reserved in place for public viewing.
    - Some counties have been provided historical markers and pathways to display the bridge.
    - One suggestion has been made that the bridge can be retained for vehicle crossing if it meets structural sufficiency, so that the traveling public can experience driving over the old bridge if they so choose.
    - Every effort should be made to keep the bridge in public ownership, either through continued use by the current owner or another government agency.
    - If marketing of the bridge to private ownership is necessary, protective covenants must be put in place for the bridge's preservation.
- Relocation of the bridge.
    - When a bridge must be moved to a new site, provisions must be made for maintenance, damage protection (natural and man made), and public accessibility.
    - The abutments or intermediate supports at the new site should match the original configuration, if possible.
    - Issues of ownership and marketing, as described above, must be considered
- Destruction of historic character, demolition.
    - This option includes rehabilitation without consideration of historic integrity. Work that harms the historic integrity of the bridge should be undertaken only if it is not possible to make the bridge safe and efficient and it cannot be moved.
    - In this event, the bridge shall be documented for the Historic American Engineering Record prior to the onset of work, unless an emergency exists.
    - If demolition is to occur, significant and ornamental features should be salvaged and reused to assist the preservation of a similar structure or for educational purposes or should be mothballed for reuse in the future.

The information presented herein is offered to assist early evaluation of alternatives for historic bridge preservation. The Nebraska Department of Transportation helps in this process through our Historic Bridge Program Office. If Federal funding is to be used in the proposed project, its availability will be dependent upon proper completion of the paperwork and processes described in this document. For additional information or assistance, please contact Environmental Section Manager.

7.6.3—Maintenance Activities on Historical Bridges

The following repairs and bridge maintenance, necessary to keep the bridge functioning, will not change the appearance or character of a bridge.

- Bridge deck patching or placing of concrete overlay.
- Replacing of truss or other structural members in kind.
- Redecking or replacing bridge rail.
- Painting of structural steel or railing.
- Rebuilding of abutment or intermediate support caps.
- Replacing of bearing devices in kind.
- Replacing of bridge expansion or fixed devices.
- Backfilling erosion around abutments.
- Providing scour protection at intermediate supports or abutments.
- Providing additional bracing to bents.
- Reestablishing berm, as needed.
- Repairing collision damage, as needed.

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Last Updated: January 13, 2025

Chapter 9 — Deck Design

9.1—GENERAL DECK CRITERIA

Concrete decks supported on longitudinal girders – except as listed in §9.2.4 – may be designed using the empirical deck design in accordance with the current AASHTO LRFD Bridge Design Specifications and the guidance in this Chapter.

Information on deck protection systems is given in §7.2.

All deck elevations on the plans shall be given at top of cast in place concrete deck unless otherwise noted.

Coordinate with other disciplines for elevations of additional surfacing.

9.2—CAST-IN-PLACE DECKS

9.2.1—Deck Thickness for New Construction and Redecks

Deck thickness, including ¹⁄₂ in. sacrificial wearing surface, shall be as shown Table 9.1. For NU Girders Table 9.2 has been compiled for ease of use

Integral sacrificial wearing surface is considered in design of all decks regardless of initial construction methodology. It is assumed that if a AC+M overlay is replaced in the future the deck will have to be ground in order to provide a good bonding surface for the replacement membrane.

A diagram of the interpretation of LRFDBDS Article 9.7.2.3 effective length and design depth is shown in Figure 9.1.

Bridge Division no longer permits 7 ¹⁄₂ in. thick decks on new and reconstruction projects.

The ratio of effective length to design depth was set at approximately 16 when creating Table 9.1 and Table 9.2. This ratio exceeds the original maximum ratio of 15 in the *Ontario Highway Bridge Design Code* (1991) where the empirical method originates but is less than subsequent research conducted by Hays et al. (1988). Bridge Division feels this is a good balance between cost effectiveness and limiting girder-to-deck stiffness ratio to reduce cracking.

Table 9.1—Deck Thickness based on Effective Length

Deck Thickness	Maximum Effective Length
8 in.	10 ft. 0 in.
8 ¹⁄₂ in.	10 ft. 8 in.
9 in.	11 ft. 4 in.
9 ¹⁄₂ in.	12 ft. 0 in.
10 in.	12 ft. 9 in.
10 ¹⁄₂ in.	13 ft. 4 in.
11 in.	13 ft. 6 in.

Table 9.2—Deck Thickness for NU Girders

Deck Thickness	Maximum Center-to-Center Spacing
8 in.	12 ft. 3 in.
8 ¹⁄₂ in.	12 ft. 11 in.
9 in.	13 ft. 7 in.
9 ¹⁄₂ in.	14 ft. 3 in.
10 in.	15 ft. 0 in.

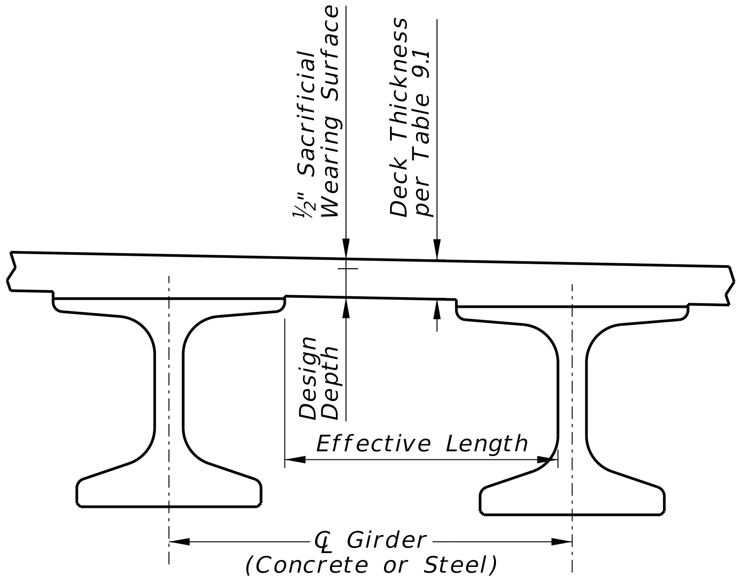


Figure 9.1—Effective Length and Design Depth

9.2.2—Reinforcement Details

All reinforcement in the deck and rails shall be epoxy coated.

Table 9.3—Deck Reinforcement

Deck Thickness	Main Reinforcement Spacing	Overhang Reinforcement between Each Full Width Bar
8 in.	12 in.	2
8 1/2 in.	12 in.	2
9 in.	11 in.	2
9 1/2 in.	11 in.	2
10 in.	10 in.	1
10 1/2 in.	10 in.	1
11 in.	9 in.	1

9.2.2.1—Transverse Bars

The clear cover for transverse bars shall be as stated in §5.3. The transverse bar spacing shall be measured along the centerline roadway and placed perpendicular to girders with bar sets provided at the end of floor where necessary for skewed structures. All reinforcement shall be #5 bars at the maximum spacing as shown in Table 9.3. Layers of reinforcement shall be staggered so the two mats do not have bars directly above one another.

Bridge Division has chosen to make reinforcement requirements for the top and bottom mats the same due to ease of detailing and construction.

9.2.2.2—Longitudinal Bars

Longitudinal bar layouts shall begin 3 in. from the edge of deck. All reinforcement shall be #5 bars at the maximum spacing as shown in Table 9.3. Layers of reinforcement shall be staggered so the two mats do not have bars directly above one another.

Additional reinforcement shall be provided in deck on structures continuous over the intermediate supports.

See additional information in §5.5.1.5.2.

Bar sizes above #6 are not permitted in the deck reinforcement until longitudinal empirical deck steel has been upsized above the intermediate supports to #6 and both layers of additional negative moment steel contain (2) #6 bars between each empirical deck rebar. Maximum bar size used in a typical reinforced concrete deck shall be #9 unless approved by Bridge Division. Minimum clear space between bars shall be 2 ³⁄₄ in.

Negative reinforcement steel should be added to the bottom mat prior to using #9 bars in a single layer of the deck reinforcement.

Upsize the longitudinal empirical deck bars to add additional moment capacity where required instead of using a full-length #5 bar in conjunction with two large supplemental bars. More medium size bars is better for crack control than one small bar and two large bars.

Lap splices are at the option of the contractor for bars up to 60 ft. For bars over 60 ft. the lap lengths shall be given in the plans and included in the plan quantity. Lap splices shall be staggered so that no two adjacent bars are spliced in the same place unless specified in the bridge plans. A detail similar to the lap detail provided on the bridge rail base sheets shall be provided on the Plans.

9.2.2.3—Skewed Decks

Additional end zone reinforcement will not be required in the deck at the turndown or integral abutments. In other situations where the skew requirements of LRFDBDS Article 9.7.2.5 applies, additional reinforcement in the deck end zones is required.

The LRFDBDS requirement for adding additional steel is not being followed due to the interaction with concrete turndowns at the abutment. The theory is that because of the stiffness at the turndown that additional reinforcement is not needed (Okumus et al., 2018).

9.2.3—Cantilever Design

The design section for negative moments and shear forces on steel I-beams and precast I-shaped concrete beams shall be taken as one-quarter the flange width from the centerline of support.

Due to the thin nature of NU girder flanges as well as the bond breaker applied to the edge of the flanges Bridge Division has chosen to use b/4 for the cantilever point on decks for both steel and concrete girders in lieu of what is shown in LRFDBDS Article 4.6.2.1.6 for concrete girders.

Minimum overhang thickness shall match the uniform thickness of the remainder of the deck.

Minimum overhang length should typically follow LRFDBDS Empirical Deck design.

For empirical method decks with design overhangs of up to 4 ft. 6 in., supplemental #6 bars shall be provided in the overhang. Two bar marks shall be created as shown in Figure 9.2. Where two supplemental bars are required between each full width bar, one of each bar mark shall be in each space. Where only a single supplemental overhang bar is required between each full width bar, the bar marks shall be alternated between each full width bar.

For design overhangs greater than 4 ft. 6 in. or traditionally designed decks, calculations shall be completed by the designer per LRFDBDS Appendix A13.4.

Designers should note that loads given in Table A13.2-1 of LRFDBDS 9th Edition are out of date. If running cantilever calculations updated loadings can be found in Transportation Research Board et al. (2024).

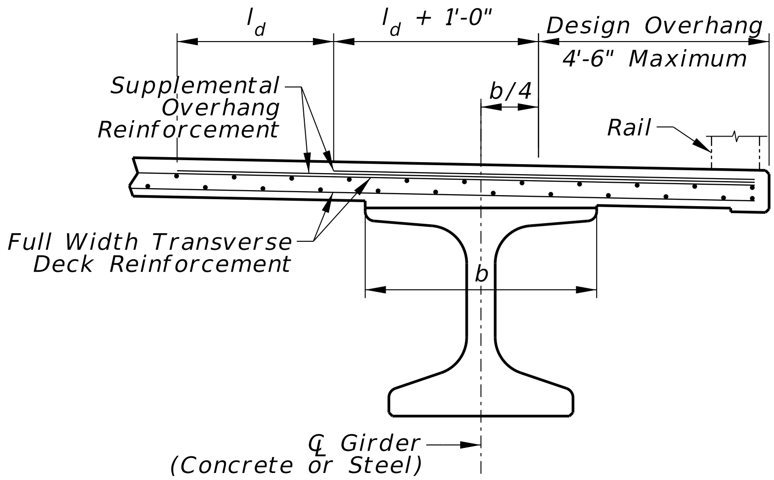


Figure 9.2—Cantilever Length and Reinforcement Section

9.2.4—IT Girder Decks

The maximum overhang length measured from the centerline of the exterior girder to the edge of the deck shall be 19 in.

9.2.4.1—Interstate and Heavy Barriers

When Inverted Tee girders are used on structures that carry interstate traffic or where barriers 39 in. or taller are used, cast-in-place deck thickness shall be 8 in. measured at the location of rod reading as shown in "Figure 9.12—SHIM SHOT IT Cell (Bridge Deck Library)".

Top and bottom reinforcement shall be #5 bars both directions. Bottom mat spacing shall be 12 in. centers each direction and top mat spacing shall at 9 in. centers each direction. Cantilever reinforcing shall be per §9.2.3 with a single bar between each #5 and the 8 in. deck thickness shall be maintained. Additional reinforcement shall be provided in the deck on structures continuous over the intermediate supports.

9.2.4.2—Other Structures

Cast-in-place deck thickness is 6 in. measured at the location of rod reading as shown in "Figure 9.12—SHIM SHOT IT Cell (Bridge Deck Library)".

Deck reinforcement is a single layer of reinforcement at mid deck thickness. All reinforcement shall be #5 bars. Transverse spacing shall be 6 in. centers and longitudinal spacing shall at 10 in. centers. No additional reinforcing is required in the cantilever and the 6 in. deck thickness shall be maintained. Additional reinforcement may be required in the deck on structures continuous over intermediate supports.

9.2.5—Construction Joints and Pour Sequencing

A pouring diagram and note shall be placed on the General Plan and Elevation Sheet of the Plans. The note is available as the cell shown in "Figure 9.13—POURSQ Cell (Bridge Deck Library)". The values in the cell text must be filled in after placing it on the sheet. When a continuous placement sequence is permitted standard note #048 shall be placed with the cell, when skip placement is mandatory standard note #049 shall be placed with the cell.

Alternate procedures for placing deck concrete may be submitted for approval by the Contractor along with a statement of the proposed method and evidence that the contractor possess the necessary equipment and facilities to accomplish the required results. The Project Manager, PCC Pavements Research and Development Engineer, and Bridge Division will determine if their plan is acceptable.

Any alternate pouring sequence must be determined before submittal of fabrication plans for steel girders and prior to casting of any bearing pedestals on substructures. The camber and blocking diagram in the plans for steel bridges is representative of the pouring diagram shown in the bridge plans.

All design checks, including uplift at supports, must be verified before the Engineer approves a revised pour sequence.

For detailing information on the construction joints see §5.4.2.

9.2.5.1—Continuous Placement Sequence

Continuous placement is the default choice for structures where it is feasible from a construction and design standpoint. It is easier for the Contractor and more likely to result in a deck with no transverse construction joints.

Optional transverse construction joints shall be detailed for placement of concrete slabs and bridge decks. The location of these transverse joints will be near the dead load moment point of contraflexure (Strength I load combination).

On continuous spans a pour shall only be terminated at the completion of a positive moment area.

9.2.5.2—Skip Placement Sequence

Skip placement sequence consists of mandatory transverse construction joints that are placed near the dead load moment point of contraflexure (Strength I load combination). All positive moment sections are placed prior to any negative moment section being placed.

Continuous for dead load steel superstructures with one or more spans exceeding 150 ft. in length, shall be detailed with skip placement sequence as mandatory.

Bridges with many spans may require an alternate pouring sequence regardless of maximum span length. Contractors are allowed to submit alternate pouring sequences regardless of maximum span length.

Structures that are unable to be poured in a continuous path along the length will be detailed with the skip placement sequence based on pour diagram calculations in §9.2.5.3.

9.2.5.3—Pouring Diagram Calculations

The minimum pour rate shall be calculated based on the following assumptions:

- The rollers and carriage travel at 80 ft./min. transverse to the finishing machine (along support skew).
- The rollers can finish in both directions of travel for non-skewed structures. For skewed structures the rollers can only finish in one direction of travel. The carriage is then returned to the first side of the pour without a finishing pass.
- The finishing machine travels 9 in. per finishing pass along the centerline of the roadway regardless of skew.
- Normal concrete remains plastic for three hours.
- Maximum concrete delivery rate is 80 yd^3/hr.

Based on these assumptions a minimum pour rate in ft./hr. along the centerline of bridge can be determined that will keep the concrete plastic between construction joints.

Initial set time is a conservative assumption from the 47BD data collected in Morcous et al. (2023).

