



Bridge Design Manual



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POLICIES AND PROCEDURES

A. GENERAL POLICY

The Colorado Department of Transportation (CDOT) Bridge Design Manual (BDM) provides the policies and procedures currently in effect for the design, rehabilitation, repair of bridges/structures and for projects that use federal or state funds. This BDM is also recommended as best practice for any project that does not contain federal or state funds and other structures that are within CDOT right-of-way (ROW). This BDM presents the minimum requirements for structure projects including the structural staff, submittals, design and construction specifications, and project processes. The BDM shall be applied to structures that require special design (i.e., modified from the CDOT M & S Standards) with the exceptions noted in Part I Exceptions, Number 4.

The latest edition of *AASHTO LRFD Bridge Design Specifications* (AASHTO) with current interim revisions is the primary document guiding the design of highway structures. Other specifications may be required for structural design but only as referenced by this BDM or by AASHTO. This CDOT BDM supplements AASHTO, as well as other applicable AASHTO documents, by providing additional direction, clarification, and requirements. Where discrepancies arise between this BDM and applicable current AASHTO specifications, this BDM will control. The State Bridge Engineer (defined in Part D) shall resolve conflicting information between standards referenced herein or any other CDOT document.

All AASHTO specifications and codes and BDM revisions shall apply to any future design projects when they are officially issued. All projects should be evaluated to ascertain the effects of using the new requirements immediately for safety, design capacity, performance, schedule, cost, contractual and other implications. If implications are minimal, design projects that are in preliminary design stages should use the latest requirements. If implications are more substantial, Shelf projects, post-FIR projects, and those projects with contractual limitations may choose to continue with the previously issued LRFD requirements. Safety revisions or standards revisions, such as crash tested bridge rail, may be required to add at any stage of the project.

Using this BDM does not relieve the Engineer of their responsibility to provide high-quality deliverables or to exercise sound engineering judgment. The Engineer is to verify all figures. Figures are shown as examples only, but the design responsibility is that of the Engineer. Staff Bridge will consider variances from the policies presented in this BDM when warranted. If different interpretations of a given article arise, guidance shall be obtained from Staff Bridge. Unless otherwise specified, the Unit Leader in coordination with the Staff Bridge Senior Design & Construction Engineer must authorize any additional modifications and variances to the BDM. Variance request examples can be found on the Bridge website.

Thorough knowledge of the contents of this BDM is essential for anyone designing structures that meet the above defined criteria.

Previous editions of the BDM and Bridge Design Technical Memorandums are now void.

B. BRIDGE DESIGN MANUAL DISTRIBUTION AND MAINTENANCE

Copies of the CDOT BDM can be obtained from the CDOT website

The Office of the Staff Bridge Branch maintains the computer files containing this BDM, coordinates revisions, and makes updates available. The Staff Bridge Branch also maintains a revision log showing all the revision dates that have transpired for each section and the person who wrote the revision.

Before starting a structural design project, the Engineer shall obtain a copy of this BDM or if the Engineer already has a manual, they shall inspect the current table of contents to make certain their copy of the BDM is up to date.

C. REVISIONS

This BDM is intended to be dynamic. Revisions will be incorporated as new material is added and as criteria and specifications change. The State Bridge Engineer shall approve and publish all revisions.

Suggestions for improving and updating this BDM are encouraged. Anyone who would like to propose revisions should informally discuss changes with other Bridge Engineers to further develop and refine ideas. All suggestions shall be submitted to the Staff Bridge Manager of Policy and Standards, who then will present the State Bridge Engineer with a preliminary draft showing the developed concept.

On deciding to pursue the revisions, the State Bridge Engineer will assign them to an Engineer. The Engineer receiving the assignment is responsible for completing the final writing, distributing the revisions to all Staff Bridge personnel for their review and comment, making revisions as appropriate based on the comments received, and submitting the final draft to the State Bridge Engineer for approval.

When a revision is made, the entire section containing the revision will be reissued. The revision date is provided in the lower right corner of the page. Whenever revisions are issued, they shall be accompanied by a cover document signed by the State Bridge Engineer.

D. DEFINITIONS

Staff Bridge Managed Structural Assets: Structures managed and assigned a structure number or structure ID.

All managed assets (bulleted items below) within CDOT ROW require a structure number. Outside of CDOT ROW, only vehicular bridges longer than 20' and tunnels require a structure number. A structure number does not denote CDOT ownership or maintenance responsibilities, only assets that CDOT tracks within our ROW.

Refer to Part E2 of this Section for information about structure number assignment.

- **Major Structures:** Bridges and culverts carrying vehicular traffic with a total length greater than 20 ft. measured along the centerline of the roadway between the inside face of abutments, inside faces of the outermost walls of culverts, or spring lines of arches. Major Structures also include culverts with multiple pipes where the clear distance between the centerlines of the exterior pipes, plus the radius of each of the exterior pipes, is greater than 20 ft.
- **Minor Structures:** Minor Structures are bridges, culverts, or a group of culverts carrying vehicular traffic that have a total length greater than or equal to 4 ft. and less than or equal to 20 ft. measured along the centerline of the roadway between the inside face of abutments, inside faces of the outermost walls of culverts, or spring lines of arches.
- **Walls:** Retaining Walls, Noise Walls, and Landscape Walls as defined below. Refer to the *CDOT Retaining and Noise Wall Inspection and Asset Management Manual* for more detailed information.
 - **Retaining Walls:** Any wall which supports or protects a CDOT facility or asset, or any wall retaining backfill measuring at least 4 ft. in height from the finished grade to the top of the wall at any point along the length of the wall.
 - **Noise Walls:** Noise Walls with structural elements including walls that do not typically retain soil. Refer to the *CDOT Retaining and Noise Wall Inspection and Asset Management Manual* if a Noise Wall retains backfill.
 - **Landscape Walls:** Walls retaining backfill measuring less than 4 ft. in height from the finished grade to the top of the wall at any point along the length of the wall.

Reinforced Slopes with no visible/inspectable facing elements are tracked under the Geohazard Program and would receive a tracking number from that group. Rock bolting although visible would also fall under the geohazard program.

- **Tunnels:** An enclosed roadway for motor vehicular traffic with vehicle access limited to portals, regardless of type of structure or method of construction. Tunnels are structures that may include lighting, ventilation, fire protection systems, and emergency egress capacity.
- **Ancillary Structures:** Ancillary structures are overhead signs, mast arm signals, and high-mast lights (height greater than 55 ft.). Cattle and deer guards are included in this category.
- **Other Structures:** A non-vehicular structure in CDOT ROW that does not fit into any of the aforementioned categories. Examples include overhead pipes, overhead cables, railroad bridges, pedestrian/bike

structures, private drive structures, overhead conveyor belts, and overhead snow sheds.

On-System / Off-System Bridges: Any bridge owned by a local agency (cities and counties) is considered Off-System with some exceptions. Any bridge owned by CDOT or Colorado Bridge Enterprise (CBE) is considered On-System. A more specific definition can be found in *Colorado Off-System Bridge Program Description and Guidelines for Selecting Bridges for Rehabilitation or Replacement Funding*.

Design Life / Service Life: The design life is the period for which a component, element, or bridge is expected to function for its designated purpose when designed, constructed, and maintained as per standards. The service life is the period for which a component, element, or bridge provides the desired function and remains in service with appropriate preservation activities. This may also be called as useful life.

State Bridge Engineer: Chief Structural Engineer for the Staff Bridge Branch of the Colorado Department of Transportation. The State Bridge Engineer is responsible for structures within CDOT ROW and federally funded off-system projects and manages CDOT's Bridge Program, which includes Major Structures, Minor Structures, Tunnels, Walls, and other highway structures, including all ancillary and miscellaneous structures on the state highway system and federally or state funded off-system projects.

Staff Bridge Manager of Policy and Standards: A CDOT Staff Bridge employee who reports to the State Bridge Engineer and manages the implementation of CDOT Bridge Design Policy and Standards used for the design of transportation structures (standards include this CDOT BDM and the documents defined in Part F). Staff Bridge Manager of Policy and Standards ensures that the Department's policy is clearly communicated, is readily referenced, and benefits the mission of the Department.

Staff Bridge Unit Leader: A CDOT Staff Bridge employee who reports to the State Bridge Engineer and manages the bridges and highway structures located in a geographical CDOT Transportation Region. Refer to the CDOT website for Region jurisdictions. Staff Bridge Unit Leader is also mentioned simply as Unit Leader in this manual.

Project Structural Engineer: A licensed professional engineer (by the State of Colorado), with structural design experience, acting in responsible charge of structural design work. Other than the sealing of plans and specifications, the activities described in this BDM pertaining to the Project Structural Engineer may be executed by a designee. There may be more than one Project Structural Engineer on a project as in the case where there is more than one structural design team working on separate Major Structures or for Design-Build where the Contractor will have a Project Structural Engineer for the Contractor's portion of the structural design work. For some Retaining Walls with significant geotechnical design issues, such as soil nail walls, tieback walls, and slurry walls, the Project Structural Engineer may be a Geotechnical Engineer.

Structural Design Engineer: A design engineer responsible for generating design calculations, construction plans, specifications, and reports. This person can be the Project Structural Engineer. The Structural Design Engineer may be referred to as the Designer or Engineer in this BDM.

Independent Design Engineer: A design engineer who develops an independent set of calculations based on the construction plans and specifications completed by the Structural Design Engineer. This includes vendor provided structural products signed and sealed by a Colorado Licensed Professional Engineer. This is a quality control task that is described in more detail in BDM Section 37.

Independent Technical Reviewer: A highly experienced engineer independent of the project team who conducts an independent technical review of the project deliverables focusing on general conformance with standard practice, AASHTO, and this BDM. This review does not involve development of detailed calculations. The review should consider other aspects of construction, such as interdisciplinary coordination, constructability, and biddability. The independent technical review is also known as an independent design review or a technical peer review. This is a quality control task that is described in more detail in BDM Section 37.

Constructability Reviewer: A construction engineer or licensed professional engineer with significant construction experience who reviews the project deliverables focusing on constructability and inspectability. This is a quality control task that is described in more detail in BDM Section 37.

CDOT Structural Reviewer: A CDOT employee with a professional engineer's license and structural design experience. This employee conducts the Department's structural design reviews on a Consultant project. The Structural Reviewer may delegate this task to a non-licensed engineer. This is a quality assurance task that is described in more detail in BDM Section 37.

Program Engineer: The immediate supervisor of the Resident Engineer.

Resident Engineer: The CDOT employee who is responsible for the administration of a project. The Resident Engineer, or their designee, can either be the preconstruction Project Manager or the construction Project Engineer, or both.

Project Engineer: As defined in CDOT's Standard Specifications for Road and Bridge Construction, the CDOT Chief Engineer's authorized representative who is responsible for the administration and satisfactory completion of a given construction contract.

Local Agency Project: Federally funded off-system transportation project executed by a public agency, local public agency, established publicly owned organization, or private interest that can legally enter into an agreement with CDOT.

Developer Project: A construction project within CDOT ROW sponsored and funded by either a private or a public entity other than Federal/State funds.

Stamped Documents & Disclaimers: Stamped documents required as part of this design manual are governed by applicable CDOT policies and State Law. Disclaimers accompanying any Professional Engineer's Stamp shall not limit CDOT's use of documents procured through CDOT contracts or for projects within or impacting CDOT assets or Right of Way. See Appendix B at the end of this Section for more information on what is required to be stamped.

E. STRUCTURES PROCESS

Design of structures involves compliance with the minimum requirements outlined in this BDM, as well as coordination with disciplines including, but not limited to, Survey, Right-of-Way, Utilities, Roadway Design, Traffic, Hydraulics, Geotechnical, and Environmental. The structures design process outlined in Appendix A of this Policies and Procedures section presents a diagram for the overall structure design and a more detailed breakdown of coordination with hydraulic design. For simplicity, the process diagram may not specifically address each aforementioned discipline; therefore, it is important to coordinate with each discipline throughout the entire project. Process diagrams for rehabilitation projects and overlays are found in Section 33. Projects involving railroad agencies will require additional submittals and longer review time.

Note that all CDOT projects and Local Agency projects with CDOT oversight are required to use CDOT ProjectWise® for storing all project files. Files shall be placed in CDOT ProjectWise within 2 weeks of any meeting, milestone or deliverable date. Files shall be in accordance with Section 6 Archiving.

1. Project Scoping for Major Structures, Walls, and Tunnels

Scoping: The Program Engineer and Resident Engineer will determine when to involve structural engineering staff in project scoping. To prevent later changes to the project scope, the Staff Bridge Branch should be involved in any scoping related to Major Structures, walls, and tunnels. When the project involves existing structures, the information available from Staff Bridge on these structures shall be used.

Project scoping should include a determination that a new structure is required or rehabilitation of an existing structure is feasible. This determination shall be confirmed during preliminary design.

On Consultant projects, CDOT's Structural Reviewer and the Consultant's Project Structural Engineer shall review the contract Scope of Work before signing the Consultant's contract. The structure activities in the Scope of Work shall be consistent with the requirements outlined in this BDM.

Schedule and Workhour Estimates: When preparing schedules and workhour estimates, the Resident Engineer shall obtain estimates for the structure tasks from the Project Structural Engineer concerning the level of work performed by Staff Bridge. The Resident Engineer shall obtain these work estimates from the CDOT Structural Reviewer on Consultant projects. The Resident Engineer will establish the final schedule and work hours; however, this decision is not to be made independent of information received from CDOT Staff Bridge.

Project Survey Request: The Project Structural Engineer shall participate in developing the project survey request to determine if any project-specific modifications to the basic information required by the CDOT Survey Manual are necessary.

2. Preliminary Design

The preliminary design for Major and Minor Structures, Pedestrian Structures, Walls, and Tunnels shall be conducted as outlined below to ensure that CDOT obtains a structure layout and type selection that achieves the project's objectives and minimizes revisions during the final design and construction phases. The Structure Selection Report presents the results of the preliminary design process. The report shall document, justify, and explain the Project Structural Engineer's structure layout and type selection. The Project Structural Engineer is responsible for ensuring that the following tasks are completed as appropriate:

a. Structure Number

All staff bridge managed structural assets must be assigned a structure number. Tunnels and Major Structures outside of CDOT ROW require structure numbers as well. If the owner does not provide a number, CDOT Bridge Asset Management Unit will assign a structure number. Tunnels, Major Structures, and other Ancillary or Miscellaneous structures are given a structure number based on the state grid system. Minor Structures and Walls are given a structure number based on highway and milepost. Structure numbers for mast arm signals are assigned based on milepost and quadrant. Any location changes for Walls (primarily the beginning) may require a new or a revised structure number. Locations should be finalized before obtaining a structure number, if possible, to minimize rework. Temporary structure numbers may be used before obtaining the final structure number, but plans issued for construction or advertisement shall use final structure numbers. For existing structures, the Project Structural Engineer shall obtain new structure number from Staff Bridge Asset Management Unit if not assigned before. For new structures, Project Structural Engineer or PM must provide structure information to CDOT Staff Bridge Asset Management Group as early as possible for the assignment of a structure number. This structure number shall be used on all subsequent correspondence and plan sheets to identify the structure. Structure numbers should be provided to the Project Engineer for inclusion into or updates of the SAP system.

The Bridge Asset Management Unit is responsible for assigning structure numbers and will make any decisions pertaining to structure numbers. To submit a request for a structure number, use the spreadsheet located on the CDOT website, Bridge Section, Forms and Form Letters. The following information, shown on the spreadsheet, is necessary before they can assign a number to a given structure:

(1) Geographical Location

- County, Highway Number (State) and Milepost (If Milepost is not known, Latitude and Longitude information may be substituted). For walls, Latitude and Longitude for the beginning (lowest mile point) of the wall is necessary.
- Feature Intersected (stream, river, highway, etc.), and facility carried.

(2) Project Number and Subaccount

(3) Existing Structure Number, if applicable

(4) Structure Information

- Type
- Material
- Bridge Typical Section
- How many spans
- Structure Length
- Width
- Designer
- Notes

b. Structure Data Collection

Obtain all data necessary for the layout and design of the structure, including, but not limited to, survey data, ROW restrictions, roadway geometry and safety criteria, utility information, hydraulic information, geotechnical recommendations, existing bridge data, accelerated bridge construction opportunities (include prefabricated items such as ACROW, InQuik, precast girders, etc.), life-cycle maintenance considerations, lighting/aesthetic requirements, and environmental clearance issues affecting the structure.

c. Foundation Investigation Request

Initiate the foundation investigation as early as practical by contacting CDOT Soils & Geotechnical Services and the Project Manager. An example format is provided in forms/form letters. On documents such as preliminary plans or aerial mapping, identify test holes with enough geometric information for the Geotechnical Engineer to locate the holes in the field. Consider the certainty of substructure and wall locations before initiating the request so that borings are located correctly and avoid additional drilling and changes to foundation recommendations. Consideration should also be given to locating borings in areas of suspected approach settlement and slope instability. See BDM Section 2.9 for more details.

d. Structure Layout and Type Selection

Compile all the site and structure data and design criteria to accomplish the following:

- Confirm that the scoping decision of constructing a new structure or rehabilitating an existing structure is still feasible
- Determine structure meets hydraulic requirements
- Determine structure type or rehabilitation type alternatives
- Evaluate layout alternatives
- Determine feasible foundation types
- Develop phase/stage construction methods
- Compute preliminary quantities and cost estimates per BDM Section 35
- Evaluate structure alternatives per criteria established in BDM Section 2
- Select the preferred alternative
- Prepare a general layout for the preferred alternative

e. Structure Selection Report

Prepare the Structure Selection Report to document and obtain approval for the preliminary structure design from the Unit Leader via the SSR QA Checklist. This report should summarize the site data and process used to select and lay out the structure. Structure Selection Reports are required for Major Structures, Minor Structures, Wall Structures, Pedestrian Structures, and Tunnels. The Structure Selection Report for a Minor Structure, Major or Minor Structure Widening, Landscape and Noise Wall, or Pedestrian Structure are typically shorter since less items affect their selection. See BDM Section 2.10 and Appendix 2A for more detailed requirements for developing the report, report contents, submission, and approval by Unit Leader.

The selection report should address any environmental concerns such as lead paint, hazardous materials, and safety concerns for designers or maintenance personnel.

When completed place the structure selection report in ProjectWise.

f. Field Inspection Review (FIR)

On obtaining initial approval, from the Unit Leader, for the structure type selection and layout, the Project Structural Engineer shall submit the general layout for inclusion in the FIR plans. After the FIR, the general layout shall be revised as needed for final detailing.

Final approval, should be obtained from the Resident Engineer for the revised general layout before proceeding with final design. The intent of this approval is to confirm design assumptions prior to final design to avoid costly re-designs at later design stages.

3. Final Design

The Project Structural Engineer shall ensure that the following tasks are completed after the FIR:

a. Revise Structure Selection Report as required

Submit a revised Structure Selection Report that incorporates comments received and accepted from the FIR submittal.

b. Perform Final Design Calculations

The Structural Design Engineer and Independent Design Engineer shall perform calculations for all structures not predesigned by M & S Standards supporting the contract documents in accordance with this BDM and noted standards. Design and independent check calculations should clearly state purpose, references, and assumptions. Ratings shall be completed during final design and checking.

c. Develop Construction Plans and Specifications

Develop Construction Plans in accordance with this BDM and the CDOT Bridge Detail Manual. The Project Structural Engineer is responsible for ensuring that all CDOT Staff Bridge Worksheets and other standards are the current version before including them in the plans.

Construction items not adequately covered by the CDOT Standard Specifications for Road and Bridge Construction and applicable CDOT Standard Special Provisions for Road and Bridge Construction will require Bridge Project Special Provisions. CDOT Standard Special Provisions and Project Special Provision Worksheets are available at CDOT's website.

The design plans and specifications for the Release for Construction submittal shall not name sole source or proprietary products unless approved by Unit Leader. Sole source or proprietary products should only be used for innovative products.

d. Final Office Review (FOR)

Complete structural plans, Standard Special Provisions, and Project Special Provisions shall be submitted for inclusion in the FOR submittal. The Project Structural Engineer shall attend the FOR meeting to obtain review comments on the structural design. After the FOR meeting, the Project Structural Engineer shall ensure the plans and specifications are revised as needed and submitted for inclusion in the final plan set.

4. Final Design Submittal

Final design submittal documents, including the Final Design Submittal Checklist found in Appendix B, shall be submitted and placed in ProjectWise prior to advertisement, or before construction for Design-Build or CM/GC projects, unless otherwise approved by the Unit Leader. All PDF documents shall be in conformance with ISO PDF/A-1b archival specifications as described in section E.6 below. The final design submittal shall include the following:

a. Construction Plans and Specifications

- (1) Plans shall be submitted in both PDF and native file format. For CDOT Projects, Microstation[®] files are required.
- (2) Specifications shall be submitted in both PDF and Microsoft Word[®] format.
- (3) Plans and specifications will be electronically sealed, per CDOT requirements but since this is a separate process, the record set will not be a structural final submittal requirement. The Project Engineer or Resident Engineer will send out the plan set and specifications for sealing for placement into CDOT ProjectWise.

b. Final Hydraulic, Geotechnical and Structure Selection Reports

The final Structure Selection Report shall include the signature of the CDOT Staff Bridge Unit Leader on in-house projects and by the CDOT Structural Reviewer on Consultant projects to indicate concurrence that conclusions in the report meet project goals and requirements.

Final Hydraulic and Geotechnical Reports must be signed and sealed by a Colorado Licensed Professional Engineer (CO PE sealed). Since sealing is a separate process, sealed reports will not be a structural final submittal requirement although a stamped version is preferred.

Geotechnical Report requirements can be found in BDM Section 2.9.2. Hydraulic (Drainage) Report requirements can be found in BDM Section 2.11.

c. Design Calculations and Independent Design Check Calculations.

Prepare design calculations and Independent Design Check Calculations for submittal. Calculations shall be:

- Clear and legible
- Organized (indexed) so individual calculations are easy to find
- Easy to follow, i.e. calculations should provide inputs (givens) and results
 - Summary of purpose of calculation - project, background, logistic, theme and basics of the calculations

- Well documented (i.e. specification references for equations & values, sketches, etc.)
- When software output is included, all inputs (sketches, etc.) should be provided as well.
- Include all native software files including input files that support the calculations. Spreadsheets, Mathcad files etc. are required only if it is in the project scope, i.e. developed for the project. (Example Program Management projects). Native files for priorly created files are not required.
- Two PDFs, one consisting of the Design Calculations and the other for Independent Design Check Calculations. Design Calculations and Independent Design Check sets shall each be sealed by a Registered Professional Engineer in the State of Colorado and shall be submitted with the Final Design submittal.

Rating calculations or analysis for phasing shall be provided as a separate document and will follow the requirements shown above. This document is to verify the color code for the existing bridge is still applicable during phased traffic loading.

Quantity calculations shall be calculated and independently checked based on the requirements in BDM Section 35. A record set of final quantity calculations for all pay items shown on the Summary of Quantities shall be provided as a separate document. This requirement does not apply to Design-Build Projects. See BDM Section 38 for more information about the differences for Design-Build projects.

- d. Load Rating Package developed in accordance with the CDOT Bridge Rating Manual.

Load rating package is required for all Major Structures and pedestrian structures greater than 20 ft span which carry maintenance or emergency vehicles. It must be CO PE sealed. Ratings for all vehicles, including Colorado Permit vehicle must be ≥ 1 for all new structures.

- e. Miscellaneous Information:

- (1) Verify a copy of the existing bridge plans is available when requested. Due to availability of electronic records, a “field information package” is no longer required unless requested.

When items are to be removed or widened (bridges, expansion joints, bearings), verify the existing plans, shop drawings and working drawings are available. Project drawings should incorporate best available information. Existing plan sets are not necessary for overlay work.

- (2) For Deck Rehabilitation projects, a sketch of the plan view for each bridge shall be provided to the largest scale that will fit on an 11”X17” and shall be provided to the Construction Manager for

delineating actual repair areas. Sketch shall include a 5' grid (horizontal and vertical) for aiding in delineating areas, north arrow and pier and abutment labels.

- f. A Final Detail Letter by CDOT and “Consultant Final Submittal Letter” by Consultants verifying that the structural plans and specifications have been prepared in accordance with CDOT’s current design standards and quality control/quality assurance procedures. An example letter can be found at <https://www.codot.gov/library/bridge/form-letters>
- g. Inspection Sketches
 - (1) Plans shall be submitted in both PDF and native file format. For CDOT Projects, Microstation® files are required.
 - (2) Project Structural Engineer or Structural Design Engineer shall place files in the inspection folder under “HQ/Staff Bridge.” If the Consultant does not have access to ProjectWise, then the CDOT Structural Reviewer shall move inspection sketches to the specified location.
- h. Structure Asset Management (SAM) Plan
 - (1) Verify that SAM plan includes all structure treatments in projects using Construction Bridge Program (CBP), Construction Culvert Program (CCP), or Construction Wall Program (CWP) funds.
- i. For Design-Builds, the Contractor shall provide an FHWA Bridge Replacement Cost Report for all bridges on the project. The report shall include element costs of the structure and the unit cost of the bridge as defined by FHWA. The report shall be submitted to CDOT for acceptance with the Released for Construction (RFC) submittal for the structure. https://www.fhwa.dot.gov/bridge/nbi/uc_criteria.cfg
- j. Intergovernmental Agreement (IGA)

When agreements are made between or involve two or more governments a copy of the IGA is needed to determine ownership and who is responsible for the maintenance of bridges. Typically, this is necessary for all managed structures built within CDOT ROW. These agreements are extremely important for long term asset management and designers should request copies of these agreements from the Project/Resident Engineer.

5. Construction

The Project Structural Engineer or Structural Design Engineer shall be available to the construction Project Engineer for assistance in interpreting the structure plans and specifications and for resolving construction problems related to the structure. Any changes or additions to the structure, as defined in the contract documents, shall be communicated to the Project Structural Engineer. BDM Section 36 describes all other construction-related procedures.

The Local Agency or Design Builder shall provide quality assurance (QA) level fabrication inspection as defined in the Local Agency Manual or Design Build RFP (Request for Proposal) unless otherwise approved by the CDOT Fabrication/Construction Unit. The Fabrication/Construction Unit shall provide fabrication inspection services when CDOT provides the construction engineering, only on projects advertised for construction by CDOT. The responsibility for fabrication inspection shall be clarified before advertising a project.

A final inspection review (owner acceptance/final walkthrough) is required after construction. See Section 36.7 of this BDM for a description.

6. Archiving

The Project Structural Engineer shall archive all pertinent documents in ProjectWise when received or by Final Inspection/Owner Acceptance Walkthrough. All PDFs with text or numerical data shall be 300 dpi, page aligned, text searchable, compressed and in conformance with ISO PDF/A-1b archival specifications. The Project Structural Engineer or Structural Design Engineer is to make sure that all documents placed in ProjectWise have their attributes updated to include all known information. At a minimum, all documents shall include a Structure Number. A full list of attributes and documents can be found in the CDOT ProjectWise Reference Manual. CDOT employees are to refer to LMS My Learning for Smart Scanning training; all others are to contact DOT_Records_Mgmt@state.co.us for training on Smart Scanning and Electronic Signatures. Repair and design build projects should also follow the archiving requirements listed. Refer to ProjectWise Reference Manual for the Bridge Project Folder Structure.

<https://www.codot.gov/business/designsupport/cadd/projectwise-reference-manual/view>

At a minimum, pertinent documents include:

- a. Design Calculations and Independent Design Check calculations
- b. Final Structure Selection Report
- c. Load Rating Package, including the electronic bridge model file
- d. Final Geotechnical Report
- e. Final Hydraulics Report
- f. Final bid documents, including Plans and Specifications in PDF format
- g. Design Build RFP Structures Section
- h. Design Build RFP Project Specials
- i. Design Build RFP Alternative Technical Concepts (ATC) after award
- j. MicroStation DGN files and related reference files. Verify that reference file association is working correctly before finishing archiving process. Provide cross-sections for walls in DGN format.

- k. Final Design Submittal Checklist (by Unit Leader)
- l. Correspondence directly affecting design and construction
- m. Final Detail Letter & Consultant Final Submittal Letter (as applicable)
- n. All construction documents, including, but not limited to, as-built drawings, working drawings, shop drawings, material certifications, and test reports
- o. Inter-Governmental Agreements (IGA's) when applicable
- p. Inspection Sketch
- q. Software Input Files (Calculations & Ratings)
- r. Bid Summary
- s. FHWA Bridge Replacement Cost Report for Design-Build projects. Share with cost estimating group (EEMA).
- t. SAM Plan verification of projects using CBP, CCP or CWP funds

F. CDOT STAFF BRIDGE PUBLICATIONS

Copies and revisions to these documents may be obtained from the CDOT website (<https://www.codot.gov/library/bridge>) or from the Office of the State Bridge Engineer.

1. CDOT Bridge Detail Manual

The CDOT Bridge Detail Manual provides the policies and procedures for developing and checking contract plans. For CADD information not covered by the Bridge Detail Manual, refer to CDOT's Office of CADD & ProjectWise Programs, and Highway Engineering Design Processes.

2. CDOT Staff Bridge Worksheets

General Use: The CDOT Staff Bridge Worksheets are pre-detailed drawings that include structure details for various bridge design policies. The details are directly applicable for most projects; however, they should be checked if project-specific modifications are necessary. The intent is to standardize details as much as possible among CDOT projects; however, it is important to understand that the accuracy and use of the drawings is the responsibility of the Project Structural Engineer. Any project changes to worksheets must stay in compliance with the specific worksheet design policy and other Staff Bridge requirements and practices. Typically, each sheet will note whether changes and which changes are acceptable without a variance. An example of this are the bridge rail worksheets. Some changes are expected depending on the deck details, but changes to the reinforcing, dimensions, and detailing of the bridge rail itself could affect their crashworthiness and requires a variance by the State Bridge Engineer in coordination with the Bridge Rail Subject Matter Experts. The worksheets do provide some minimum requirements, such as concrete footers for MSE walls, so any planned

changes should be discussed with the Unit Leader in coordination with the appropriate Subject Matter Expert.

All applications of these Worksheets shall originate from the file posted on CDOT's website. Note that Worksheet numbers are for identification only and shall be removed at the same time the designer's, detailer's, and checker's initials are placed on the sheet.

In general, the CDOT Standard Plans (M & S Standards) do not provide standard details used for bridges. There are exceptions to this. For this reason, and because structural details often depend on the roadway design standards, familiarity with the M & S Standards and the Staff Bridge Worksheets is essential.

Distribution and Maintenance: CDOT Staff Bridge maintains the master files, coordinates revisions, and posts them to CDOT's website. Staff Bridge will maintain a revision log showing all the revision dates that have transpired for each Worksheet and the engineers and detailers who made the revisions. This information is available to anyone for reference.

Revisions: The CDOT Staff Bridge Worksheets are intended to be dynamic. Revisions will be incorporated as new material is added and as criteria and specifications change. The State Bridge Engineer shall approve all revisions to the master files.

Suggestions for improving and updating the Worksheets are encouraged. Anyone who would like to propose revisions should informally discuss the changes with other bridge engineers and detailers to further develop and refine ideas. All suggestions shall be submitted to the Staff Bridge Manager of Policy and Standards. The State Bridge Engineer should then be presented with a preliminary draft showing the developed concept.

On deciding to pursue the revisions, the State Bridge Engineer will assign them to an engineer and a detailer. The Engineer receiving the assignment is responsible for completing the final design, distributing the revisions to all Staff Bridge personnel for their review and comment, making revisions as appropriate based on the comments received, and submitting the final draft to the State Bridge Engineer for approval.

Revised and new Worksheets shall have their effective date given in the upper left revision block of the drawing. On receiving new and revised Worksheets, Staff Bridge will update the master files and the revision log. The effective dates on the drawings and in the revision log provide a ready means to check if a given copy is up to date.

Engineers making revisions to the CDOT Staff Bridge Worksheets should submit design notes documenting their revisions to the Staff Bridge Manager of Policy and Standards. These notes shall describe the changes, identify why they were made, and provide supporting calculations as appropriate. The Structural Design Engineer and the Independent Design Engineer are to sign the notes.

3. Bridge Rating Manual

The Bridge Rating Manual and Bridge Rating Technical Memorandums provide the policies and procedures for performing and submitting the structural capacity rating of bridges.

4. Project Special Provisions

General: Contract documents primarily consist of plan sheets and construction specifications. Structural engineers are responsible for the construction specifications and the plan sheets, applicable to their structure. Construction specifications consist of the CDOT Standard Specifications for Road and Bridge Construction, the Standard Special Provisions, and the Project Special Provisions. See CDOT Standard Specification 101.72 and 101.73 for more information.

If there is a discrepancy with the plans and specifications, the order of precedence is as follows (see Standard Specification 105.09):

(a) Special Provisions

- (i) Project Special Provisions
- (ii) Standard Special Provisions

(b) Plans

- (i) Detailed Plans
- (ii) CDOT M & S Standard Drawings

(c) Standard Specifications for Road and Bridge Construction

Since the Standard Special Provisions and the Project Special Provisions take precedence over the plan sheets, the Project Structural Engineer carefully prepares and reviews them. The plans should refer to the Special Provisions where applicable.