Take the hypothetical bridge shown in Figure 9.3. The structure has a 28 ft. clear width, 10° skew, and 24.85 $ft.^2$ average cross section area for deck (including the girder haunches).

- the finishing machine requires a pass each direction across the bridge even though it only finishes in one direction due to the skew.
$$\frac{\frac{28\,ft.}{\cos 10°} \times 2}{80\,\frac{ft.}{min.}} = 0.71\frac{min.}{pass}$$

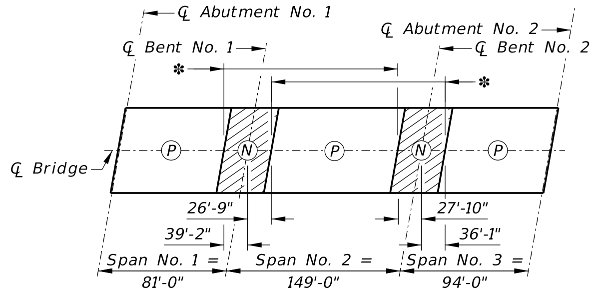
- maximum finishing rate
$$\frac{9\frac{in.}{pass} \times \frac{1\,ft.}{12\,in.}}{0.71\,\frac{min.}{pass} \times \frac{1\,hr.}{60\,min.}} = 63.3\frac{ft.}{hr.}$$

- maximum concrete delivery rate
$$\frac{80\frac{yd.^3}{hr.} \times \left(\frac{3\,ft.}{1\,yd.}\right)^3}{24.85\,ft.^2} = 86.9\frac{ft.}{hr.}$$

- minimum pour rate to ensure plastic concrete along entire pour. The longest positive plus negative section, regardless of chosen pour direction needs to remain plastic during the pour.
$$\frac{39.17ft. + 149\,ft. - 27.82\,ft.}{3\,hr.} = 53.4\frac{ft.}{hr.}$$

Therefore the plans shall show a minimum pour rate of 53.4 ft./hr. along with the pouring diagram as shown in Figure 9.3.

Should the pour rate to ensure plastic concrete exceed the maximum finishing or delivery rates, skip pouring shall be denoted as mandatory in the pouring diagram.



✻ Optional Construction Joints (Pour in direction of arrows)
Contractor shall maintain a minimum placement and finishing rate of at least 53.4 feet/hour along ℄ Bridge.
Should the Contractor not possess the necessary equipment and facilities to maintain the minimum placement and finishing rate, the slab shall be placed in sections. All Positive moment sections of the slab shall be placed followed by placement of the Negative moment sections of the slab.
Alternate procedures for placing deck concrete may be submitted for approval by the Contractor.

Figure 9.3—Example Pouring Diagram

Alternatives if the contractor cannot maintain this pour rate or concrete cannot be placed and finished fast enough to maintain plasticity between construction joints, listed in order of preference

A. Skip pouring, see §9.2.5.2
B. Admixtures to increase the length of time concrete remains plastic. This option is less preferred due to the inherent lack of certainty with admixture performance.
C. Add a longitudinal deck construction joint

Minimum pour rates determined using this methodology are likely to be significantly higher than the 20 ft./hr. for bridge decks specified in paragraph 3.d. of subsection 706.03 in the Standard Specifications. The pour rate required must be given on the plans for it to govern over the minimum pour rate given in the Standard Specifications.

9.2.6—Drip Bead Detail

A drip bead shall be placed on all bridge decks and concrete slab bridges. The cell shown in "Figure 9.14—DRIPBnew Cell (Bridge Deck Library)" shall be used for open rail bridges. The cell shown in "Figure 9.15—DRIPB Cell (Bridge Deck Library)" shall be used for closed rail bridges, approaches, and bridge decks with a single layer of reinforcement.

9.2.7—Roadway Crown

Crown of the bridge deck shall be shown on all Plans. The cell shown in "Figure 9.16—CROWN Cell (Bridge Deck Library)", is available for a standard 2% crown.

Bottom of the deck shall be set as a straight line between girder shims.

9.2.8—Phased Decks

Bridge decks to be built under phasing shall meet the empirical deck cantilever requirements for the overhang that is open to traffic or as loading requires. Consideration should be provided for phased traffic and temporary barriers for existing and new decks in cantilever condition.

Designs shall include closure pours whenever the differential dead load deflection exceeds 2 in. Preliminary analysis of girder deflections may be necessary to determine if phased construction is feasible. Girder deflections at each phase shall be considered when adjusting the shim to the deck. When closure pours are required, standard note #010 shall be shown and the details showing the left out separators shall be shown in the typical cross section.

Typical closure pour width shall be 4 ft when using standard clear cover specified in §5.3.

Bottom clear cover on transverse reinforcement may be adjusted to reduce splice length at construction joints, but core depth must follow LRFDBDS requirements for Empirical Deck Design.

For example, take the girder phasing shown in Figure 9.4.

Deflections to account for on Girder C

- Non-composite deflections
  - Self weight of the girder
  - Prestressing (if applicable) and losses per §5.2.1.5.
  - Camber cut/formed into steel girders
  - Phase I deck pour (consider actual pour width)
- Partial width composite deflections (consider actual Phase I pour width in determining moment of inertia at this phase)
  - Temporary safety barrier
  - Wearing surface applied to Phase I prior to Phase II construction (if applicable, some wearing surfaces are applied full width after the bridge deck is fully poured)
  - A portion of Phase I permanent rail

Deflections to account for on Girder D

- Self weight of the girder
- Prestressing (if applicable) and losses per §5.2.1.5.
- Camber cut/formed into steel girders

The difference between these two deflection values is the differential deflection between Girder C and Girder D.

Greater than 2 in. deflection listed in the deflections for shims table on the girder data sheet does not necessarily mean a structure requires a closure pour.

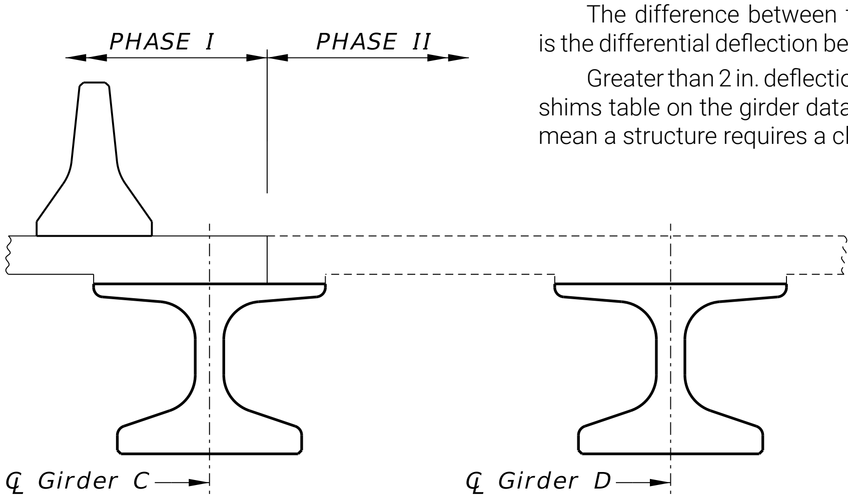


Figure 9.4—Differential Deflection Cross Section (for illustrative purposes)

9.2.9—Stay in Place Forms

Metal stay in place forms are allowed if identified as allowed on the BDS. The forms shall support the weight of the plastic concrete deck and any corresponding construction loads. Shop drawings shall be submitted to NDOT as information only. After construction these forms provide no structural function. All corresponding bridge elements shall be designed for stay-in-place forms load based on §3.3. Refer to Standard Specifications (§704.03.6.g) for more information.

9.2.10—Widening

Match new reinforcement to the greater of:

- Existing deck reinforcement
- Reinforcement for a newly constructed bridge deck per §9.2.2.

Overhang reinforcement shall be per §9.2.3.

9.3—NU DECK

Reserved for future use.

For more information see Morcous et al. (2013).

9.4—COMPOSITE ACTION ON CONCRETE GIRDERS

Where a haunch tall enough to prevent the SWWR projecting from the girder from developing within the CIP deck properly (per §9.4.2) is detected (either in design phase or during shim shots) supplemental reinforcement shall be provided.

Supplemental reinforcing "hat bars" may be necessary following shim survey measurements to satisfy requirements. Supplemental hat bar plan and quantity along with additional shim concrete quantities shall be provided after the results of the girder survey.

It is recommended to issue some estimated amount of supplemental reinforcement with the letting set so that construction can continue without waiting for procurement of the supplemental reinforcement.

Supplemental reinforcement spacing need not match spacing of SWWR wires.

9.4.1—Design Assumptions

Shear friction calculations (per LRFDBDS Article 5.7.4) shall be run for:

- The interface between the precast girder and the bottom of cast-in-place haunch
- The interface between the top of the cast-in-place haunch and the bottom of the cast-in-place deck
- Other interfaces as required, such as for threaded rod connected I-Girders discussed in §5.5.3.

Reduced interface width shall be accounted for in interfaces where full bond stress may not be achieved.

NDOT base sheets show 8 in. each side of NU Girder top flanges to be smooth with bond breaker.

9.4.2—Reinforcement Development

To be considered effective for horizontal shear transfer reinforcement must be embedded within the core of the slab, above the bottom mat of longitudinal reinforcement in the deck.

Preferred supplemental reinforcement where provided SWWR in girders is not tall enough to engage the CIP can be seen in Figure 9.5.

The 5 in. standard extension of precast stirrups provides enough development length near the location of minimum haunch to satisfy anchorage requirements. Where thicker haunches are anticipated designers may investigate the feasibility of using larger projections to avoid hat bars.

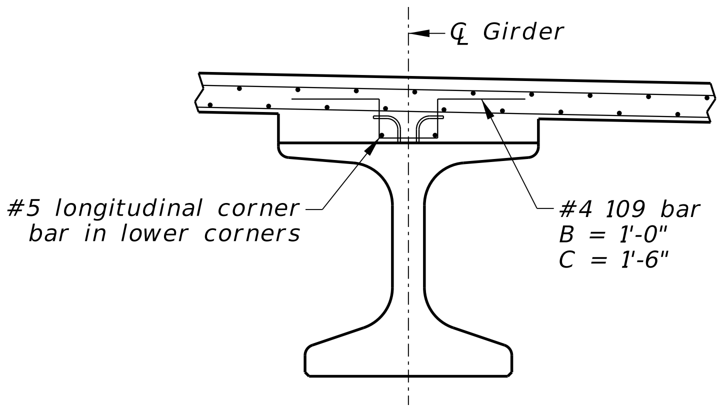


Figure 9.5—Preferred Hat Bar Detail

9.5—TURNDOWNS AND DIAPHRAGMS

A #6 reinforcing bar shall be placed in the hooks of the extended prestressing strand to improve the effectiveness of the anchorage of the reinforcement. The length of reinforcing placed in the hook shall be the width of the girder bottom flange, except at locations where splayed strands are used. Full length bars are not permitted on bridges with skew as they will not be able to simultaneously anchor strands and run parallel to the skewed substructure.

Bars in the corner of extended strands do not need to be developed to be used for anchorage.

9.5.1—Blockouts for Anchor Rods

Mandatory block outs shall be shown on the plans in concrete turndowns/diaphragms that utilize anchor rods see Figure 9.6 for recommended layout.

See §14.2.10 for discussion of when anchor rods may be required at bearings.

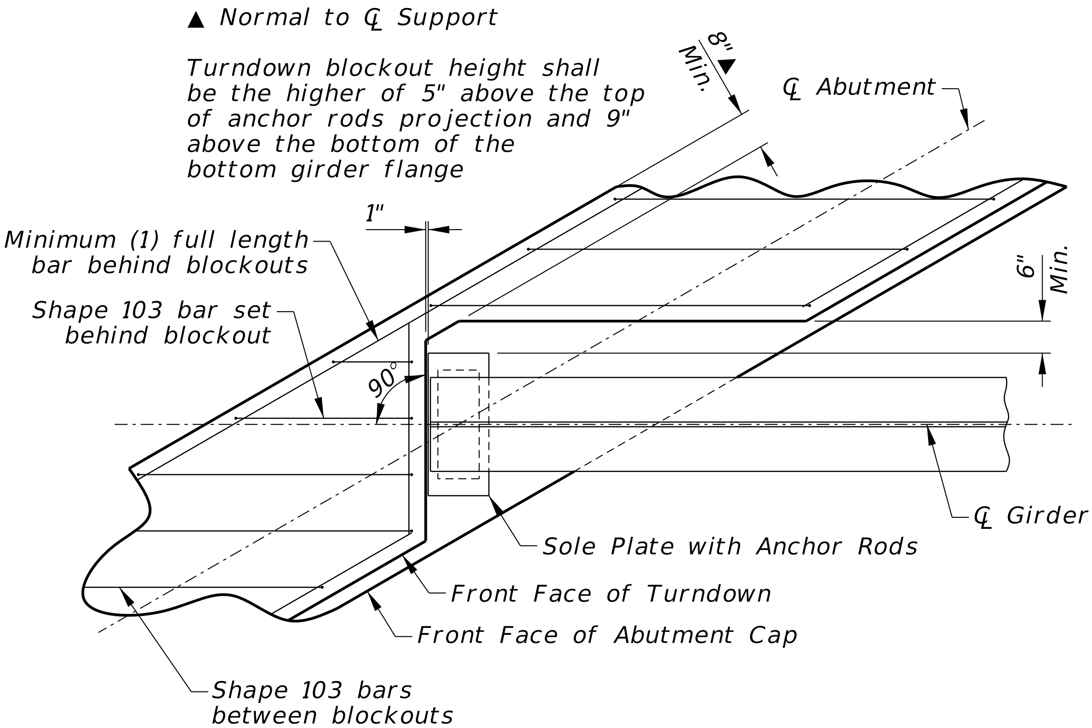


Figure 9.6—Anchor Rod Blockouts

Schematic details for turndowns at abutments and intermediate support diaphragms can be seen in Figure 9.7 through Figure 9.11 and can be used as detail guidelines.

Details shown are the minimum reinforcement and designers should calculate the required reinforcement on a case-by-case basis.

9.5.2—Slab Turndown at Abutments

The front face of a slab turndown shall be setback 2 in. minimum from the front face of the abutment cap. The back face of a slab turndown shall be flush with the back face of the abutment cap for abutment caps not greater than four feet wide. Slab turndowns shall be detailed with a 4 in. chamfer from bottom of bridge deck to front face of turndown.

The minimum approach slab seat length shall be 1 ft. 0 in. long measured along the centerline of the Roadway. A mandatory construction joint shall be detailed at an elevation between the approach slab seat and bottom of the bridge deck.

Extended strands on shallow girders at abutments should be detailed to miss approach slab seat blockouts. Use 180° hooks in lieu of 90° if necessary.

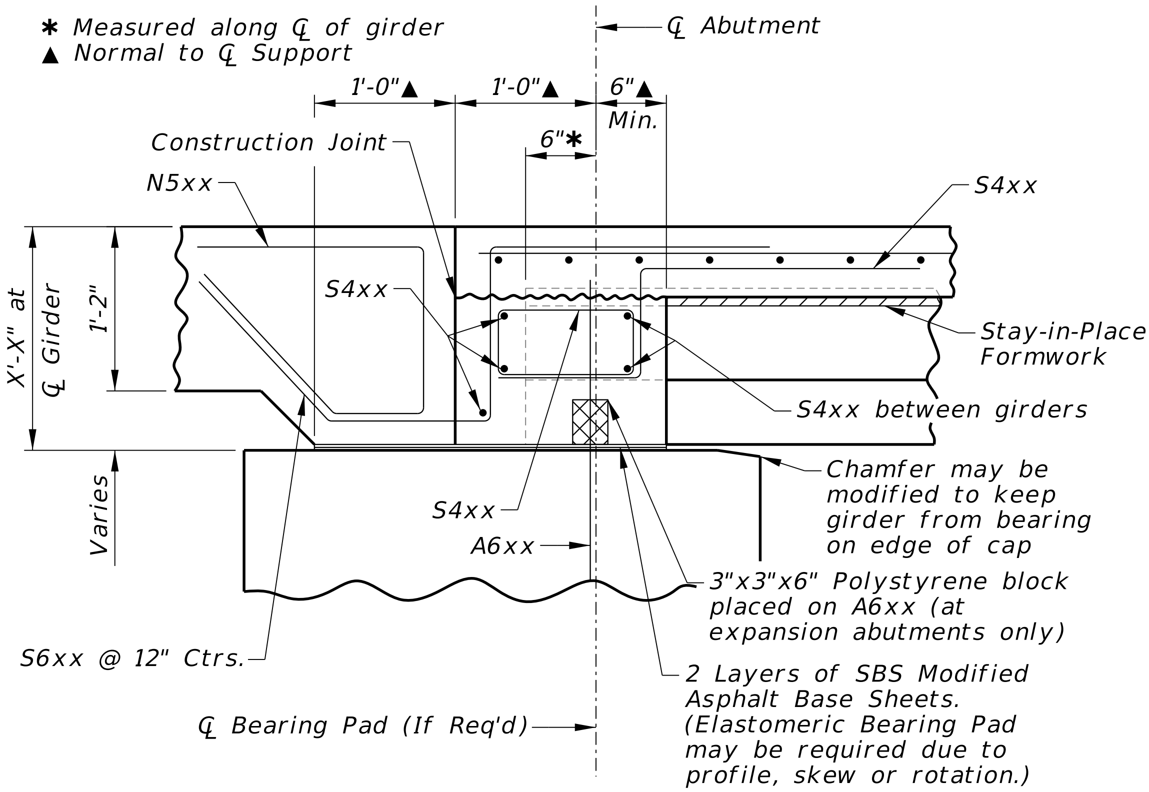


Figure 9.7—IT Girder Slab Turndown (Required on IT 300, Optional on IT 400)

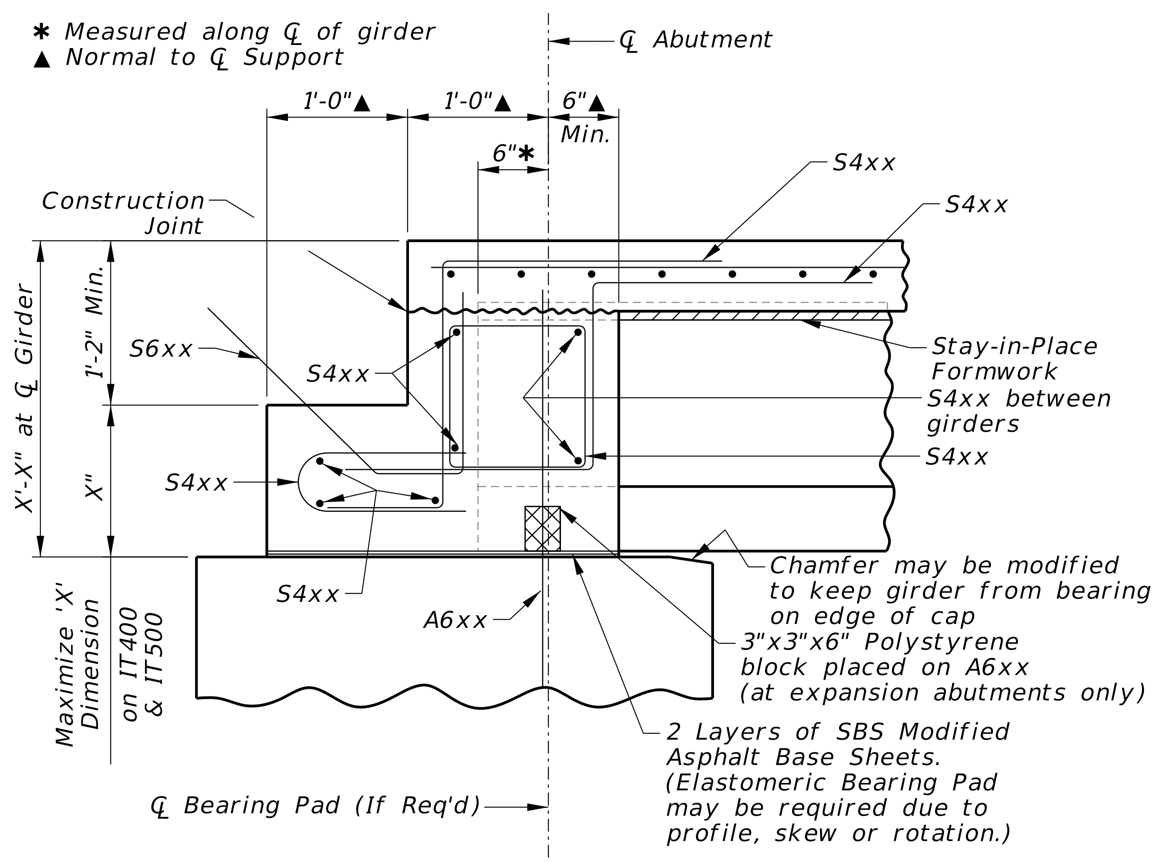


Figure 9.8—IT Girder Slab Turndown (Required on IT 500 and Deeper, Optional on IT 400)

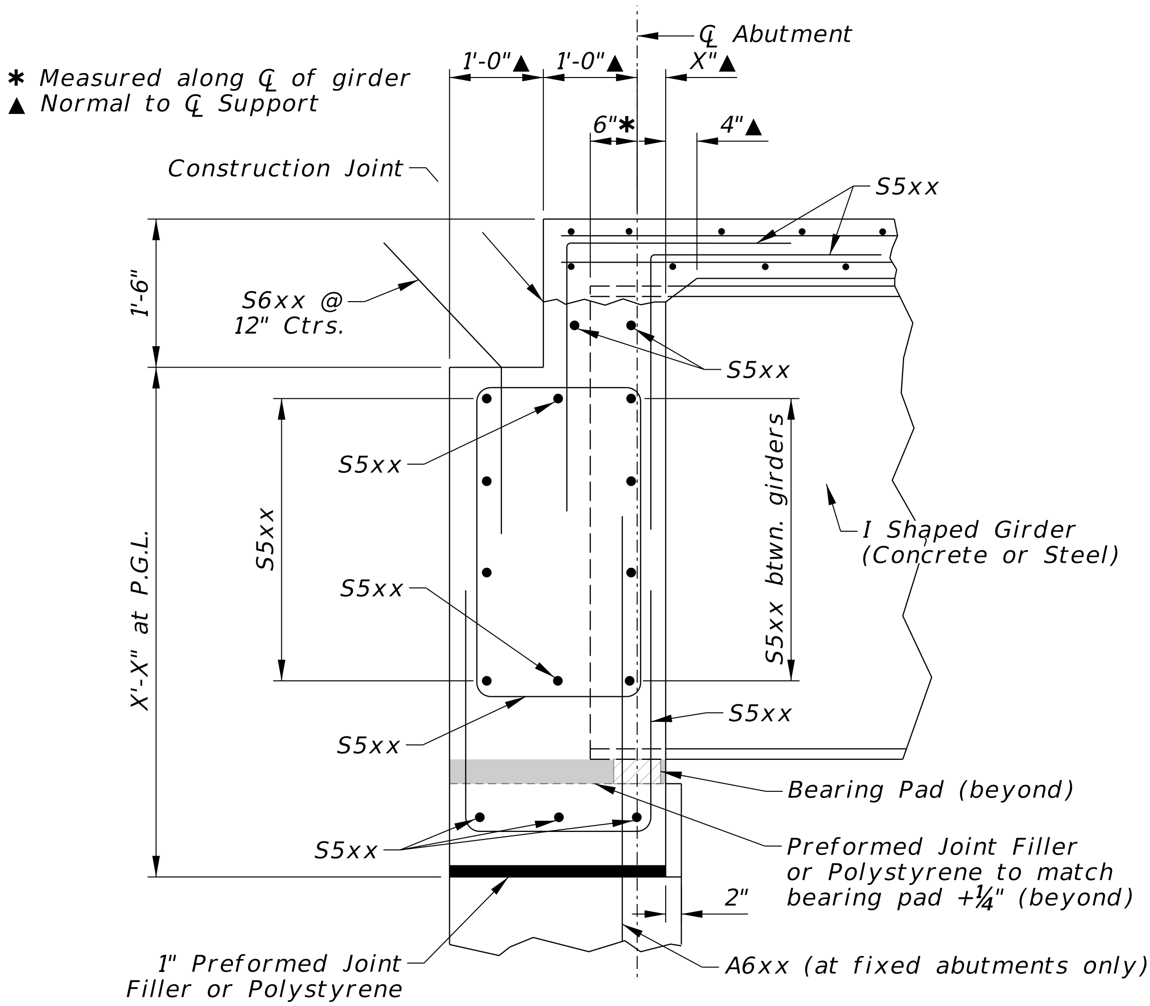


Figure 9.9—I Shaped Girder Slab Turndown (Concrete or Steel)

9.5.3—Diaphragm at Intermediate Supports

Ahead station and back station faces of diaphragms shall be set back 2 in. from the respective face of the substructure cap. Diaphragms shall be detailed with 4" chamfer from bottom of bridge deck to ahead station and back station faces of diaphragm.

A mandatory construction joint shall be detailed at a location approximately ⅓ of the girder depth below the bottom of the bridge deck.

A flush concrete Diaphragm, where the end of the diaphragm matches the exterior girder profile, shall be used at all overpass structures where the lower roadway is of State Functional Classification Major Arterial or higher. When a flush concrete diaphragm is used the cell shown in "Figure 9.17—Ext. Strand Splay Cell (NU Details Library)" is provided for use on the plans. For all other locations, the concrete Diaphragm shall be extended beyond the exterior girder.

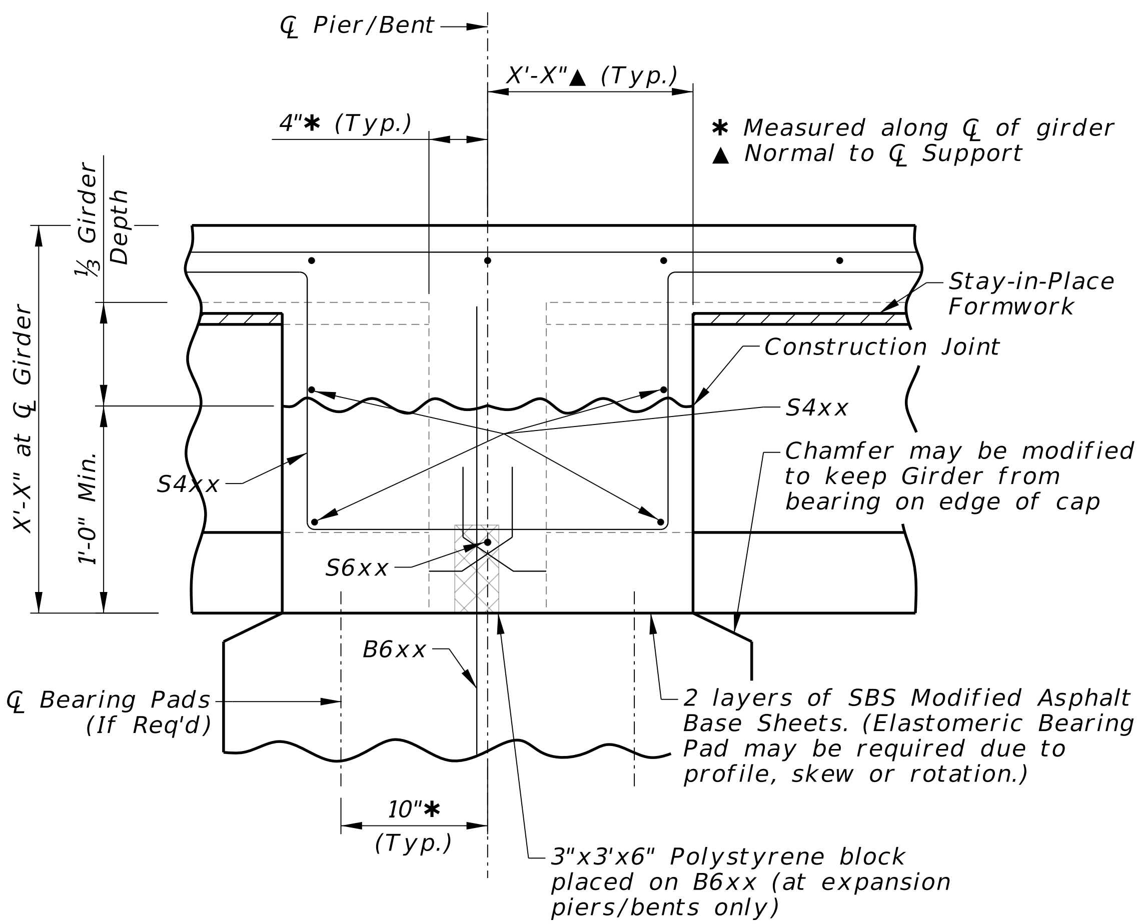


Figure 9.10—IT Girder Diaphragm

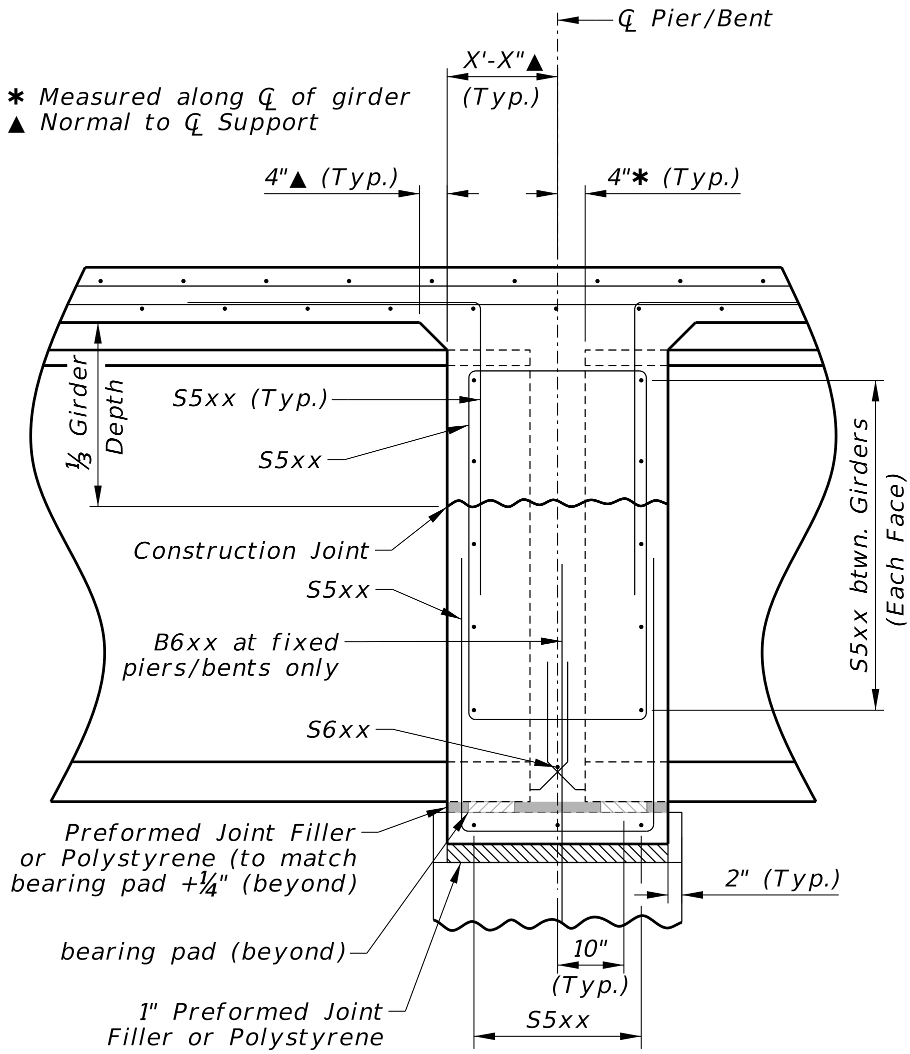


Figure 9.11—I Shaped Girder Diaphragm (Concrete or Steel)