Developing the Project Special Provisions is an integral part of the structure design. To assist design engineers, Staff Bridge makes available on the CDOT website the most commonly used Project Special Provisions related to structures. Chapter 16.1 of the CDOT Roadway Design Guide provides additional information on Project Special Provisions.

All structural-related Project Special Provisions should originate from the file located on CDOT's website if there is a provision covering the subject area. The master files shall not be modified without approval of the State Bridge Engineer in coordination with the Staff Bridge Manager of Policy and Standards.

Distribution and Maintenance: CDOT Staff Bridge maintains the master files, posts them to CDOT's website, and coordinates revisions to the master files. Staff Bridge will also maintain a revision log with each Project Special Provision.

The revision log lists all the revisions that have transpired for the Project Special Provision by showing the date and author of the revision, accompanied by a brief explanation of the revision. Where appropriate, the explanation includes instructions for using the Project Special Provision.

Revisions: Most Project Special Provisions kept on file require little or no revision for most projects (e.g., the Removal of Portions of Present Structure provision), while others are project-specific and require heavy revision (e.g., the Alter and Erect Structural Steel provision).

Revisions made to prepare a Project Special Provision for a specific project shall be made from the copy of the master file posted to CDOT's website. This is necessary to minimize errors and to account for the latest policies for the subject area.

Errors and omissions in the master files or needed improvements are to be reported to the Staff Bridge Manager of Policy and Standards. The State Bridge Engineer will assign the necessary changes to an engineer. The engineer receiving the assignment is responsible for completing the final writing, updating the revision log to include the information described above, and submitting the final draft to the State Bridge Engineer.

5. Deck Geometry Manual

The CDOT Bridge Geometry Program computes coordinates and elevations at various locations on the bridge deck and approach slabs used by the Contractor during construction. The point locations include edges of deck and approach slabs; bridge rail inside face; support centerlines and centerlines of bearing at support locations; and centerlines of girders. Results are provided where girders intersect supports and fractional points along the girders. The bridge deck geometry program shall be used on all CDOT bridges unless the Unit Leader approves an alternate method for deck evaluation tabulation.

6. Staff Bridge Records

Existing structure records maintained by Staff Bridge Asset Management serve several functions for structural design. Bridge design engineers primarily use them to evaluate existing structures for rehabilitation, replacement, or impact to a project in which it is located.

Structure Folders: Every structure has a file whose contents include the bridge inspection reports, the Structure Inventory and Appraisal Report (SIA), and a summary of the structural capacity rating. CDOT personnel (and Consultants with Staff Bridge permission) may access these folders at:

<https://sites.google.com/a/state.co.us/cdotstructures/?pli=1>

As-built Construction Files: The project plans and other construction documents are stored on ProjectWise® for the life of the structure. If these

files are not available on ProjectWise®, contact Staff Bridge Asset Management.

CDOT Structure Inventory Coding Guide: This guide lists and explains the structure inventory and appraisal items.

Field Log of Structures: This catalog lists all CDOT structures by highway number and gives several attributes of each structure.

7. Retaining and Noise Wall Inspection and Asset Management Manual

A manual describing the requirements for CDOT's Retaining and Noise Walls Inspection and Asset Management Program. The purpose of this program is to establish and maintain a comprehensive inventory of all wall assets that could potentially affect public safety, CDOT owned roads, and ROW. In addition, the program outlines inspection requirements, risk identifiers, and project funding and maintenance needs. The manual establishes consistent condition ratings and coding guidelines for the wall inventory.

G. CDOT STANDARDS PUBLISHED OUTSIDE STAFF BRIDGE

Copies and revisions to these documents may be obtained from the CDOT website (<https://www.codot.gov/>).

1. CDOT Standard Specifications and Special Provisions

- CDOT Standard Specifications for Road and Bridge Construction
- CDOT Standard Special Provisions
- CDOT Project Special Provision Worksheets and Samples
- CDOT Design/Build Special Provisions
- CDOT Innovative Contract Provisions

2. CDOT Design and Construction Manuals

- CDOT Survey Manual
- CDOT Roadway Design Guide
- CDOT Materials and Geotechnical Documents
- CDOT Drainage Design Manual
- CDOT Construction Manual

3. CDOT M & S Standard Drawings

H. STANDARDS PUBLISHED OUTSIDE CDOT

- *AASHTO LRFD Bridge Design Specifications*
- *AASHTO Standard Specifications for Highway Bridges* (Note: This document is not permitted for design of new structures.)
- *AASHTO LRFD Bridge Construction Specifications*

- *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges*
- *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges*
- *AASHTO Guidelines for Steel Girder Bridge Analysis*
- *AASHTO Manual for Bridge Evaluation*
- *AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*
- *AASHTO Guide Specifications for LRFD Seismic Bridge Design*
- *AASHTO/AWS D1.5M/D1.5:2015 Bridge Welding Code*
- American Railroad Engineering and Maintenance-of-Way Association (AREMA) *Manual for Railway Engineering (MRE) (Current Edition)*

I. EXCEPTIONS

The following are exceptions to the policies above:

1. Structures (e.g., concrete box culverts) and sign bridges for which the Department's M & S Standards are used are excluded from the final design requirements previously described in Part E, Number 3, of the Policies and Procedures (i.e., final design calculations, developing plans and specifications). A bridge load rating sheet is still required for concrete box culverts that are major structures based on the information in the M & S Standards.
2. Sign bridges, cantilevers, and butterflies extending over traffic are excluded from the preliminary design requirements stated in Part E, Number 2, Items c through e (e.g., foundation investigations, structure layout, and structure type selection).
3. The requirements in this BDM apply to Design-Build projects, except the FOR tasks in Part E, Number 3, Item d (FOR meeting). In addition, the quantity calculation requirements in BDM Section 35 will not apply to the Contractor's design work. See BDM Section 38 for more information on the differences for Design-Build projects.
4. Exceptions for special design structures based on S-Standards that use BDM criteria include:
 - a. Structure Type Selection reports are generally not required; however, a design memorandum is recommended to document how the structure differs from a standard design and to outline the design methodology
 - b. Hydraulic and Geotechnical reports are only required based on design needs.
 - c. Special design structures should be reviewed at project meetings.

J. LOCAL AGENCY PROJECTS, DEVELOPER PROJECTS, AND UTILITY AND SPECIAL USE PERMITS WITHIN CDOT ROW

1. General Services for All Local Agency Projects, Developer Projects, and Access Permits

For Local Agency projects, developer projects, and access permits with no federal or state funding within CDOT ROW, Staff Bridge will provide technical assistance, when requested, to Local Agencies, developers, Consultant design engineers, CDOT Regions, and Federal Highway Administration (FHWA). This assistance will involve answering specific questions and facilitating the use of CDOT structures-related documents. This assistance will be provided by the Staff Bridge PE II assigned to the Region where the project is located. This person will be the CDOT Structural Reviewer for the project.

2. Requirements for Local Agency Projects

The requirements in this BDM apply to all local agency projects using federal/state funds. In addition, Local Agency Checklist attached in Appendix C shall also be provided. For more information regarding Local Agency Projects, refer to the CDOT Local Agency Manual.

Staff Bridge will provide reviews of the structure plans and specifications to help ensure that the Department's written minimum requirements for safety, inspection access, and geometry are satisfied and that the new construction has no adverse impact on CDOT facilities. For bridges off of the National Highway System, some CDOT practices may be omitted with CDOT approval, by the State Bridge Engineer, through the variance process. Currently these variances only consist of the requirements of approach slabs on dirt or gravel roads. Crash tested rail below the TL-4 level will be considered on a case-by-case basis. The review will include helping to ensure that CDOT's written minimum requirements for structure durability are satisfied. Examples of these requirements include those related to corrosion protection and the use of bridge expansion devices.

A final inspection will be required for the structures that carry or could affect the highway system. See Section 36.7 for a description.

Submittals for archiving shall be as listed in section E of this chapter with the following exceptions. Any other exceptions will have to be approved by the Unit Leader.

- SAM plan is not needed.
- Electronic stamping is optional.
- The Final Bid Documents (Plans and Specifications) (CO PE sealed). Plans shall be submitted in both PDF and native file format. Microstation® files are preferred, but CDOT recognizes that many Local Agencies use AutoCAD® exclusively and, therefore, the latter is acceptable. Verify that reference file association is working correctly before finishing archiving process.

3. Requirements for Utility and Special Use Permits

Staff Bridge will provide reviews of the structure plans and specifications to help ensure that the Department's written minimum requirements for safety, inspection access, and geometry are satisfied and that the new construction has no adverse impact on CDOT facilities. A structure number is required for any structure within CDOT ROW except small cell structures. If the structure is owned and maintained by the Local Agency, their Structure Number may be used.

A preliminary plans/scope and/or design criteria submittal and concurrence from Staff Bridge is suggested to avoid any delays to the permit. The following are required to be submitted as a minimum for structures. Additional submittals and/or final walkthrough may be required based on the complexity of the project. Design documents shall be stamped by a Colorado licensed professional engineer. Unless the vehicular road is private, any structures supporting or extending over the vehicular road shall follow all requirements in section E with exceptions shown in section I. Selection reports are typically not required although they are preferred.

a. *Stamped final plans and specifications*

i. Stamped plans and specifications shall be submitted for all new structure(s) and any structure(s) being modified. Stamped Plans and specifications are not required for conduit placement based on electrical codes and manufacturer's recommendations. Addition of new utilities in existing conduits or replacement of existing utilities in kind do not require a PE Stamp.

b. Stamped design calculations

i. Stamped Design Calculations shall be submitted for new structure(s) and any structure(s) being modified. Calculations shall show design loads and codes used in the design. If an existing structure is modified, structural analysis of the existing structure after modification shall be submitted.

ii. Stamped Design calculations are not required if designs are following M & S standards.

iii. Stamped Design calculations are not required for conduit support spacing provided by the National Electric Code, nor for anchors that meet the manufacturers published performance criteria and installation requirements. Tables showing conduit support spacing shall be submitted. Any anchors shall be submitted with manufacturer's embedment requirements or independent design calculations.

c. Stamped Geotech and Hydraulic reports (as applicable to and available for the project)

- d. Load Rating package (If existing CDOT structure is modified causing 3% or more increase in operating rating or if the proposed vehicular structure crosses or supports a highway) For more details, see CDOT Rating Manual.
- e. Provide as-built plans and construction drawings including working and shop drawings after construction for archiving purpose.

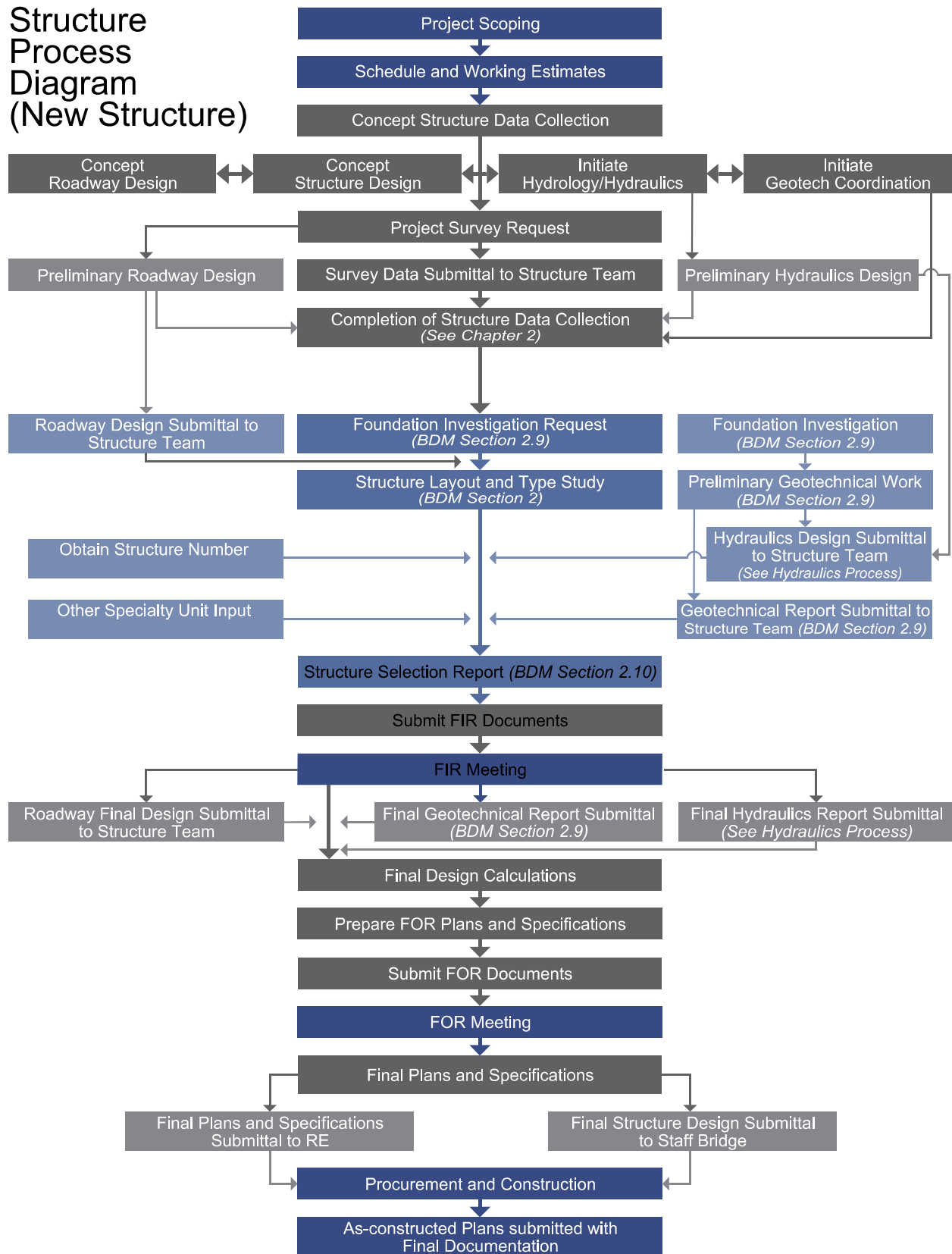
Plans shall show CDOT ROW lines and any easements during construction.

K. REFERENCE

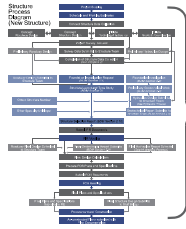
FHWA, Bridge Preservation Guide, Spring 2018.

APPENDIX A - STRUCTURES PROCESS DIAGRAM

Structure Process Diagram (New Structure)



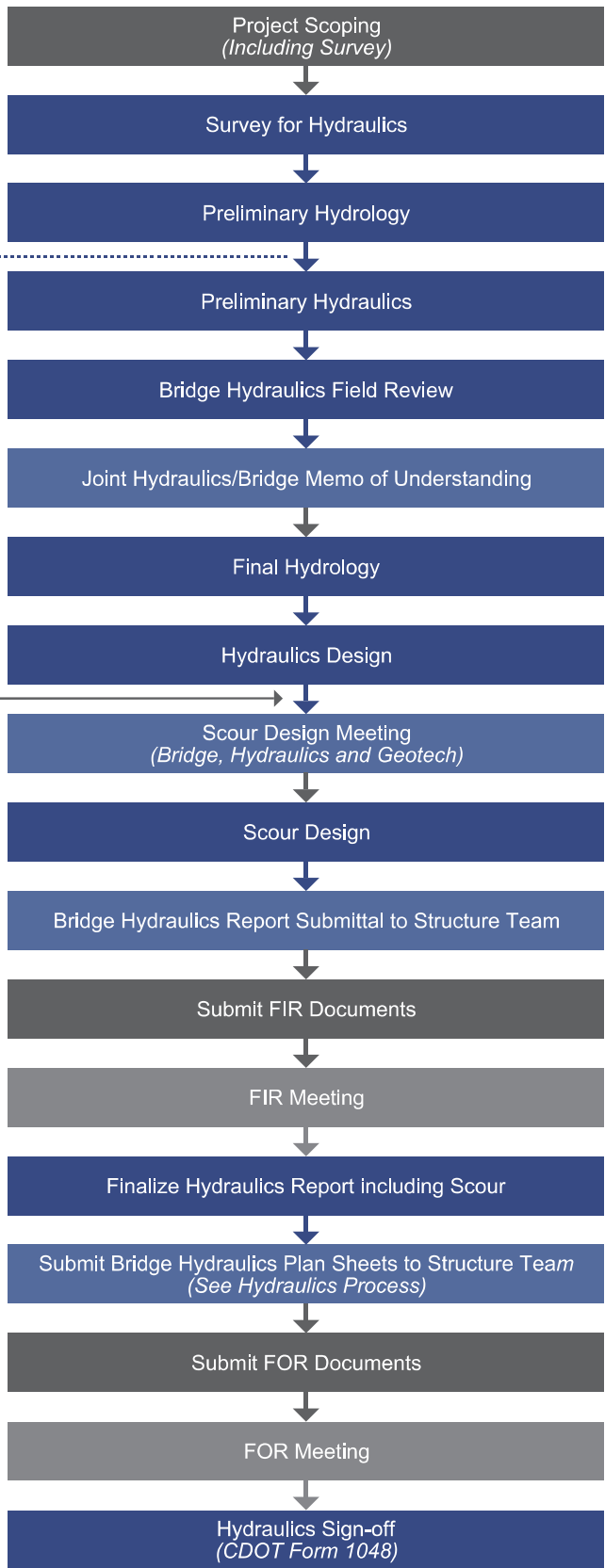
Hydraulics Coordination Process



See Structures Process Diagram

Foundation Investigation

Preliminary Geotechnical Work (BDM Section 2.9)



APPENDIX B - FINAL SUBMITTAL CHECKLIST

This checklist is to serve as quality control/assurance of the finals submittal process as defined in the Bridge Design Manual Policy and Procedures (BDM P&P). This checklist must be signed by the Staff Bridge Unit Leader, prior to advertisement of the project or before construction for a Design-Build or CM/GC project. Checked boxes indicate files are available in ProjectWise and are attributed.

Structure Number(s): _____

Construction Plans and Specifications

- Plan PDF
- Plan Native format files
- Specifications PDF □N/A: _____
- Specifications Word document(s) □N/A: _____

Reports

- Final Hydraulic Report □N/A: _____
- Final Geotechnical Report □N/A: _____
- FHWA Bridge Replacement Cost Report □N/A: _____
(Design/Build Only)
 - Element costs included
 - Bridge Unit Cost included
- Final Structure Selection Report □N/A: _____

Calculations

- Design Calculations (signed and sealed) □N/A: _____
 - Native format files received
- Independent Design Check Calculations (signed and sealed) □N/A: _____
 - Native format files received
- Rating calculations during phasing (signed and sealed) □N/A: _____
- Quantity calculations and quantity check & comparison □N/A: _____

Load Rating

- Load Rating package PDF (signed and sealed) □N/A: _____
- Load Rating XML file □N/A: _____
- Rating Package submitted to Rating Group □N/A: _____

Miscellaneous Information

- Existing bridge plans □N/A: _____
- Existing Shop Drawings □N/A: _____
- Existing Working Drawings □N/A: _____

Final Detail Letter / Consultant Final Submittal Letter

Document verifying structural plans and specifications have been prepared in accordance with CDOT's current design standards and QA/QC procedures and all documentation is archived

Inspection Sketch

- Inspection Sketch PDF □N/A: _____
- Inspection Sketch Native format file □N/A: _____

Structure Asset Management (SAM) Plan

- Project verified on SAM Plan (CBP Funding) □N/A: _____

Intergovernmental Agreement(s)

- IGA(s) attached □N/A: _____

CDOT Staff Bridge Quality Assurance Sign-off

By signing this checklist Staff Bridge Unit Leader acknowledges receipt of a complete package of the Final Submittal documents and that the package is in conformance with the CDOT Bridge Design Manual requirements.

Print Name

Signature

Date

APPENDIX C – LOCAL AGENCY BRIDGE PROJECT CHECKLIST

Local Agency Bridge Project Checklist

PROJECT INFORMATION:

Project Code#: _____ Subaccount#: _____ Project Description: _____
Highway: _____ MP: _____ City: _____ County: _____ CDOT Region: _____
Design Submittal Status: Conceptual Preliminary FIR FOR Final
Remark: _____
Prepared by Engineer: _____ Company: _____ Date: _____

SCOPING:

Rehab vs Replacement Survey Conceptual Plan

Remarks: _____

PRELIMINARY DESIGN:

Structure Number Existing Structure details, if applicable Preliminary Geotechnical Report
 Preliminary Hydraulics Report Structure Selection Report Utility Design Structure Layout

Remarks: _____

FIR SUBMITTAL:

FIR Plans FIR Specifications FIR Cost Estimate

Remarks: _____

FOR SUBMITTAL:

FOR Plans FOR Specifications FOR Quantities Final Geotechnical Report
 Final Hydraulics Report Revised Structure Selection Report, if required

Remarks: _____

FINAL SUBMITTAL:

PE Sealed PS&E PE Sealed Load Rating Package PE Sealed Design Calculations
 PE Sealed Independent Design Check Calculations Quantity Estimate Calculations
 Independent Quantity Check Calculations Bridge Geometry Run Inspection Sketch for Bridges
 PE Sealed Geotechnical Report PE Sealed Hydraulics Report CADD Files Software Input Files
 Revised Structure Selection Report if required IGA , if within CDOT ROW Final Submittal Letter

Remarks: _____

NOTE TO DESIGNER:

1. See CDOT Bridge Design Manual, Section J in Policies and Procedures Chapter for requirements.
2. For more details on final submittal documents, see CDOT BDM, Section E.4 in Policies and Procedures at the following link: <https://www.codot.gov/library/bridge/bridge-manuals/lrfd-bridge-design-manual/bdm-policies-and-procedures/view>
3. This checklist shall be submitted along with each submittal package by the structural designer.

SECTION 1 INTRODUCTION

1.1 GENERAL REQUIREMENTS

The intent of the CDOT Bridge Design Manual (BDM) is to complement current *American Association of State Highway and Transportation Officials Load and Resistance Factor Design Bridge Design Specifications with current interim edition* (AASHTO) and to provide interpretations applicable to the design of Colorado projects. This BDM also establishes CDOT policies and describes preferred practices and procedures in the state of Colorado. Whenever conflicts between AASHTO and this BDM arise, policies established in this BDM shall govern.

1.2 DEFINITIONS

1.2.1 Bridge Definition

A bridge is a structure that spans over a road, railway, river, or other obstacle to provide passage for pedestrians and vehicles from one side to the other.

1.2.2 Culvert Definition

In general, a culvert is a structure, conduit, or drain that passes underneath a road, railroad track, or other obstruction to allow water to be directed away from travel corridors. Some large culverts can carry pedestrian and vehicle traffic inside.

1.2.3 Glossary of Terms

For additional acronyms and abbreviations, refer to CDOT Standard Plans M-100-2.

AASHTO – American Association of State Highway and Transportation Officials. For the purpose of the CDOT BDM, “AASHTO” will refer specifically to *Load and Resistance Factor Design Bridge Design Specifications*.

ABC – Accelerated Bridge Construction

Abutment – A structure that supports the end of a bridge, provides lateral support for fill material on which the roadway rests immediately adjacent to the bridge, and transfers the loads from the superstructure into the ground.

Alignment – Control line used to determine the direction of travel in the roadway.

Approach Slab – A concrete slab that provides a transition between roadway pavement and the bridge and is used to alleviate problems with settlement of the bridge approaches relative to the bridge deck.

ASD – Allowable Stress Design

Batter – Inclination of a vertical surface (typically wall or pile) in relation to a horizontal plane.

BDM – CDOT Bridge Design Manual

Bent – A structure that supports the superstructure at each end of a span.

Bent Angle – Angle between the centerline of a support and a layout line (see Figure 4-1). This angle is typically used as a bridge description skew and a culvert skew.

CBT girders – Colorado Bulb Tee Girders

Clear Zone – The total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles.

Diaphragm (integral) – Concrete block encasing free ends of girders at abutments or piers. Usually provided to resist lateral forces and to ensure proper load distribution to points of support.

Diaphragm (intermediate) – A vertically oriented solid transverse member connecting adjacent longitudinal flexural components to transfer and distribute vertical and lateral loads and to provide stability to the compression flanges.

Efflorescence – White deposit on concrete caused by the crystallization of soluble salts brought to the surface by moisture in the concrete.

Embankment – A bank of earth constructed above the natural ground surface to carry a road.

End Block – An increase in web width at the girder end intended to provide adequate bearing.

ERS – Earthquake resisting system

FHWA – Federal Highway Administration

FIR – Field Inspection Review, occurs at approximately 30% project completion.

FOR – Final Office Review, occurs at approximately 90% project completion.

Freeboard – Clearance between the lowest point of the bridge superstructure and the design water surface elevation immediately upstream of the bridge.

Girder – A main horizontal structural member that supports vertical loads.

GRS – Geosynthetic Reinforced Soil

Haunch – The section of concrete between the top of girder and the underside of deck.

HCL – Horizontal Control Line

HLMR – High-Load Multi-Rotational bearings (pot, spherical, and disc bearings)

LCCA – Life Cycle Cost Analysis

LFD – Load Factor Design

Life Cycle – The period of time used for the calculation of LCCA. A bridge is expected to be in operation in excess of this period.

LRFD – Load and Resistance Factor Design

Milepost – A post placed along a roadway to mark a distance in miles.

MOT (Maintenance of Traffic) – Traffic flow alternatives used to allow construction.

MSE – Mechanically Stabilized Earth

NBIS – National Bridge Inspection Standards

OSHA – Occupational Safety and Health Administration

PDA – Pile Driving Analyzer

Pier – The part of a bridge structure that provides intermediate support to a superstructure.

PPC – Polyester Polymer Concrete

P&S Submittal – Construction plans and specifications

PTFE – Polytetrafluoroethylene (typically used for sliding bearings)

QA/QC – Quality Assurance and Quality Control

QMP – Quality Management Plan

Refined Analysis – Detailed, sophisticated structural modeling approach that typically involves computerized finite element analysis.

RFC – Release for Construction

RFI – Request for Information

Riprap – Protective covering material deposited on river stream beds or banks to prevent erosion and scour.

ROW (Right of Way) – A privately owned strip of land granted or reserved by the owner for construction of facilities, such as highways, railroads, power lines, and other infrastructure.

Sacrificial Anode – The anode in a cathodic protection system used to inhibit the object's corrosion.

SC – Site Class

SDC – Seismic Design Category

Skew Angle – Angle between the centerline of a support and a line normal to the layout line (see Figure 4-1). This angle is typically used in Structure Inspection Reports and Bridge Geometry.

Sleeper Slab – A strip of concrete that supports the free end of the approach slab.

SPT (Standard Penetration Test) – An in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil.

Staff Bridge Branch – A branch of CDOT tasked with setting overall policies and procedures for bridges and bridge-related structures, providing direction, and reviewing and approving plans for the individual projects in the state of Colorado. Review and approval shall be by CDOT Designer/Reviewer or Unit Leader.

Substructure – The part of a bridge structure supporting the superstructure that includes elements such as piers and abutments.

Superstructure – The part of a bridge structure that directly supports traffic loads and includes elements such as bridge rail, bridge deck, and girders.

WEAP – Wave Equation Analysis of Pile driving

Wingwall – A retaining wall adjacent to an abutment or a culvert that serves to retain earth in an embankment.

1.3 DESIGN SPECIFICATIONS

1.3.1 Load and Resistance Factor Design (LRFD)

Load and Resistance Factor Design (LRFD) is a current design method that shall be used for all new structure designs in the state of Colorado. It is a reliability-based design methodology in which force effects caused by factored loads are not permitted to exceed the factored resistance of the components. Load and resistance factors are used to take into account statistical probability of both the variability of loads and the uncertainty of material properties.

Load modifier η_i is not permitted to be less than 1.00 under any conditions for ductility, redundancy or operational importance.

AASHTO
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1.3.2 Load Factor Design (LFD)

Load Factor Design (LFD), also known as Ultimate Strength Design, is a design method that incorporates safety provisions by separately accounting for uncertainties relative to load and resistance. Unit Leader may allow the use of this design method on some rehabilitation and widening projects where the original structure was designed in LFD. The intent to use the LFD method shall be documented in the Structure Selection Report and approved by Unit Leader before beginning the design process.

1.3.3 Allowable Stress Design (ASD)

Allowable Stress Design (ASD), also known as Service Load Design Method or Working Stress Design, uses uniform factors of safety to account for uncertainty in both applied loads and structure capacity. This method is allowed on rehabilitation and widening projects only where the original structure was designed in ASD to avoid conflicts between different design philosophies. The intent to use ASD method shall be documented in the Structure Selection Report and approved by Unit Leader before beginning the design process.

SECTION 2 GENERAL DESIGN AND LOCATION FEATURES

2.1 GENERAL

This section addresses structure configuration, clearance requirements, aesthetic guidelines, structure investigation, and selection report requirements.

2.2 LOCATION FEATURES

2.2.1 Alignment

Care shall be taken during the preliminary design phase to ensure that horizontal and vertical alignments of the proposed bridge satisfy project objectives and minimum requirements of this BDM. Careful consideration of all feasible alternatives will minimize revisions at later stages. Structure layout alternatives shall be evaluated based on economic, engineering, environmental, construction, aesthetics, ease of inspection, cost of maintenance, traffic safety, bridge security, and utility avoidance factors.

2.2.1.1 Horizontal Alignment

Abutments and piers on curved bridges shall be placed to optimize the span lengths and girder spacings. Use of 1° increments is preferred when setting bridge skew, especially when not restricted by the existing construction. When a bridge is on a curve with a large radius, it is appropriate to consider a slightly wider straight bridge to lower the cost of construction. The Designer shall perform overhang design for the worst case cantilever.

When appropriate, preliminary design shall consider the possibility of future bridge widening.

Where practical, ends of approach slabs on bridges with skew $\geq 30^\circ$ should be set square to the roadway to facilitate construction and to minimize direct impact on the joint by snowplows.

2.2.1.2 Vertical Alignment

The Designer shall consider all local constraints and code requirements to ensure safety and to minimize interference with traffic under the bridge. Ultimate roadway configuration should be considered when setting bridge vertical alignment to accommodate any future bridge widenings.

2.2.1.3 Screed Elevations

The plans shall provide elevations of the bridge deck, approach slabs, and roadway approaches. Roadway approach and approach slab information is intended to avoid misalignments between the roadway and bridge. At a minimum, deck elevations should be provided at 1/10 points along control line, girder lines, crown line/centerline of the bridge, flow line, and edges of deck. At a minimum, approach slab elevations should be provided at 1/2 points along the approach slab crown and flow lines.

Bridge geometry shall be presented in the plans in accordance with Chapter 14 of the CDOT Bridge Detail Manual and CDOT Structural Worksheets.

2.2.2 Vertical Clearances

Required minimum vertical clearances to bridges passing over the rural and urban principal arterial routes shall be 16.50 ft. The minimum vertical clearance from the roadway to pedestrian bridges, utility bridges, and overhead sign supports shall be 17.50 ft.

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Vertical clearance over low speed, low volume undercrossings (i.e., collector roads, streets, and private entrance crossings) may be modified to 15.00 ft. minimum with approval from State Bridge Engineer and concurrence of the owner of the low speed, low volume or private undercrossing. Any structures with clearance less than 16.50 ft. shall be signed as part of the project.

These values include 6 in. clearance for future overlays, structure deflections, snow on the road, vehicles oriented other than plumb, effects of sag vertical curves, future expansion, etc., which can be modified at the Owner's request. Provided values should be true over the entire roadway width, including shoulders. If construction requirements restrict the vertical or horizontal clearances to values lower than required at final design, Staff Bridge shall notify the Permit Department.

For vertical clearance from a pedestrian or bicycle path to an overhead obstruction, refer to Section 31.4.2 of this BDM.

Vertical clearance over waterways should be established based on hydrology and hydraulics explorations and shall also consider applicable watercraft clearance requirements. The Designer is required to provide adequate freeboard based on hydraulics elevations provided by the Hydraulics Engineer. At a minimum, freeboard for 100-year flood should be 2 ft for low to moderate debris streams with velocities > 6 fps. See Chapter 10 of the CDOT Drainage Manual for additional clearance information. When minimum freeboard clearances are not feasible, a hydraulic variance will be required. If freeboard requirements are not met, bridge and bridge connections shall be designed for any additional lateral loading due to the lack of clearance.

2.2.3 Horizontal Clearances

Horizontal clearances shall conform to AASHTO and *A Policy on Geometric Design of Highways and Streets*.

**AASHTO
2.3.3.3**

Figure 2-1 to 2-3 summarize the minimum requirements for horizontal clearances. These are preferred configurations and should be evaluated and modified as appropriate. Modifications may be appropriate based on the location of the existing drainage features and the cost benefits of balancing or adjusting span lengths.

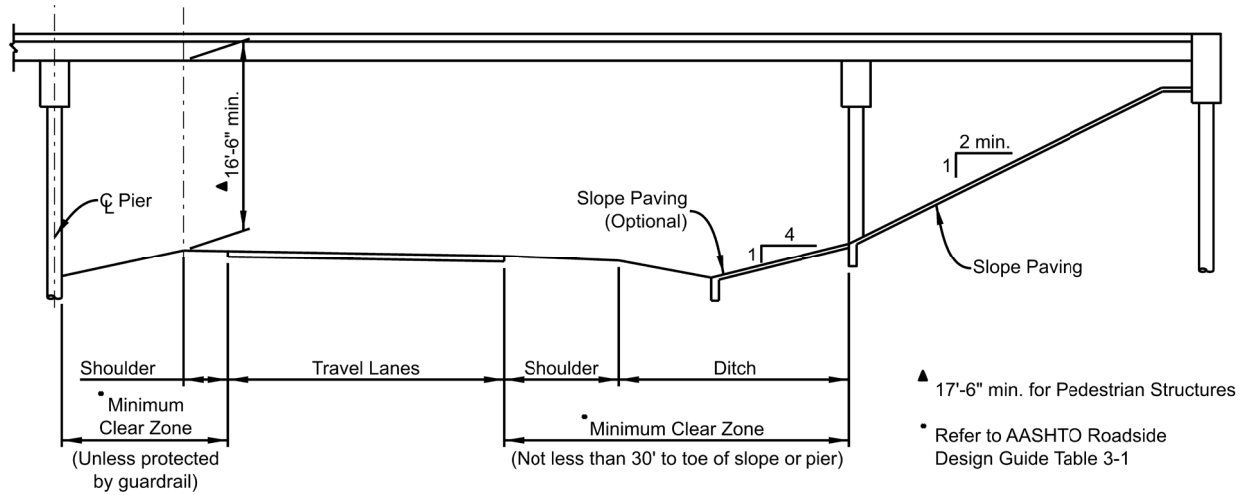


Figure 2-1: Bridge Clearances – High Speed Roadway
Design speed > 45 MPH

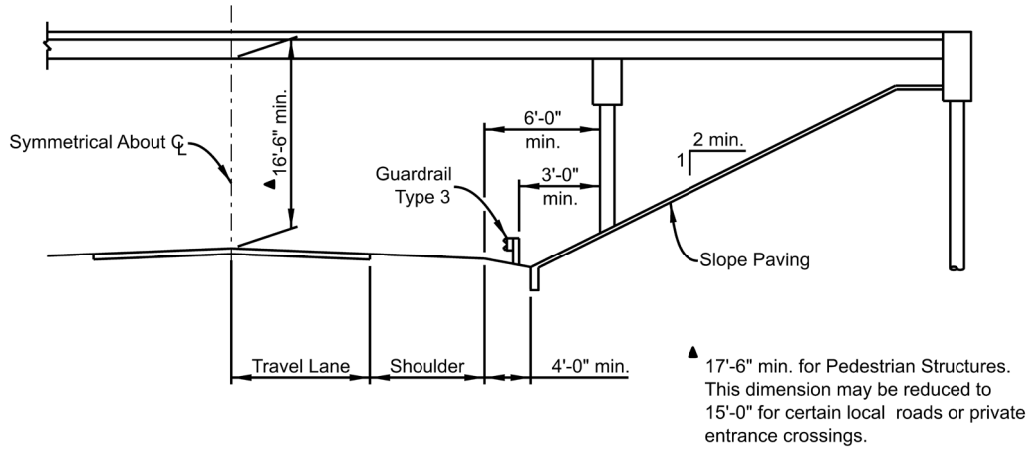


Figure 2-2: Bridge Clearances – Low Speed Roadway
Design Speed ≤ 45 MPH

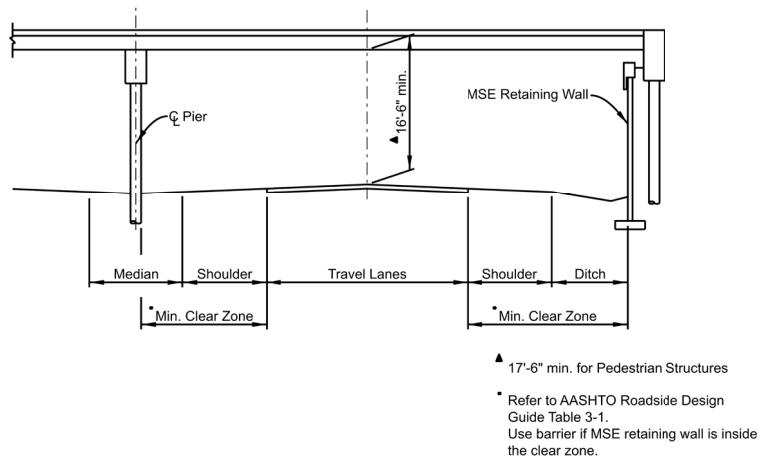


Figure 2-3: Bridge Clearances with MSE Retaining Wall
All undercrossings and roadways

2.2.4 Criteria for Deflection

Design shall follow deflection criteria outlined in AASHTO 2.5.2.6.2. When these criteria cannot be met, a variance request shall be submitted to the Unit Leader.

**AASHTO
2.5.2.6.2**

2.2.5 Sidewalks

For an attached sidewalk on a vehicle bridge, the clear walkway shall be 5 ft. minimum but in no case shall it be narrower than the approaching sidewalk. Additional width may be required in an urban area or for a shared pedestrian-bikeway facility. Curb height of the raised sidewalk on the bridge should not be less than 6 in. above the final grade. If the deck does not have an asphalt layer, the sidewalk height should be increased to 9 in. to account for future overlays. Raised sidewalk shall be connected to the deck using fully developed reinforcement.

**AASHTO
13.11.2**

When requested by Owner or when pedestrian walkways are provided on high speed, high volume bridges, walkways shall be protected with a combination of inboard traffic barrier (Bridge Rail Type 9 or Type 10 MASH) and outboard pedestrian railing. Any other rail shall be approved by the State Bridge Engineer in coordination with the Bridge Rail SMEs. High speed roadways are defined as those with a speed limit greater than 45 mph. Refer to Figure 2-4 for sidewalk details. When pedestrian traffic is high, a separate pedestrian bridge shall be considered based on a combination of factors such as cost, safety, phasing and site conditions. Roadway / trail designers shall assist in performing applicable studies as required. For high speed, high volume roadways, the Project Engineer decides whether or not to separate traffic and pedestrian facilities. See Section 2.4.2 for fencing information.

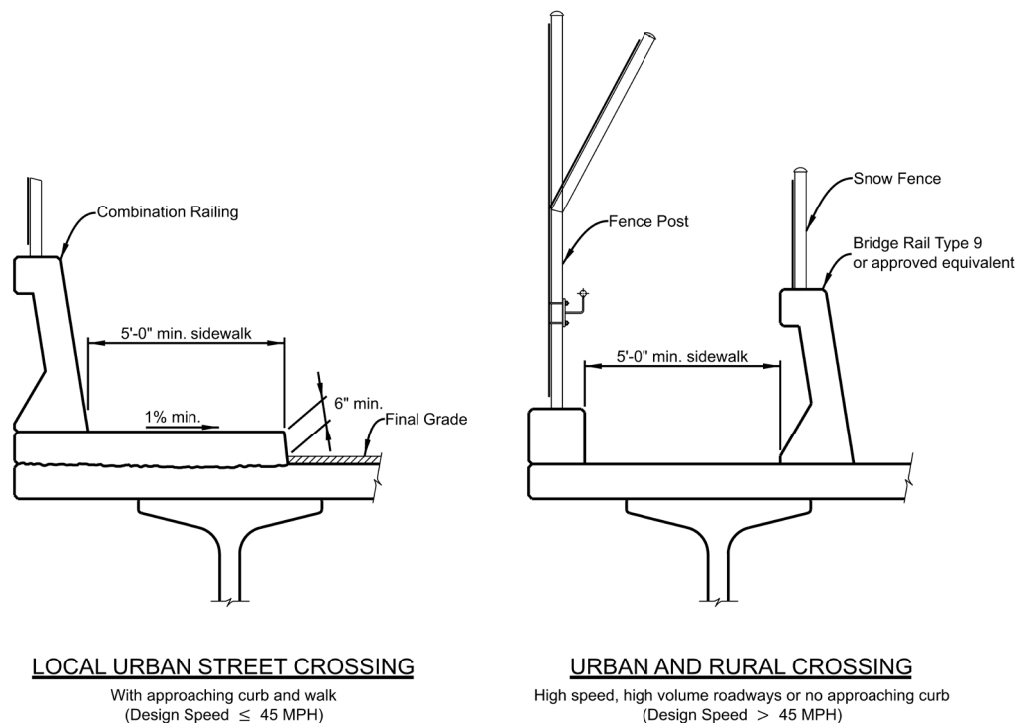


Figure 2-4: Standard Sidewalk Details

2.2.6 Environmental Considerations

Minimizing and mitigating environmental impacts of any construction project shall be given the highest priority. All proposed projects shall be evaluated for all possible environmental impacts at the preliminary stages of the design. Engineers and Contractors shall comply with state and federal laws concerning all environmental issues, including, but not limited to:

AASHTO 2.3.4

- Ecological impacts on wetlands
- Water pollution and contaminated materials
- Erosion and sediment control
- Streams and floodplains encroachment
- Removal of embankment stabilizing vegetation
- Fish/wildlife habitation or migration routes
- Unstable slopes
- Noise/vibration control policy
- Hazardous materials and solid waste
- Asbestos containing materials/soils
- Transportation and discharge of hazardous materials
- Spill reporting
- Impact on local communities
- Historic/archaeological/paleontological sites

2.3 AESTHETICS

2.3.1 General Requirements

Aesthetic enhancements are defined as items not necessary for the load carrying capability of a bridge or a structure, such as facades, monuments, and artwork. The level of aesthetic treatment will vary from project to project depending on the importance of the structure, sensitivity of the setting, construction budget, location, historical value, and Owner's preferences.

AASHTO 2.5.5

CDOT has a Visual Resources Program (VRP), administered by the CDOT Environmental Programs Branch, Landscape Architecture Section. The CDOT Landscape Architect's primary goal regarding aesthetics is to objectively measure, following standard methodology, the visual impacts for projects supporting the transportation system to meet legal requirements and regulations, while maintaining and improving scenic quality. The primary value of the Landscape Architect in structure design is to create beautiful and pleasing structures in harmony with the surroundings. Early in the project's development process, the Engineer shall coordinate with the Landscape Architect for project aesthetics requirements. The Landscape Architect will review and provide aesthetic recommendations for any bridge/structures based on the VRP principles of integrating, designing, and recommending visual solutions for bridges and other structures. "A beautiful design can only be achieved if the aesthetics design is developed as an essential part of the total concept" (Transportation Research Board and National Research Council, 1991, p. 8, Bridge Aesthetics Around the World, Washington DC).

Bridge and structures will be reviewed from several vantage points. The CDOT Landscape Architect shall determine the users and neighbors/community and consider views of the road, both from and of the road, to generate a sense of place, security, and context of scale by incorporating Context Sensitive Solutions (CSS). Context sensitive design acknowledges a concern for local architectural identity and investment.

The approach to CSS aesthetics is an iterative process with many aspects of aesthetic design review, to build on a framework for color, scale, style, direction, proportion, shape, form, balance, etc. Corridor themes are established by developing a complementary appearance between varying bridge/structure types and components along the corridor. Every structure will receive architectural and aesthetic design reviews and recommendations during the design and construction phase.

If the bridge is a part of a specific corridor, it must be visually consistent with the overall scheme of the corridor. Cost-effective aesthetic treatment can be achieved by using color coating, staining, colored concrete, form liners, rustications, and other methods. CDOT practice limits aesthetic treatment costs to less than 5 percent on any individual project and 2 percent at the statewide program level, unless outside funding is provided. Veneers are generally discouraged based on safety, durability, and maintenance concerns.

Aesthetic enhancements shall not be attached to the main load carrying members of the structure, that is, girders, pier caps, columns, etc., without approval from Unit Leader. Any attachment of aesthetic enhancements to the structure shall be detailed and/or designed to prevent deterioration (e.g., corrosion) that may damage or degrade any component of the structure. Aesthetic enhancements shall not be placed in locations or consist of components that in any way limit access to or inhibit the inspection of the structure. Any permanent aesthetic enhancements within CDOT ROW shall not impact the safety of the traveling public.

The requesting entity shall maintain and repair the aesthetic enhancements. An IGA is needed to determine appropriate responsibility to maintain which bridge/structures/components and should be archived by the Engineer for future reference. Access to CDOT ROW to maintain or repair the aesthetic enhancements in the future would be requested and approved through the CDOT Access Management Permit Office.

2.3.2 Lighting

The placement and type of lighting poles and fixtures can have a major visual impact on the overall appearance of the bridge. Poles should be set such that they are visually complementary to the structure.

Superstructure-mounted highway lighting shall be avoided wherever possible. The Designer shall investigate the possibility of mounting the lighting on an extended pier cap. If superstructure-mounted lighting cannot be avoided, it shall be located as closely to a pier as is practical to limit movement and vibration.

Underdeck lighting should be provided on bridges over roadways and trails when requested by the region. It is preferable to place the underdeck lighting on

substructure elements rather than directly on the deck to allow easier deck repairs and replacement.

When lighting for pedestrian bridges is provided on poles, it should be independent of the bridge structure where possible. Other lighting options can be evaluated on a case-by-case basis. Pedestrian lighting should be incorporated into local underpasses. Structural plans should be coordinated with electrical plans for conduit splices and locations, expansion/deflection coupler locations considering longitudinal and lateral deflections, fixture locations, and mounts to avoid conflicts during construction. Any bridge lighting configuration must be readily accessible for inspection and maintenance. All junction boxes maintained by Xcel Energy must comply with Xcel Energy requirements. Xcel Energy junction box size is typically 18 in. x 8 in. x 6 in.

2.3.3 Form Liners and Veneers

Both form liners and veneers can be used to create desired architectural surface treatments, such as intricate patterns, stamps, murals, etc., to increase the aesthetics of the bridge or retaining wall. The use of integral aesthetics, such as form liners, rather than rock veneers, is preferred to limit possible delamination and flying debris from impact. The Unit Leader shall approve any attachment of architectural enhancements to the load carrying members. All such attachments shall be adequately designed and detailed on the plans.

For vertical and nearly vertical concrete surfaces with rustications that are accessible to pedestrians, practical means should be considered to make these surfaces unattractive for climbing. To reduce the construction labor required to make these rustications, they should be made in dimensions that use standard lumber sizes with a minimum number of cuts. In all cases, grooves should have at least one beveled edge to facilitate removal of the lumber strips used to form them. Figure 2-5 shows examples of unacceptable configurations and suggested details. This does not apply to standard prefabricated form liners with vertical flute configurations that have proven to be practical from previous use.

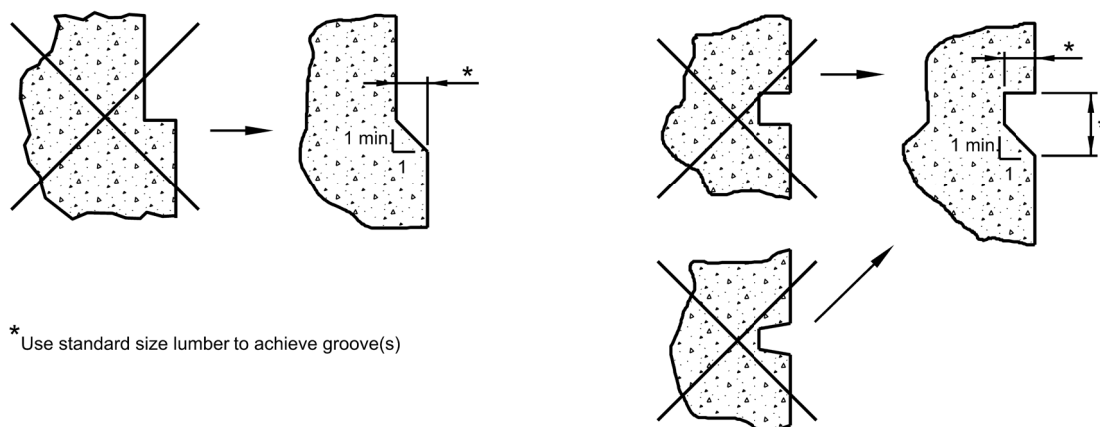


Figure 2-5: Vertical Concrete Surface Details

2.4 RAILING AND FENCING

The overall height and shape of the safety barrier and fence should meet or exceed AASHTO standards. Railings and fencing installed on retaining walls should be consistent with or complementary to those found on adjacent bridges.

AASHTO 13.4

To improve durability, the use of weathering steel for railing and fencing is discouraged. Galvanizing of steel portions of safety barriers and fences is the minimum standard required. In cases where steel portions of safety barriers and fences are to be painted for aesthetic or other reasons, it must be done in addition to galvanizing using a duplex coating system.

Chain link fence is not required to be painted, but for aesthetic purposes a vinyl coating can be added.

It is recommended that dissimilar metals be avoided to improve durability. The use of aluminum or galvanizing steel are acceptable but combinations should be avoided.

2.4.1 Railing

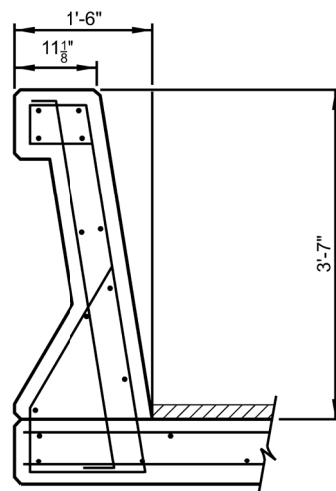
2.4.1.1 Traffic Railing

MASH compliant and crashworthy railing systems shall be used adjacent to vehicle traffic. Bridge rail Type 9 and Type 10 MASH are CDOT's MASH compliant bridge rails and shall be used for all new construction. New construction consists of new bridges and bridges being rehabilitated or widened in a manner that requires removal of the existing bridge railing. Refer to CDOT Staff Bridge Worksheets for details of bridge traffic railings. Any changes made to bridge rail and/or the transition worksheets will require a variance from the State Bridge Engineer in coordination with the Bridge Rail SMEs. Any modified or non-CDOT bridge rails proposed for projects should be crash tested or suitably evaluated to ensure MASH compliance and crashworthiness.

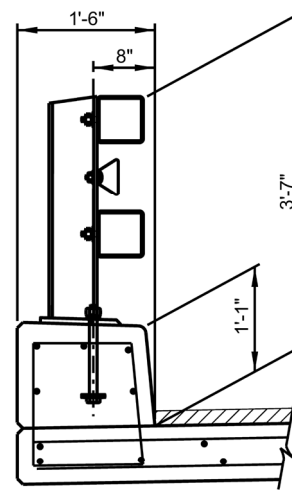
AASHTO 13.7

All existing bridge rails that meet the current AASHTO *and* MASH criteria may remain in place. If the bridge falls within the limits of a Federal-Aid project, if sufficient funds exist, or if it is an essential repair, rails should be modified to meet these standards or be replaced with a MASH compliant rail or rehabilitated to meet MASH compliance.

The Type 9 and Type 10 MASH bridge rails, shown on Figure 2-6, are typically used for nearly all new construction on state highway projects. These rails offer the overall optimum solutions given safety, cost, maintenance, appearance, and guardrail compatibility issues. For local agency projects a test level lower than TL-4 may be approved by State Bridge Engineer in coordination with the Bridge Rail SMEs based on design speed, ADT and other safety factors. Bridge rails and transitions at any test level shall be MASH compliant and crashworthy.

**BRIDGE RAIL TYPE 9**

(See B-606-9 for more details)

**BRIDGE RAIL TYPE 10 MASH**

(See B-606-10 MASH for more details)

Figure 2-6: Bridge Rail Types 9 and 10 MASH

2.4.1.2 Pedestrian Railing

The height of pedestrian railing should not be less than 42 in. Openings between horizontal or vertical members on pedestrian railings shall be small enough that a 4 in. sphere (IBC requirement) cannot pass through them. This value shall be used in lieu of AASHTO requirements and shall apply to the full height of the railing.

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2.4.1.3 Bicycle Railing

Where specific protection of bicyclists is deemed necessary the minimum height of railing used to protect a bicyclist shall be 42 in., measured from the top of the riding surface. Bicycle railing with horizontal members is preferred when allowed. Chain link fence may be used in lieu of bicycle railing. Smooth rub rail shall be attached to the barrier or fence at a handlebar height of 42 in., unless a smooth surface is already provided. See AASHTO Bicycle Facilities Guide and CDOT Roadway Design Guide for more information.

**AASHTO
13.9.1**

2.4.1.4 Combination Railing

Combination railing is a type of traffic railing that also satisfies the height and opening requirements of either the pedestrian or the bicycle railings. Where combination railing (pedestrian/bicycle and traffic) is provided, the combination rail modified from CDOT MASH compliant rails shall not interfere with the crashworthiness of the MASH compliant rails or cause other hazards (such as spearing) to the traveling public.

**AASHTO
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2.4.1.5 Safety Railing

Safety railing is intended to provide limited fall protection and visual identification of the vertical drops. The top of safety railing should be at least 42 in. above

walking/working surface. Intermediate members (such as balusters), when used between posts, shall not be more than 19 in. apart.

2.4.1.6 Handrail

Handrail is intended to meet ADA requirements (See Section 31.7) and may be required in addition to Pedestrian and Bicycle Railing. When required, handrail shall be placed at a height between 34 and 38 inches above finished grade. Handrail may be similar to bicycle railing details.

2.4.2 Fencing

2.4.2.1 Chain Link Fence

All bridges with pedestrian or bicyclist access that cross roadways or railway tracks shall be provided with chain link fabric fence or other approved fencing to prevent objects from being thrown onto the road below. Fencing other than those noted below shall be approved by the Unit Leader. The addition of fencing to the bridge rail may affect its crashworthiness and test level criteria.

The maximum size opening for chain link fabric shall be 2 in. Other approved fencing includes the use of picket fences with a maximum clear opening of 4 in. between pickets. Fencing should extend, as a minimum, 30 ft. beyond the outside shoulder line on the traveled way below, the bridge rail length, or as required per railroad criteria. Bridges with pedestrian walkways over traffic should have pedestrian fencing on the barrier or the curb.

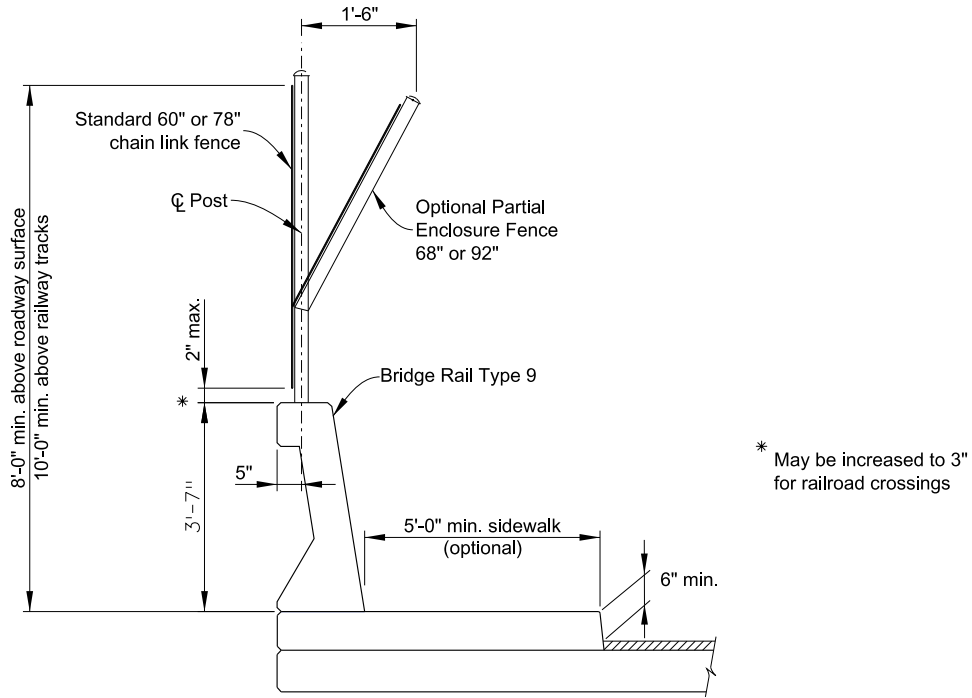
Partial enclosure pedestrian fence should be considered at locations where there is a history of objects being thrown over the fence. The Designer should coordinate with the Region to determine these locations. The minimum overall height of the barrier and fence above roadway surface should be 8 ft. Fence above railway tracks shall be 10 ft. for vertical fence and 8 ft. for partial enclosure fence. Refer to Figure 2-7 for more details.

Horizontal pipe members should be avoided as they are a spearing hazard.

2.4.2.2 Snow Fence

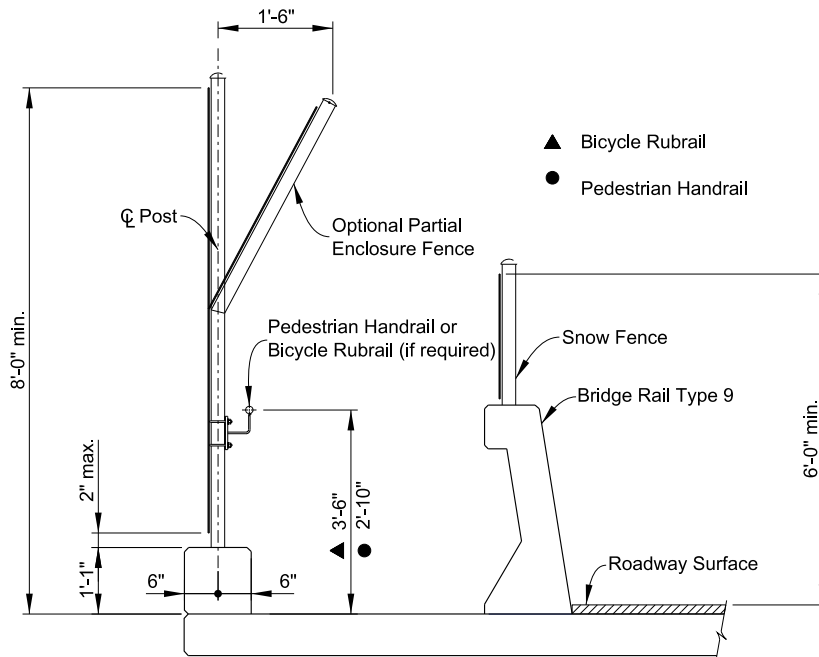
A snow fence prevents snow from splashing over the barrier during snow removal. Snow fencing shall be required over highways, over railroad facilities, and on other bridges per Region requests.

Snow fencing may be used over pedestrian paths per Region requests. The minimum height of the snow protection should be 36 in. with 3/8 in. mesh and should extend, as a minimum, 30 ft. beyond the outside shoulder line on the traveled way below, the bridge rail length, or as required per railroad criteria. Designer shall verify the limits of snow fencing with Traffic Safety for any obstructions to sight distance.



FENCE, SPEED ≤ 45 MPH

With optional approach curb and walk
 See B-607-5, B-607-6B, B-607-778, and B-607-8B for additional details



FENCE, SPEED > 45 MPH

High speed, high volume roadways or no approach curb
 See B-607-3 for additional details

Figure 2-7: Fencing Types

2.4.2.3 Drop-Off Protection

Drop-off protection is categorized as follows:

- Pedestrian Protection** – Pedestrian railing shall be provided for any sidewalk or shared use path adjacent to a wall with a drop-off greater than 30 in. Safety railing or fencing shall be provided for walls with drop-offs greater than 30 in. that are generally accessible by the public but not adjacent to sidewalks or paths.
- Shared Path Users Protection** - It is preferred that a 5 ft separation be provided between a shared use path and drop-offs or embankments with slopes greater than 4:1. Otherwise a suitable barrier such as a railing or fence shall be provided at the top to the slope. Refer to Chapter 14 of the CDOT Roadway Design Guide for more details on shared use path next to embankment slopes protection requirements. Safety railing or fencing shall also be provided for walls with drop-offs greater than 30 in. that are not adjacent to sidewalks or paths but are generally accessible by the public."
- CDOT Maintenance Personal Protection** – Fall protection, including safety railing, fencing (chain link or 3 cable), or tie-off points, as approved by CDOT Maintenance, shall be provided at all wall drop-offs greater than or equal to 4 ft. in areas restricted to public access by either location or fencing.

Safety railing and fencing in all cases shall be capable of resisting 200 lbs. of force, applied to the top of the longitudinal element acting in any direction.

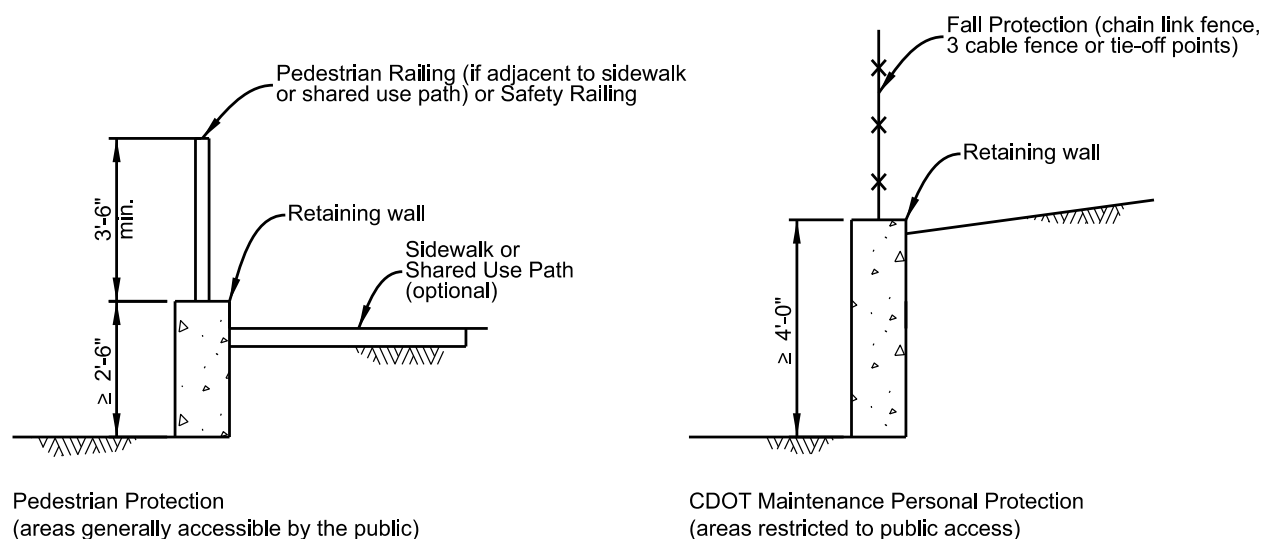


Figure 2-8: Drop-Off Protection

2.5 RAILROAD REQUIREMENTS

2.5.1 General Requirements

New bridges designed to overpass a railroad should be designed per AASHTO specifications, except for clearance requirements, which shall conform to the

American Railway Engineering and Maintenance of Way Association (AREMA). When designing a bridge carrying a railroad, using the railroad's preferred design and materials may accelerate the review process but a Structure Selection Report is still required. Some local railroad agencies, such as BNSF and UPRR, have set requirements that are more conservative than those outlined in AREMA. These requirements should be met, if required. However, if adherence to the local railroad's requirements results in an impractical or a non-cost-effective design, the Owner should be notified and the decision should be made on a case-by-case basis. Railroads typically specify intermediate reviews of the design e.g. 30%, 60% and 100% etc. depending on the contract requirements. These submittals may require extensive interruption in the design process while the railroad reviews the design for compliance. It is important for a checker to be assigned to the project early so any design discrepancies can be dealt with prior to each submittal.

2.5.2 Vertical Clearance

All highway bridges over non-electrified railroads are required to have a minimum vertical clearance of 23 ft above the top of rail per AREMA guidelines. Note that greater clearances are required for tracks on a curve. For details, refer to *AREMA Manual for Railway Engineering*, Chapter 28. Typically, local railroad request clearance greater than 23' (23'-4" for UPRR, 23'-6" for BNSF). The Structure Selection Report should evaluate and discuss the difference in cost if a vertical clearance larger than the normal practice specified in the design guidelines is requested. If the cost is minimal, the project will fund the difference. If the cost is excessive, the local railroad should be required to fund the additional cost. Please refer to FHWA memo 130416 for discussion of funding eligibility for additional information. The railroad shall document or justify by special site conditions the need for clearances greater than those shown or referenced herein.

2.5.3 Horizontal Clearance

It is preferable to keep bridge piers outside the railroad ROW or the 25 ft clear zone, measured perpendicular to the centerline of the track. Piers located less than 25 ft from the centerline of the outside track shall meet the requirements to qualify as heavy construction or are to be protected by a reinforced concrete crash wall. Absolute minimum horizontal clearance to the face of the pier protection wall should meet *AREMA Manual for Railway Engineering* or local railroad's requirements.

2.5.4 Construction Clearance

Minimum vertical temporary construction clearances shall be 21'-6" (22'-6" AREMA) above the top of the high rail. Greater temporary clearances may be required on a project-by-project basis. Minimum horizontal construction clearances measured perpendicular to centerline of track to nearest obstruction (formwork, equipment, stockpile materials, etc.) should satisfy requirements set by the local railroad. Any excavation work within these limits requires approval of the railroad.

2.5.5 Protection and Screening

All highway bridges over any railroad shall include a fence with a barrier, approved by the railroad, on both sides of the structure, extending to the limits of the railroad ROW.