9.6—REFERENCES

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Transportation Research Board, Holt, J. M., Lopez, M. D., Murphy, T. P., Steelman, J. S., Rosenbaugh, S. K., Loken, A. E., Faller, R. K., Galvan, M., Bloschock, M., & National Cooperative Highway Research Program. (2024). *NCHRP Research Report 1109: Bridge Railing Design Requirements*. National Academies Press. https://doi.org/10.17226/27893

9.7—REFERENCED CELLS

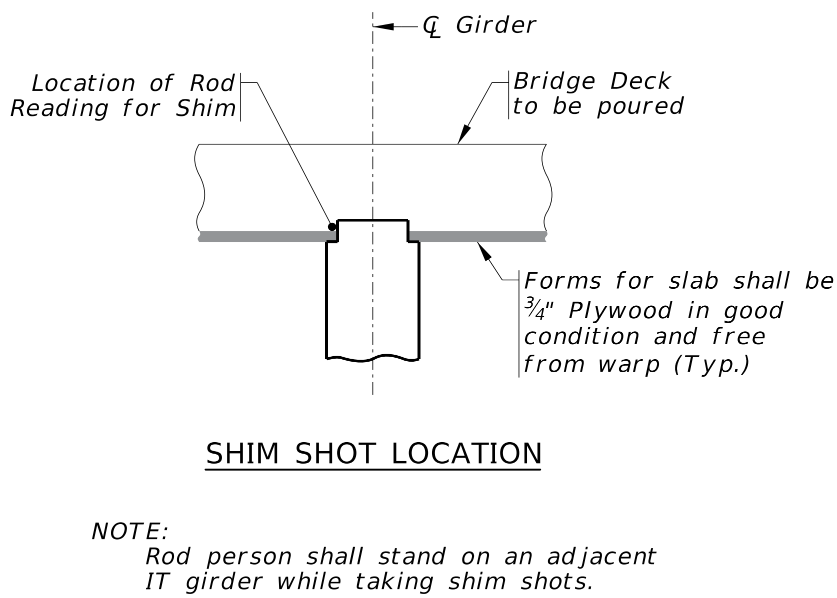


Figure 9.12—SHIM SHOT IT Cell (Bridge Deck Library)

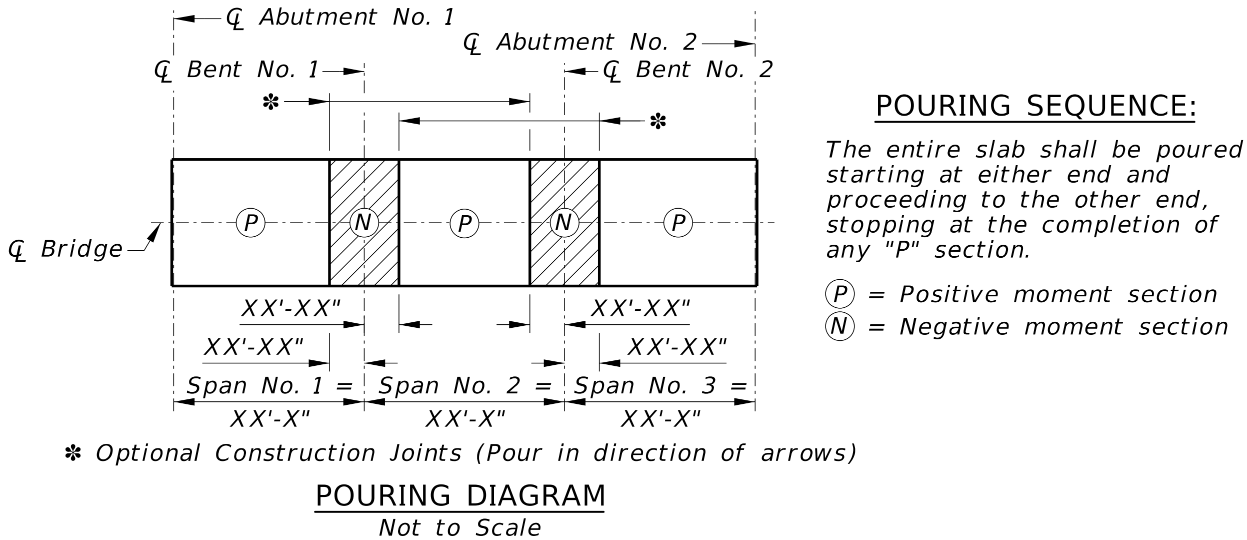


Figure 9.13—POURSQ Cell (Bridge Deck Library)

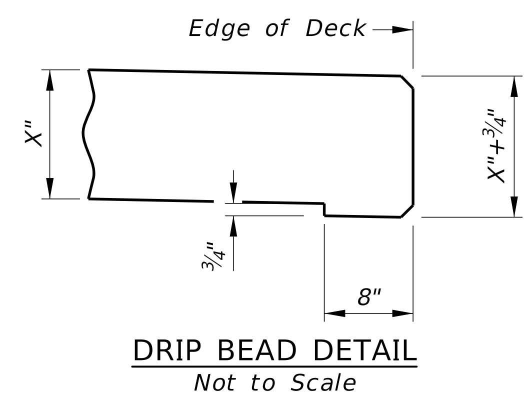


Figure 9.14—DRIPBnew Cell (Bridge Deck Library)

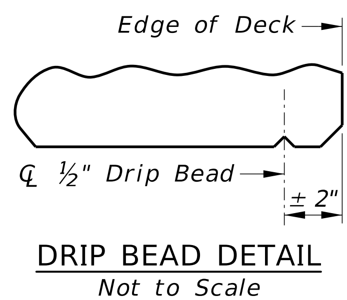


Figure 9.15—DRIPB Cell (Bridge Deck Library)

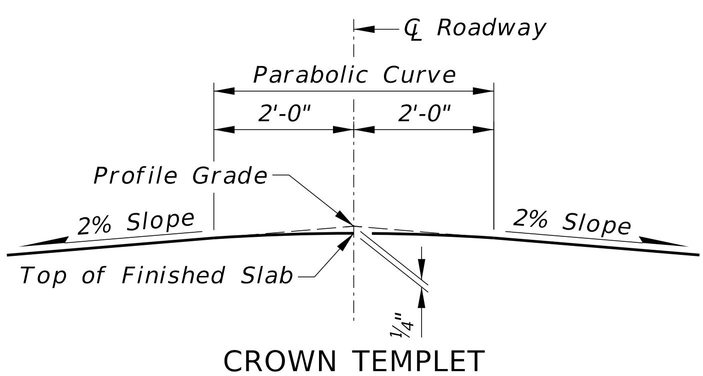


Figure 9.16—CROWN Cell (Bridge Deck Library)

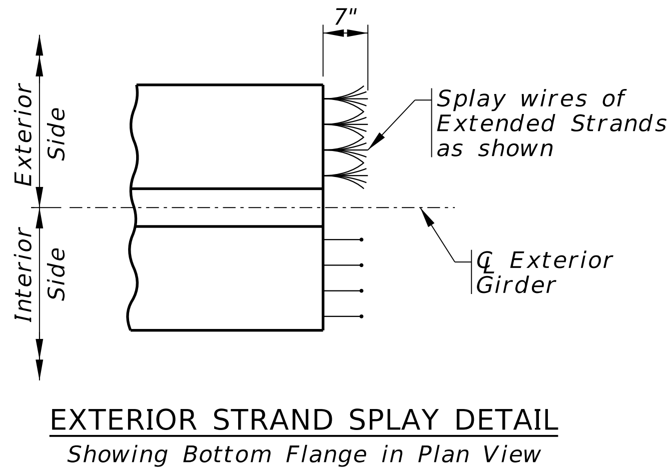


Figure 9.17—Ext. Strand Splay Cell (NU Details Library)

Last Updated: November 12, 2024

Chapter 13 — Railings

13.1—OVERVIEW

Standard weights for each rail type are provided in Table 3.1 in §3.3.2.

13.1.1—Design Criteria

13.1.1.1—New Construction

13.1.1.1.1—Rail Type

Final rail selection will be determined at data sheet approval, but general guidelines are:

- Use Closed Rail for Grade Separations.
- Use Closed Rail for stream crossings less than 300 ft with hydraulic section approval.
- Use Open Concrete Rail (OCR) all for stream crossings 300 ft. and greater.

The 300 ft. minimum limit for use of OCR was chosen to capture most of the river crossings in the state. Open Concrete Rails provide operational advantages for snow removal as well as improved deck drainage, particularly on bridges with narrow shoulders. These benefits become more prevalent for longer bridge structures. For longer bridge structures that span both roadways and stream crossings, the Designer should consider using the closed section only where required over recreational trails, sidewalks, vehicle, or railroad traffic.

Existing bridges with open concrete rails in the state inventory are associated with an increased prevalence of deck edge deterioration developing over time. When factoring in the lifetime costs of repairs needed to maintain open rail bridges, the benefits of closed rail become apparent.

13.1.1.1.2—Rail Height

- 42 in. Rails
  - Use 42 in. NU Rail on structures carrying Interstate and expressway traffic.
  - Use 42 in. NU Rail as a minimum over Railroads, see §2.3 for additional information.
  - Use 42 in. NU Rail as a separation barrier for bridges with sidewalks.

Typically, the 42 in. rail has been used on high ADT and urban area structures.

It is not preferred to use 42 in. open rail on short stream crossings.

The 42 in. rail was originally tested at TL-5 using the previous standard of NCHRP 350. It is still approved for TL-5 use under the current standard of AASHTO MASH if used without a thick (such as AC+M) overlay. LRFDBDS Article 13.7.3.2 requires a 42 in. tall rail to qualify for TL-5.

LRFDBDS Article 13.8.1 requires 42 in. minimum rail height above the top of sidewalk. Previous practice was to install a steel Pedestrian Barrier Rail on top of lower height rails to make up the difference. This is not needed for 42 in. rails. Pedestrian Barrier Rails may still be necessary in certain situations, refer to §13.5.

- 39 in. Rails
  - Use 39 in. Concrete Rail on all Interstate grade separations and State Highway Structures. The 39 in. Single Slope Closed Rail (SSCR) is preferred.

The 39 in. rail was crash tested in 2018 with the report approved and published in 2021 at AASHTO MASH TL-4 level, including additional height for a 3 in. overlay (Rosenbaugh et al., 2021).

- 34 in. Rails
  - Use 34 in. NU Rail may be used on allowable Non-System Bridges. Not permitted on curved bridges.

The 34 in. NU Closed rail requires more concrete than the 39 in. SSCR rail. Therefore, for closed rail, the 39 in. SSCR is preferred even when 34 in. NU Closed Rail is allowable.

For non-State owned structures, other MASH-tested rails may be used at an appropriate test level based on traffic volume and roadway design needs. An example of a TL-2 bridge rail for low volume road can be found in Rosenbaugh et al. (2020a).

13.1.1.2—Existing Bridges

Existing bridges should be investigated to determine deviations from the standard rail layouts. See §7.3.2 for additional information regarding existing Bridge Rails.

When widening an existing Bridge to one side, it is preferable to use a new MASH tested railing in the new construction. Consideration should be given to upgrading the non widened side to a new rail using engineering judgment.

13.1.2—Closed Rail Requirements

Closed Concrete Rail shall be used on all approach sections. If necessary, Roadway Design will provide inlets and surfacing to handle the drainage.

For bridges with sidewalks, all separation barriers on the bridge and approach sections shall be closed to prevent icing of the sidewalk.

For closed rails requiring deck drains on bridges with sidewalks, ensure that there is no conflict between the deck drains and the girder lines. This shall be determined during preliminary design.

All closed rail bridges will be reviewed by Bridge Hydraulics for Floor Drains. In certain situations, Bridge Hydraulics may recommend use of Open Concrete Rail instead, see §8.1.2 for more information.

Some of the factors Bridge Hydraulics considers when evaluating deck drainage are:

- Shoulder Width. In general, a Bridge with narrow shoulders (<4 ft.) requires more floor drains than a wider bridge at the same site.
- Longitudinal slope. A bridge with a slope of less than 0.5% is considered flat, which is less efficient at moving drainage off the deck than a bridge with a steeper grade.
- Location. The amount of rainfall in the western part of the state is less than that in the eastern part of the state, which may impact floor drain decisions.

13.1.3—Embankment Protection for Open Rails

Embankments above high water elevations are subject to compounding erosion from Open Concrete Rail drainage. Designers should be aware of erosion problems that may exist on-site and should recommend appropriate action to protect stream embankments.

13.1.4—General

Rail systems will be laid out and reinforced as shown on the Bridge Rail Base Sheets in Appendix B.

The height of the vertical leg of the "L" shaped reinforcement will be sized to fit the bridge deck. The vertical leg will be designed to provide 3 in. minimum clearance to the top of the rail. Angled Bar Types shall be provided for cross slopes greater than 2.5% to eliminate field bending of the "L".

Concrete Rail Sections are available as cells for use in detailing, see §13.8.

Quantities for the portion of concrete rail placed on the bridge deck will be listed as subitems under the bridge Pay Items. Quantities for the portion of concrete rail placed on the bridge approach slab will be listed as subitems under the Pay Items for the approach slab.

13.2—39 IN. RAILS

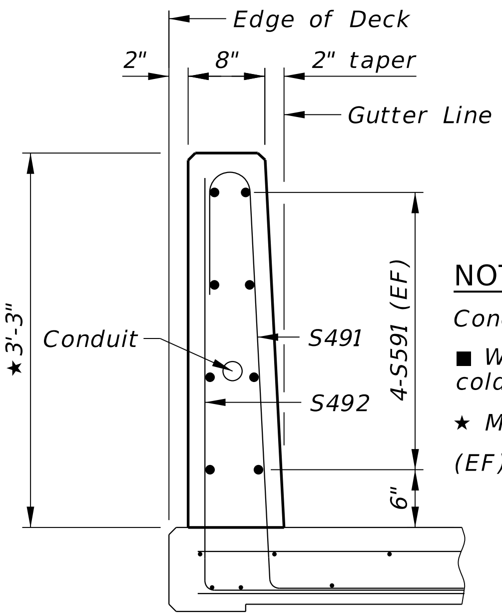
The 39 in. rail series were developed according to the criteria set forth in AASHTO's Manual for Evaluating Safety Hardware (MASH).

13.2.1—Single Slope Closed Rail

The 39 in. single slope rail is 8 in. wide at the top, sloping along the front face to 10 in. wide at the base of the rail (Rosenbaugh et al., 2021).

The distance from gutter line to edge of deck shall be 1 ft. 0 in. when this rail type is used.

The shape of the 39 in SSCR was designed to allow slip-forming.



NOTES

Concrete rail and post will be built plumb.

■ *When pouring concrete rails, a mandatory chamfered cold joint must be formed at the end of floor.*

★ *Measured at front face of rail.*

(EF) = Each Face (FF) = Front Face (BF) = Back Face

TYPICAL SECTION OF RAIL

Figure 13.1—39 in. SSCR Bridge Rail Reinforcing

13.2.2—Open Concrete Rail

The 39 in. OCR consists of a 14 in. wide × 27 in. tall beam, with 3 ft. long × 10 in × 1 ft. tall interior posts and 6 ft. long × 10 in. × 1 ft. tall end posts (DeLone et al., 2023).