If the structure over the railroad tracks is subject to snow removal, one of the following must be provided: barrier rail with height not less than 42 in. or a snow fence or splashboard extending to the limits of the railroad ROW. Splashboards shall be included in the cost of Fence Chain Link (Special).

Some local electrified lines require arc flash shielding at the bottom of concrete girders. Coordinate with the Owner's design standards for protective shielding details and grounding requirements.

2.5.6 Collision

Refer to the *AREMA Manual for Railway Engineering* or local railroad guidelines for heavy construction piers and crash wall requirements. Criteria regarding vehicle and railway collision loads on structures found in AASHTO are also applicable to the design of crash walls, as appropriate.

2.6 INSPECTION ACCESS

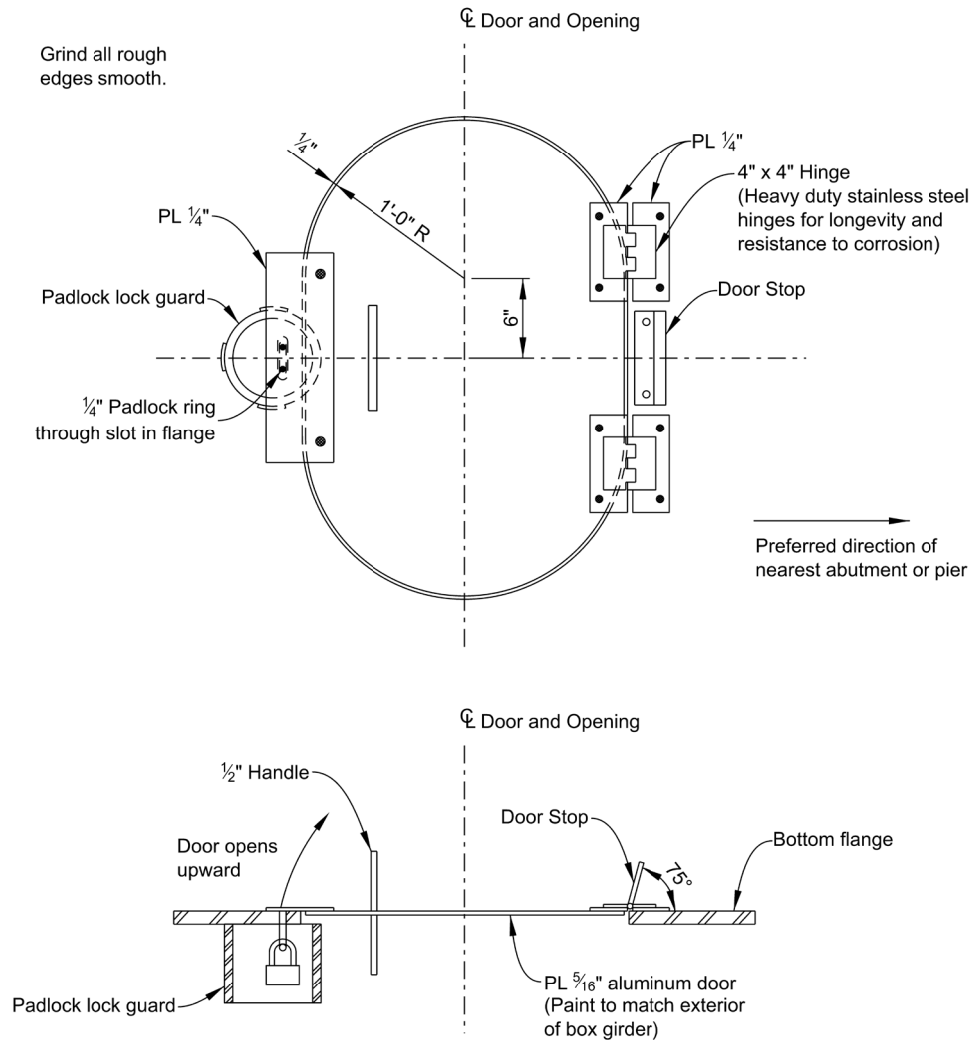
All bridge girders, bearings, external tendons, and fracture critical details shall be made accessible for long-term inspection from the ground, from walkways installed within the girder bays, or by means of a Below Bridge Access Vehicle (BBAV). Areas that are to undergo inspections shall be provided with handles and ladder stops as applicable. Bridges requiring BBAV access shall have a minimum clear distance of 12 ft. between the outside edge of barrier and any obstruction (building, parallel structure, etc.) for access. Bridges accommodated by regular inspection vehicles need 6 ft. minimum, up to 10 ft. preferred of lateral shoulder clearance.

All steel box girders, cast-in-place concrete box girders and precast concrete tub girders with an inside depth of 5 ft. or more shall be made accessible for interior inspection. Bottom flange or slab access doors shall swing into the girder and, when possible, shall be placed at locations that do not impact traffic under the bridge. Lock protectors, doorstops, and tie off hooks inside the girders shall be provided. Steel box girders, cast-in-place concrete box girders and precast concrete tub girders without access shall have 4 in. (minimum) diameter weep holes for camera access at 10 ft. maximum spacing.

Access doors into the girder shall be aluminum, providing a 2 ft. by 3 ft. minimum opening, and shall open to the inside of the box girders. The doors shall be locked by a single padlock protected by a lock guard. Neither bolts nor screws may be substituted for the padlock. An example access door for steel box girders is shown on Figure 2-9 and on Staff Bridge Worksheet B-618-2 for concrete box girders.

To prevent corrosion between the aluminum door and the adjacent steel, the plans should call for shop coating, as a minimum, of the aluminum to steel surfaces on painted girders. The Designer may call for rubber shims at the interfaces with unpainted ASTM A588 steel if desired.

For payment, the aluminum plate should be included in the work for the girder. It should not receive a separate pay item. The plans should call for ASTM B209 aluminum plate, alloy number 6061-T6. Additional material specifications are not needed.



ACCESS DOOR DETAIL

Door shall be aluminum ASTM B209 alloy No. 6061-T6.
 Other hardware and plates are ASTM A36 steel.
 Door and associated hardware to be included in
 Item 509 Structural Steel.

Figure 2-9: Access Door Detail

Traffic, required ladder heights or BBAV reaches, and other obstacles shall be considered when locating access doors. Where possible, access doors near abutments should be placed 3 ft. minimum to 4 ft. maximum clear from top of ground to allow entry without a ladder. Where a ladder must be used above slope paving, support cleats or level areas for the ladder shall be provided in the slope paving.

Access through diaphragms within boxes shall be provided by openings with a minimum area of 5.70 ft² and a minimum width dimension of at least 24 in. The bottom of the opening through diaphragms within boxes shall not exceed 2.50 ft. from the bottom of the girder unless details for passing through higher openings are provided; for example, step platforms or climbing handles up the side of the diaphragm, and, if necessary, along the bottom of the deck. If possible, the bottom of diaphragm access should be flat for ease of use.

Attachments to diaphragms, such as bearing stiffeners, and to other possible projections shall be detailed so that they will not present a hazard to someone passing through the box. Using k-type bracing shall provide an opening through steel box girder intermediate cross frames.

2.7 FORMWORK

All internal formwork, waste, and debris shall be removed from precast and cast in place girders that are made accessible for internal inspection by means of an access door or a camera. For shallow cast-in-place box girders with no access door, pour should occur in two stages to allow formwork removal (unless approved by Unit Leader.)

A note shall be placed on the plans to phase the construction and remove internal formwork for both cast in place and precast girders that require internal inspection.

2.8 UTILITIES

A request for permission to attach utilities to existing bridge structures should be coordinated through the District Utility Engineer, who should submit the request, in writing, to Staff Bridge. Such requests shall state the following:

**AASHTO
2.5.2.5**

- Proposed schedule for installation
- Location of the conduits
- Type of conduit sleeve required
- Size, spacing, capacity, and number of inserts

When attending the FIR meeting, the Designer should inquire as to what utilities and conduits for future use the bridge will carry to assure that they are accommodated. The Utility Group/coordinator should provide information on the size and number of conduits needed for proposed utilities as well as required future or spare utility conduits. The bridge plans shall indicate the size, spacing, and capacity of the utilities and the basis of payment for installation. The Designer shall verify and show the locations of pull boxes and j-boxes to allow future use. Pull boxes or other method shall be provided for all utility lines. See BDM Section 2.3.2 for the typical size of Xcel pull boxes. Unless utilities provide more defined guidance, pull boxes shall be provided every 150 ft. in length or 360 degrees of turns of the conduit. Buried conduit is to have 500 ft. of length between pull boxes. Each utility may have further guidance. The Designer shall coordinate with the lighting and utility discipline for additional requirements.

Utilities should be installed either inside the concrete barrier or underneath the bridge deck with the blockouts provided through abutments and pier diaphragms. CDOT prefers to install small utility conduits inside the barrier whenever practical.

For aesthetic and safety reasons, conduits on new bridges will not be permitted to be installed under deck overhangs or on bridge railings.

Installation of utilities on bridges in service shall be coordinated with Unit Leader. Typically, spare conduits are used when utilities are added while a bridge is in service. If spare conduits are not available, the Vendor will need to provide anchoring details for approval by Unit Leader. Details will need to be evaluated for any detriment to longevity or durability of the existing bridge.

Whenever utilities are installed externally, hanger or support spacing will depend on the size and material of piping supported, e.g., 2 in. PVC conduit may require 5 ft. ± spacing, while 2 in. steel conduit may require 10 ft. ± spacing. Spacing shall be designed to limit deflection to less than ½ in. The utility owner should specify spacing, and it shall be coordinated with Staff Bridge. Information about weight of piping and heaviest conductor can be found in the National Electrical Code.

Blockouts shall be sized to accommodate only those utilities to be installed during bridge construction. Blockouts for the installation of "future" utilities shall not be provided. Blockouts shall not extend below the bottom of the girders. It is preferable to avoid utilities with rigid pipes through integral abutment. When such installations cannot be avoided, the effects of the abutment backfill settling and the effects of superstructure translational and rotational movements need to be considered in the design and properly detailed.

Waterlines, gas lines, and other safety issue utilities shall not be located within tubs or boxes unless approved by Unit Leader. If a waterline is approved for use inside tubs or boxes, relief or drainage valves shall not be located within the girder and a full length casing for the utility is preferred.

ITS utility boxes need to be coordinated if a sound wall will inhibit access.

2.9 FOUNDATION INVESTIGATION

2.9.1 General

Geotechnical explorations will meet requirements of AASHTO Table 10.4.2-1 and the CDOT [Geotechnical Design Manual](#). The proposed subsurface investigation, including means and methods, should be disclosed and fully

discussed with the Structural Engineer of Record prior to start of work. Typical boring spacings are every 200' for walls and a minimum of one boring at each bridge bent depending on the consistency of the geologic stratum.

AASHTO 10.4

2.9.2 Geotechnical Report Requirements

Minimum requirements for the Geotechnical Report deliverables for bridges, retaining walls, and box culverts, as well as the Geotechnical Report Checklist, designed to assist a reviewer, can be found in the CDOT Geotechnical Design Manual.

AASHTO 10.4

2.9.3 Code

All geotechnical design information shall be provided in LRFD format. Preliminary design may be provided in Allowable Stress Design (ASD) format but shall not

be used for final design. Exceptions can be made for bridge widenings where the original design was done in LFD.

2.9.4 Global Stability

Stability requirements, particularly global stability of walls and tall wall abutments, shall satisfy the requirements of the Geotechnical Design Manual and AASHTO. The Geotechnical Engineer of Record shall perform the overall global stability calculations. Structural Engineer of Record is to verify that these calculations are completed.

Loss of support due to erosion of riprap layers, soil removed during design and extreme scour events, pavement structure replacement (wearing surface and base course layers), future utility excavations, etc., should be considered in design.

2.9.5 Deliverable

Final sealed Geotechnical Reports for all new structures shall be provided to CDOT Staff Bridge. Preliminary foundation recommendations should be provided when possible.

2.10 STRUCTURE SELECTION REPORT

2.10.1 General Requirements

The Structure Selection Report presents the results of the preliminary design process and represents good stewardship of available funding. To find feasible solutions, constraints such as serviceability requirements (deflection, settlement, etc.) and spatial limitations (ROW, underground easement, etc.) should be defined as comprehensively as possible. All solutions shall be evaluated, compared or discussed regarding feasibility, advantages and disadvantages. Although the selection report provides overall project requirements and restrictions regarding the applicable structures, the primary purpose is to document the possible structures and recommend the optimal structure to meet project requirements. Cost savings should be analyzed based on a project total cost, not a bridge cost specifically. For example, a \$1 million savings in the bridge should not lead to a \$2 million increase in project costs. Ideally, structures with the highest rank should be adopted for detailed design, and the rest can be used as design alternatives. The Structure Selection Report shall document, justify, and explain Project Structural Engineers' structure layout and type selection. If the Designer anticipates the need for the refined method analysis, this should also be documented in the Structure Selection Report (refer to Section 4.1 for information on refined analysis requirements).

If the structure selection process indicates two options are not definitive in the recommended solution, two designs may be shown in the bid package. Providing two options as an alternative provides more competition in the bidding process, as an example, concrete vs steel or precast concrete vs cast-in-place concrete. A Project Special Provision will need to be included in the specifications. Coordination with Unit Leader should be performed and approval obtained prior to proceeding with this option in addition to a discussion included in the Structure Selection Report.

For projects with multiple bridges or structures, common material may be summarized for the project as a whole and need not be repeated in each Structure Selection Report unless there are bridge or structure-specific differences.

The Structure Selection Report for all structures shall be submitted to CDOT for review and comment by the Project Design Team. For structures that are part of Federal-Aid projects or National Highway System Projects, a Structure Selection Report shall also be submitted to the FHWA Division Bridge Engineer. Allow at least two weeks before the FIR meeting or as scheduled otherwise for report review in the project schedule.

Appendix 2A includes the Structure Selection Report Checklist that shall be used as a general guideline for Designers as to what topics to consider when writing Structure Selection Reports. This list may not be all inclusive for topics that affect the structure selection. If items are not applicable that may be left from the report and the overall report shortened.

Staff Bridge Unit Leaders or designees are to use the checklist during their QA process. After the process is completed, the Staff Bridge Unit Leader will sign the provided Structure Selection Report QA Checklist to acknowledge approval and to document in writing an acceptance of the recommended structure type, layout, and all design deviations from CDOT Structural Standards. This should be done before FIR documents are submitted to the Region. The structure type in final design should match the Structure Selection Report. Otherwise, amendment to the report or a revised report shall be submitted before FOR for approval.

Structure selection includes the following steps:

1. The Design Team evaluates all feasible alternatives through discussion, tables, supporting drawings, etc., and prepares the Structure Selection report. It is recommended to meet and discuss the bridge with Staff Bridge and Region representatives.
2. Report undergoes QA/QC procedure before being submitted.
3. The Design Team submits the Structure Selection Report to the Unit Leader for review. Unit Leader performs review of the Report and signs off on the Structure Selection Report QA Checklist to acknowledge approval.
4. Structure Selection Report is submitted to the Region and to the FHWA Division Bridge Engineer (if applicable) for review and acceptance.
5. The Design Team updates Structure Selection Report as required per final geotechnical and hydraulics reports.

2.10.2 Major Structures

The definition of the term *Major Structures* is found in the Policies and Procedures section of this BDM.

2.10.2.1 Bridges

Different span arrangements and appropriate superstructure types should be evaluated and findings presented in the Structure Selection Report. Site conditions, phasing, bridge length, and required minimum horizontal and vertical clearances will influence most decisions. The following are other factors that shall be considered during the preliminary design phase:

- Construction cost
- Life cycle cost
- Possible future widenings
- Ultimate roadway section below
- Capacity of girders during phase construction
- Speed of construction and maintenance

Refer to Appendix 2A, Structure Selection Report Checklist, for more criteria to be considered.

Adherence to the span-to-depth ratios in accordance with AASHTO Table 2.5.2.6.3-1 (Traditional Minimum Depth for Constant Depth Superstructures), is not required but the table is a good starting point for preliminary design.

**AASHTO
2.5.2.6.3**

In the Structure Selection Report, the Designer shall evaluate, confirm, and document the stability of the existing bridge when it is used in a partial width configuration as part of the new construction phasing. A separate rating may be required for the configuration of the existing bridge during phased construction to verify sufficient load capacity.

2.10.2.2 Culverts

A culvert is considered a major structure if its total length is greater than 20 ft. measured along the center of the roadway between the inside faces of the outside walls or spring lines of arches. It may also include multiple pipes where the clear distance between the centerlines of the exterior pipes, plus the radius of each of the exterior pipes, is 20 ft. or more.

A culvert is used in lieu of a bridge based on estimated construction and maintenance costs when viable hydraulically. Culverts may have a lower initial cost due to foundation conditions, shorter spans and shallower than bridges. In general, culverts have less aesthetic value and are potentially more damaging to streams than bridge structures, but much more durable and maintenance and inspection friendly. These factors should be considered when making a decision to choose a culvert over a bridge. Section 5.4.13 of this BDM outlines culvert design criteria.

2.10.3 Minor Structures

The Structure Selection Report for minor structures shall be provided with applicable sections. The definition of the term *minor structures* is found in the Policies and Procedures section of this BDM.

2.10.4 Wall Structures

The definition of the three categories of *walls*: retaining walls, bridge walls, and noise walls, is found in the Policies and Procedures section of this BDM.

2.10.4.1 *Retaining Walls*

The following considerations may affect the selection of a wall structure:

- Construction cost
- Spatial constraints
- Behavior constraints
- Constructability
- Maintenance
- Schedule
- Aesthetics (Corridor requirements)
- Environmental concerns
- Durability
- Scour Potential
- Available standard designs

The selection process shall be documented as evidence to support the decision. The wall Structure Selection Report shall be a stand-alone report with a cover letter, and site plan clearly indicating the names and locations of the walls.

For walls that support a highway and are affected by scour, the selection report shall document the cost of achieving stability for the 100 & 500 year scour compared to replacement cost. Structural engineer should discuss with the region for alternate criteria and resiliency requirements.

2.10.4.2 *Bridge Walls*

Bridge wall selection may be included in the Structure Selection Report for the bridge and not as a separate report if there are no additional retaining walls on the project. Selection considerations are the same as listed for retaining walls.

2.10.4.3 *Noise Walls*

Noise walls require a Structure Selection Report. Noise Wall discussions can be a standalone document or included in the Environmental Concerns portion of the Bridge or Wall Structure Selection Report. Refer to Chapter 18 of the CDOT Roadway Design Guide for additional noise wall requirements and discussion.

2.10.5 Overhead Sign Structures

Overhead sign structures do not require a Structure Selection Report unless requested by the Project Manager.

2.10.6 Tunnels

The definition of the term tunnels is found in the Policies and Procedures section of this BDM.

2.10.6.1 Tunnels

Tunnels can typically be constructed with several different methods such as: bottom up or cut and cover, top down, and use of boring machines. The Structure Selection Report shall evaluate the various methods of construction and any other criteria that may affect their design, maintenance, and construction. Site conditions, phasing, span, length, and required minimum horizontal and vertical clearances will influence most decisions. The following are other factors that shall be considered during the preliminary design phase:

- Construction cost
- Life cycle cost
- Possible future widenings
- Phase construction impacts
- Speed of construction and maintenance
- Construction methods
- Emergency egress
- Need for air recirculation

2.10.7 Accelerated Bridge Construction

The Accelerated Bridge Construction (ABC) design and construction method uses several technologies to facilitate accelerated construction, such as rapid embankment construction, prefabricated bridge elements, various structural placement methods, fast track contracting, etc. This method of design and construction usually results in an overall decrease in construction time when compared to the historic construction methods used to build bridges. The ABC Matrix shall be evaluated and included in the Structure Selection Report for all structures. For more details, refer to Section 39 of this BDM and to the [FHWA Accelerated Bridge Construction Manual](#).

2.10.8 Life Cycle Cost Analysis

The structure selection process may consider the life cycle cost analysis (LCCA), which tracks cost values that cover the full cycle of the structure from the initial design to the end of the analysis period. The Designer shall assume all new bridges will last 100 years if all requirements are followed. Since approximately 1990, CDOT has been performing the LCCA and has tracked many cost factors. The following represent some of the factors engineers should determine:

- Design cost
- Construction cost
- Traffic control cost
- Maintenance cost
- Rehabilitation cost
- User cost

For recommended default cost values to be used for CDOT projects, refer to the latest [Cost Data](#) books published by CDOT and available online. For appropriate interest rate values refer to the latest CDOT pavement design manual available online.

2.10.9 Aesthetics

Aesthetic value shall be evaluated in a structure selection process for high profile structures and structures with corridor aesthetic requirements.

2.11 HYDROLOGY AND HYDRAULICS

2.11.1 Drainage Report Requirements

Hydraulic analysis and the Drainage Report shall meet requirements of AASHTO and the CDOT [Drainage Design Manual](#). The format of the Drainage Report is expected to vary based on a project's needs.

2.11.2 Scour

Scour shall be considered when designing any structure located in a streambed or impacted by streamflow. All bridges should be designed to withstand 100-year and 500-year storm scour events without failing. Design for 100-year storm shall be performed at the service and strength limit states, and 500-year storm scour shall be considered only for the extreme event limit state analysis. The General Layout and Hydraulics sheets shall show scour limits, elevations, and velocities of these storm events. If the 500-year flow would overtop the structure, the Designer should determine the appropriate AASHTO loads and groupings to apply during the stability analysis.

Walls that support the highway should be designed to withstand 100-year and 500-year storm scour events without failing unless approved by CDOT Resident Engineer/ Region. Region will evaluate the resiliency requirements for retaining walls.

Based on FHWA's model study, in instances where neither contraction scour nor general degradation is expected to be significant, there is no benefit to be gained from reducing local scour by placing the top of the footing supported by piles at an elevation other than flush with the streambed. As a rule, the disturbance of the streambed beyond the level described herein is discouraged.

Where substantial scour is predicted, the piles with pile caps may be designed to place the top of the pile cap below the estimated contraction scour depth where practical.

**AASHTO
2.6.4.4.2**

In general, spread footing foundations shall not be used for stream crossings. However, when shallow scour-resistant bedrock is present, spread footings may be considered as a foundation option provided they are embedded 6" min. into the bedrock. When considering this approach, Designers should consult with the project geotechnical and hydraulic engineers to evaluate the suitability of the bedrock present and get written approval from Unit Leader. When spread footings are placed into rock the sides of the footing should not be formed and then backfilled but should be placed to the rock.

Outlet Scour Protection and Roadway Overtopping & Revetment for culverts, which is covered in the CDOT Drainage Design Manual, is a hydraulics design issue and uses different criteria and definitions than typical bridge scour. The Structural Designer should coordinate with the hydraulic designer to make sure adequate requirements are met.

2.11.3 Deck Drainage Requirements

All bridges shall be investigated for drainage requirements. Bridge deck shall be kept watertight and deck drains should be placed at the interval required by design to intercept water surface and keep it away from expansion devices and bearings. Special attention for deck drainage is needed for decks with super elevation transitions. The FHWA publication, Design of Bridge Deck Drainage, Hydraulic Engineering Circular No. 21 (HEC-21) (Publication No. FHWA-SA-92-010, May 1993), shall be used for the design of bridge drainage systems. The hydraulic design frequency shall be 5 years rather than the frequencies specified in HEC-21. The structural engineer shall coordinate with the Hydraulics Engineer and Environmental Scientist to create appropriate details and required spacing of drains as needed to meet their requirements.

Water exiting bridge drains shall not flow onto girder flanges, bearings, pier caps, abutment caps, roadways, railroad templates, or pedestrian/bikeways. Pipe drains, scuppers, and grated inlet drains shall extend below bottom of girders to assure that drainage is kept off steel girder flanges. If possible, drains should not be positioned above riprap. When drains must be placed over riprap, special filter fabric shall be placed under the riprap. This filter fabric shall be highly permeable and non-biodegradable. The bridge designer should coordinate with the Hydraulics Engineer and show an appropriately sized energy dissipater at the bottom of the bridge drain system to minimize scour.

Curb drains and pipe drains require approval from the CDOT Environmental Department. When allowed, curb drains shall provide a continuous curb for wheel impact. When allowed, pipe drains shall have a minimum diameter of 8 in. and internal grates 2 in. below the surface or be covered by a grate designed for 16 kip wheel load. Inlet grates shall be removable for cleaning. Project-specific details shall be included.

Approach slab drains shall be provided on the high side of expansion devices located at the end of approach slabs. The purpose of the approach slab drain is to minimize flow over the joint. The approach slab drain should be detailed such that the approach slab drain is not affected from the anticipated bridge movement. The location and size of the approach slab drain shall be designed and coordinated with the roadway engineer and hydraulic engineer.

When a drain is placed within the limits of the sidewalk, it shall be pedestrian and bicycle friendly.

Cleanouts shall be added to any closed pipe run to facilitate easier cleaning by maintenance. These should be reviewed at FOR by maintenance personnel for concurrence with the detailing and locations.

2.12 BRIDGE SECURITY

The Structure Selection Report will include discussion and recommendations on providing security measures for all Major Structures defined by CDOT and FHWA as structures with national importance and needs for protection. The Designer will coordinate with Staff Bridge at the preliminary phase of the design to develop both operational and engineering solutions to the proposed security measures

and to ensure that security solutions will be met in design, construction, and operation stages.

2.13 APPROACH SLABS

Approach slabs are used to improve rideability and mitigate problems with settlement of the bridge approaches relative to the bridge deck and shall be provided on all vehicular bridges, except as noted below, or unless approved by State Bridge Engineer.

Concrete approach slabs are not required on bridges with GRS abutments that do not have an expansion device, as differential settlement between abutment and roadway approach is not expected to be significant. Asphalt pavement approach should be installed to allow minor grade corrections.

Approach slabs are not required on pedestrian bridges unless the Owner requests them.

The Designer should evaluate the use of approach slabs on concrete box culverts with no or minimal fill cover based on settlement concerns. An alternate may be to utilize MSE fill in the roadway section adjacent to the culvert to deal with possible settlement issues.

In all cases, the concrete approach slab shall be anchored to the abutment. Approach slab notches shall be provided on all abutments, even if an approach slab will not be placed with the original construction (see Section 11 of this BDM for details). Refer to BDM Section 14 for expansion joint requirements.

Elevations of the approach slabs shall be coordinated with the roadway approaches to avoid misalignment and must be provided in the plans in accordance with Section 2.2.1.3 of this BDM.

Roadway drains shall be placed in approach slabs to prevent flows across the expansion joint. Bridge designer shall coordinate with hydraulic engineer to determine location, number and size of drain inlets.

2.14 PIGEON PROOFING

Bridge areas with inspection requirements (such as bearings, abutment and pier caps) and roadway/pedestrian areas (such as utility pipes above pedestrian trails and sidewalks) should be protected from bird droppings when requested by the region or Staff Bridge. Methods to minimize potential pigeon roosting and nesting areas include plates, grating, nets, spikes, electric systems, and wires. Bird control and nest removal shall be taken into consideration when planning long-term maintenance and inspections.

2.15 SPREAD FOOTING EMBEDMENT

Bottoms of spread footings shall be embedded below the local or regional frost depth, with a minimum embedment of 3 ft.

2.16 DAMAGE AVOIDANCE DETAILS

CDOT Structural Worksheets and practice contain many details categorized as "Damage Avoidance Details" (DAD) or best management practices. In most

cases, these details are intended to provide added durability to the structures, but in some cases they are required for design considerations as well. The minimum requirements are shown on the current worksheets published on the Staff Bridge website. As with any worksheet, designers may elect to improve on some of the shown details for project-specific requirements, but some changes will require Unit Leader and State Bridge Engineer approval as noted in the worksheets or within this Design Manual.

Common damage avoidance details such as the drip groove at the edge of the deck and wall copings are intended to minimize water damage and shall not be revised. The coping details shown in the worksheets for MSE walls in tight ROW situations are intended to deal with trickle flows and increase durability. This detail may be revised, especially if water is kept away from the walls using the preferred method of separate ditches and swales. The two longer straps at the top of MSE walls are a damage avoidance detail primarily for impact loads but also serve for seismic purposes. The detailing of the rail anchor slab over the MSE wall facing is a design detail intended to separate impact loads from the wall facing. Wall copings are primarily intended to stabilize block walls during seismic events. Shiplap panel joints with fabric backing are used to accommodate settlement issues as well as seismic issues. Geomembrane over the MSE prism has the dual purpose of controlling design loads and facing durability. The FRP Bar or stopper at the base of precast panel walls is a seismic detail. The use of a concrete footer for MSE wall facings is another damage avoidance detail that will require approval to remove.

2.17 DISSIMILAR METALS

Dissimilar metals in contact with each other shall be avoided if possible. Electrolytic isolation shall be provided to prevent contact of dissimilar metals. Dissimilar metal-to-metal or aluminum-to-concrete post or rail installations shall have contact surfaces separated by an approved protective coating. Protective Coating shall be approved by Corrosion Systems SME. Asphaltic paint shall not be permitted to remain on surfaces to be exposed or to receive a sealant or paint.

APPENDIX 2A - STRUCTURE SELECTION REPORT QA CHECKLIST

(An interactive PDF Form version of the
Checklist is available on CDOT website)

Structure Selection Report QA Checklist

This checklist is to serve as quality assurance of the structure selection process. The sections in the report need not be in the same order as this checklist. This checklist must be signed by Staff Bridge Unit Leader or designee prior to submittal of FIR documents to the Region.

Structure Number(s): _____

Cover Sheet

- Name of the Project and Site Address
- Structure Number(s)
- Property Owner Name and Contact Information
- Report Preparer Name and Contact Information
- Submittal and Revision Dates as Applicable

Executive Summary

- Project Description
- Structure Recommendations

Site Description and Design Features

- Existing Structure(s) N/A: _____
- Vicinity Map N/A: _____
- ROW Impact N/A: _____
- Traffic Detour N/A: _____
- Constructability & Construction Phasing N/A: _____
- Utilities N/A: _____
- Geotechnical Summary N/A: _____
- Hydraulics Summary N/A: _____
- Environmental Concerns N/A: _____
- Roadway Design Features
 - Cross Section
 - Vertical Alignment
 - Horizontal Alignment

Structural Design Criteria

- Design Specifications
- Loading N/A: _____
 - Collision Load
 - Earthquake Load
- Deck Drainage N/A: _____
- Aesthetic Requirements N/A: _____
- Possible Future Widening N/A: _____

Structure Selection

- Selection Criteria
- Rehabilitation Alternatives N/A: _____
 - Inspection Summary
 - Load Testing Requirements N/A: _____

Add figures/sketches to the following sections as needed:

- Structure Layout Alternatives
 - Vertical Clearances
 - Horizontal Clearances
 - Skew

- Span Configurations
- Superstructure Alternatives N/A: _____
 - Concrete Girder Alternatives
 - Steel Girder Alternatives
 - Deck Drains
- Substructure Alternatives N/A: _____
 - Abutment Alternatives (GRS, Integral, Semi-integral, etc.)
 - Pier Alternatives
- Wall Alternatives N/A: _____
- Constructability & Construction Phasing
- ABC Design (include pre-scoping ABC rating results from spreadsheet found on the CDOT website)
- Maintenance and Durability
- Corrosive Resistance
- Summary of Structure Type Evaluation Table
- Construction Costs (including costs of alternatives)

Other

Figures and Appendices

- Alternative Typical Sections (if not provided in the report)
- General Layout of the Selected Structure
- Summary of Quantities and Cost Estimate Tables

List of Variances

Requested Variance: _____

Approved? Yes No

Requested Variance: _____

Approved? Yes No

Requested Variance: _____

Approved? Yes No

If you need more space, use an additional sheet(s) of paper.

CDOT Staff Bridge Quality Assurance Sign-off

By signing this checklist Staff Bridge Unit Leader acknowledges approval of the Structure Selection Report findings, recommendations, and all design deviations from the CDOT Structural Standards and design criteria.

Print Name

Signature

Date

SECTION 3 LOADS AND LOAD FACTORS

3.1 GENERAL REQUIREMENTS

The following section is provided as CDOT practice for loads and load factors. The Designer shall coordinate with Staff Bridge regarding project-specific circumstances warranting deviations from standard practices referenced herein.

This section is complementary to the current CDOT Bridge Rating Manual, CDOT Bridge Detailing Manual, CDOT Standard Specifications for Road and Bridge Construction, and current Bridge Structural Worksheets.

3.2 CODE REQUIREMENTS

Unless otherwise modified by this section, the minimum requirement for loads and load factors shall be in accordance with Section 3 of AASHTO. This section of the BDM is intended to supplement AASHTO code requirements. Any requests to vary from methodologies presented herein will be discussed with Staff Bridge.

3.3 CONSTRUCTION LOADING

Construction loads act on the structure only during construction and are often not accurately known at the time of design. If specific construction loads have been assumed as a part of the design, these loads shall be documented in the plans. Otherwise, the Contractor's Engineer shall determine the magnitude and applicability of construction loads and provide falsework and temporary supports as necessary to ensure the stability and constructability of the structure during construction.

**AASHTO
C3.4.2.1**

Transient construction loads shall meet all legal load limits or be approved by CDOT's permit office for both new and existing structures.

3.4 DEAD LOADS

3.4.1 Stay-in-Place Metal Deck Forms

In accordance with Section 9.13.3 of this BDM, form flutes shall not be filled with concrete. A minimum of 5 psf (non-composite) shall be used to account for stay-in-place metal deck forms, when they are allowed.

3.4.2 Wearing Surface

The following unit weight shall be used in the design of CDOT structures:

Asphalt Unit Weight: 146.67 lb/ft³

This unit weight results in 36.67 psf for 3-inch asphalt overlays. This unit weight is equivalent to the roadway standard of using 110 pounds per square yard per inch of thickness for quantities.

3.4.3 Utilities

Utility loads shall include the dead load of both the basic utility and all connections, supports, casings, and other required appurtenances.

Waterlines carried in a casing shall be evaluated at the extreme event level for the potential of waterline failure, resulting in the casing being filled with water.

An allowance of 5 pounds per square foot of composite load shall be included for new bridges within urban areas to account for future utilities. For rural bridges, the potential for future utilities should be discussed with the Local Agency and the CDOT Project Manager. Refer to Section 4.4 of this BDM for distribution of utility loads.

3.4.4 Girder Concrete

3.4.4.1 Concrete Unit Weight

The unreinforced concrete unit weight for use in calculating dead loads shall be 145 pcf per AASHTO Table 3.5.1-1.

- For reinforced CIP concrete, a minimum of 5 pcf is added to the unreinforced weight to account for reinforcing which results in the typical 150 pcf.
- For shop produced precast girders, a minimum of 5 pcf shall be added to the unreinforced weight to account for reinforcing. The unreinforced weight for load purposes shall be calculated per AASHTO Table 3.5.1-1 using the final girder strength f_c . For Class PS concrete to account for variability in actual concrete strength, a minimum unit weight of 155 pcf shall be used.

3.4.4.2 Weight of Curved Precast U Girders

The Designer is responsible for accounting for the increased self-weight due to inside faces of webs being chorded for curved precast U girders. The Designer should confer with local suppliers concerning the inside web form geometry required for specific project parameters.

3.5 COLLISION LOAD

3.5.1 Policy

CDOT structures shall be evaluated for Collision Force (CT) as detailed in Sections 3.5.2, 3.5.3, and 3.5.4 of this BDM. In certain cases, structures may be deemed exempt from CT loads based on the criteria within the commentary of AASHTO 3.6.5.1, including Equation C3.6.5.1-1 and Table C3.6.5.1-1. Exemption from CT loads will be allowed only with Unit Leader approval in coordination with the State Bridge Engineer and should be documented in the Structure Selection Report.

3.5.2 New Bridges

The preferred strategy for new bridges is to meet the clearance and protection requirements set forth in AASHTO. Exposed supporting elements of new bridges that can be hit by errant or oversized vehicles shall be designed for a

AASHTO
3.6.5

Collision Force (CT) of 600 kip. The application shall be in accordance with AASHTO.

This design criterion typically applies to pier columns and non-redundant through type superstructure elements, such as through trusses or through arches.

Columns subject to train impact shall be designed in accordance with the *AREMA Manual for Railway Engineering* and the *UPRR/BNSF Guideline for Railroad Grade Separation Projects*.

3.5.3 Existing Structures

Existing structures shall be evaluated for CT loads in accordance with AASHTO. The preferred strategy for existing structures is to meet the clearance and protection requirements. If clearance and protection are impractical, the columns shall be evaluated for a CT force of 600 kip. The application shall be in accordance with AASHTO.

**AASHTO
3.6.5**

The Engineer shall consider retrofitting the column system to achieve the required load capacity. The existing foundation should be evaluated, along with the column system, to ensure proper load carrying capacity.

The structure may be alternatively checked for adequate redundancy to resist collapse from the loss of the members that have inadequate strength to resist the CT load. This is done by modeling the structure without the inadequate members, with the structure subjected to a load of at least 1.0 DL and 0.5 LL+I.

3.5.4 Temporary Works

Temporary falsework towers that are within 30 ft. of through traffic shall be designed to resist a 600 kip impact load without collapse of the supported structure or shall be protected by concrete barriers or rigid steel barriers with a minimum 2-ft. shoulder. In cases where loss of the temporary tower would cause collapse of the supported structure the tower shall be protected with a barrier and have a 2-ft shoulder.

The barriers shall have a minimum 2-ft. clear zone of intrusion from the tower to the back face of the barrier. For speeds between 35 mph and 45 mph, the barrier shall either be at least 54 in. tall or have a 10-ft. clear zone of intrusion and be at least 42 in. tall. If the speed is expected to be over 45 mph, if the ADTT exceeds 10,000 vehicles per day, or if the through traffic is railroad or light rail traffic, then the barrier shall have the strength, stability, and geometry required for a TL-5 barrier. Guardrails protecting falsework towers or piers shall continue at full rail height for at least 30 ft. either side of the tower and shall be configured with full height rigid barriers to prevent vehicles from running around the rail end and hitting the tower from the opposite side of the rail. If ends transition into lower approach rails rather than crash cushions or barrels, that approach rail shall be a rigid rail type (such as Type 7) and shall not end for at least an additional 170 ft. This extension of the approach rail prevents a vehicle mounting and straddling a barrier from reaching the tower or pier.

3.6 VEHICULAR LIVE LOAD

Vehicular live load shall be in accordance with AASHTO.

AASHTO
3.6.1.2

Bridges should be designed such that future removal of medians and/or sidewalks is considered in the design and rating of the bridge. Simultaneous loading of the sidewalk dead load and vehicle live load is recommended when barrier separation is not present to cover the likelihood of errant trucks mounting the sidewalks or medians. Pedestrian load need not be applied in addition to the vehicle live load in this case.

Live load factors for Service III shall be in accordance with AASHTO Table 3.4.1-4. See Section 5.5.1 of this BDM for further explanation of applicability of the different live load factors.

The Colorado Permit Vehicle shall be evaluated at Strength II. Figure 3-1 shows the axle weights and axle configuration that represent the Colorado Permit Vehicle. This vehicle is used to determine the Overload Color Code for bridges. It is a moving live load using the same live load distribution factors, number of lanes loaded, and impact factors as the HL-93 truck.

Deck slabs do not need to be designed for the Colorado Permit Vehicle wheel loads.

An operating rating for the Permit Vehicle shall be provided on the Bridge Rating Summary Sheet (see the CDOT Bridge Rating Manual).

Additional design vehicles, such as Specialized Hauling Vehicles (SHVs), Notational Rating Load (NRL), and other legal loads shall be evaluated in accordance with the CDOT Bridge Rating Manual and the *AASHTO Manual for Bridge Evaluation*.

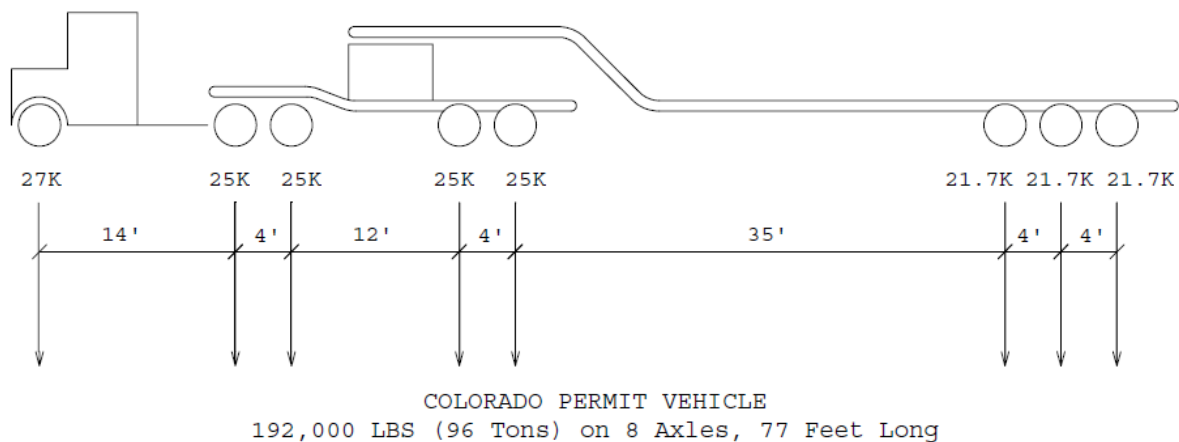


Figure 3-1: Colorado Permit Vehicle

3.7 VEHICULAR LIVE LOAD ON CULVERTS

CDOT considers surcharge from lane loads in the design of box culverts. To maintain consistency with CDOT's M-standards, surcharge loads from lanes shall be applied to the walls and bottom slabs of culverts using the Boussinesq stress distribution.

AASHTO
3.6.1.2.6 and
3.6.1.3.3

Thrust may be considered in the design of box culverts (precast or cast-in-place). If thrust is considered in the design, the rating is to incorporate thrust and design assumptions are to be included within the design plans.

For arch culverts, soil structure interaction with refined analysis shall be used for vehicular load and for identifying positive arch action.

3.8 DECK OVERHANG LOAD

Bridge deck overhangs shall be designed for horizontal loads resulting from vehicle collision in accordance with AASHTO. For deck overhang greater than 1/3 of the girder spacing, special attention shall be paid to shear capacity and concrete screed machine load during deck pour.

AASHTO
3.6.1.3.4

The AASHTO Chapter methodology for determining impact loads on the overhang as shown in the BDM examples is very conservative. Recent research has shown that the combined impact tension load on deck reinforcing at the flowline of concrete barriers may be as low as 14.8 kip/ft for a TL-4 rail. This value would be increased 100% at rail expansion joints. These reduced values are due to torsion and yield line capacities not currently shown in the code.

3.9 CENTRIFUGAL FORCES

For piers and abutments with a pin connection between the superstructure and substructure, centrifugal forces may be assumed to act horizontally at the roadway surface. For piers and abutments with a moment resisting connection between superstructure and substructure, the eccentricity of the centrifugal force shall be considered. Centrifugal forces shall be distributed to substructure elements based on their relative individual longitudinal stiffness.

AASHTO
3.6.3

3.10 BRAKING FORCE

For piers and abutments with a pin connection between the superstructure and substructure, braking forces may be assumed to act horizontally at the roadway surface. For piers and abutments with a moment resisting connection between superstructure and substructure, the eccentricity of the braking force shall be considered. Braking forces shall be distributed to substructure elements based on their relative individual longitudinal stiffness.

AASHTO
3.6.4

CDOT has experienced loss of backfill material (voids) behind abutments of existing bridges due to water intrusion over time. In addition, cyclical temperature movements of bridges may cause gaps between backfill and abutments. Due to these considerations, relying on passive earth pressure behind abutments to resist braking loads is cautioned. If passive earth pressure behind abutments is considered, AASHTO Table C3.11.1-1 should be used to estimate the participation of passive earth pressure relative to pier stiffness.

3.11 FATIGUE LOAD

Due to the uncertainty of future traffic volumes, the maximum ADT per lane of 20,000 vehicles shall be used when evaluating fatigue. In lieu of site-specific fraction of truck traffic data, the values of AASHTO Table C3.6.1.4.2-1 may be applied to obtain ADTT for use in Equation 3.6.1.4.2-1.

**AASHTO
3.6.1.4.2**

3.12 STREAM FORCES AND SCOUR EFFECTS

Stream forces shall be designed in accordance with Section 3.7.3 of AASHTO. Debris raft loads need only be applied on structures within high debris channels as determined by the Hydraulic Engineer.

**AASHTO
3.7.2, 3.7.3,
and 3.7.5**

Scour of bridge foundations should be evaluated at two levels:

- Strength I – Evaluate 100-year scour in conjunction with maximum dead load factors, live load, and stream forces. If the 100-year scour limits undermine beyond the back of abutment, impeding live load from approaching the structure, live load may be reduced.
- Extreme Event II – Evaluate 500-year scour in conjunction with minimum dead load factors and stream forces. Live load may be reduced if the approaches to the bridge are impassable due to scour. The extreme event check should verify that the bridge will not collapse.

All other service, strength, and extreme event combinations need not be checked concurrent with the 100-year or 500-year scour limits.

3.13 SEISMIC LOADING

For bridges and other structures within Seismic Zone 1, the minimum connection requirements of AASHTO shall apply.

**AASHTO
3.10.9.2**

For all other seismic zones, both force-based and displacement-based analysis methods are allowed. A geotechnical investigation must be completed for bridges to determine the site class of the foundation materials. When using Extreme Event I, the load factor on live load should be 0.50. The 0.50 live load factor signifies a low probability of the concurrence of the maximum vehicular live load and the extreme event case.

Seismic analysis is not required for mechanically reinforced earth (MSE) and Geosynthetic Reinforced Soil (GRS) walls if Staff Bridge Structural Worksheets are used. These worksheets contain damage avoidance details such as rail anchor slab/beam, coping, and shiplap panel joints that cannot be revised without approval by Unit Leader in coordination with the MSE Wall SMEs. See Section 2.16 of this BDM for further details.

3.14 TEMPERATURE / THERMAL FORCES

Structures shall be designed for the temperature ranges detailed in Section 14 of this BDM.

3.15 EARTH PRESSURES AND SETTLEMENT EFFECTS

Appropriate earth pressures and predicted settlement should be provided in a geotechnical investigation. The Geotechnical Engineer shall evaluate criteria for settlement periods and potential down drag effects.

Consideration should be given to lateral earth pressures from surcharge loads in accordance with AASHTO 3.11.6, modified on a project-specific basis. For structures that support vehicular live loads within the stated criteria of AASHTO 3.11.6.4, the load factor on the surcharge shall be in accordance with LS in AASHTO Table 3.4.1-1. For walls designed for a nominal surcharge to account for backfilling operations, the load factor on the assumed surcharge may be taken as 1.50. The lower load factor represents the temporary nature of this surcharge effect and reflects the construction load factor in AASHTO 3.4.2.1.

**AASHTO
3.11.6**

A combination of mechanically reinforced earth (MSE) with a non-collapsible void or a gap with low density polystyrene can be considered when reduced earth pressure effects are required.

Settlement shall be evaluated at the service limit state with a load factor of 1.0 applied to all applicable loads. Transient loads may be omitted from settlement analysis.

Effects of abutment settlement on bridges using Geosynthetic Reinforced Soil (GRS) abutments shall be evaluated during the structure selection stage. See Section 11 of this BDM for additional requirements.

3.16 PEDESTRIAN LOADING

Pedestrian load should be considered in accordance with AASHTO and the AASHTO *LRFD Guide Specifications for the Design of Pedestrian Bridges*.

**AASHTO
3.6.1.6**

3.17 BLAST LOADING

The potential for blast loading shall be evaluated and documented during the structure selection process and coordinated with Staff Bridge on a project-specific basis.

**AASHTO
3.15**

3.18 WIND LOADS

Wind loads shall be in accordance with AASHTO. Staff Bridge shall be consulted for structures within special wind regions not covered by AASHTO.

AASHTO 3.8

3.19 FENCE LOADS

Table 3-1 shows the minimum load for which fences on bridges and other structures shall be designed unless site conditions justify a different load condition. Refer to Section 13 of this BDM for additional information. Calculated load values were generated using the *Chain Link Fence Wind Load Guide*, 2007. Snow loads are based on energy momentum equations by calculating the power available from a snowplow moving at 45 to 50 mph to determine the maximum amount of snow that could be continuously thrown. This provides the momentum of the snow thrown per second. Dividing the momentum by time yields the snow impact loads that are shown in Table 3-1.

Table 3-1: Fence Loads

Fence Type	Chain Link Opening	Wind Load	Snow Impact Load*
36" Chain Link splash guard	3/8"	31 psf	96 plf 1'-6" up from bottom of fence
60" Chain Link	1"	14 psf	96 plf 1'-6" up from bottom of fence
68" Chain Link	2"	8 psf	96 plf 1'-6" up from bottom of fence
92" Chain Link	2"	8 psf	96 plf 1'-6" up from bottom of fence

* The required mesh opening for CDOT snow fence is 3/8".

3.20 REFERENCES

Chain Link Fence Manufacturers Institute. 2007. *Chain Link Fence Wind Load Guide*.

SECTION 4 STRUCTURAL ANALYSIS AND EVALUATION

4.1 GENERAL REQUIREMENTS

Section 4 discusses the preferred methods of structural analysis, design, and evaluation of bridges. The section is limited to the modeling of structures and the determination of member stresses, forces, and deformations. The primary analysis goals for the Designer are to satisfy force equilibrium and to identify a load path to adequately transfer the loads to the foundations.

Bridges are to be analyzed in accordance with AASHTO 4.5.2.2, except for extreme limit states or with approval from Unit Leader in coordination with the State Bridge Engineer.

In most cases, the Designer should use simple models using distribution equations from AASHTO and reasonable assumptions. Complex structures may require refined analysis, but refinement should not be used unless necessary. Any cost savings realized by refined analysis may be negated by the additional efforts needed for the independent design check and the rating. Each bridge design must consider the need for a satisfactory bridge rating, further supporting the need for simpler, more straightforward calculations versus refined analysis.

4.2 CODE REQUIREMENTS

AASHTO lists multiple acceptable methods of analysis options, allowing the Designer to choose their preference. Staff Bridge does not require, prefer, or forbid any specific method. The Designer must be knowledgeable about the design specifics and the analysis parameters of the chosen approach.

The Designer must validate all computer software before it is implemented into the design. Using a software program does not relieve the Designer of the responsibility to properly apply and interpret results. Staff Bridge does not support a preapproved list of software but reserves the right to disallow any software on a regular or case-by-case basis. A list of specialized software shall be noted in the Structures Selection report and shall be approved by the Unit Leader in coordination with the Software SMEs.

4.3 MODELING METHODS

AASHTO allows the contribution of continuous composite barriers in service and fatigue limit states for the calculation of the structural cross section of the exterior girder. Staff Bridge's preference is not to use the composite section for new designs, but these sections may be considered in the evaluation or design for rehabilitation. The Designer should not consider continuous composite barriers in section properties without approval from Unit Leader in coordination with the Bridge Rail SMEs.

Uplift at bearings is not allowed unless approval is obtained from Unit Leader. Hold downs or anchorages are required if uplift is permitted in the design. There may be additional requirements for bearings when uplift is permitted, as outlined in Section 14 of this BDM.

Calculations are to follow a clear and detailed process. Spreadsheets should show all equations, assumptions, design parameters, and references. When modeling integral abutments, the Designer is to model the connection between the superstructure and the substructure as a pin connection. The reason for this is that integral abutments are not intended to transfer moment from superstructure to substructure. Modeling the connection this way prevents moment from being transferred into the substructure elements and eliminates the need for negative moment design at the deck level.

Time-dependent material effects shall be modeled as outlined in Section 5 of this BDM. Using code prescribed equations for these effects will account for the impacts of creep, shrinkage, and relaxation.

Redistribution of moments in continuous bridges is allowed.

Unit Leader must review and approve non-standard resistance factors for unique materials prior to implementation.

Staff Bridge allows the use of cracked section properties in the analysis of both superstructure and substructure. The Designer should be aware that in some situations the use of 0.5 value for γ_{TU} , γ_{CR} , and γ_{SH} load factors no longer applies in conjunction with cracked section properties.

AASHTO 3.4.1

When using moment magnification, the calculations shall follow AASHTO.

**AASHTO
4.5.3.2.2**

4.4 DEAD LOAD DISTRIBUTION

Non-composite dead load should be distributed to the girders based on tributary width for straight bridges. Non-composite dead load on curved I-girders may be distributed uniformly to all girders, as long as intermediate diaphragms or cross frames are provided and have been designed as primary members per AASHTO. CDOT allows composite dead loads to be distributed evenly to all girders; however, the Designer must use engineering judgment in determining the distribution of heavier concentrated line loads such as utilities, parapets, sidewalks, barriers, etc.

**AASHTO
C4.6.1.2.4b**

4.5 LIVE LOAD DISTRIBUTION

Theoretically, live load distribution factors (LLDF) change for each variance in the cross section; this could result in more refinement than necessary. The Designer must decide how often to calculate the LLDF along the span. All LLDF used in the design must be included in the Bridge Load Rating Package, developed in accordance with the CDOT Bridge Rating Manual.

AASHTO Table 4.6.2.2.1-3 provides simplified values to be substituted when calculating the LLDF in corresponding tables in AASHTO 4.6.2.2. The State Bridge Engineer has approved Table 4.6.2.2.1-3 for use to simplify calculations.

**AASHTO
4.6.2.2**

When calculating LLDF, a refined analysis may be required whenever a variable falls outside the "Range of Applicability" as provided in the various LLDF tables of AASHTO. Approval from Unit Leader may be obtained to waive the need for the refined analysis if the value of the parameter is close to the

**AASHTO
4.6.2.2.1**

limit provided in the LLDF tables. Lever Rule may be used as a conservative alternative.

LLDFs for culverts and three-sided boxes shall be calculated as outlined in Section 12, Buried Structures and Tunnel Liners, of this BDM.

The use of our 5" minimum deck over side by side girders allows the designer to utilize the distribution factors based on F type (AASHTO Table 4.6.2.2.1-1) girder arrangements per AASHTO 5.12.2.3.3(f). For normal traffic bridges utilizing adjacent box girders, shear keys shall not be used.

4.5.1 Exterior Girder Live Load Distribution

The LLDF of specific multi-girder cross sections reported in AASHTO were calculated without consideration of interior diaphragms or cross frames within spans, or the effects of those members on the exterior girders. AASHTO Equation C4.6.2.2.2d-1 shall be checked for exterior girders when rigid cross frames are present between girders that would cause the entire superstructure to behave as a rigid body.

AASHTO
C4.6.2.2.2d

4.6 SKEW EFFECTS ON BRIDGES

Staff Bridge prefers bridge skews less than 50 degrees. Bridges with large skew angles can produce differential deflection between adjacent girders and unpredictable transfer of load from interior girders to exterior girders. Simple analysis will not be enough to correctly calculate deflection and load based on diaphragm and deck stiffness variations; therefore, Staff Bridge prefers a refined analysis to correctly model the effects of the large skew angles. AASHTO provides correction factors for LLDF for shear; care must be taken to not apply adjusted factors manually when software models the skewed supports and makes adjustments automatically. Refer to Figure 4-1 for the definition of skew angles.

AASHTO
4.6.2.2.3c

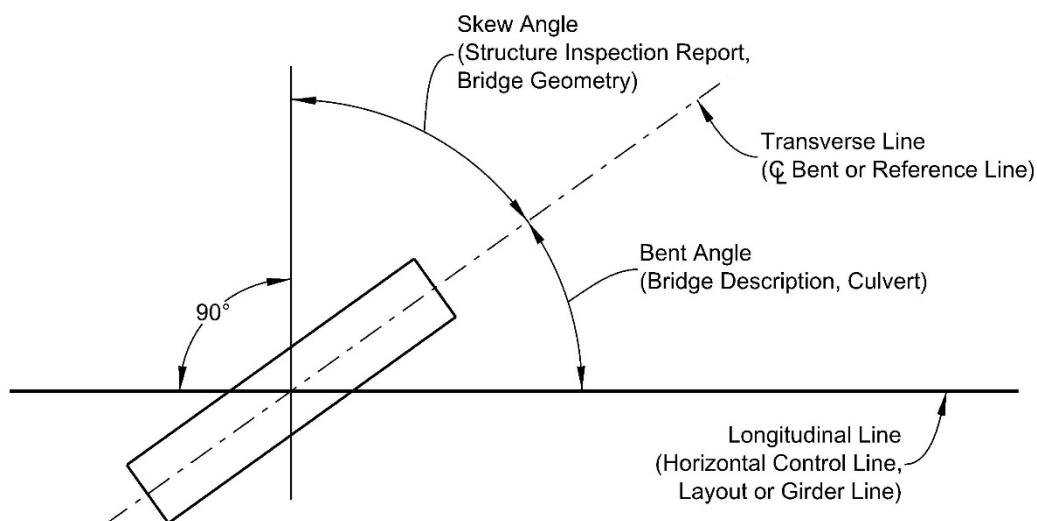


Figure 4-1: Skew Angle Definition

4.7 FOUNDATION STIFFNESS AND SOIL-PILE INTERACTION

The following guidelines supplement the general information given in AASHTO regarding modeling foundation boundary conditions.

AASHTO 4.5.4

For non-complex bridges with a length of 300 ft. or less that do not require a seismic analysis, Designers may use an assumed depth to fixity method to model pile and drilled shafts for lateral foundation analysis. In this case, the length used for determining lateral force effects, un-braced length, beam-column buckling analysis, and field welding requirements (BDM Section 10.5.3), may be based on engineering judgment founded on successful past practice.

For complex bridges, such as curved, highly skewed, and where an individual substructure stiffness varies significantly from the group, any bridge over 300 ft, or bridges that require a seismic analysis, CDOT prefers that Designers account for foundation stiffness in a more refined manner. This may be accomplished with the use of direct soil springs, equivalent spring constants, or equivalent depth to fixity calibrated with a soil/structure interaction analysis.

SECTION 5 CONCRETE STRUCTURES

5.1 GENERAL REQUIREMENTS

The provisions in this section apply to the design of reinforced concrete and prestressed concrete.

5.2 CODE REQUIREMENTS

Designs shall be consistent with AASHTO, unless modified herein.

5.3 MATERIAL PROPERTIES

5.3.1 Concrete Classes

5.3.1.1 Cast-in-Place Concrete

Table 5-1 and Table 5-2 show CDOT's most commonly specified classes of cast-in-place (CIP) concrete, typical design 28-day compressive strengths, and typical uses. See CDOT Standard Specifications for more information on concrete classes. Class DF shall be used to replace Class D concrete on elements exposed to de-icing salts (Splash Zone) in new bridge structures. Do not substitute Class BZ or S with Class DF. If stainless reinforcing is used, the Class DF requirement may be waived. Class DF is not required for prestressed elements or precast wall panels since the element is generally in compression and will limit crack size and chloride intrusion. When substructure elements are in separate construction pours such as columns and pier caps, the use of Class DF for those elements outside of the splash zone are not required, i.e. the quantities for the column could be Class DF while the quantities for the pier cap could be shown as Class D. Class DF should be considered for elements that are in areas that are exposed to leakage such as behind or near inlets.

Table 5-1: Common Concrete Classes and Strengths

Concrete Class	D, DF, DR	BZ	S35	S40	S50	Shotcrete
f'c (ksi)	4.5	4	5	5.8	7.25	4.5

Table 5-2: Typical CIP Concrete Applications

Structural Element	Typical Concrete Class
CIP Reinforced Concrete	D or DF
CIP Post-Tensioned Concrete	D or DF, S35, or S40
Drilled Shafts	BZ
Spliced Girder Bridge Closure Pours	D or DF, S35, S40, or S50*
Initial Facing for Soil Nail Walls and Top-Down Caisson Walls	Shotcrete
Concrete Patching	DR

*It is CDOT's preference to avoid designs using Class S50 concrete due to past difficulty in meeting the required cracking tendency test. In cases where the supplier is known during design, S50 concrete may be evaluated for feasibility.

5.3.1.2 Precast Concrete

Shop produced precast concrete girders shall be Class PS concrete and shall be limited to the following maximum design strengths:

- $f'_{ci} = 6.5$ ksi
- $f'_c = 8.5$ ksi

Plans shall show minimum strengths required to meet design requirements. These design strengths shall be used for all strength and service design checks.

Higher design values of f'_c and f'_{ci} may be permitted for special cases, after conferring with local precast suppliers and with approval from Unit Leader in coordination with the State Bridge Engineer.

Class DC concrete is a dry cast method of concrete used for precast box culverts.

5.3.1.3 Lightweight Concrete

It is CDOT's preference to avoid the use of lightweight concrete due to difficulty in passing aggregate tests and associated concerns regarding freeze-thaw durability. However, when the supplier is known during design, lightweight concrete is permitted for use provided a suitable mix passing ASTM C66 and C672 requirements is submitted for approval by the supplier to CDOT Materials. Approval for the use of lightweight concrete by the Unit Leader in coordination with the concrete SMEs is contingent on the passing mix design. The rationale for using lightweight concrete shall be documented in the Structure Selection Report.

5.3.2 Modulus of Elasticity

The unreinforced concrete unit weight for use in calculating the modulus of elasticity shall be per AASHTO Table 3.5.1-1 and C5.4.2.4.

**AASHTO
5.4.2.4**

5.3.3 Relative Humidity

When calculating creep and shrinkage coefficients, relative ambient humidity shall be taken as 55 percent.

**AASHTO
5.4.2.3**

5.3.4 Reinforcement

5.3.4.1 Mild Steel

Mild steel should typically be designed with a yield strength of 60 ksi. However, the use of 75 ksi rebar is allowed to assist in meeting the seismic transverse reinforcement detailing requirements when required in Seismic Zone 1 (see Section 5.4.9 for more information).

Use of epoxy-coated mild steel is the standard of practice where corrosion resistant reinforcement is required per Section 5.4.5, but alternates such as stainless steel should be considered per Section 5.3.4.3.

5.3.4.2 *Welded Wire Fabric*

Reinforcement for CIP concrete should generally be detailed as rebar in the contract plans, except for shotcrete wall facing where it is typically advantageous to specify welded wire fabric (WWF). In other structure elements where WWF may be an economical substitution, it may be noted as an allowable substitution at the Contractor's option.

5.3.4.3 *Stainless Steel and Corrosion Resistant Alloy Steel (CRAS)*

Both stainless steel and CRAS are acceptable alternatives to epoxy-coated mild steel. When the Designer elects to use either of these for a project, it shall be documented in the Structure Selection Report. The Designer is responsible for determining appropriate lap lengths.

5.3.4.4 *Glass FRP Rebar*

Glass FRP rebar shall not be used unless approved by Unit Leader.

5.3.4.5 *Epoxy Anchored Systems*

Expansion type concrete anchors are undesirable because of the vibration and pullout concerns. Instead, drilled-in-place anchor bolts bonded to the supporting concrete with an approved two-part epoxy system may be used. Two-part epoxy systems shall be approved by Concrete SME and CDOT Materials.

If the anchor is in continuous tension, the Designer shall use only an epoxy system if it is approved for use in continual tension loading. Project approval will be by the Unit Leader in coordination with the concrete SMEs and CDOT Materials using NTPEP, APL or project specific material submittals. Many epoxy systems are not allowed if the anchor is in continuous tension. Refer to ACI 318 and ACI 355.4 for more information on use of post-installed adhesive anchors.

5.3.5 **Prestressing Strand and Bars**

Prestressing strand shall be 0.60 in. diameter, low-relaxation strand, with a design ultimate tensile strength of 270 ksi. One exception to this requirement is for precast panel deck forms for which strands shall be no larger than 3/8 in. diameter. Prestressing bars shall have a design ultimate tensile strength of 150 ksi.

AASHTO 5.4.4

Post Tensioning Institute does not permit the use of tensioned galvanized bars because during curing the zinc layer may react with the alkaline grout and may generate hydrogen. Hydrogen can reduce the ductility of steel bars. Effective long-term corrosion protection is provided by grouting uncoated bars inside plastic ducts. The alkaline cement grout passivates the bar surface and the plastic duct acts as a moisture barrier. Such corrosion protection requires special anchorage details to maintain threadability and corrosion protection. CDOT has adopted this policy.

5.3.6 Concrete Inserts

Material of concrete inserts/embeds that will be part of the permanent structure shall match the material used for the attachments (e.g., bolts). Dissimilar materials shall be avoided to prevent corrosion issues. Galvanized or stainless steel inserts are preferred.

5.4 REINFORCED CONCRETE

5.4.1 Bar Size Availability

Reinforcing bars larger than #11 (that is, #14 and #18) may be used to eliminate reinforcement congestion if availability from suppliers is verified through the Engineering Estimates and Market Analysis Unit.

5.4.2 Development and Splice Lengths

Development lengths shall be calculated per AASHTO.

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The general notes sheet of the bridge plans shall no longer contain lap splice tables. The following tables are provided for Designer use in selecting lap splices for epoxy coated bars in slabs, walls, and footings, or other non-stirrup contained reinforcing.

Table 5-3: Minimum Lap Length for Epoxy-Coated Slab, Wall, or Footing Bars Spaced at 6.0 in. min. on Center with 2.0 in. min. Clear Cover and $f'c = 4.5$ ksi

#4	#5	#6	#7	#8	#9	#10	#11
1'-10"	2'-3"	3'-4"	3'-11"	4'-5"	5'-6"	6'-10"	8'-2"

Table 5-4: Minimum Lap Length for Epoxy-Coated Slab, Wall, or Footing Bars Spaced at 6.0 in. min. on Center with 1.0 in. min. Clear Cover and $f'c = 4.5$ ksi

#4	#5	#6	#7	#8	#9	#10	#11
2'-3"	3'-4"	4'-7"	5'-11"	7'-5"	9'-0"	10'-11"	12'-11"

For the same size bar in both top and bottom mat, the more conservative of the two tables shall be shown for ease of construction inspection. Table 5-4 lap splice values may be shown on the deck reinforcing sheet as applicable for both top and bottom mats of reinforcing bars, conservatively. The Designer may also choose to individually detail lap splices for deck rebar to take advantage of the smaller lap lengths required for top slab bars.

All other required lap lengths shall be detailed in the contract plans. Appendix 5A contains design aid tables for calculating development and lap splice lengths for reinforcing not meeting the criteria of Table 5-3 or Table 5-4.