The system uses a 9 ft. maximum center to center post spacing over the length of the bridge. At the end posts the maximum clear span of the rail beam is 6 ft.

Typically the end span length is adjusted and the 9 ft. post spacing is maintained over the remainder of the bridge length. Where this results in unreasonably short end spans either the 9 ft. spacing can be adjusted full length or the end two spans of the rail can both be adjusted.

Vertical and horizontal layout dimensions shall be measured at the front face of rail.

The distance from Front face of Rail to edge of deck shall be 1 ft. 4 in. when this rail type is used.

To protect substructure elements, place a 6 in. concrete curb between rail posts over the interior supports as shown in the Base Sheet details.

For repair activities coinciding with AC+M overlay placing an FRP angle between posts to protect intermediate supports is also acceptable For other construction placing a 6 ft. end post centered over the interior supports is also acceptable alternative.

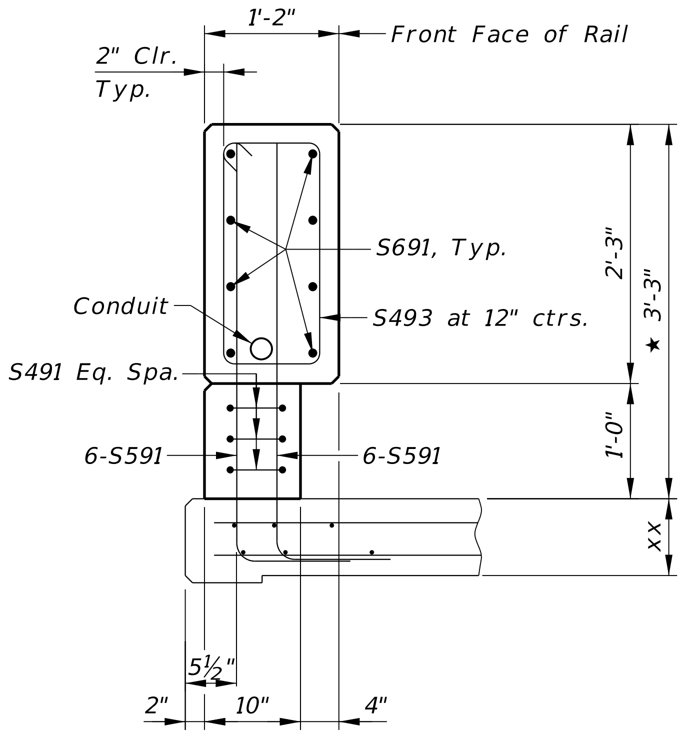


Figure 13.2—39 in. OCR Bridge Rail Reinforcing

13.2.3—Buttress Details

Designers are advised that the 39 in. Rail series utilize a different Buttress design than was used in the previous generation of Rails. The design has an upper and lower horizontal taper, as well as 4 in. vertical height transition on the end (Rosenbaugh et al., 2020b).

The Control Bolt location is 10 in. from the end of the Rail, which is 11 in. closer to the end of the rail than the previous generation of Rails.

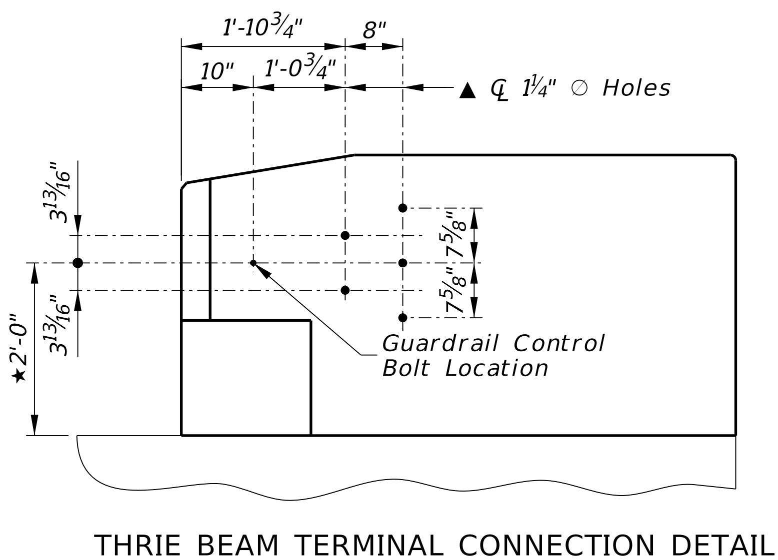


Figure 13.3—39 in. Rail Buttress — Thrie Beam Terminal Layout

13.3—NU RAILS

The NU Rail Series were developed according to the criteria set forth in NCHRP Report 350, "Recommended Procedures for the Safety Performance Evaluation of Highway Features."

Where 34 in. rails are specified, both the primary and alternate cells should be used on the plans in order to permit the use of steel forms purchased by some contractors. When showing existing rails in plans, inspections photos should be used to determine which profile was constructed.

Vertical and horizontal layout dimensions for the NU Rail series shall be measured at the front face of rail.

The distance from Front face of Rail to edge of deck shall be 1 ft. 4 in. when this rail type is used.

13.3.1—NU Open Concrete Rail

The 34 in. OCR consists of a 14 in. wide × 23 in. tall beam, with 2 ft. 6 in. long × 10 ¹⁄₂ in. wide × 11 in. tall interior posts and 4 ft. long × 10 ¹⁄₂ in. wide × 11 in. tall end posts.

The system uses a 8 ft. maximum center to center post spacing over the length of the bridge. At the end posts the maximum clear span of the rail beam is 5 ft. 6 in.

Typically the end span length is adjusted and the 8 ft. post spacing is maintained over the remainder of the bridge length. Where this results in unreasonably short end spans either the 8 ft. spacing can be adjusted full length or the end two spans of the rail can both be adjusted.

The 42 in. OCR is the same as the alternate layout of the 34 in. rail with an additional 8 in. tall extension

The extension has a setback to prevent a vehicle occupant's head extending outside of the window during an oblique small car impact from contacting the barrier face (Polivka et al., 2005)

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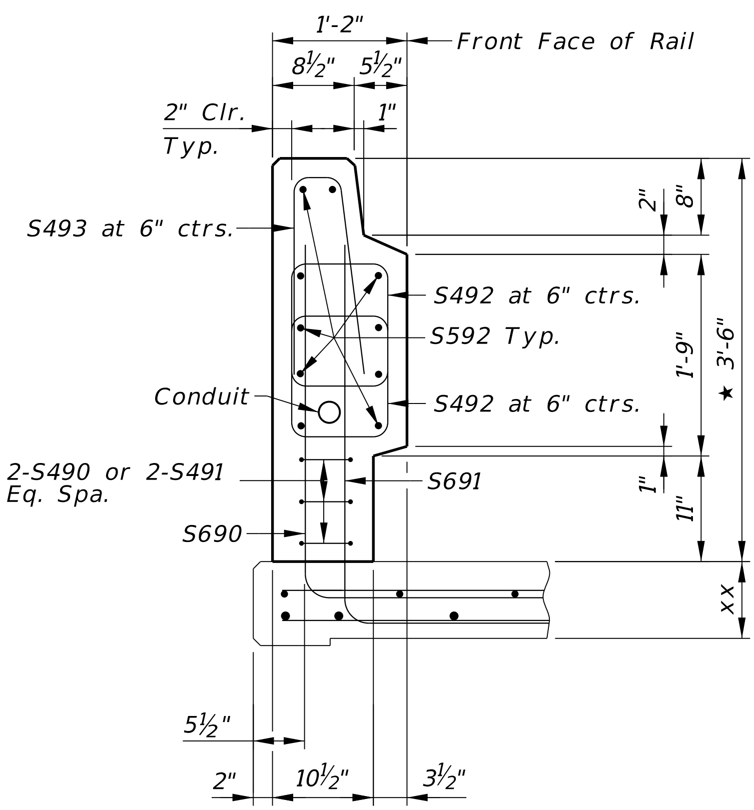

TYPICAL SECTION OF RAIL

Figure 13.4—NU Bridge Rail Reinforcing (shown here for 42 in. rail, see Base Sheets for 34 in. rail)

13.3.2—Buttress Details

Designers shall use the buttress design consistent with this generation of Rails (NCHRP 350).

The Control Bolt location is 1 ft. 9 in. from the end of the Rail.

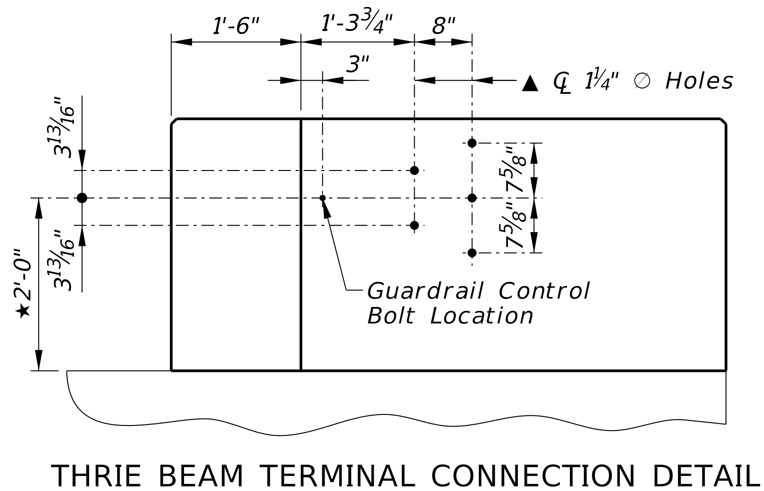

Figure 13.5—NU Bridge Rail Buttress — Thrie Beam Terminal Layout

13.4—LEGACY RAILS

13.4.1—Overview

Legacy Rails are concrete bridge rail types that are no longer specified for new construction on System Bridges but are acceptable to remain in place. For these rail types, cells are available for detailing in the Barriers and Rails cell library.

When it is necessary to repair, replace or extend these rail types on a bridge repair project, it is considered acceptable to match the existing geometry and reinforcement layout of the existing rail.

Per LRFDBDS Article 13.7.3.2, the minimum height of an existing concrete bridge rail to remain in place is 27 in. from the top of the wearing surface at the FF of the rail.

Base sheets for these rail types are not available in OBM; however, the Base Sheets from the previous generation of MicroStation are still applicable. When remodeling these rail types, use engineering judgment to determine appropriate modifications from the available Base Sheets. For example, some existing Jersey Barriers are 15 in., they don't need to be widened to 16 in.

13.4.2—29 in. Nebraska Rail

The 29 in. Nebraska open rail consists of a 14 in. wide × 16 in. tall beam, with 2 ft. long × 11 in. wide × 13 in. tall interior posts and 3 ft. long × 11 in. wide × 13 in. tall end posts. The 29 in. Nebraska closed rail variant has the same cross section geometry without discrete posts.

The system uses a 8 ft. maximum center to center post spacing over the length of the bridge. At the end posts the maximum clear span of the rail beam is 6 ft. At locations where the rail beam must continue beyond a post up to a 1 ft 0 in. cantilever is acceptable.

Some existing bridges use a previous version of the 29 in. tall Nebraska Rail that used a 14 in wide x 12 in tall beam, with 11 in. long × 11 in. wide × 17 in. tall posts. This rail shall not be used anymore. Only use the version described in Figure 13.6 when permissible.

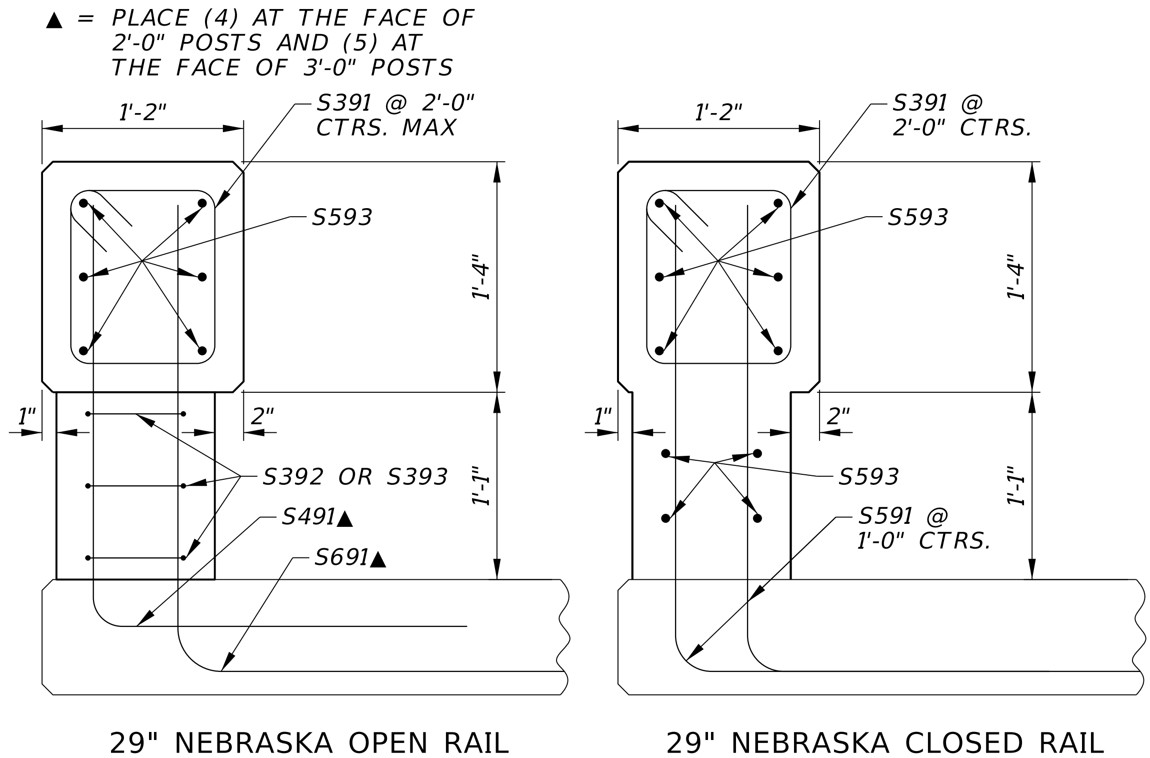

Figure 13.6—29 in. Nebraska Rail Reinforcing Sections

13.4.3—Concrete Barrier

The concrete barrier consists of a 32 in. or 42 in. tall New Jersey shaped section. All layout dimensions for the barrier will be measured at the gutter line (front face) of the barrier.

All longitudinal bars in the concrete barrier will be # 4 bars. In addition, there are four continuous bars placed in the bridge deck.

The 123, 125, and 126 type stirrups will match a spacing provided in the bridge deck, up to a maximum spacing of 1 ft. 3 in.