5.4.3 Clear Cover

Concrete cover to main reinforcing bars shall be provided per AASHTO Table 5.10.1-1 and its accompanying notes, except as modified herein. For

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minimum clear cover specified in the table, use “severe to moderate exposure” for all cases excepted as noted in this section.

- The AASHTO provision for reducing concrete cover in the table by 0.5 in. for stirrups and ties shall apply only to precast girder faces and the minimum clear cover for precast girder faces shall be 1.5 in. or as shown in the worksheets.
- The minimum cover for reinforcing steel for concrete cast against earth shall be 3 in. for uncoated, epoxy coated, or galvanized bars.
- For CIP slabs not cast against earth or CIP deck bottoms, 1 in. minimum cover shall be used.
- For CIP piles, use “corrosive environments” for all cases.
- For drilled shafts on bridges, refer to Table 5-5 for the minimum required cover. The increased covers are adopted from FHWA’s recommendations due to constructability issues that may occur when lesser values of cover are specified for large diameter caissons.
- For elements with rustications, such as columns or abutments, required cover at innermost face of rustications may be reduced by 0.5 in.

Table 5-5: Minimum Clear Cover for Drilled Shafts on Bridges

Drilled Shaft Diameter, D (ft.)	Minimum Concrete Cover (in.)
$D \leq 3$	3
$3 < D < 5$	4
$D \geq 5$	6

5.4.4 Spacing

Reinforcement spacing requirements shall be per AASHTO, except as modified herein.

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Mild reinforcing bars shall have minimum clear spacing of at least 2 in. for both CIP and precast members (this includes bundled and lapped bars) unless noted otherwise in worksheets. This deviation from AASHTO results from past concrete consolidation issues encountered in Colorado.

5.4.5 Corrosion Protection Requirements

Reinforcing in structural elements that may be subjected to anti-icing or deicing chemicals shall be corrosion resistant (epoxy-coated mild steel, stainless steel, or CRAS). This includes, but is not limited to, all layers of reinforcing in the following elements and bars projecting therein:

- All deck slabs, approach slabs, CIP slab superstructures, and top flanges of CIP box girder bridges used as decks, regardless of wearing surface provided
- Concrete box culvert (CBC) top slabs with 2 ft. or less fill on top

- All abutment and pier diaphragms, abutment caps, and abutment wingwalls
- Pier caps and columns located under an expansion joint
- Retaining wall elements and pier columns located within the splash zone
- Ends of girders within 8 ft. of an expansion joint

5.4.6 Splash Zone Definition

The splash zone extends 10 ft. from the edge of the roadway shoulder, as shown in Figure 5-1 and includes the deck and superstructure elements from above.

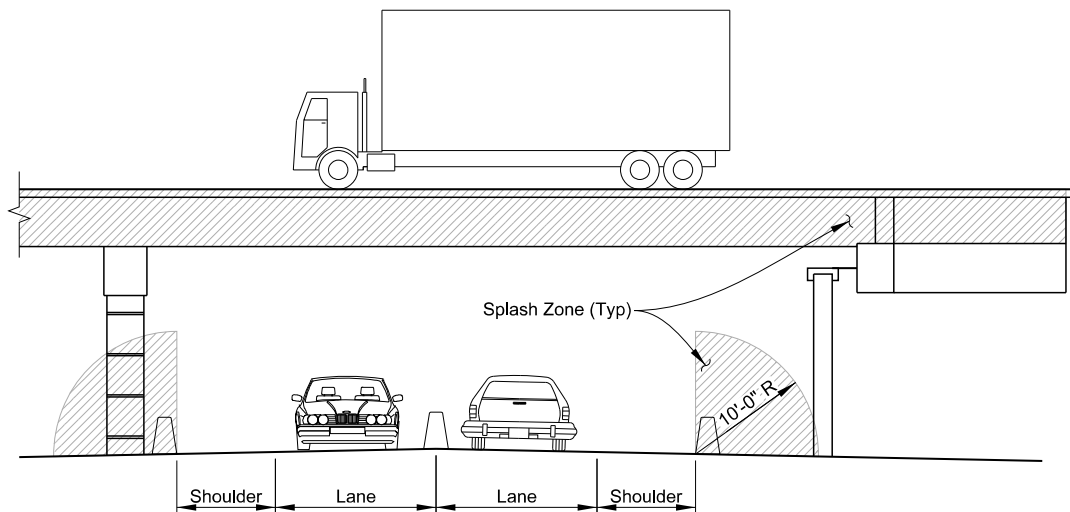


Figure 5-1: Splash Zone

5.4.7 Crack Control Factors

When calculating maximum spacing for crack control, an exposure factor of 0.75 shall be used for reinforcement that is required to be corrosion resistant, except for decks. For all other reinforcement, including decks complying with the wearing surface requirements of Section 9 of this BDM, 1.0 may be used.

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5.4.8 Mass Concrete

Large volumes of concrete sometimes have an increased potential to generate heat resulting in temperature-related cracking. This is typically an issue for concrete placements with least dimension greater than 6 ft., including, but not limited to, spread footings, thick walls, or bridge piers. In such cases, the Designer should consider requiring the Contractor to submit a thermal control plan. See ACI Manual of Concrete Practice Publication 207 for more information.

5.4.9 Seismic Detailing

Per AASHTO, for bridges in Seismic Zone 1 where S_{D1} is greater than or equal to 0.1, seismic detailing of columns and caissons shall be required for transverse reinforcement in potential hinge zones. When seismic detailing is

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5.11.2

required for round columns or caissons, spirals are preferred over seismic hoops.

5.4.10 Drilled Shaft and Round Column Shear Reinforcing

For shear reinforcing within drilled shafts and round columns that does not require seismic detailing per BDM Section 5.4.9, hoops containing a lap splice are generally more economical than spirals in the CDOT market.

5.4.11 Pier Cap Reinforcing Details

Cap reinforcement shall be placed below both mats of slab steel and below the main girder reinforcement in mildly reinforced girder bridges. In post-tensioned bridges, the cap reinforcement shall be placed below both mats of slab steel or between the mats of slab steel, if necessary, to provide clearance for post-tensioning ducts.

Hooks on integral cap shear stirrups shall be bent away from the centerline of the cap. The hooks shall enclose a cap reinforcement bar and the stirrups shall be adequately developed. To ensure proper concrete cover for stirrup hooks, hooks shall be below the top mat of slab steel. Figure 5-2 and Figure 5-3 provide details.

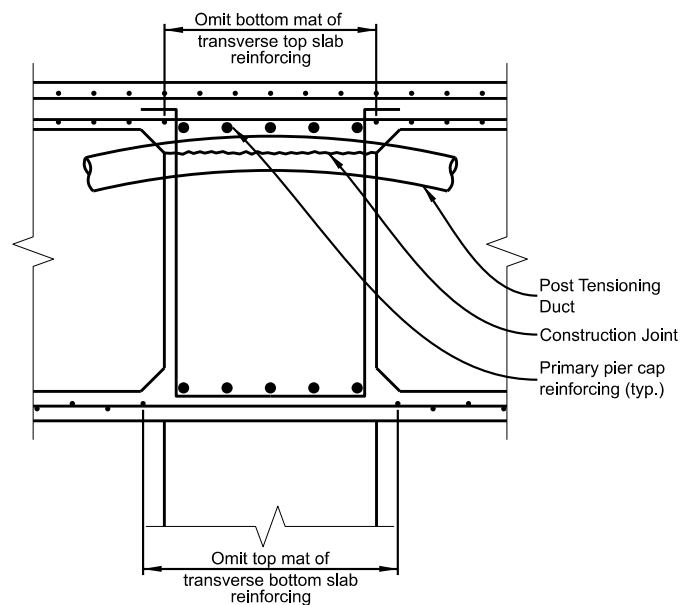


Figure 5-2: Pier Caps in Post-Tensioned Bridges with a Skew Angle of 20 Degrees or Less and Deck Reinforcing Parallel to Cap

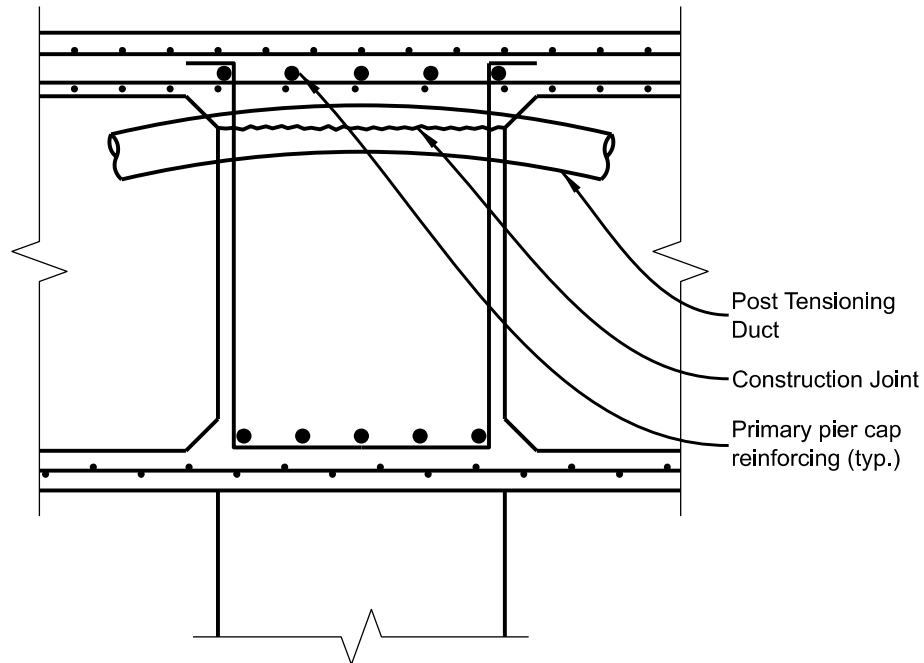


Figure 5-3: Pier Caps in Post-Tensioned Bridges with a Skew Angle Greater Than 20 Degrees and Deck Reinforcing Not Parallel to Cap

For precast girder bridges, cap reinforcement shall be enclosed in closed stirrups, as shown in Figure 5-4 and Figure 5-5. Stirrups shall be adequately developed.

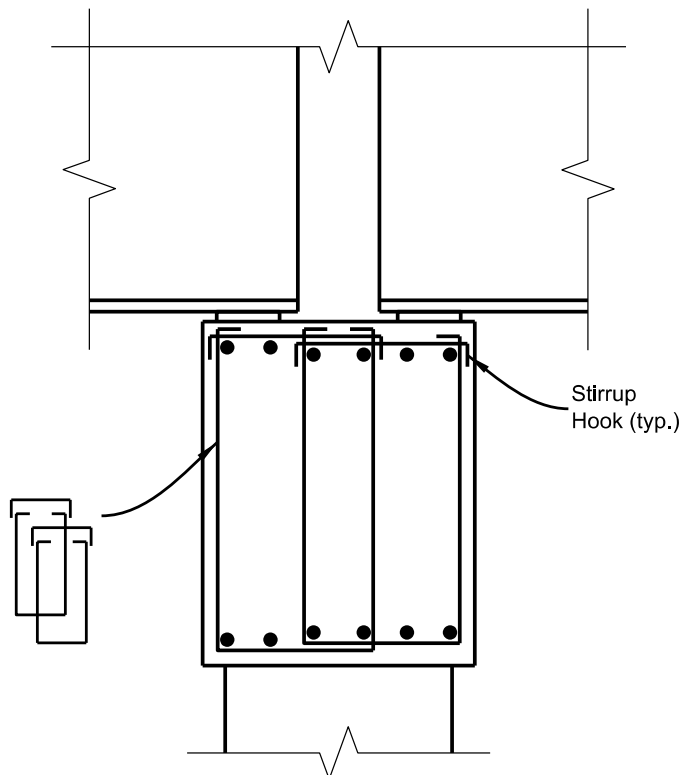
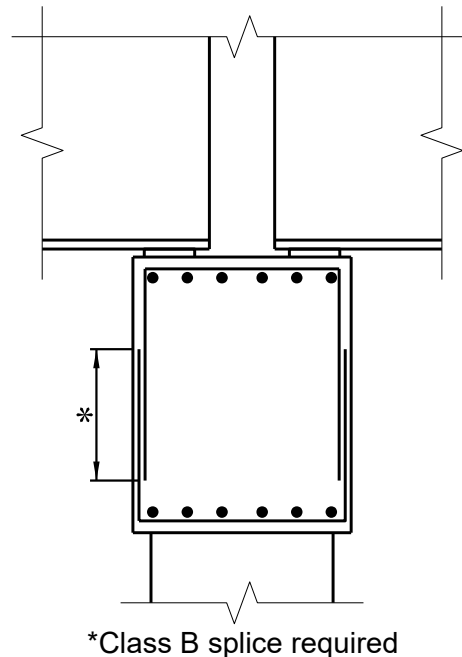


Figure 5-4: Pier Caps in Precast Girder Bridges with Constant-Depth Cap



**Figure 5-5: Pier Caps in Precast Girder Bridges with Variable-Depth Cap
(side steel not shown for clarity)**

5.4.12 Combination of Flexural and Axial Effects

Members subjected to flexure and compression may be analyzed using the method of creating an influence diagram using equilibrium and strain compatibility. Many commercial structural design software programs use this approach to create interaction diagrams. Alternatively, AASHTO approximate expressions may be used.

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5.5 PRESTRESSING

5.5.1 General

5.5.1.1 Transformed Section Properties and Elastic Gains

AASHTO allows the use of transformed section properties. The Designer should note that when calculating concrete stresses using transformed section properties, the effects of losses and gains due to elastic deformations are implicitly accounted for. Commercial software that calculates elastic gains separately in conjunction with using transformed section properties shall not be used.

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Prestressed concrete components designed using the refined estimates of time-dependent losses as specified in AASHTO in conjunction with taking advantage of the elastic gain shall use the increased SVC III live-load factor of 1.0. This increased live load factor also applies to designs using transformed section properties since elastic gains from live load are implicitly accounted for. When elastic gains are not taken advantage of, a live-load factor of 0.8 may be used for SVC III.

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If elastic gains due to slab shrinkage are taken advantage of, the corresponding girder moment due to slab shrinkage shall be considered in the girder stress calculations. Alternatively, the slab shrinkage elastic gain and the corresponding girder moment may be disregarded.

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5.5.1.2 Intermediate Diaphragms

The Construction Layout sheet shall show the location of intermediate diaphragms for CBT girders.

The Designer is responsible for providing a design that considers stability at the AASHTO Strength III limit state of the girders during construction, especially the stability of exterior girders that may be exposed to wind loads before the deck pour. Additional diaphragms or modifications to CDOT's standard diaphragm details may be needed for special situations. Additional diaphragms or modifications to the standard details should not be used unless determined necessary by calculation.

The Designer should check that the resultant of factored construction loads falls within the area of the leveling pad and that the compression in the portion of the pad loaded in these cases is less than the pad strength. If the resultant falls outside the pad or if the compression strength of the pad is exceeded, additional diaphragms should be provided to reduce eccentricity by causing the girders to overturn in concert.

5.5.1.3 Concrete Stresses

Girders shall be designed such that there is no tension in the concrete under dead load acting alone, at service limit state, and after losses. This provision applies to the pre-compressed tensile zones only as required by the AASHTO tensile stress limits in prestressed concrete. The top ends of girder for a simple span, or simple made continuous bridge, are often under long term tension caused by the prestressing. However, the section is not a pre-compressed tensile zone, so the no tension limit does not apply.

Per AASHTO, compression stresses shall be limited to $0.65 f'_c$ at release. This provision is cited in the BDM due to it being a relatively recent change in AASHTO.

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5.5.1.4 Design Jacking Force

The maximum design jacking force in all prestressing strands (pretensioned or post-tensioned) shall be no more than 75 percent of the ultimate tensile strength of the strand.

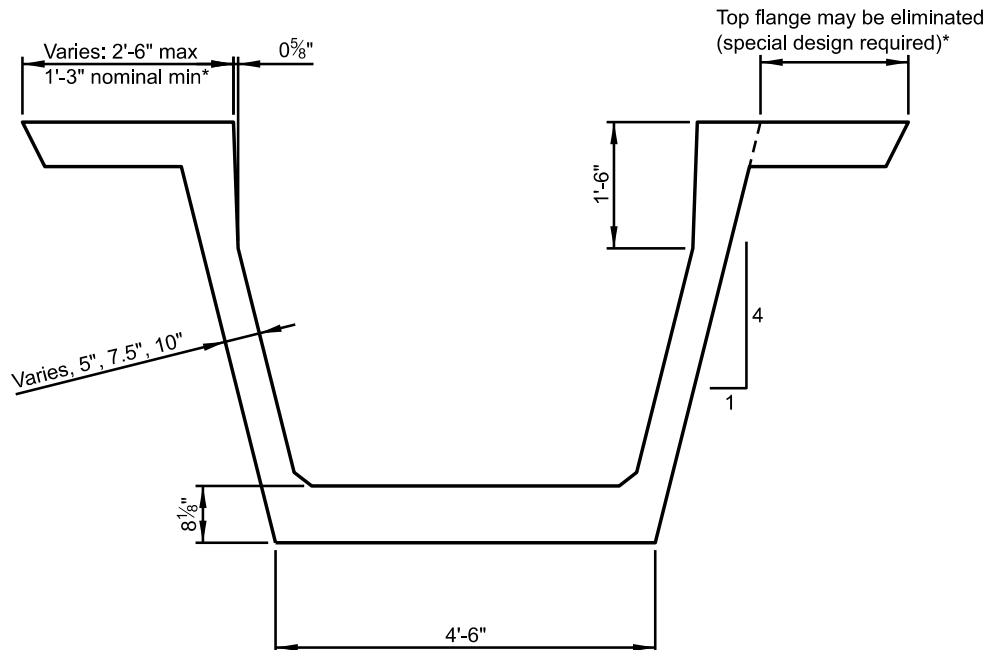
5.5.1.5 Standard Girder Shapes

Table 5-6 identifies the standard properties of CBT girders. See CDOT standard girder worksheets for standard CBT girder dimensions.

Table 5-6: Standard CBT Properties

Section	Depth	A (sq. in.)	I _x (in. ⁴)	Y _b (in.)
CBT37.5	37.5	792	151579	18.5
CBT45	45	845	240424	22.08
CBT54	54	908	378473	26.40
CBT63	63	971	553233	30.74
CBT72	72	1034	767268	35.10
CBT81	81	1097	1023130	39.48
CBT90	90	1160	1323390	43.87

Figure 5-6 identifies the standard dimensions of precast U girders.



*When setting the top flange width of U girders, the Designer shall consider the loss of concrete width for interface shear resistance due to the support requirements for partial depth precast deck panels. While the top flange of U girders may be eliminated entirely from a fabrication standpoint, the limited remaining interface width may preclude using partial depth precast deck panels.

Figure 5-6: Standard U Girder Dimensions

Leap bridge concrete software girder library files are located in CDOT Bridge homepage under Bridge Manuals & Documents section. Designers should contact local suppliers for the following information, which may vary by supplier:

- Pretensioned strand locations
- U girder radius limitations

- U girder height options
- Non-standard CBT girder height options
- U girder and CBT girder thickened bottom flange options
- U girder anchorage blister options

The maximum harped strand height for CBT girders is generally 60" to 66" and is dependent on the girder length/precast bed configuration. For fabrication efficiency, debonded strands are preferred over harped strands and girders less than 100 feet should not use harped strands.

For skewed bridges, the ends of CBT should not be skewed, but the top flange may be clipped to maintain clearance. Wider caps and diaphragms may be considered.

End blocks shall be used for box girders. End blocks are not required for typical applications of the CBT or U girders, but an internal diaphragm of some type is required at the ends of U girders to deal with bearing loads and splaying loads from self-weight and handling.

For box girders harped designs shall not be used. Provide designs with debonded strands only. For skewed bridges, skew the ends of the box girders. The transverse reinforcing steel area in precast box girder flanges shall, as a minimum, be equal to the minimum required shear reinforcing steel for one web. If the top flange of the box is intended to serve as precast stay-in-place formwork for the final deck, this reinforcing shall be designed as the bottom mat of the deck.

5.5.1.6 *Maximum Stirrup Spacing*

Maximum stirrup spacing in prestressed girders shall be 18 in.

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5.5.1.7 *Negative Moment Reinforcement*

For simple made continuous bridges and spliced bridges, the negative moment reinforcing shall be sized for the moment at face of support. The face of support varies depending on pier details and shall be assumed as follows:

**AASHTO
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- For integral pier caps, the face of support is the face of pier cap.
- Where pier diaphragms are integrally connected to the pier cap, the face of support is the face of diaphragm.
- For pier diaphragms that use the typical CDOT pin detail with a single line of dowels between it and the pier cap, the face of support shall be taken as the centerline of pier.
- For other situations, the Designer is responsible for determining the appropriate face of support.

Longitudinal reinforcing for negative moment placed near the top of deck may be accomplished one of two ways:

- Continuing the typical top longitudinal deck steel over the pier and bundling to the typical bars with larger bars where needed.
- Discontinuing the top longitudinal deck steel and continuing with larger bars where needed. Two bar bundles may be used for the peak negative moment region for this option.

When partial depth precast deck panels are permitted on the project, bottom longitudinal reinforcing in the deck shall not be used for composite girder negative moment capacity calculations.

See Section 9 of this BDM for the minimum clearance required between deck reinforcing and the top of partial depth precast deck panels.

5.5.1.8 *Shipping and Handling*

Per AASHTO, the fabricator is responsible for the shipping and handling design. However, when the Designer specifies temporary girder support locations on the plans, the Designer is responsible for designing the girder for the force effects resulting from that support condition.

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5.5.1.9 *Shipping Weights and Lengths*

For typical locations along the Front Range urban corridor, typical maximum girder length and shipping weights are 154 ft. and 240 kip, respectively. For lengths or weights exceeding these limits and for project site locations where delivery routes may have constraints, such as sharp curved roads and/or tunnels, the Designer shall coordinate with local suppliers to determine the dimensional and weight limitations of the proposed girders.

5.5.1.10 *Partial Prestressing*

Partial prestressing is not addressed in AASHTO. Partial prestressing as a design strategy may be allowed with approval from Unit Leader and concrete SMEs.

Partial prestressing refers to situations where the prestressing is insufficient to reduce flexural tensile stresses to the Service III or temporary tensile stress limits. When partial prestressing is used, expected crack openings shall be controlled to an appropriate limit in the Service I load case. This control may be provided by distribution of bonded reinforcement with an area of at least 1 percent of the area of the tensile zone or by limiting tensile stresses or tensile strains. Also, when partial prestressing is used, live and dead load deflections shall be calculated using the appropriate cracked section properties. Strength shall be checked in all relevant load cases, including construction and handling loads. In the instance of partial prestressing, either compressive stress limits may be applied at the service loads or ultimate strength limits may be applied.

For sheltered locations not subject to deicing salts, rain, snow, or direct sunlight, 0.024 in. may be an acceptable crack opening at the reinforcing depth. For locations subjected to the above elements, 0.016 in. may be taken as an acceptable crack opening.

5.5.2 Pretensioned Concrete

5.5.2.1 Girder Haunch, Camber, and Dead Load Deflections

A. General

The Designer is responsible for setting the thickness of the haunch at supports, such that an adequate haunch is maintained along the length of the girder considering the estimated girder camber with tolerance, dead load deflections, deck profile grade and cross slope, and required precast deck panel clearance when applicable.

For side-by-side box or slab girders, the haunch is synonymous with the deck. In this case, the Designer is responsible for setting the deck thickness at supports and verifying that adequate deck thickness is maintained along the length of girder, considering the applicable factors noted previously for girder haunches.

B. Minimum Haunch

The minimum haunch at supports shall be 1.5 in. where partial depth precast deck panels are permitted. This allows the required 1 in. vertical clearance underneath the panels, plus 0.5 in. of tolerance that accounts for girder depth variation and/or bearing seat height variability. Where partial depth precast deck panels are not permitted, the minimum haunch at supports shall be 0.5 in.

The minimum estimated haunch between supports shall be 1 in. where partial depth precast deck panels are permitted and may be taken as zero where partial depth precast deck panels are not permitted.

For side-by-side box or slab girders, the minimum deck thickness specified at supports shall be 5 in., in accordance with Section 9.5 of this BDM. The minimum estimated deck thickness between supports shall also be maintained at 5 in.

All minimum haunch requirements above must be met along the entire width of the top flange of the girder, not only at centerline. The Designer must take into consideration cross slope effect on the haunch variance.

C. Maximum Haunch

There is no limiting maximum haunch either at supports or for the estimated haunches between supports. For haunches with a side face dimension estimated at 8 in. or greater, minimum temperature and shrinkage reinforcement shall be added to the side faces of the haunch.

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D. Camber Estimates

Release and erection cambers should be estimated using the plan specified concrete design strength minimums per Section 5.3.1.2. |

When using camber calculations where the age is a factor for the camber at the time of deck pour, the age of the girder shall be assumed to be 60 days. See Section 5.7.2 for situations where this age may be assumed to be less than 60 days.

Tolerances for girder cambers with respect to estimating minimum and maximum haunches are as follows:

- For CBT girders, camber shall be assumed to be up to 20 percent over or 50 percent under the predicted camber.
- For slab and box girders, camber shall be assumed to be up to 50 percent over or 50 percent under the predicted camber.
- For design cambers greater than 1", the camber tolerance shall be taken as no less than ± 1 in. For camber designs less than 1", camber tolerance may be less than 1" with approval by Unit Leader. This is typically restricted to widening with site constraints.

These camber tolerances are based on fabrication data for girders with cambers greater than 1" and a 90% confidence range on the envelopes as shown in Table 5-7. Data for design cambers under 1" have a higher camber variance.

If a girder line can be eliminated by reducing over and/or under camber tolerances, a variance shall be requested.

Table 5-7: Camber Data

Girder Type		CBT Girder	BX Girder
Count		192	871
Minimum		-44.44%	-90.33%
Maximum		55.96%	209.09%
Mean		5.85%	-3.75%
Standard Deviation		19.18%	32.10%
Confidence Range	99.7%	63.4% / -51.7%	92.6% / -100.1%
	99%	55.2% / -43.5%	78.9% / -86.4%
	95%	43.4% / -31.7%	59.2% / -66.7%
	90%	37.4% / -25.7%	49.1% / -56.6%
	85%	33.5% / -21.8%	42.5% / -50.0%

E. Dead Load Deflections

Dead load deflections may be calculated assuming no long-term increase in deflection beyond construction. This assumption may be used for the dead load deflection reported on the girder sheet, for estimating haunches, and for setting deck grades. The long term multipliers shall not be used in the girder design.

F. Deck Profile Grade Effect

The deck profile grade ordinate shall be taken as the difference between a chord of profile grade from bearing to bearing and the actual profile grade at any point along the chord line.

This ordinate will add to the haunch thickness if profile grade is higher than the chord line and, conversely, will subtract from the haunch thickness if profile grade is lower than the chord line.

G. Design Considerations

Side-by-side box girders shall be designed for service and strength criteria using the range of deck thicknesses expected considering the assumed tolerances for box girder cambers. The dead load deflection reported on the plans and used to set deck grades shall be calculated with the deck thickness resulting from the predicted girder camber. The deck concrete quantity may also be based on this deck thickness. The use of our 5" minimum deck over the girder allows the designer to utilize the distribution factors based on F type (AASHTO Table 4.6.2.2.1-1) girder arrangements per AASHTO 5.12.2.3.3(f) so shear keys are generally not allowed.

Other girders may be conservatively designed assuming the maximum estimated haunch due to an under-cambered girder for all calculations.

Girder sag is not permitted for any girder type, unless there is prior approval by the Unit Leader. Sag is considered prevented when the girder camber remaining after deducting the under-camber tolerance and the dead load deflection is greater than or equal to zero. The dead load deflection used for this check need not be magnified by long-term effects. For side-by-side boxes, the dead load deflection for this check shall be based on the increased deck weight resulting from the girder being under-cambered.

In lieu of considering over-camber tolerance in the design of side-by-side box girder bridges, the bearing seats may be lowered by the over-camber tolerance amount. Shims shall be provided where the total shim stack height equals the over-camber tolerance amount. If the girders are over-cambered, shims may be removed as necessary to maintain a 5 in. minimum deck thickness.

A weighted average haunch (or slab depth for side-by-side boxes) may be used for dead load calculations for girder design. The equation below is derived for the midspan moment effect assuming the haunch (or slab) varies parabolically with the apex (either concave or convex) at midspan:

$$(D_1 + 10*D_2 + D_3) / 12 \quad \text{Eq. 5.1}$$

A volume-based average haunch (or slab depth for side-by-side boxes) may be used for the concrete quantity. The equation below is derived assuming the top of girder is chorded between the end of girder and midspan:

$$(D_1 + 2*D_2 + D_3) / 4 \quad \text{Eq. 5.2}$$

Where D_1 is the depth over one bearing, D_2 at midspan, and D_3 over the other bearing.

See Example 7 for detailed examples of setting girder haunches and verifying the above criteria.

5.5.2.2 *Hold Down Limits*

Harped strands shall be designed so that the hold-down force does not exceed 4.0 kip per strand.

5.5.2.3 *Partially Debonded Strands*

If debonding is used, the design shall follow the debonding criteria outlined in AASHTO 5.9.4.3.3. Minimum Plan Requirements

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The contract plans for pretensioned members shall specify:

- Jacking force
- Area of prestressing steel
- Minimum concrete strength at jacking and at 28 days
- Center of gravity of prestressing force path
- Final force
- Dead load deflection
- Expected cambers (release and before deck pour)
- Estimated haunch at midspan (estimated deck thickness for side-by-side box girders)

5.5.3 **Post-Tensioned Concrete**

5.5.3.1 *Anchorage*

The post-tensioning supplier is responsible for the design of the local zone, including the anchorage device itself and confinement reinforcement. The Design Engineer is responsible for all other anchorage-related designs, including the general zone. The Designer shall verify that all anchorage design assumptions are correctly represented on the plans to aid the supplier in the design of the local zone to coordinate with the design of the general zone.

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Composite anchorages shall not be permitted. Multi-plane anchorages may be used.

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The design jacking force of strands shall be 75 percent of the ultimate tensile strength of the tendon for the design of the post-tensioned member. For the design of anchorages, including the local and general zones, the anchorage force shall be based on 80 percent of the ultimate tensile strength of the tendon. This allows reserve capacity for increasing the jacking force to the AASHTO limit, if needed, during construction.

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The plans shall show the configuration of the anchorages and the arrangement of ducts at typical high and low points appropriate for the duct and strand size noted on the plans. The arrangement of anchorages shall permit a center-to-center anchorage spacing of at least $\sqrt{(2.2P_j / f'_{ci})}$ in. and a spacing from the center of each anchorage to the nearest concrete edge of at least half that value. If web flares are needed for this arrangement, they shall be dimensioned in the plans and included in the quantities.

5.5.3.2 Post-tensioning Ducts

A. Spacing

- Minimum clear spacing of ducts shall be the greater of 40 percent of the nominal duct diameter or 1.5 in.
- Bundled ducts shall not be used without approval from Unit Leader in coordination with the Fabrication/Construction Unit.

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B. Clear Cover

- For cast-in-place bridges, the minimum clear cover to ducts shall be the greater of 75 percent of the nominal duct diameter or 3 in.
- For precast girder bridges, the minimum clear cover to ducts shall be the greater of 50 percent of the nominal duct diameter or 2 in. An exception to this is post-tensioned CBT girders, which have demonstrated good past performance with a minimum of 1.75 in. clear cover.
- Clear cover for ducts curved in plan shall meet the greater of the applicable criteria above, or the confinement criteria as specified in AASHTO.

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C. Eccentricity

Eccentricity of strand within ducts shall be considered when modeling the tendons. In lieu of using the eccentricities specified in AASHTO Figure C5.9.1.6-1, manufacturer-specific eccentricity may be used if known during design.

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5.5.3.3 Monostrands

Monostrand tendons shall be of a fully encapsulated waterproof construction whether permanent or temporary.

Permanent monostrand tendons placed in any of the locations listed below shall be of a type certified by their manufacturer for chloride contaminated environments:

- In decks or haunches above girders
- When any part of the tendon is within a horizontal distance equal to the structure depth of an expansion joint or within 6 in. of the back face of an integral abutment
- When tendon is used in below ground construction

Monostrands and bundles of up to 4 monostrands in plant produced members using a highly fluid small aggregate concrete, or using a moderately fluid small aggregate concrete with form vibrators, shall have a clear spacing of at least 1.25 in.

Field produced members or members not using form vibrators or a fluid small aggregate concrete shall have a clear spacing between monostrands or bundles of monostrands of at least 1.5 in.

5.5.3.4 *Unbonded Tendon Redundancy*

An unbonded tendon is any tendon that is not bonded to the structure throughout its length in its final installed condition. Common examples are monostrand tendons and multi-strand tendons used in externally post-tensioned precast segmental box girders.

For each girder, any two unbonded tendons shall be assumed to be failed. The moment strength provided by the remaining tendons and reinforcement shall be at least 80 percent of that required by the Strength I load combination. The same provision applies to any 13.5 ft. width of slab. The 13.5 ft. limit is a conservative limit based on the arching capability of the slab.

At the discretion of the Designer, for longitudinal tendons in multiple girder systems in which there are adequate load paths between the girders, the entire connected cross section may be considered a single girder element.