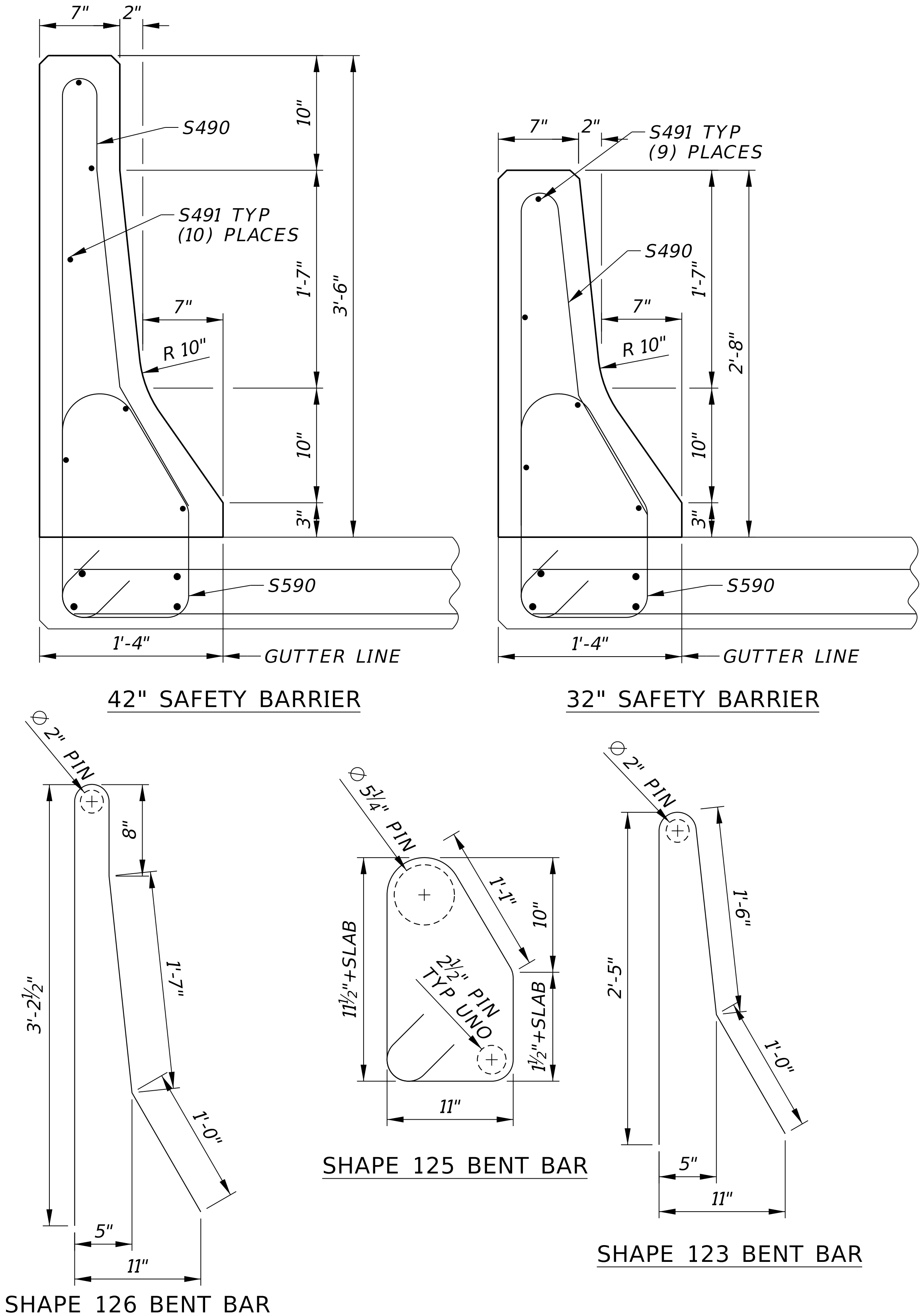

Figure 13.7—Concrete Barrier Reinforcing Sections

13.4.4—Thrie Beam Guardrail Connections for Legacy Rails

13.4.4.1—Use of Standard End Sections (NCHRP 350)

The standard End Section for the Legacy Rails on Approach Slab is shown in Figure 13.8. This section utilizes fully developed #6 L bars at approx. 5 in. centers in each face for the deck to rail connection.

When designing rail updates on an existing bridge, the standard end section should be utilized as much as possible.

See §7.3.2.1 for guidance regarding the location of the guardrail connection.

In most situations, the typical Standard End Section length of 5 ft. shall be used. However, because this system was designed with an allowance for a 2 ft. blockout on each side of the expansion joint, the absolute minimum length of a Standard End Section is 3 ft.

Designers are encouraged to remove and rebuild a portion of the deck edge, wingwall, or both as needed to install a standard end section, in lieu of using adhesive anchor bars (also known as dowels) to make the rail connection. The design of adhesive anchors take into account failure modes that do not apply to cast-in fully developed reinforcement.

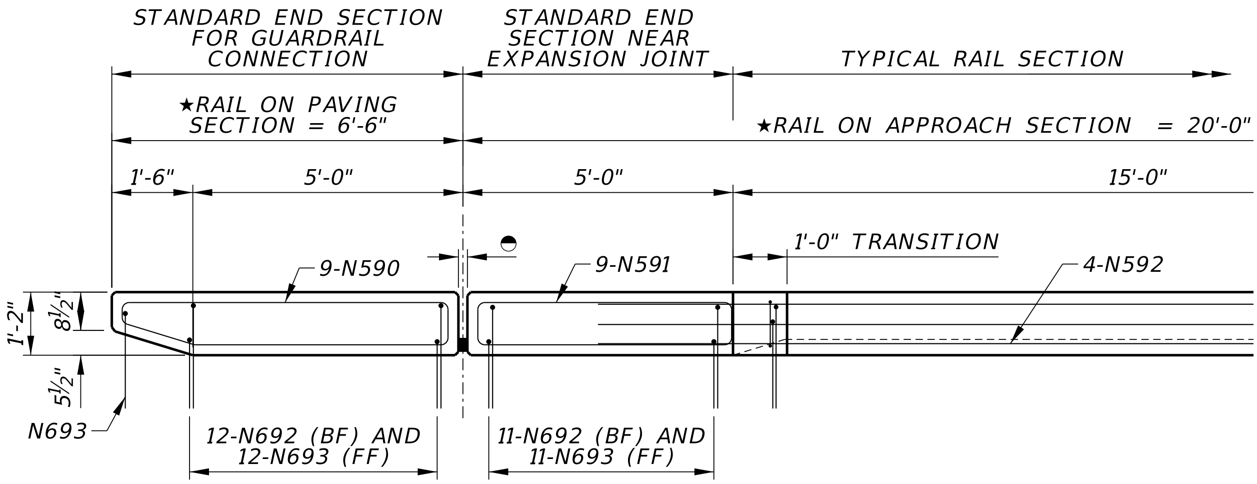

Figure 13.8—Plan View of Standard Legacy Rail End Sections (NCHRP 350)

13.4.4.2—Thrie-Beam Guardrail Connections

All layout dimensions for guardrail connections will be measured at the Gutter Line / Front Face of Rail.

New guardrail connections shall consist of a Rectangular End Section with a 1 ft 6 in. taper added to the end.

The height of the End Section shall be 35 in. at the Guardrail Connection, measured from the top of the concrete slab. The Control Bolt location is 1 ft. 9 in. from the end of the Rail, shown to be installed 24 in. from the top of the slab, see Figure 13.10.

For existing Bridge Rails, designers shall transition the height of the existing rail as needed to provide 35 in. at the End Section. This height transition shall be no steeper than 1V:8H. Transitions from existing rail geometry to the constant width End Section shall be done as shown in the Rail Base Sheets.

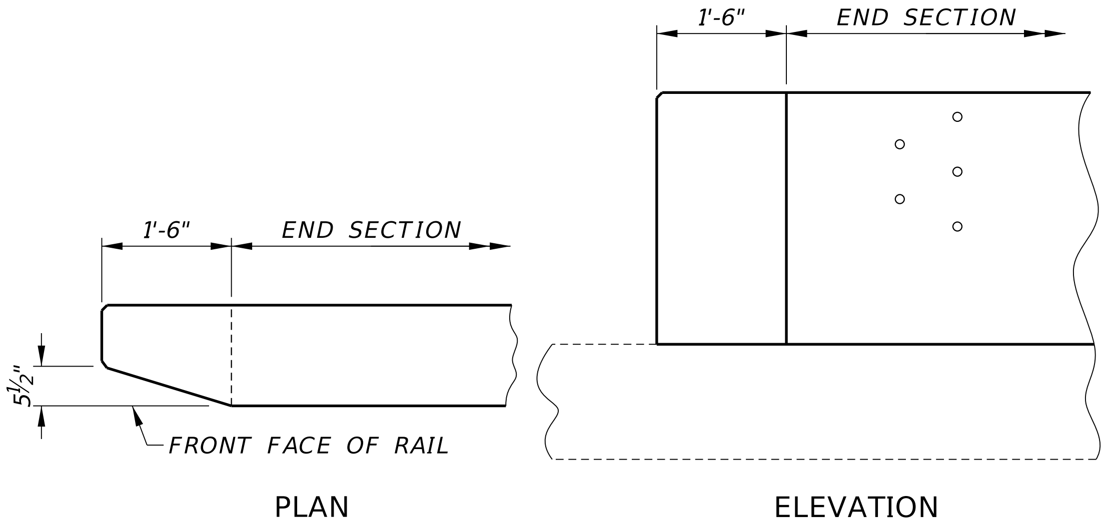

Figure 13.9—Legacy Rails — Guardrail End Section Dimensions

★ *Measured at front face of rail.*

(EF) = Each Face (FF) = Front Face (BF) = Back Face

▲ *As an alternate method, the contractor shall furnish and cast into the concrete an approved welded assembly consisting of threaded inserts, held accurately to the template of the holes shown. Inserts are to be complete with galvanized plate washers and galvanized ⅞" ∅ x 2" cap screws. The insert assembly shall be a standard product of a reputable manufacturer of such items and be capable of resisting a shear load of 80,000 lbs.*

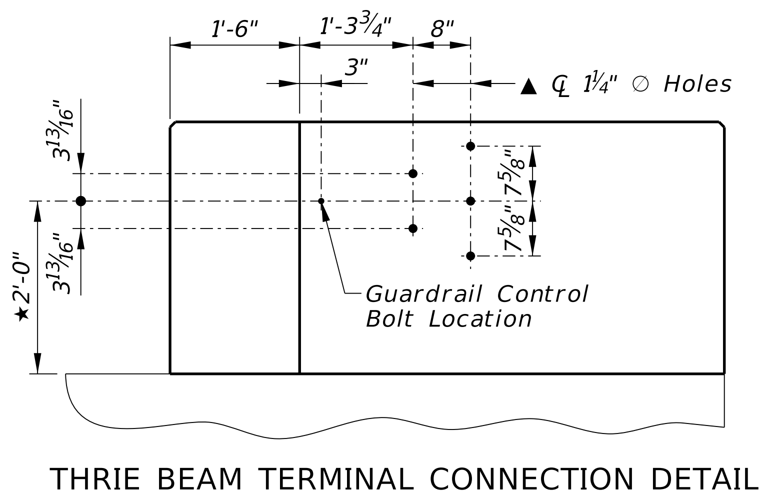

Figure 13.10—Legacy Rails — Thrie Beam Terminal Layout

13.5—PEDESTRIAN BARRIER RAILS

End treatments for pedestrian barriers shall be coordinated with Roadway Division.

Pedestrian Barriers on Legacy Rails is allowed only as in-kind replacement or on existing bridges where the use of 42 in. tall rail adjacent to the sidewalk would result in different sized rail on each side of the bridge. A Base Sheet is available for the Pedestrian Barrier Rail.

See §§ 2.10 and C13.1.1.1.2 for sidewalk barrier recommendations.

13.6—FENCE

13.6.1—Pedestrian Fence

Pedestrian fences are required adjacent to sidewalks or trails on bridges.

Provide one of the following for bridges that are over roadway traffic:

- 8 ft. 3 in. high, curved chain-link fence
- minimum 10 ft. high straight, anti-climb mesh

This is to provide protection from falling debris for the traffic below.

Provide a 6 ft. high straight, chain-link fence or anti-climb mesh for bridges that are not over roadway traffic or railroad.

Some municipalities have their own local fence rules that may need to be coordinated with these rules.

13.6.2—Railroad Protection Fence

Railroad protection fences are required within the limits of the Railroad Right-of-Way or a minimum of 25 ft. beyond the centerline of the outermost existing track, future track, or access road, whichever is greater.

Railroad protection fences shall be provided on both sides of structures over railroads.

For edges of structures without pedestrian access provide a minimum 10 ft. high straight chainlink or anti-climb mesh fence.For edges of structures with a sidewalk or trail provide one of the following:

- 8 ft. 3 in. high, curved chain-link fence
- minimum 10 ft. high straight, anti-climb mesh

13.6.3—Fence Layout

All fence layouts should be located on the bridge using dimensions from the end(s) of the bridge floor at the centerline of the fence. The maximum post spacing shall be 8 ft. for all posts. For fence types that require bracing, the first bay of each end shall be 8 ft. and the second bay shall vary to accommodate any odd lengths required.

All fence heights are measured from the top of the sidewalk.

13.6.4—Expansion Joints

Expansion joints shall be provided in the all longitudinal members and handrail at all bridge expansion locations in the bridge deck and at end of floor (where present). Typical fence bracing should be provided on both sides of expansion joints.

Expansion gaps should be designed for a fixed dimension (i.e., no adjustments for temperature at time of installation). Expansion gaps greater than 2 in. should use a longer pipe on the inside of the joint.

13.6.5—Base Sheets

Base sheets are currently available for:

- 8 ft. 3 in. high curved pedestrian fence (chain link type) which has 6 ft. high sections at the ends.
- 10 ft. 6 in. high railroad protection fence

The following minimum information shall be populated into the base sheets:

- The clear dimension of the sidewalk should be indicated on the "Typical Section Thru Fence."
- "Typical Section Thru Fence" will indicate the proper traffic barrier used on the bridge project.
- The fence layout will indicate the odd post spacing and number of 8 ft. spaces.
- The expansion gap distance should be shown in the "Expansion Joint / Splice Detail." This information should be placed with the note for intermediate splice dimensions.

Designers should note that the minimum clear dimension of the sidewalk is on the approach slab when using the 39 in. SSCR, see Figure 13.11.

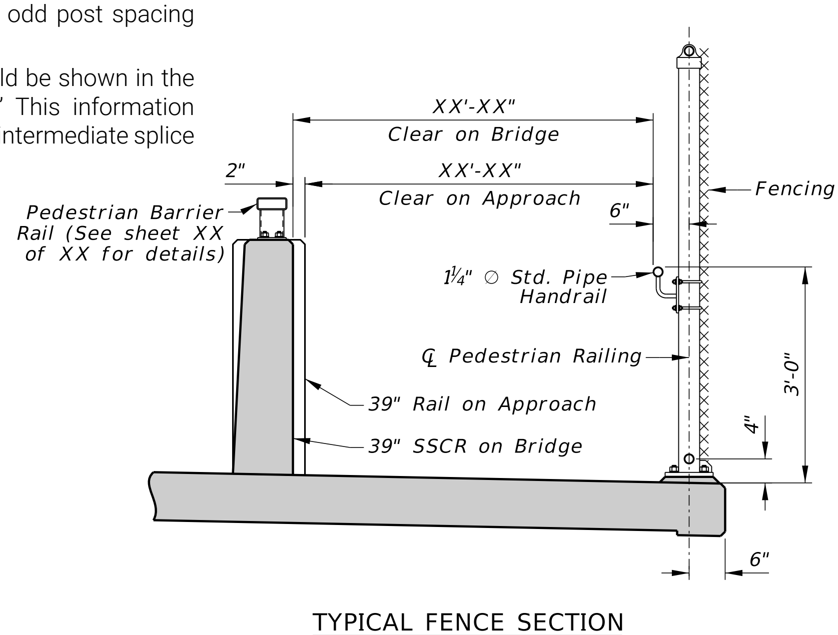

Figure 13.11—39 in. SSCR Sidewalk Clear Width

13.7—REFERENCES

DeLone, J. A., Faller, R. K., Rasmussen, J. D., Rosenbaugh, S. K., & Bielenberg, R. W. (2023). *Development of a MASH Test Level 4 Open Concrete Bridge Rail* (Technical Report TRP-03-406a-23). Midwest Roadside Safety Facility (MwRSF) Nebraska Transportation Center University of Nebraska-Lincoln, Iowa Department of Transportation, Kansas Department of Transportation, Nebraska Department of Transportation, South Dakota Department of Transportation, & Virginia Department of Transportation. https://mwrsf.unl.edu/researchhub/files/Report485/TRP-03-406a-23.pdf