5.5.3.5 *Transverse Post-Tensioning in Adjacent Precast Box Girders*

For railroad bridges, adjacent box girders without a CIP deck are permitted with the use of transverse post-tensioning, per AREMA. For the design of adjacent box girders without a CIP deck, the design guidance of PCI Bridge Design Manual Section 8.9 may be used.

Adjacent box girders without a CIP deck are not permitted for traffic bridges. For normal traffic bridges utilizing adjacent box girders, shear keys shall not be used. The use of our 5" minimum deck over the girder allows the designer to utilize the distribution factors based on F type (AASHTO Table 4.6.2.2.1-1) girder arrangements per AASHTO 5.12.2.3.3(f).

5.5.3.6 *Severe Exposure Category for Tension Limits*

The following locations shall be considered the severe exposure category for AASHTO concrete tension limits:

**AASHTO
5.9.2.3.2b**

- Tops of decks that are post-tensioned
- Top flanges acting as the deck for CIP post-tensioned girders

Tops of girders for post-tensioned spliced bridges need not be classified as severe exposure.

5.5.3.7 *Through-the-Thickness Web Reinforcing*

Through-the-thickness reinforcing equal to 9 percent of the area of the tendon shall be provided for the following situations:

- To arrest propagation of through-the-thickness cracks driven by misalignment at construction joints. The specified amount of reinforcing shall be placed near each tendon passing through a joint. The reinforcement shall be split between each side of the joint.
- Due to potential through-the-thickness forces at thickness transitions at the beginning and end of transitions of web or flange thickness. The specified amount of reinforcing shall be located near the tendon and split between each side of the transition beginning or end.

Through-the-thickness reinforcement shall be anchored as close to the face of the concrete as practical. Headed studs or stud-rails may be used in lieu of reinforcing.

Figure 5-7 and Figure 5-8 illustrate through-the-thickness reinforcement.

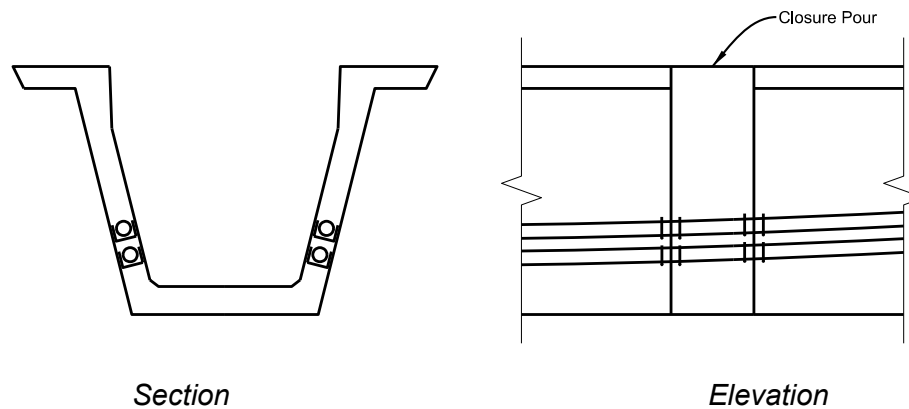


Figure 5-7: Through-the-Thickness Steel at Construction Joints

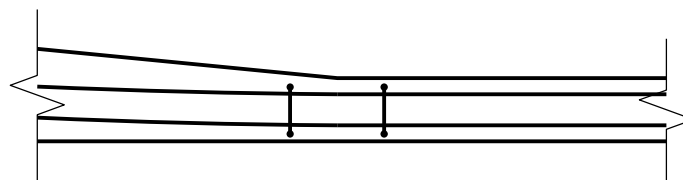


Figure 5-8: Plan View of Through-the-Thickness Reinforcing at a Web Thickness Transition

5.5.3.8 Horizontally Curved Tendons

Reinforcing requirements for horizontally curved tendons shall be per AASHTO.

**AASHTO
5.9.5.4.3**

5.5.3.9 Minimum Plan Requirements

The contract plans for post-tensioned members shall specify:

- Jacking force
- Area of prestressing steel
- Minimum concrete strength at jacking and at 28 days
- Center of gravity of prestressing force path
- Jacking ends
- Anchor sets
- Friction constants
- Long-term losses assumed in the design
- Strand and duct size assumed in the design
- Net long-term deflections and expected cambers

- Estimated haunches at midspans (for spliced girders only)

5.6 LONGITUDINAL REINFORCEMENT FOR SHEAR

5.6.1 General

AASHTO Equation 5.7.3.5-1 accounts for increased tension in longitudinal reinforcement caused by shear. The applicability of this interaction equation depends on support and loading conditions. This section is provided as further clarification of AASHTO.

**AASHTO
5.7.3.5**

5.6.2 Direct Loading and Supports

Direct loading may be assumed where the load introduces compression directly onto the compression face of the member.

Direct supports may be assumed where the reaction introduces direct compression directly onto the compression face of the member.

In simple-made-continuous bridges, pier caps that are detailed as pinned connections to the pier diaphragms may be classified as direct supports.

Figure 5-9 presents examples of direct support and direct loading conditions.

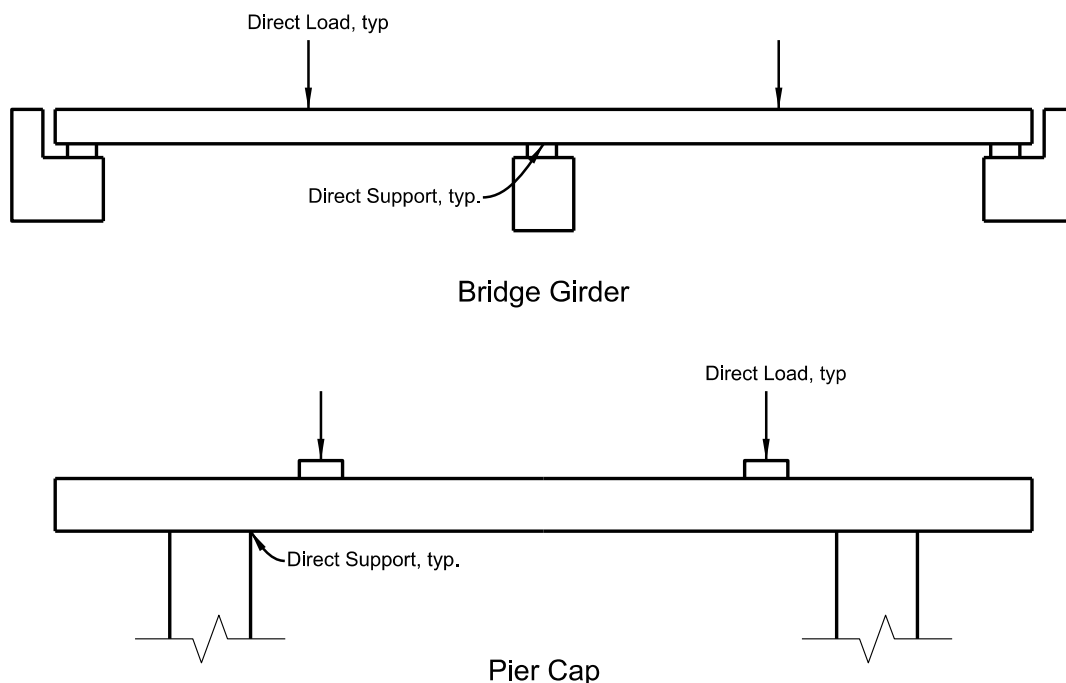


Figure 5-9: Examples of Direct Supporting and Loading Conditions

For direct support/loading conditions, the following provisions apply:

- Checking interaction is not required at or near direct supports or at other locations of maximum moment, such as at or near midspan. At these locations, the longitudinal reinforcement needed for moment demand alone need not be exceeded.

- Interaction shall be checked where longitudinal reinforcement is reduced along the member. In this case, the equation shall be checked at 10th points that are away from the maximum moment locations and at locations of reinforcement reduction.
- In summary of the previous two notes, if the maximum needed flexural reinforcement is continuous through the member and not reduced, checking the interaction equation is not required.

5.6.3 Indirect Loading and Supports

Any load or support that is not classified as a direct load or support shall be classified as indirect. For practical purposes in bridge design, this primarily happens at integral pier caps.

In simple-made-continuous bridges, pier caps that are detailed as integrally connected to the pier diaphragm shall be assumed to be indirect supports.

Figure 5-10 shows an example of an indirect support. In this case, the girders shall be considered indirectly supported, and the pier cap shall be considered indirectly loaded. The pier cap may be considered directly supported by the columns.

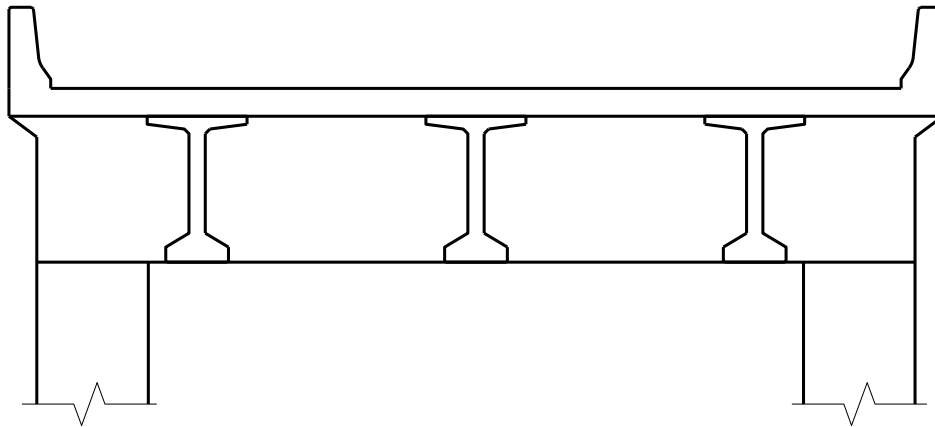


Figure 5-10: Indirect Support/Loading – Integral Pier Cap

For indirect support and loading conditions for a typical integral pier cap, the following provisions apply:

- Interaction shall be checked in the girder at the face of the integral pier cap, at 10th points, and at places of rebar termination.
- Interaction does not need to be checked in the girder at midspan if it is directly loaded.
- Interaction shall be checked in the pier cap at points of maximum positive moment, at 10th points, and at locations where positive moment reinforcement is terminated.
- Interaction need not be checked in the pier cap at or near the face of column, as this is at a direct support. But if negative moment reinforcement

is reduced, then interaction shall be checked at 10th points away from the direct support and at locations of rebar termination.

5.6.4 Simply Supported Girder Ends

AASHTO Equation 5.7.3.5-2 shall be satisfied at the inside edge of the bearing area of simple supports. Girders supported by the typical CDOT integral abutment are required to meet this provision.

5.7 SIMPLE SPAN PRESTRESSED MADE CONTINUOUS

5.7.1 General

These provisions apply to multi-span bridges composed of simple-span precast girders with continuity diaphragms cast with the deck between ends of girders at interior supports. These bridges shall be designed using the specific provisions for this structure type per AASHTO, except as amended herein.

**AASHTO
5.12.3.3**

5.7.2 Age of Girder When Continuity Is Established

The plans shall specify the minimum age of the precast girder when continuity is established (deck and continuity diaphragm placed).

**AASHTO
5.12.3.3.4**

For standard designs, the minimum age before establishing continuity shall be 60 days. If waiting 60 days for deck/diaphragm placement has negative impacts to the project schedule, the minimum age may be specified as less than 60 days. In either case, the following simplifications shall apply:

- Positive restraint moment caused by girder creep and shrinkage and deck slab shrinkage shall be taken to be zero.
- Computation of restraint moments shall not be required.
- A positive moment connection shall be designed to resist $1.2M_{cr}$.

5.7.3 Degree of Continuity at Various Limit States

The connection between precast girders at a continuity diaphragm may be considered fully effective if the plans require that the age of the precast girders be at least 60 days before deck/diaphragm pour.

**AASHTO
5.12.3.3.5**

If the precast girder connection between precast girders at a continuity diaphragm does not satisfy this requirement, the joint shall be considered non-effective.

Superstructures with fully effective connections at interior supports may be designed as fully continuous structures for all loads applied after continuity is established for both service and strength limit states.

Superstructures with non-effective connections at interior supports shall have designs enveloped for the worst-case force effects between simple span and continuous behavior for all loads applied after continuity is established for all limit states. For example, simple span behavior will govern positive moment regions, and continuous behavior will govern negative moment regions.

The provisions in AASHTO for partially effective continuity diaphragms shall be disregarded.

5.8 PRECAST SPLICED BRIDGES

5.8.1 General

Precast spliced bridges are structures using precast girders fabricated in segments that are joined or spliced longitudinally to form girders in the final structure. These bridges shall be designed using the specific provisions for this structure type per AASHTO, except as amended herein.

AASHTO
5.12.3.4

5.8.2 Girder Age Restrictions

The contract documents shall show the minimum age of girder segments at the time of post-tensioning. The age may be specified as the average age of segments per girder line.

Where expansion joint movements are at or near the full joint design capacity, the contract documents shall show the minimum time required to elapse after post-tensioning and before installing expansion joints.

5.8.3 Joints Between Segments

Match-cast joints shall not be used between segments unless approval is obtained from Unit Leader in coordination with the Fabrication/Construction Unit.

AASHTO
5.12.3.4.2

5.8.4 Details of Closure Joints

The width of closure joints shall not be less than 2 ft.

AASHTO
5.12.3.4.2b

5.8.5 Segment Design

Where girder segments are handled before the application of prestressing, the provisions of AASHTO 5.6.7 shall apply until post-tensioning is applied.

AASHTO
5.12.3.4.3

Refer to Section 5.5.1.8 for additional segment shipping and handling design requirements.

5.8.6 Consideration of Future Deck Replacement

To facilitate future deck replacement, the follow criteria shall apply:

AASHTO
5.12.3.4.4

1. All post-tensioning tendons should be located fully within the girder.
2. All tendons shall be stressed before deck placement.
3. All temporary girder supports shall be removed prior to deck placement.

Deviations from items 2 and 3 may be permitted with approval from Unit Leader in coordination with the State Bridge Engineer. In this case, an analysis of the feasibility of future deck replacement shall be accomplished, and a future deck replacement plan shall be provided in the bridge design plans. The deck replacement plan shall delineate the construction steps necessary for deck replacement including, but not limited to, the following, as applicable:

- Special requirements for deck removal sequencing
- Temporary girder supports required and accompanying reactions

- Additional post-tensioning required (if this is required, accommodations for future post-tensioning shall be detailed into the plans)
- Special requirements for deck placement sequencing

5.8.7 Girder Camber, Haunch, and Dead Load Deflections

The provisions of Section 5.5.2 for pretensioned girder bridges should generally be followed for spliced girder bridges with the following additional considerations.

The total girder camber is the superimposed total of the individual segment camber, the camber resulting from continuity post-tensioning, and the camber induced through the setting of temporary support bottom-of-girder elevations.

The dead load deflection reported on the plans shall include long-term effects. The long-term effects shall be estimated in conjunction with a time-dependent, staged construction analysis method. The long-term dead load deflection shall be used for setting deck grades, setting and estimating girder haunches, and verifying overall girder camber.

CDOT has not experienced the same severity of issues regarding camber variability and associated girder sag for spliced bridges as it has for pretensioned girder bridges. For spliced bridges, the Designer is responsible for determining appropriate camber tolerances used for setting and estimating girder haunches and for verifying adequate final girder camber.

5.9 CAST-IN-PLACE CONCRETE GIRDERS

5.9.1 General

CIP box and T-beam girders constructed on falsework shall be designed using the specific provisions for CIP girders per AASHTO, except as amended herein.

**AASHTO
5.12.3.5**

5.9.2 Box Girder Bottom Slab Slope

Except for crowned roadways, the bottom slab should be made parallel to the top slab. For crowned roadways, the bottom slab should be made horizontal.

5.9.3 Box Girder Formwork Load

Design shall include the additional dead load for deck formwork to be left in place. This formwork load shall be applied over a width equal to exterior web to exterior web.

5.9.4 Web Reinforcement

One-piece "U" stirrups shall not be used in box girder webs.

For post-tensioned girders, each web face shall contain continuous longitudinal reinforcement of at least 0.20 in²/ft, spaced at 12 in. max.

5.10 SEGMENTAL BOX GIRDERS

5.10.1 General

Segmental box girder bridges are composed of multiple box girder segments where the width of each segment is typically the full width of the bridge. The segments are post-tensioned together longitudinally to act as one continuous structure. Segmental structures shall be designed using the specific structure type provisions per AASHTO, except as amended herein.

**AASHTO
5.12.5**

5.10.2 Provision for Future Dead Load or Deflection Adjustment

The AASHTO provision for detailing segmental structures to accommodate future unbonded tendons that provide at least 10 percent of the positive and negative moment post-tensioning force may be waived in spans for which the long-term deflection is less than 0.5 percent of the span length. Long-term deflection variability can be easily affected by the unpredictability of ultimate creep and shrinkage coefficients, prestressing, losses, and weight; especially if the future wearing surface occurs early. The addition of 10 percent future tensioning to segmental spans with this magnitude of stiffness would not change long-term cambers significantly.

**AASHTO
5.12.5.3.9c**

This waiver is contingent upon the bridge being designed for a future wearing surface in accordance with Section 3 of this BDM.

APPENDIX 5A- DEVELOPMENT LENGTH & LAP SPLICE LENGTH DESIGN AIDS

Tables for development length and lap splices are provided for the following cases:

- Table 5A-1: Tension Development Length of Deformed Bars
- Table 5A-2: Tension Development Length of Epoxy-Coated Bars (Coating Factor = 1.5)
- Table 5A-3: Tension Development Length of Epoxy-Coated Bars (Coating Factor = 1.2)
- Table 5A-4: Compression Development Length and Minimum Lap Splices in Compression
- Table 5A-5: Tension Development Length of 90 and 180 Degree Standard Hooks
- Table 5A-6: Class B Tension Lap Splice Lengths of Deformed Bars
- Table 5A-7: Class B Tension Lap Splice Lengths of Epoxy-Coated Bars (Coating Factor = 1.5)
- Table 5A-8: Class B Tension Lap Splice Lengths of Epoxy-Coated Bars (Coating Factor = 1.2)

The Designer is responsible for calculating development lengths and lap splices for situations not covered by these tables.

Table 5A-1: Tension Development Length of Deformed Bars

		Tension Development Length (L_d) of Uncoated Deformed Bars (in.)							
		$\lambda_{rc} = 0.4$		$\lambda_{rc} = 0.6$		$\lambda_{rc} = 0.8$		$\lambda_{rc} = 1.0$	
Bar #	L_{db}	Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	25.5	14	12	20	16	27	21	34	26
4	33.9	18	14	27	21	36	28	45	34
5	42.4	23	17	34	26	45	34	56	43
6	50.9	27	21	40	31	53	41	67	51
7	59.4	31	24	47	36	62	48	78	60
8	67.9	36	28	53	41	71	55	89	68
9	76.6	40	31	60	46	80	62	100	77
10	86.2	45	35	68	52	90	69	113	87
11	95.7	50	39	75	58	100	77	125	96
14	114.9	60	46	90	69	120	92	150	115
18	153.2	80	62	120	92	160	123	200	154

Notes:

1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension development length is 12 in.
4. See AASHTO 5.10.8.2.1.

Calculation Variables:

$$\text{Tension Development Length, } L_d = L_{db} * \lambda_{rl} * \lambda_{cf} * \lambda_{rc} * \lambda_{er} / \lambda$$

$$\text{Basic Tension Development Length, } L_{db} = 2.4d_b * f_y / \sqrt{f_c}$$

$$\text{Reinforcement Location Factor, } \lambda_{rl} = 1.3 \text{ For top bars}$$

$$\lambda_{rl} = 1.0 \text{ For others}$$

$$\text{Coating Factor, } \lambda_{cf} = 1.0$$

$$\lambda_{rl} * \lambda_{cf} = 1.3 \text{ For top bars}$$

$$\lambda_{rl} * \lambda_{cf} = 1.0 \text{ For others}$$

$$\text{Excess Reinforcement Factor, } \lambda_{er} = 1.0$$

$$\text{Concrete Density Modification Factor, } \lambda = 1.0$$

$$\text{Reinforcing Steel Yield Strength, } f_y = 60 \text{ ksi}$$

$$\text{Compressive Strength of Concrete, } f_c = 4.5 \text{ ksi}$$

$$d_b = \text{bar diameter}$$

$$\text{Reinforcement Confinement Factor, } \lambda_{rc}: \text{ User shall calculate}$$

Table 5A-2: Tension Development Length of Epoxy-Coated Bars (Coating Factor = 1.5)

Tension Development Length (L_d) of Epoxy-Coated Steel Reinforcing Bars (in.) $\lambda_{cf} = 1.5$ (cover less than $3 \cdot d_b$ or clear spacing between bars less than $6 \cdot d_b$)									
		$\lambda_{rc} = 0.4$		$\lambda_{rc} = 0.6$		$\lambda_{rc} = 0.8$		$\lambda_{rc} = 1.0$	
Bar #	L_{db}	Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	25.5	18	16	26	23	35	31	44	39
4	33.9	24	21	35	31	47	41	58	51
5	42.4	29	26	44	39	58	51	73	64
6	50.9	35	31	52	46	70	62	87	77
7	59.4	41	36	61	54	81	72	101	90
8	67.9	47	41	70	62	93	82	116	102
9	76.6	53	46	79	69	105	92	131	115
10	86.2	59	52	88	78	118	104	147	130
11	95.7	66	58	98	87	131	115	163	144
14	114.9	79	69	118	104	157	138	196	173
18	153.2	105	92	157	138	209	184	261	230

Notes:

1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension development length is 12 in.
4. See AASHTO 5.10.8.2.1.

Calculation Variables:

$$\begin{aligned} \text{Tension Development Length, } L_d &= L_{db} \cdot \lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er} / \lambda \\ \text{Basic Tension Development Length, } L_{db} &= 2.4 d_b \cdot f_y / \sqrt{f_c} \\ \text{Reinforcement Location Factor, } \lambda_{rl} &= 1.3 \text{ For top bars} \\ &= 1.0 \text{ For others} \\ \text{Coating Factor, } \lambda_{cf} &= 1.5 \\ &= 1.7 \text{ For top bars (max. of 1.7)} \\ &= 1.5 \text{ For others (max. of 1.7)} \\ \text{Excess Reinforcement Factor, } \lambda_{er} &= 1.0 \\ \text{Concrete Density Modification Factor, } \lambda &= 1.0 \\ \text{Reinforcing Steel Yield Strength, } f_y &= 60 \text{ ksi} \\ \text{Compressive Strength of Concrete, } f_c &= 4.5 \text{ ksi} \\ d_b &= \text{bar diameter} \\ \text{Reinforcement Confinement Factor, } \lambda_{rc} &= \text{User shall calculate} \end{aligned}$$

Table 5A-3: Tension Development Length of Epoxy-Coated Bars (Coating Factor = 1.2)

		Tension Development Length (L_d) of Epoxy-Coated Steel Reinforcing Bars (in.) $\lambda_{cf} = 1.2$ (cover at least $3 \cdot d_b$ and clear spacing between bars at least $6 \cdot d_b$)							
		$\lambda_{rc} = 0.4$		$\lambda_{rc} = 0.6$		$\lambda_{rc} = 0.8$		$\lambda_{rc} = 1.0$	
Bar #	L_{db}	Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	25.5	16	13	24	19	32	25	40	31
4	33.9	22	17	32	25	43	33	53	41
5	42.4	27	21	40	31	53	41	67	51
6	50.9	32	25	48	37	64	49	80	62
7	59.4	38	29	56	43	75	58	93	72
8	67.9	43	33	64	49	85	66	106	82
9	76.6	48	37	72	56	96	74	120	92
10	86.2	54	42	81	63	108	83	135	104
11	95.7	60	46	90	69	120	92	150	115
14	114.9	72	56	108	83	144	111	180	138
18	153.2	96	74	144	111	192	148	240	184

Notes:

1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension development length is 12 in.
4. See AASHTO 5.10.8.2.1.

Calculation Variables:

$$\text{Tension Development Length, } L_d = L_{db} \cdot \lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er} / \lambda$$

$$\text{Basic Tension Development Length, } L_{db} = 2.4 d_b \cdot f_y / \sqrt{f_c}$$

$$\text{Reinforcement Location Factor, } \lambda_{rl} = 1.3 \text{ For top bars}$$

$$\lambda_{rl} = 1.0 \text{ For others}$$

$$\text{Coating Factor, } \lambda_{cf} = 1.2$$

$$\lambda_{rl} \cdot \lambda_{cf} = 1.6 \text{ For top bars (max. of 1.7)}$$

$$\lambda_{rl} \cdot \lambda_{cf} = 1.2 \text{ For others (max. of 1.7)}$$

$$\text{Excess Reinforcement Factor, } \lambda_{er} = 1.0$$

$$\text{Concrete Density Modification Factor, } \lambda = 1.0$$

$$\text{Reinforcing Steel Yield Strength, } f_y = 60 \text{ ksi}$$

$$\text{Compressive Strength of Concrete, } f_c = 4.5 \text{ ksi}$$

$$d_b = \text{bar diameter}$$

$$\text{Reinforcement Confinement Factor, } \lambda_{rc}: \text{ User shall calculate}$$

Table 5A-4: Compression Development Length and Minimum Lap Splices in Compression

Bar #	Min. Compressive Development Length (L_{db}) (in.)		Min. Compression Lap Splice, (L_c)(in.)
	$f'_c = 4.0$ ksi	$f'_c = 4.5$ ksi	$f'_c \geq 4.0$ ksi
3	8.00	8.00	12.00
4	9.45	9.00	15.00
5	11.81	11.25	18.75
6	14.18	13.50	22.50
7	16.54	15.75	26.25
8	18.90	18.00	30.00
9	21.32	20.30	33.84
10	24.00	22.86	38.10
11	26.65	25.38	42.30
14	32.00	30.47	50.79
18	42.66	40.63	67.71

Notes:

1. Values based on use of normal weight concrete.
2. Values based on use of grade 60 reinforcement.
3. The minimum compression development length is 8 in.
4. The minimum compression lap splice length is 12 in.
5. Where bars of different sizes are lap spliced in compression, the splice length shall not be less than the development length of the larger bar or the splice length of the smaller bar.
6. See AASHTO 5.10.8.2.2 and 5.10.8.4.5.

Calculation Variables:

$$\begin{aligned} \text{Basic Development Length, } L_{db} &= 0.63 \cdot d_b \cdot f_y / \sqrt{f'_c} \\ L_{db}(\text{lower limit}) &= 0.3 \cdot d_b \cdot f_y \\ \text{Minimum Compression Lap Splice, } L_c &= m \cdot (0.9 \cdot f_y - 24) \cdot d_b \\ \text{Modification Factor, } m &= 1.0 \end{aligned}$$

Table 5A-5: Tension Development Length of 90 and 180 Degree Standard Hooks

		Standard Hook Tension Development Length L_{dh} (in.)			
Bar #	L_{hb} (in.)	Cover Factor $\lambda_{RC} = 1.0$		Cover Factor $\lambda_{RC} = 0.8$	
		Spacing Factor $\lambda_{rc} = 1.0$	Spacing Factor $\lambda_{rc} = 0.8$	Spacing Factor $\lambda_{rc} = 1.0$	Spacing Factor $\lambda_{rc} = 0.8$
3	6.7	6.72	6.00	6.00	6.00
4	9.0	8.96	7.17	7.17	6.00
5	11.2	11.20	8.96	8.96	7.17
6	13.4	13.44	10.75	10.75	8.60
7	15.7	15.67	12.54	12.54	10.03
8	17.9	17.91	14.33	14.33	11.46
9	20.2	20.21	16.17	16.17	12.93
10	22.7	22.75	18.20	18.20	14.56
11	25.3	25.26	20.21	20.21	16.17

Notes:

1. Values based on use of normal weight concrete.
2. The minimum development length is 6 in.
3. See AASHTO 5.10.8.2.4.

Calculation Variables:

$$\text{Basic Development Length, } L_{hb} = 38*d_b/60*f_y/(\lambda*\text{sqrt}(f'_c))$$

$$L_{hb}(\text{lower limit}) = 8*d_b$$

$$\text{Concrete Density Modification factor, } \lambda \text{ taken as } 1.0$$

$$\text{Reinforcing Steel Yield Strength, } f_y = 60 \text{ ksi}$$

$$\text{Compressive Strength of Concrete, } f'_c = 4.5 \text{ ksi}$$

Table 5A-6: Class B Tension Lap Splice Lengths of Deformed Bars

Class B Tension Lap Splice Length of Uncoated Deformed Bars (in.)								
$\lambda_{rc} = 0.4$		$\lambda_{rc} = 0.6$		$\lambda_{rc} = 0.8$		$\lambda_{rc} = 1.0$		
Bar #	Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	17.21	15.60	25.81	19.86	34.42	26.47	43.02	33.09
4	22.94	17.65	34.42	26.47	45.89	35.30	57.36	44.12
5	28.68	22.06	43.02	33.09	57.36	44.12	71.70	55.15
6	34.42	26.47	51.62	39.71	68.83	52.95	86.04	66.19
7	40.15	30.89	60.23	46.33	80.30	61.77	100.38	77.22
8	45.89	35.30	68.83	52.95	91.78	70.60	114.72	88.25
9	51.76	39.82	77.64	59.73	103.52	79.63	129.41	99.54
10	58.28	44.83	87.42	67.24	116.56	89.66	145.70	112.07
11	64.70	49.77	97.05	74.66	129.41	99.54	161.76	124.43

Notes:

1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension lap splice length is 12 in.
4. See AASHTO 5.10.8.4.3a.

Calculation Variables:

Class B Lap Splice Length = $1.3 \cdot L_d$
 Reinforcing Steel Yield Strength, f_y = 60 ksi
 Compressive Strength of Concrete, f_c = 4.5ksi
 Reinforcement Confinement Factor, λ_{rc} : User shall calculate

**Table 5A-7: Class B Tension Lap Splice Lengths of Epoxy-Coated Bars
(Coating Factor = 1.5)**

Class B Tension Lap Splice Length of Epoxy-Coated Steel Reinforcing Bars (in.) $\lambda_{cf} = 1.5$ (cover less than $3 \cdot d_b$ or clear spacing between bars less than $6 \cdot d_b$)								
$\lambda_{rc} = 0.4$		$\lambda_{rc} = 0.6$		$\lambda_{rc} = 0.8$		$\lambda_{rc} = 1.0$		
Bar #	Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	22.50	19.86	33.75	29.78	45.01	39.71	56.26	49.64
4	30.00	26.47	45.01	39.71	60.01	52.95	75.01	66.19
5	37.50	33.09	56.26	49.64	75.01	66.19	93.76	82.73
6	45.01	39.71	67.51	59.57	90.01	79.42	112.51	99.28
7	52.51	46.33	78.76	69.49	105.01	92.66	131.27	115.82
8	60.01	52.95	90.01	79.42	120.02	105.90	150.02	132.37
9	67.69	59.73	101.53	89.59	135.38	119.45	169.22	149.31
10	76.21	67.24	114.32	100.87	152.42	134.49	190.53	168.11
11	84.61	74.66	126.92	111.99	169.22	149.31	211.53	186.64

Notes:

1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension lap splice length is 12 in.
4. λ_{rc} is the Reinforcement Confinement Factor (user shall calculate).
5. See AASHTO 5.10.8.4.3a.

Calculation Variables:

$$\begin{aligned} \text{Class B Lap Splice Length} &= 1.3 \cdot L_d \\ \text{Reinforcing Steel Yield Strength, } f_y &= 60 \text{ ksi} \\ \text{Compressive Strength of Concrete, } f_c &= 4.5 \text{ ksi} \end{aligned}$$

**Table 5A-8: Class B Tension Lap Splice Lengths of Epoxy-Coated Bars
(Coating Factor = 1.2)**

Class B Tension Lap Splice Length of Epoxy-Coated Steel Reinforcing Bars (in.) $\lambda_{cf} = 1.2$ (cover at least $3 \cdot db$ and clear spacing between bars at least $6 \cdot db$)								
$\lambda_{rc} = 0.4$		$\lambda_{rc} = 0.6$		$\lambda_{rc} = 0.8$		$\lambda_{rc} = 1.0$		
Bar #	Top Bars	Others	Top Bars	Others	Top Bars	Others	Top Bars	Others
3	20.65	15.88	30.97	23.83	41.30	31.77	51.62	39.71
4	27.53	21.18	41.30	31.77	55.07	42.36	68.83	52.95
5	34.42	26.47	51.62	39.71	68.83	52.95	86.04	66.19
6	41.30	31.77	61.95	47.65	82.60	63.54	103.25	79.42
7	48.18	37.06	72.27	55.60	96.37	74.13	120.46	92.66
8	55.07	42.36	82.60	63.54	110.13	84.72	137.67	105.90
9	62.11	47.78	93.17	71.67	124.23	95.56	155.29	119.45
10	69.93	53.80	104.90	80.69	139.87	107.59	174.83	134.49
11	77.64	59.73	116.46	89.59	155.29	119.45	194.11	149.31

Notes:

1. Values based on use of normal weight concrete.
2. Top bars are horizontal bars placed so more than 12 in. of fresh concrete is cast below the reinforcement.
3. The minimum tension lap splice length is 12 in.
4. λ_{rc} is the Reinforcement Confinement Factor (user shall calculate).
5. See AASHTO 5.10.8.4.3a.