Polivka, K. A., Faller, R. K., Holloway, J. C., Rohde, J. R., & Sicking, D. L. (2005). *Development, Testing, and Evaluation of NDOR's TL-5 Aesthetic Open Concrete Bridge Rail* (Technical Report TRP-03-148-05). Midwest Roadside Safety Facility (MwRSF) Nebraska Transportation Center University of Nebraska-Lincoln & Nebraska Department of Roads. https://mwrsf.unl.edu/researchhub/files/Report82/TRP-03-148-05.pdf

Rosenbaugh, S. K., DeLone, J. A., Faller, R. K., & Bielenberg, R. W. (2020a). *Development and Testing of a Bridge Rail for Low-Volume Roads* (Technical Report TRP-03-407-20). Midwest Roadside Safety Facility (MwRSF) Nebraska Transportation Center University of Nebraska-Lincoln. https://mwrsf.unl.edu/researchhub/files/Report339/TRP-03-407-20.pdf

Rosenbaugh, S. K., Faller, R. K., Asselin, N., & Hartwell, J. A. (2020b). *Development of a Standardized Buttress for Approach Guardrail Transitions* (Technical Report TRP-03-369-20). Midwest Roadside Safety Facility (MwRSF) Nebraska Transportation Center University of Nebraska-Lincoln. https://mwrsf.unl.edu/researchhub/files/Report415/TRP-03-369-20.pdf

Rosenbaugh, S. K., Faller, R. K., Dixon, J., Loken, A., Rasmussen, J. D., & Flores, J. (2021). *Development and Testing of an Optimized MASH TL-4 Bridge Rail* (Technical Report TRP-03-415-21). Midwest Roadside Safety Facility (MwRSF) Nebraska Transportation Center University of Nebraska-Lincoln. https://mwrsf.unl.edu/researchhub/files/Report433/TRP-03-415-21.pdf

13.8—REFERENCED CELLS

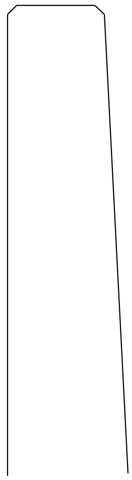

Figure 13.12— Rail Sec 39 SSCL Cell (Barriers and Rails Library)

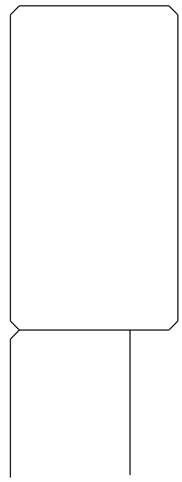

Figure 13.13—Rail Sec 39 OCBR Cell (Barriers and Rails Library)

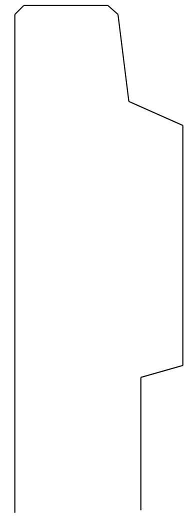

Figure 13.14—Rail Sec 42 NUCL Cell (Barriers and Rails Library)

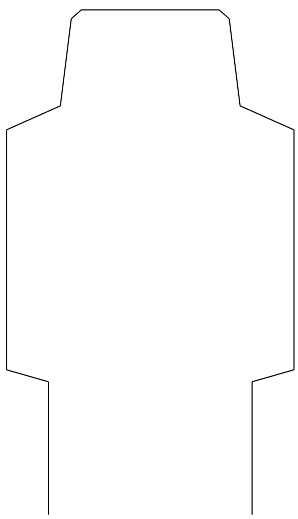

Figure 13.15—Rail Sec 42 NUMD Cell (Barriers and Rails Library)

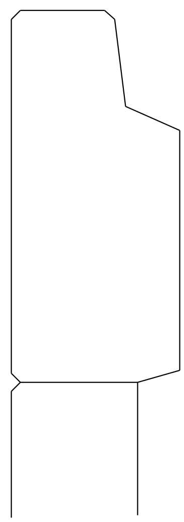

Figure 13.16—Rail Sec 42 NUOP Cell (Barriers and Rails Library)

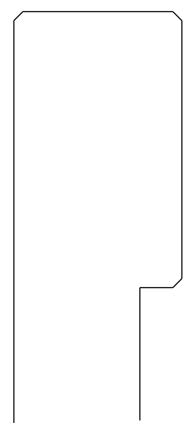

Figure 13.17—Rail Sec 34 NUCL Cell (Barriers and Rails Library)

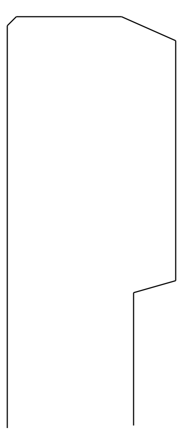

Figure 13.18—Rail Sec 34 NUCL Alt Cell (Barriers and Rails Library)

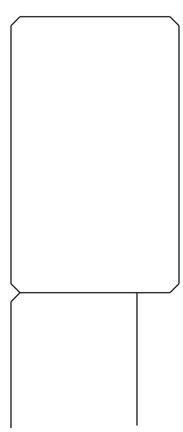

Figure 13.19—Rail Sec 34 NUOP Cell (Barriers and Rails Library)

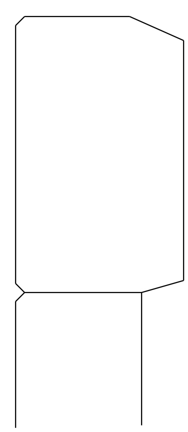

Figure 13.20—Rail Sec 34 NUOP Alt Cell (Barriers and Rails Library)

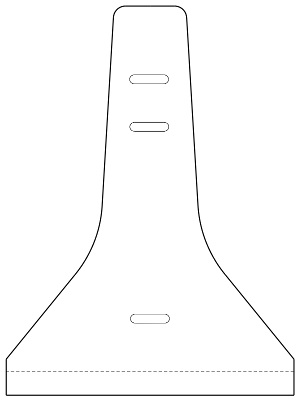

Figure 13.21—WP Barrier Sec Cell (Barriers and Rails Library)

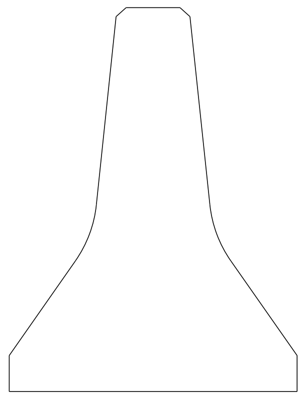

Figure 13.22—Temp Barrier Sec Cell (Barriers and Rails Library)

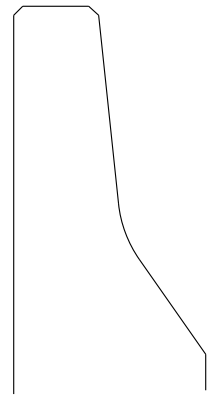

Figure 13.23—Ex Barrier Sec 32 NJ Cell (Barriers and Rails Library)

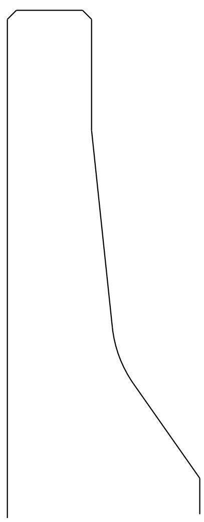

Figure 13.24—Ex Barrier Sec 42 NJ Cell (Barriers and Rails Library)

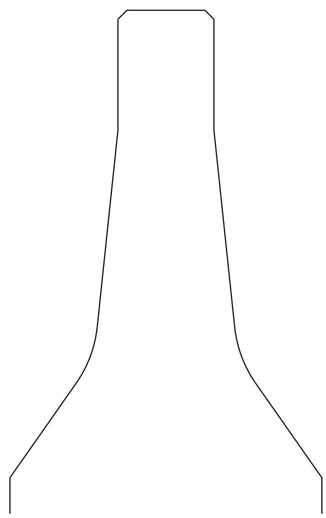

Figure 13.25—Ex Barrier Sec 42 NJMD Cell (Barriers and Rails Library)

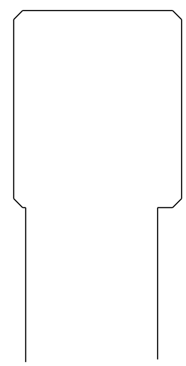

Figure 13.26—Ex Rail Sec 29 NEBCL Cell (Barriers and Rails Library)

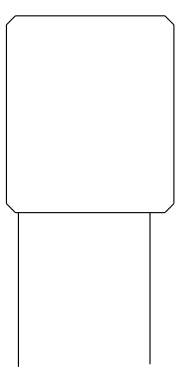

Figure 13.27—Ex Rail Sec 29 NEBOP Cell (Barriers and Rails Library)

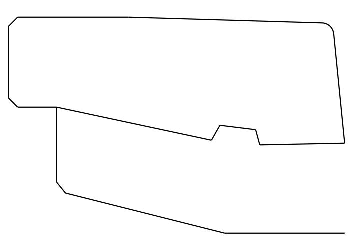

Figure 13.28—Ex Curb Sec 10.5 Cell (Barriers and Rails Library)

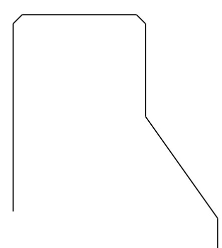

Figure 13.29—Ex Curb Sec 20 Cell (Barriers and Rails Library)

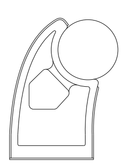

Figure 13.30—Ex Rail Sec 1755C Cell (Barriers and Rails Library)

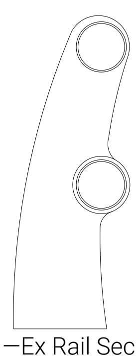

Figure 13.31—Ex Rail Sec 1753C Cell (Barriers and Rails Library)

Last Updated: January 13, 2025

Appendix C — Plan Assembly & Drafting

C.1—GENERAL PLAN INFO

Reserved for future use.

C.2—TITLE BLOCKS

Reserved for future use.

C.3—PLAN SHEET REQUIREMENTS

Reserved for future use.

C.4—PLAN REVISIONS

Reserved for future use.

C.5—STANDARD NOTES

The standard notes listed in this section are up-to-date at the time of publication. For the most up-to-date standard notes, see "NDOT Production" ProjectWise Datasource. It is the designer's responsibility to incorporate the most up-to-date notes in their designs.

A cell was created for each note subsection in the cell libraries named "OBD Front Sheet Notes" and "OBD Geology." A pdf of the cell library for the notes can also be found under Standard Plans\Bridge\OpenBridge Documentation.

C.5.1—Front Sheet Notes

C.5.1.1—General

001 This structure is designed in accordance with the AASHTO LRFD Bridge Design Specifications, XXXXX Edition, including subsequent interim revisions.

002 The Contractor may substitute any one of the alternate designs shown on the plans for the original design. All quantities are based on the original design and no additions or deductions will be allowed for the use of an alternate design.

003 The concrete bridge deck is designed by the empirical design method.

004 The superstructure and substructure are designed for a future wearing surface of three inches of asphalt, 35 psf.

005 The superstructure is designed for the allowance of stay-in-place forms (5 psf) between girders.

006 All dimensions shown are in horizontal plane only. No allowances have been made for vertical curve or roadway cross slope.

007 Unless noted "NOT TO SCALE", all details are drawn using a constant scale in accordance with NDOT Bridge Scaling Policy.

008 Girder shims that will be provided to the Contractor account for the dead load deflection due to weight of the slab, rail/barrier, wearing surface (if present), and median (if present) only. The Contractor is responsible for making the necessary adjustments for the specific forming system used to achieve the slab grades and elevations shown on the plans.

009 The shim shots may be taken before or after the turndowns and diaphragms are poured.

010 The finishing machine shall be supported by the girders of the phase that is being poured. When closure pours are required, the bridge finishing machine shall be supported on the completed Phase I and Phase II slabs. Intermediate cross frames and diaphragms under closure pours shall be left out, or bolts left loose, until after the Phase II pour.

C.5.1.2—Excavation

020 The Pay Item, "EXCAVATION (ESTABLISHED QUANTITY)", shall include the channel excavation/fill through the bridge as shown on the plans.

021 The Pay Item, "EARTHWORK MEASURED IN EMBANKMENT", shall include the channel excavation/fill through the bridge as shown on the plans.

022 Any excavation required for Riprap below the new channel cross section shall be subsidiary to the respective Riprap Pay Item.

C.5.1.3—Substructure

030 All structural steel shall conform to the requirements of ASTM A709, Grade 36.

031 The Pay Item, "STRUCTURAL STEEL FOR SUBSTRUCTURE", shall include any of the following applicable items: tie rods, turnbuckles, sheet pile bent plates, nose armor angles, steel channel framing at integral Abutments.

032 Tie Rods shall conform to ASTM A709, Grade 36 steel. Turnbuckles shall conform to ASTM A668, Class C.

033 After fabrication, all structural steel for substructure, except steel channel framing at integral Abutments, shall be galvanized according to ASTM A123.

C.5.1.4—Concrete & Reinforcement

040 Concrete for slab, approach slabs, and rails/barriers shall be Class "47BD", with a 28-day strength of 4,000 psi.

041 Concrete for slab, approach slabs, diaphragms, turndowns, and rails/barriers shall be Class "47BD", with a 28-day strength of 4,000 psi.

042 All other cast-in-place concrete shall be Class "47B" concrete, with a 28-day strength of 3,000 psi.

043 Chamfer all exposed edges of concrete.

044 Unless noted as "Optional", all construction joints shown are mandatory.

045 All reinforcing steel shall be epoxy coated and conform to the requirements of ASTM A615, Grade 60 steel.

046 The minimum clearance, measured from the face of the concrete to the surface of any reinforcing bar, shall be 3" for substructure elements and 2" for superstructure elements, except where otherwise noted.

047 Field bend and/or clip reinforcing bars to maintain minimum clearance. Epoxy coat clipped ends.

048 Contractor shall maintain a minimum placement and finishing rate of at least XXX feet/hour along CL Bridge. Should the Contractor not possess the necessary equipment and facilities to maintain the minimum placement and finishing rate, the slab shall be placed in sections. All Positive moment sections of the slab shall be placed followed by placement of the Negative moment sections of the slab. Alternate procedures for placing deck concrete may be submitted for approval by the Contractor.

049 All Positive moment sections of the slab shall be placed followed by placement of the Negative moment sections of the slab.

C.5.1.5—Prestressed Girders

060 The prestressed girders have been designed assuming 100% continuity at the interior supports for live load.

061 Prestressed concrete girders must be at least 9 days old before they can be set on the Bridge substructure.

062 Surveying for shim shots, forming the bridge deck, turndowns, diaphragms, and placing construction material on the girders is not allowed until the girders have reached design strength and are at least 30 days old.

063 All girder lines and spans, between expansion joints, shall be set before the shims are calculated.

064 Shim shots are valid for 60 days. If the deck is not placed within 60 days, shim shots must be retaken, shims may be adjusted, and all costs shall be subsidiary to the Pay Item "CLASS 47BD-4000 CONCRETE FOR BRIDGE".

065 The Contractor must provide any temporary intermediate diaphragms and/or bracing necessary to provide lateral and torsional stability for the girders during construction of the concrete slab.

066 The temporary intermediate diaphragms/bracing shall be removed after the concrete slab has attained 75% of its design strength. The cost for furnishing, installing, and removing the temporary intermediate diaphragms and/or bracing shall be subsidiary to the Pay Item, "CLASS 47BD-4000 CONCRETE FOR BRIDGE".

C.5.1.6—Steel Girders

080 All structural steel for rolled beams, stiffeners, separators, and all splice material shall conform to the requirements of ASTM A709, Grade 50W weathering steel.

081 All structural steel for girder flanges, webs, stiffeners, separators, and all splice material shall conform to the requirements of ASTM A709, Grade 50W weathering steel.

082 All structural steel for girder flanges, webs and splice material marked “HPS” shall conform to the requirements of ASTM A709, Grade 70W High Performance Steel. All other structural steel shall conform to the requirements of ASTM A709, Grade 50W.

083 Nuts, bolts, and washers used in the assembly of weathering steel shall be Type 3.

084 All bearing stiffeners and girder ends, except at field splices, shall be vertical after final erection. All other stiffeners and all field splices shall be normal to the top flange.

085 During girder fabrication, the flanges at the splice must line up within ⅛" of parallel to the adjacent flanges without applying external force before the splice is drilled.

086 Where Charpy V-notch testing is required the impact energy requirements shall be determined using Temperature Zone 2.

087 All rolled beams shall be placed with mill camber upwards.

088 The Contractor may eliminate any bolted field splice and/or butt welded flange splice by extending the heavier of the two sections so connected. The Contractor shall make all necessary adjustments in bearings and bearing seat elevations caused by these changes. These changes and a revised blocking diagram shall be shown on the shop plans and will be subject to approval by the Engineer. No change in the contract price or quantities will be made for this change.

089 As an alternate design, all intermediate stiffener plates may be omitted if X" web plates are used in place of X" web plates shown.

090 Butt splices will be permitted for flange or web plates exceeding 60 feet in length. The locations of the splice shall be shown on the shop plans and will be subject to approval by the Engineer.

091 During girder fabrication, the final camber tolerance shall not exceed those in Table 5.3 of AWS D1.5. “S” is the length of girder between splices.

092 Where the entire slab is not expected to be placed in one day, the Contractor shall submit an alternate proposed slab pouring sequence to the Bridge Division at the preconstruction conference so that new camber and shims may be calculated.

093 All fasteners shall be ⅞" Φ high strength bolts, ASTM F3125 Gr. A325. Fasteners for flange splices shall be supplied with sufficient grip length to exclude threads from the shear planes. All other fasteners (web splices and separator connections) may be detailed with the threads included in the shear planes.

094 Field tack welding of form hangers or miscellaneous hardware to any part of the steel girder, except for shear connectors, shall be prohibited.

095 When assembling the girders, they shall be set according to the blocking diagram before any bolts are tightened to a snug-tight condition.

096 Field splices shall be clean and free of all foreign matter before field assembly. The plates shall be in full contact when the bolts are tightened to a snug-tight condition.

097 The Contractor must provide any temporary bracing required to support the girder web and flanges against all torsional or lateral forces until the structure is fully erected. The girders with cross-frames for this bridge are designed to resist torsional and lateral forces caused by temporary construction loads applied to the fully erected steel framework prior to achieving composite action with the deck.

098 Girders and cross-frames or diaphragms are to be detailed, fabricated and erected for a no load fit (NLF) condition.

099 Girders and cross-frames or diaphragms are to be detailed, fabricated and erected for a steel dead load fit (SDLF) condition.

C.5.1.7—Miscellaneous

110 All materials, equipment, tools, labor, and incidentals necessary to complete the work, not paid for directly, shall be considered subsidiary to other items for which payment is made.

111 Details, quantities, or information for all Group 9 items contained in these bridge plans are for use "BY OTHERS."

112 Membranes are paid for by the plan view area of covered deck and approaches.

113 For integral abutments: No form work, reinforcing steel, or construction loads shall be placed on the girders (concrete girders need to be 30 days old and have reached their design strength) until the abutment concrete has set for 72 hours or reached a minimum compressive strength of 2,000 psi.

114 See Roadway Plans and Special Provisions for details and information regarding optional Contractor's Access Bridge or Access Crossing.

C.5.1.8—Repair/Rehab

120 Before ordering any materials, the Contractor shall make a detailed field inspection of the structure verifying all dimensions and reporting to the Engineer any discrepancies between the field measurements and those shown on the plans.

121 All materials removed shall become the property of the Contractor and shall be removed from the project site.

122 The State does not guarantee that these repair plans or the As-built plans depict the actual site conditions and shall not be liable for any discrepancies.

123 Dimensions shown were obtained from the as-built Plans. The Engineer shall establish control points from the existing structure as needed.

124 The Contractor shall place a 1" deep saw cut at the limits of concrete removal to facilitate a clean, smooth line when breaking back existing concrete.

125 All existing concrete surfaces to be in contact with the new construction shall be thoroughly roughened and cleaned before placing any new concrete.

126 Existing unbroken concrete surfaces to be in contact with the new concrete shall be scarified to an amplitude of $\frac{1}{4}$".

127 Use surface saturated dry condition when placing new concrete against old concrete.

128 Damage to existing structures, consequent to the Contractors operations, shall be repaired at the Contractor's expense, under the direction of the Engineer.

129 Actual field conditions may require repair more or less than what is depicted in the plans. The final areas to be repaired shall be determined by the Engineer. The Bridge Office shall be notified when field conditions impede the implementation of these plans or vary significantly from what is shown.

C.5.1.9—Utilities

140 (Utility Company) shall furnish all PVC sleeves, conduit, inserts, and hardware required for the utility attachment to the bridge. All material to be installed by the Bridge Contractor shall be delivered to the bridge site by the utility company within 72 hours after notification from the Bridge Contractor. The contact person for (Utility Company) is XXXXX and can be reached at (XXX) XXX-XXXX.

141 The Contractor shall install the sleeves and inserts as shown on the plans. The Contractor shall install the conduit from the face of the abutment to the toe of the slopes under the approach slabs but will not be required to install the hangers and conduit between abutments. The installation by the Bridge Contractor will not be paid for directly but shall be subsidiary to the Pay Item, "CLASS 47BD-4000 CONCRETE FOR BRIDGE".

C.5.2—Geology Sheet Notes

C.5.2.1—General

150 All pile spacing is given at the bottom of concrete.

151 Piers/Bents are designed for scour to Elev. XXXX.XX for 100-Year Flood. Piers/Bents are checked for scour to Elev. XXXX.XX for 500-Year Flood.

152 Abutment piling followed by the letter "B" shall be battered at X:12.

153 Pier/Bent piling followed by the letter "B" shall be battered at X:12.

C.5.2.2—Borings

160 The borings, as logged on the plans, represent the character of the subsoil at the location indicated. No guarantee is made that the subsoil conditions vary uniformly between or outside the given location.

161 Figures beside the column of borings indicate the number of blows required to drive a standard penetrometer, of 2" O.D., the second and third 6 inches using a 140 lb. weight falling 30 inches, in accordance with ASTM D1586 procedures.

162 * indicates refusal, greater than 50 blows in 6 inches

C.5.2.3—Bearing Piles

170 Structural steel for all "H" piles shall conform to ASTM A709, Grade 50.

171 Prefabricated cast steel points will be required on all HP piles in this structure. They shall conform to the requirements of ASTM A27 Grade 70-36 or ASTM A148 Grade 90-60 and be listed on the NDOT Approved Products List.

172 Concrete piling shall be prestressed concrete piles, Type I.

173 All concrete for prestressed concrete bearing piles shall have a minimum 28-day compressive strength of 5,000 psi.

174 As alternate, cast-in-place concrete piles may be used, provided that the Contractor shall be responsible for furnishing piling of sufficient length to obtain the penetration and bearing value required by the Geotechnical Engineer.

175 All concrete for cast-in-place concrete piles shall be Class "47B" with a minimum 28-day compressive strength of 3,000 psi and 6" to 8" slump. The slump will be increased by adding plasticizer as required.

176 All exposed pipe piles shall be filled with concrete. This concrete shall be Class "47B" with a minimum 28-day compressive strength of 3,000 psi. This concrete shall be subsidiary to the Pay Item, "PIPE PILING".

C.5.2.4—Test Piles

190 Test piles shall be driven, as shown in the TEST PILE DATA table.

191 Pile order lengths, except for those shown for the test piles, are tentative. The final order lengths shall be based on the results obtained from the test pile driving.

192 The driving of the test pile will be monitored with a Pile Driving Analyzer. The use of the Pile Driving Analyzer will require the Contractor to set up the hammer for driving. The Contractor shall bolt two accelerometers and two strain transducers to the pile before driving is started.

193 The holes or anchors for the accelerometers and strain transducers will have been predrilled by Department personnel while the pile is still on the ground. The Contractor may be required to stop the hammer for wave speed determination after the first few blows.

194 The Contractor shall drive the pile until the transducers are near the surface of the ground, or as directed by the Engineer, at which time the Contractor shall stop the hammer for the removal of the accelerometers and strain transducers. The Contractor shall then continue driving the pile to cut-off or as directed by the Engineer.

195 The time delay in driving each pile being monitored by the Pile Driving Analyzer will normally range from 30 to 60 minutes.

196 The Contractor shall provide access to the pile driving area for the Engineer's equipment vehicle (light truck). The work performed by the Contractor, in conjunction with the use of the Pile Driving Analyzer, as described herein, shall not be paid for directly, but shall be considered subsidiary to items for which direct payment is made.

197 Final order lengths will be provided by the Engineer to the Contractor within three working days after the test pile driving is complete.

C.5.2.5—MSE Wall Piling

210 Abutment and grade beam piling may be driven before or after the construction of the MSE walls.

211 If the piling are to be driven before the construction of the MSE wall, the Contractor shall place Corrugated Metal Pipe (CMP) sleeves around each piling prior to constructing the wall.

212 If the piling are to be driven after the construction of the MSE wall, the Contractor shall place CMP sleeves at the exact location of each pile. This ensures that after the completion of the MSE wall, the Contractor can drive the piles through the sleeves.

213 The CMP sleeves shall be maintained in a plumb position during construction of the MSE wall. Furnishing and placing of the CMP sleeves shall be included as part of the work for the MSE wall.

214 After all piling for the abutment and grade beam are driven and the MSE wall is complete, the Contractor shall fill the space between the piling and the CMP sleeves with dry, clean sand. Backfilling with sand shall be considered subsidiary to the respective piling Pay Item.

C.5.2.6—Integral Abutments

230 All abutment piles, excluding wing pile, shall be started in holes predrilled to Elev. XXXX.XX. The minimum diameter of the holes for the pile shall be XX inches.

231 Piles shall be placed in the drilled holes, driven to design bearing and the void between the hole wall and the pile shall be backfilled with dry, clean sand. Predrilled holes shall not be backfilled until all abutment and wing piles are driven. Drilling, disposal of removed soil, and providing and backfilling with sand will be considered subsidiary to payment for steel piling.

C.6—PAY ITEMS

Reserved for future use.

C.7—PLAN SHEET SCALING

Bridge Plan details should be drawn at full size in OBM design models, and then referenced into the standard NDOT Bridge border (ARCH D, 24 in. × 36 in.) at an appropriate, proportional scale factor. CADD software includes a number of appropriate scales to use. Suggested scale factors are provided in Table C.1.

Table C.1—Common Bridge Detail Scales

Architectural		Engineering	
Detail Scale	Master : Ref	Detail Scale	Master : Ref
Full Size	1 : 1	1" = 5'-0"	1 : 60
6" = 1'-0"	1 : 2	1" = 10'-0"	1 : 120
3" = 1'-0"	1 : 4	1" = 20'-0"	1 : 240
1 1/2" = 1'-0"	1 : 8	1" = 30'-0"	1 : 360
1" = 1'-0"	1 : 12	1" = 40'-0"	1 : 480
3/4" = 1'-0"	1 : 16	1" = 50'-0"	1 : 600
1/2" = 1'-0"	1 : 24	1" = 60'-0"	1 : 720
3/8" = 1'-0"	1 : 32	1" = 100'-0"	1 : 1200
1/4" = 1'-0"	1 : 48	1" = 200'-0"	1 : 2400
3/16" = 1'-0"	1 : 64		
1/8" = 1'-0"	1 : 96		
1/16" = 1'-0"	1 : 192		
1/32" = 1'-0"	1 : 384		

Designers are not limited to these factors and may choose custom scale factors as appropriate. Multiple details can be placed within a border using a scale that best suits the particular detail.

Bridge Standard Note #007 shall be placed on all plans.

C.7.1—Proportional Bridge Detail

For details scaled into the border in this manner, it will not be necessary to address scale below the title. In the Figure C.1, the detail was placed on the plan sheet at a scale of 1 1/2" =1'-0" (it has been reduced in size to fit in this manual).

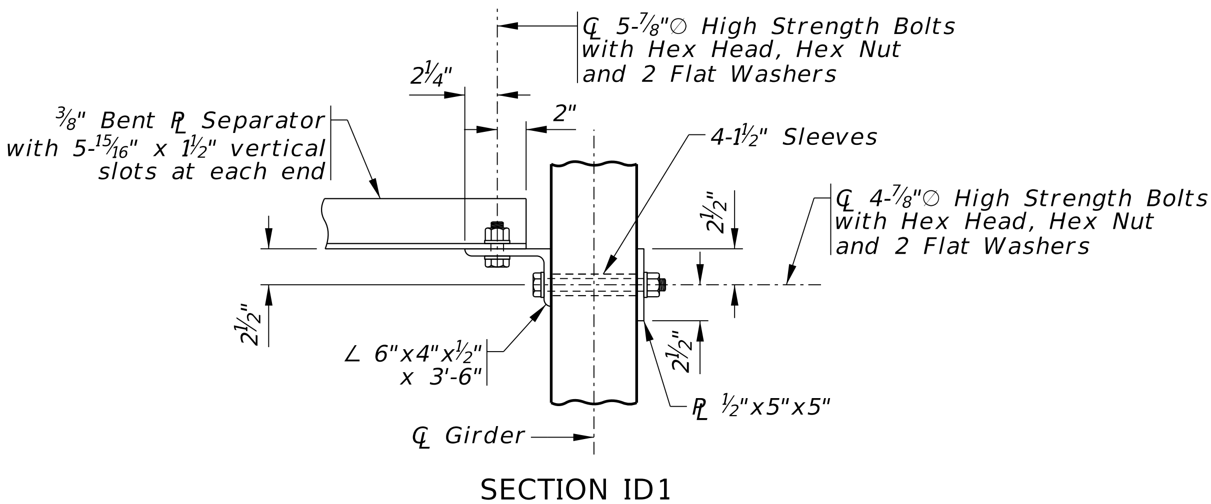

Figure C.1—Example of Proportional Detail

C.7.2—Non-Proportional Bridge Detail

Only in special cases should the term Not to Scale be placed below the title of the detail. These situations include standard drawings that are meant to apply to a large number of bridge plans, or cases where it is appropriate to exaggerate either the X or Y scale of a drawing for clarity. In Figure C.2, the detail was drawn with scale two times larger in the X direction than it has in the Y direction.

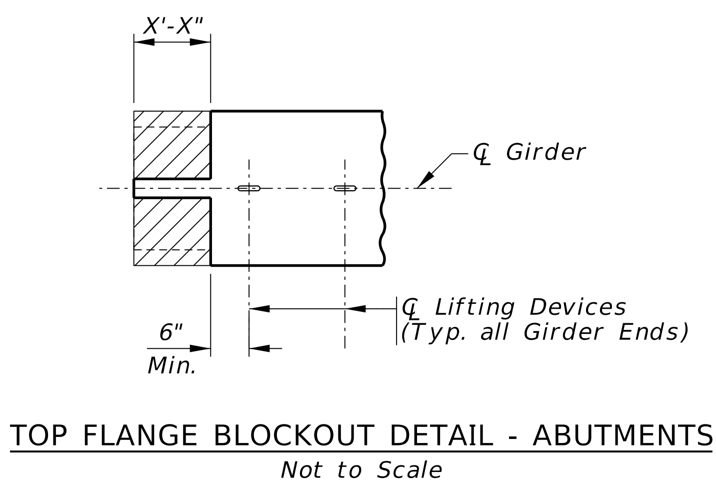

Figure C.2—Example of Non-Proportional Detail

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NEBRASKA

Good Life. Great Journey.

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