Calculation Variables:

$$\begin{aligned} \text{Class B Lap Splice Length} &= 1.3 \cdot L_d \\ \text{Reinforcing Steel Yield Strength, } f_y &= 60 \text{ ksi} \\ \text{Compressive Strength of Concrete, } f_c &= 4.5 \text{ ksi} \end{aligned}$$

APPENDIX 5B - GIRDER PRELIMINARY DESIGN AIDS

General

The following table and graphs are design aids to assist with the selection of girder types and spacing for preliminary design only. See Section 6 for span capabilities of standardized steel girders.

Design assumptions for the table and the graphs are the same, except the f'_{ci} in the table may be up to 8,500 psi at the time of post-tensioning for spliced spans.

Table 5B-1

The span capabilities shown may be limited by maximum shipping weight (see Section 5.5.1.9) or site-specific limitations. For the table, the following assumptions apply:

- No splices in simple span
- One splice in end spans
- Two splices in interior spans

Haunched pier segments were not assumed but may be feasible. Pier segments may require a thickened top flange and a thickened web. Economic spliced span capabilities were based on 4 ft. clear between flanges.

The box section properties shown are for 6 in. webs, 6 in. bottom flange, and 4 in. top flange. Actual box depths used on a project should optimize use of the available superstructure depth.

Figures 5B-1 through 5B-3

The graphs are intended to provide a quick means to compare relative costs between options. Actual cost estimates should reflect unit costs based on specific project constraints and current market conditions.

Table 5B-1: Economic Span Capabilities

PRECAST SECTION PROPERTIES							ECONOMIC SPAN CAPABILITIES			
APPROXIMATE							SIMPLE SPAN		SPLICED	
NAME	WIDTH IN	AREA IN ²	CG IN	INERTIA IN ⁴	EMS IN	EE IN	FROM FT	TO FT	END FT	INT FT
BT84	43	948	41.7	875207	5	22	120	172	200	240
BT72	43	864	35.8	594437	5	20	106	178	180	210
BT63	43	801	31.4	425875	5	18	90	162	160	190
BT54	43	738	27	289236	5	16	72	-143	140	170
BT42	43	654	21.1	153066	5	14	55	-114	114	130
BX44	72	1128	20.5	319160	3	~9	116	133	N/A	N/A
BX44	48	906	20.7	224630	3	~12	75	128	140	170
BX35	72	1038	16.1	177917	3	~7	95	-128	N/A	N/A
BX35	48	780	16.6	129108	3	~10	65	108	110	130
BX24	72	906	11.1	68313	3	~6	-79	-88	N/A	N/A
BX24	48	666	11.3	46880	3	~7	44	-79	N/A	N/A
BX18	72	834	8.4	31885	3	~5	-65	-71	N/A	N/A
BX18	48	594	8.5	21557	3	~6	36	-65	N/A	N/A
SL16	72	1152	8	24576	2.4	2.4	41	-47	N/A	N/A
SL14	72	1008	7	16464	2.4	2.4	36	-42	N/A	N/A
SL12	72	864	6	10368	1.9	1.9	31	-40	N/A	N/A
SL10	72	720	5	6000	1.8	1.8	25	-37	N/A	N/A
SL8	72	576	4	3072	1.8	1.8	24	-31	N/A	N/A
SL6	72	432	3	1296	1.7	1.7	14	-24	N/A	N/A
SL4	72	288	2	384	1.7	1.7	0	14	N/A	N/A

- Designates a span length which requires continuity to control live load deflection.

N/A Designates sections that typically cannot benefit from spliced design.

~ Designates typical EE if harping is used. Path may be harped and/or sleeved strands and/or bottom slab thickening used near supports to control stresses.

EMS and EE may vary due to design requirements and shop capabilities, representative values are shown.

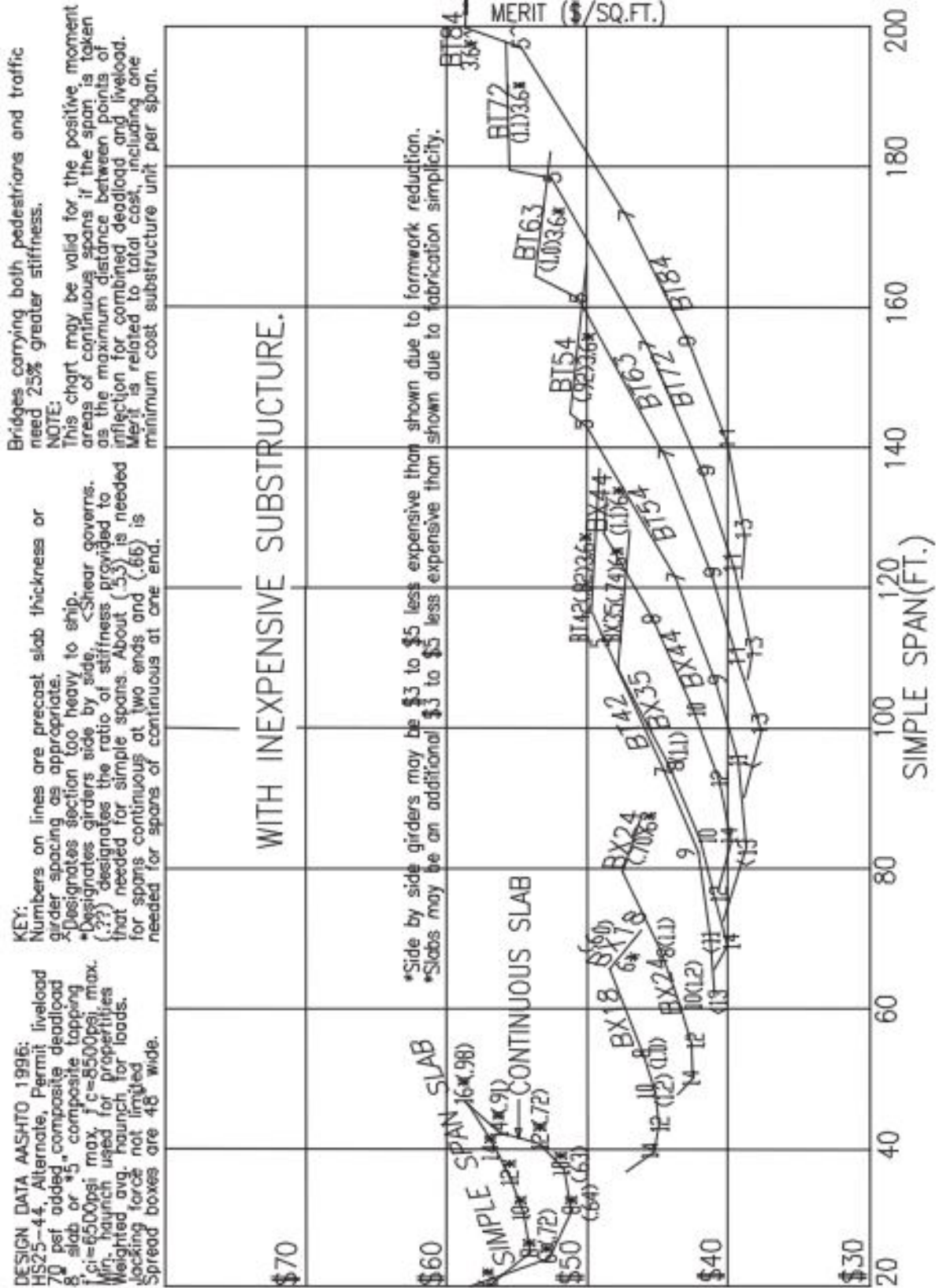


Figure 5B-1: Simple Span Girder Capabilities with Inexpensive Substructures

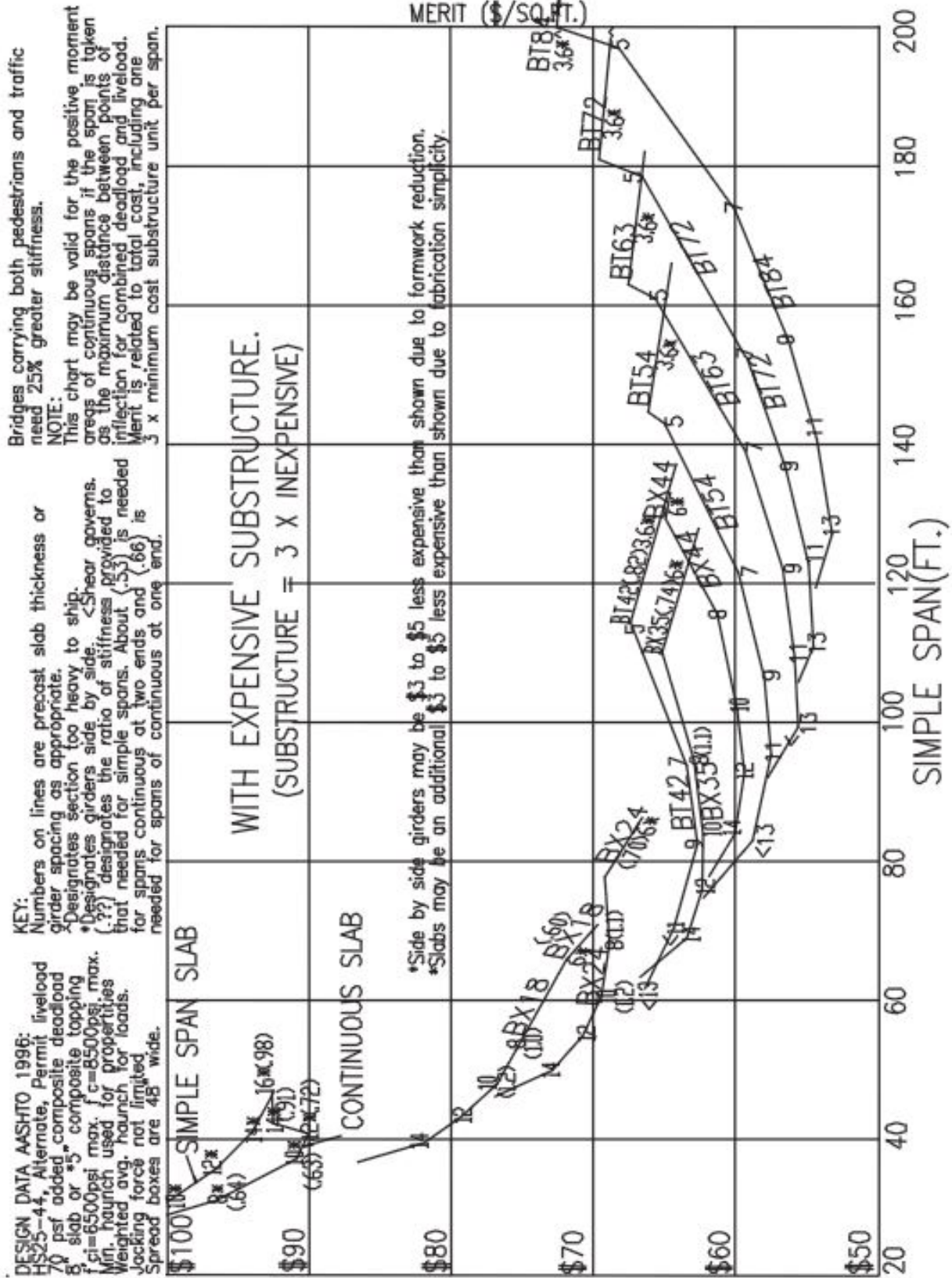


Figure 5B-2: Simple Spans Girder Capabilities with Expensive Substructures

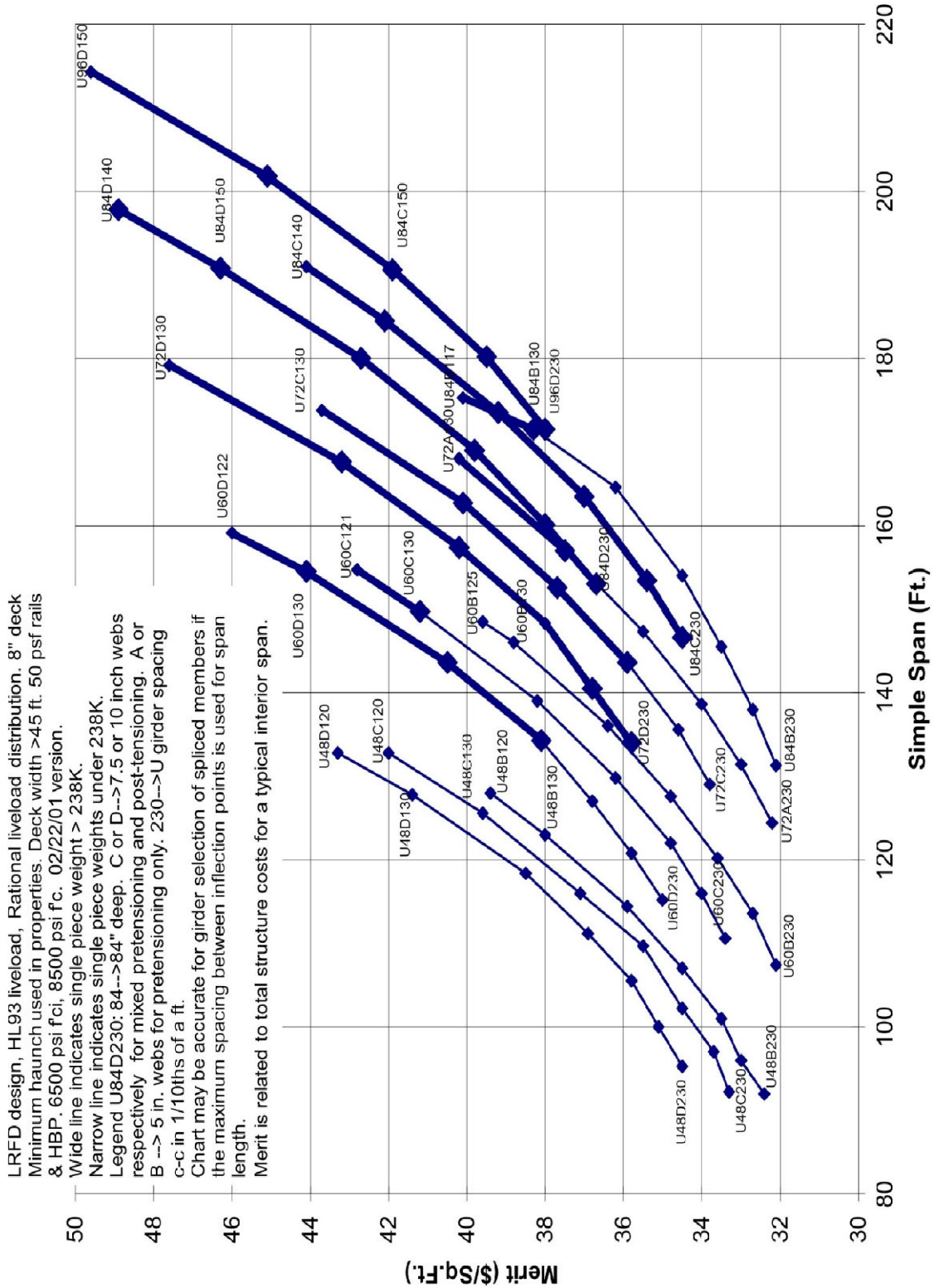
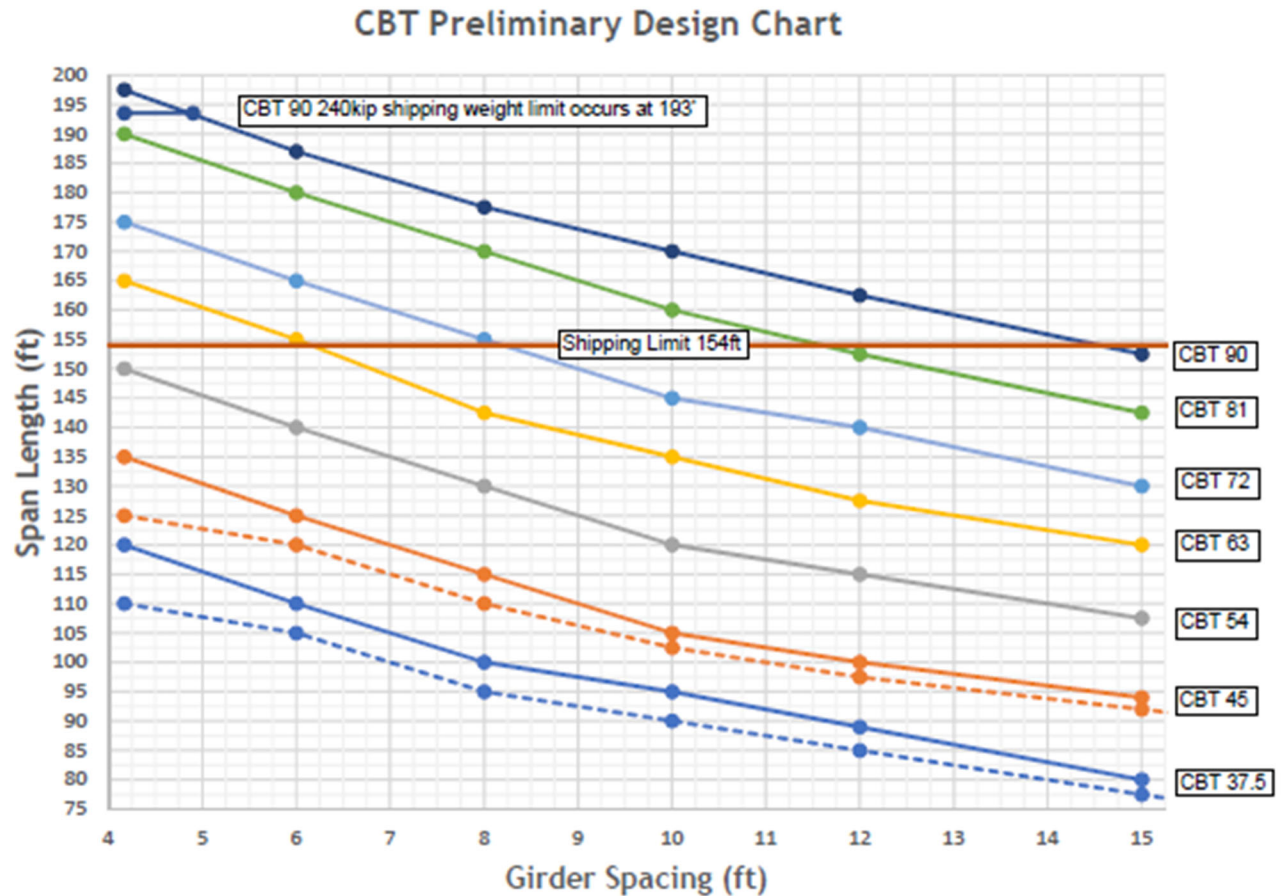


Figure 5B-3: Proposed Colorado U Girders

**Notes:**

Design Criteria: LRFD 9th edition, simple spans assumed with no skew. Design aid may be used for continuous spans per LRFD 5.12.3.3.

Criteria complies with CDOT BDM standard practices and policies including but not limited to concrete strengths, camber under tolerance, hold down forces, zero tension under dead load, etc.

Solid lines denote harped strands, dashed lines denote straight strands

Complies with local manufacturers' capabilities

Assumed loads:

DC Noncomposite: 4" total weighted average haunch (1" used for section properties)

DC Composite: Type 9 bridge rail

DW Composite: 42psf between curbs for 3" HMA overlay and utilities

LL: HL-93 & Colorado Permit Vehicle

Figure 5B-4: Colorado CBT Girders

SECTION 6 STEEL STRUCTURES

6.1 GENERAL REQUIREMENTS

The following section is provided as CDOT practice for steel structure design. The Designer shall coordinate with Staff Bridge regarding project-specific circumstances warranting deviations from standard practices referenced herein.

This section is complementary to the CDOT Bridge Detail Manual, CDOT Standard Specifications for Road and Bridge Construction, Bridge Structural Worksheets, and AASHTO/AWS D1.5M/D1.5 Bridge Welding Code.

CDOT strongly recommends constructability reviews for new steel structures; review Section 37 of this BDM for more information.

Refer to BDM Section 10 – Foundations for steel piling design.

Refer to BDM Section 33 – Preservation and Rehabilitation of Structures for rehabilitation of steel structures.

The following recommended resources for steel bridge design include design examples of I-girder and tub/box girder design:

- American Institute of Steel Construction (AISC) / National Steel Bridge Alliance (NSBA) website (<https://www.aisc.org/nsba/>)
- FHWA website (<https://www.fhwa.dot.gov/bridge/steel.cfm>)

Found on the NSBA website, the “Short Span Steel Alliance Standards” should be used only for preliminary design and are not acceptable for final design. Refer to Section 37 of this BDM for acceptable final design calculations.

6.2 CODE REQUIREMENTS

Designs shall be consistent with AASHTO, unless modified herein.

**AASHTO
4.4**

6.3 MATERIAL PROPERTIES

The economics of design, expected length, and location of a bridge generally govern the choice of girder material. Steel girders shall be rolled I-beams, welded plate I-girders, or tub/box girders.

6.3.1 Steel Components

All structural steel components, including structural steel, bolts, nuts, washers, and shear connectors, shall be in accordance with AASHTO steel grades, strengths, available thicknesses, and properties.

**AASHTO
6.4.1**

Assume 50 ksi as the default steel yield strength. CDOT allows hybrid sections. During the design phase, the Designer should contact fabricators and NSBA to verify that the design cross section is the most economical.

Generally, unpainted weathering steel may be specified for CDOT bridge girders, with the following exceptions:

1. Highway overpasses, with minimum vertical clearance, subject to a “tunnel” effect, where vertical abutments are used and full height retaining walls continue parallel to the abutment. This situation does not allow roadway spray with deicing salt to dissipate by air currents.
2. Low level water crossings where the girder has less than 8 ft. clearance to the Ordinary High Water elevation. This situation could result in prolonged periods of wetness of the steel.

When specifying unpainted weathering steel, the last 6 ft. of girders on either side of an expansion joint shall be painted, equivalent to Federal Standard 595B Color No. 30045 (weathered steel color).

Unpainted weathering steel shall not be used for railings. See Section 2.4 and Section 13 of this BDM for minimum railing and fencing steel requirements.

Refer to Section 14 of this BDM for additional information on bearing design.

6.3.1.1 Bolts

ASTM F3125 Grade A325 high strength bolts are preferred. ASTM F3125 Grade A490 bolts should be used only when necessary. It is preferred practice not to mix A325 and A490 bolt types in the same connection type. However, if the use of mixed bolt grades is justified, it is recommended that different bolt diameters be used to distinguish between the grades during construction.

A490 bolts shall not be hot-dip galvanized. If a zinc coating is required, it must follow the mechanically deposited process.

The twist-off versions of Grade A325 and A490, F1852, and F2280, respectively, are acceptable options in structural steel joints.

6.4 FATIGUE AND FRACTURE CONSIDERATIONS

6.4.1 Fatigue

Fatigue shall be categorized as load- or distortion-induced fatigue. Refer to AASHTO LRFD for fatigue design criteria for steel components and details.

**AASHTO
6.6.1**

6.4.2 Fracture

Refer to AASHTO LRFD for members that require mandatory Charpy V-Notch testing. If needed, Section 509 of CDOT Standard Specifications may be revised with a Project Special Provision to resolve any differences with AASHTO over which components and connections require Charpy V-Notch testing. The Designer shall clearly identify on the contract plans all components and connections requiring Charpy V-Notch testing.

**AASHTO
6.6.2**

The Designer shall clearly identify on contract documents all main members and/or details that are to receive non-destructive testing.

A Fracture Critical Member (FCM) is defined as a component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to safely carry a minimum level of traffic (live load) in its damaged condition. The Designer has the responsibility to clearly identify all FCMs on the contract plans. All FCMs shall be fabricated to AASHTO/AWS D1.5M/D1.5 Bridge Welding Code.

As a default, Staff Bridge considers the following (but not limited to) fracture critical members:

- Flanges and webs in tension on single-box girder bridges
- Flanges in tension in two-box girder bridges
- Girders or tension sub-elements in a built-up member on a bridge with fewer than three girders

**AASHTO
6.11.5**

The Designer has the option to perform a rigorous analysis with assumed cracked components to confirm the strength and stability of a damaged structure. However, the loading cases to be evaluated, the location of potential cracks, the appropriate level of live loads, the degree to which dynamic effects associated with a fracture are included, the refinement of models, and the choice of element type shall all be agreed upon with Staff Bridge. The ability of a software product to adequately capture the complexity of the analysis shall be considered and mutually agreed upon with Staff Bridge.

6.5 GENERAL DIMENSION AND DETAIL REQUIREMENTS

6.5.1 General

Steel structure components shall be in accordance with the CDOT Bridge Detail Manual and AASHTO LRFD.

6.5.2 Dead Load Camber

The general requirements for camber shall be in accordance with AASHTO LRFD, unless modified in this Section.

**AASHTO
6.7.2**

Steel girders should be cut or heat cambered to counteract service dead load deflections and vertical profile requirements as needed.

A tabulation showing dead load deflections for the girder only, slab only, and total shall be shown with the Girder Elevation, if “Camber and Dead Load Deflection” sheets are not used.

For straight skewed I-girder bridges and horizontally curved I-girder bridges, the Designer should clearly state in the contract documents the intended erected position of girders and fit condition. The preference is to use Steel Dead Load Fit conditions, but the Designer should consider the economic implications of using other conditions. The selected fit condition must be either recommended or acceptable in NSBA’s “Steel I-Girder Bridge Fit” Table 3 and Table 4 (shown on Figure 6-1). The complete document is found on NSBA’s website; the link is provided at the beginning of this section.

Table 1 Common Fit Conditions

Loading Condition Fit	Construction Stage Fit	Description	Practice
No-Load Fit (NLF)	Fully-Cambered Fit	The cross-frames are detailed to fit to the girders in their fabricated, plumb, fully-cambered position under zero dead load.	The fabricator (detailer) sets the drops using the no-load elevations of the girders (i.e., the fully cambered girder profiles).
Steel Dead Load Fit (SDLF)	Erected Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge steel dead load at the completion of the erection.	The fabricator (detailer) sets the drops using the girder vertical elevations at steel dead load, calculated as the fully cambered girder profiles minus the steel dead load deflections.
Total Dead Load Fit (TDLF)	Final Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge total dead load.	The fabricator (detailer) sets the drops using the girder vertical elevations at total dead load, which are equal to the fully cambered girder profiles minus the total dead load deflections.

Table 3 Recommended Fit Conditions for Straight I-Girder Bridges
(including Curved I-Girder Bridges with L/R in all spans ≤ 0.03)¹

Square Bridges and Skewed Bridges up to 20 deg Skew			
	Recommended	Acceptable	Avoid
Any span length	Any		None
Skewed Bridges with Skew > 20 deg and $I_S \leq 0.30$ +/-			
	Recommended	Acceptable	Avoid
Any span length	TDLF or SDLF		NLF
Skewed Bridges with Skew > 20 deg and $I_S > 0.30$ +/-			
	Recommended	Acceptable	Avoid
Span lengths up to 200 ft +/-	SDLF	TDLF	NLF
Span lengths greater than 200 ft +/-	SDLF		TDLF & NLF

Table 4 Recommended Fit Conditions for Horizontally Curved I-Girder Bridges
($(L/R)_{MAX} > 0.03$)¹

Radial or Skewed Supports			
	Recommended	Acceptable	Avoid
$(L/R)_{MAX} \geq 0.2$	NLF ^{2,3}	SDLF ⁴	TDLF
All other cases	SDLF	NLF	TDLF

Note 1: For the various recommended fit conditions presented in Tables 3 and 4, the span length and skew index limits should be considered approximate guidelines and should be evaluated in the full context of the geometric and structural complexity of the given bridge.

Note 2: The recommendation transitions to NLF at or above a maximum L/R of 0.2 because research on these types of bridges (NCHRP 2015) shows that the increase in the cross-frame forces from SDLF detailing can become more significant as the degree of curvature increases. NLF matches the normal analysis methods used in the design and will provide a better match between predicted forces and displacements than SDLF when the steel dead load displacements become large.

Note 3: The recommendation to use NLF detailing does not necessarily imply the need to use temporary shoring, nor does the use of SDLF or TDLF detailing imply that temporary shoring cannot be used. The choice of detailing method affects the nature and magnitude of the bridge's internal dead load forces as well as the "fit-up" forces which the erector may need to apply to assemble the structural steel. The nature and magnitude of these forces are also influenced by the use of temporary shoring. Bridges erected without temporary shoring can be detailed for NLF and successfully erected if the fit-up forces are manageable. Likewise, bridges which are to be erected using some form of temporary shoring can be detailed for SDLF or TDLF and successfully erected if the fit-up forces are manageable.

Note 4: SLDF detailing is considered acceptable in these cases if the additive locked-in force effects are considered

L = span length, bearing to bearing along the centerline of the bridge
R = radius of the centerline of the bridge cross-section
 I_S = skew index

AASHTO
Eq. 4.6.3.3.2-2

Figure 6-1: Recommended Fit Conditions

Because box girders are inherently stiff in torsion, it is difficult to achieve fit-up of Total Dead Load Fit conditions. As a result, external cross-frames are typically detailed and fabricated to fit to the girder geometry under No-Load Fit or Steel Dead Load Fit conditions, depending on the intended erection sequencing.

**AASHTO
6.7.2**

For curved or skewed box girder bridges where a line girder analysis was not used, report deflections along individual webs, not along the centerline of the girder.

6.5.3 Minimum Thickness of Steel

Refer to AASHTO LRFD and AASHTO/NSBA G12.1, *Guidelines to Design for Constructability*, for minimum thicknesses of steel elements.

**AASHTO
6.7.3**

6.5.4 Diaphragms and Cross-Frames

Refer to AASHTO LRFD for the design and placement of diaphragms or cross-frames for main I-beam and box section members.

**AASHTO
6.7.4**

CDOT prefers bolted connections. No tack welding is allowed before bolting.

Per AASHTO, all members included in the structural model that are used to determine girder force effects shall be designed as primary members. This includes all diaphragms or cross-frames in horizontally curved and heavily skewed bridges.

In general, for bridges with skew angles of 20° or less, the diaphragms or cross-frames shall be placed parallel to the centerline of the support.

For bridges with skew angles greater than 20°, the diaphragms or cross-frames shall be placed perpendicular to the main members.

Where a support line at an interior pier is skewed more than 20° from normal, elimination of the diaphragms or cross-frames along the skewed interior support line may be considered. Verify with Staff Bridge that this is an acceptable option.

6.5.4.1 Box Section Members

The need for temporary or permanent intermediate internal diaphragms or cross-frames, external diaphragms or cross-frames, top lateral bracing, or other means shall be investigated to ensure that deformations of the box section are controlled. If temporary intermediate diaphragms are specified, they shall be removed once the entire deck is poured and has achieved its full design strength.

6.6 I-SECTION FLEXURAL MEMBERS

Refer to Section 9.4.2 of this BDM for design of concrete decks and overhangs.

CDOT does not allow chorded girders, except for a simple made continuous design (see Section 6.6.6).

6.6.1 Composite Sections

The Designer shall refer to AASHTO LRFD to calculate composite section properties in positive and negative moment regions.

**AASHTO
6.6.1.2.1,
6.10.1.1.1b & c,
& 6.10.4.2**

In situations where AASHTO does not consider the concrete deck when calculating composite girder section properties in negative moment regions, only longitudinal reinforcing in the top mat, within the effective deck width, shall be considered effective. If a project does not allow precast deck panels, then the bottom longitudinal reinforcing may also be considered effective when calculating said section properties.

6.6.2 Minimum Negative Flexure Concrete Deck Reinforcement

Refer to AASHTO LRFD for the minimum negative moment flexure reinforcement.

**AASHTO
6.10.1.7**

6.6.3 Non-composite Sections

CDOT does not permit the use of non-composite sections in positive moment locations. If the Designer finds that it is not economical to use composite sections in negative moment regions, shear connectors and longitudinal reinforcing shall satisfy requirements in AASHTO LRFD for the contra-flexure points.

**AASHTO
6.10.10.3**

6.6.4 Constructability

Satisfy all requirements in AASHTO LRFD for primary members at all critical construction phases.

**AASHTO
6.10.3**

6.6.4.1 Wind Loads During Construction

The Designer shall verify stability of girders due to wind loading on girders during construction. Be aware that not all design software addresses these checks. Refer to *AASHTO Guide Specifications for Wind Loads on Bridges During Construction* for guidance.

6.6.4.2 Deck Placement

Sections in positive flexure that are composite in the final condition, but are non-composite during construction, shall be investigated for flexure during the various stages of deck placement.

**AASHTO
6.10.3.4**

Cantilevered brackets placed along the exterior girders typically support concrete deck overhang construction loads. The overhang brackets with construction loads, such as screeds, can result in excessive deflections and rotation on exterior girders. The Designer may assume a deck overhang bracket configuration shown in Figure 6-2, with the brackets extending to the bottom flange, which is preferred. Alternatively, the brackets may bear on the girder web if means are provided to ensure that the web is not damaged and the associated deformations permit proper placement of the concrete deck.

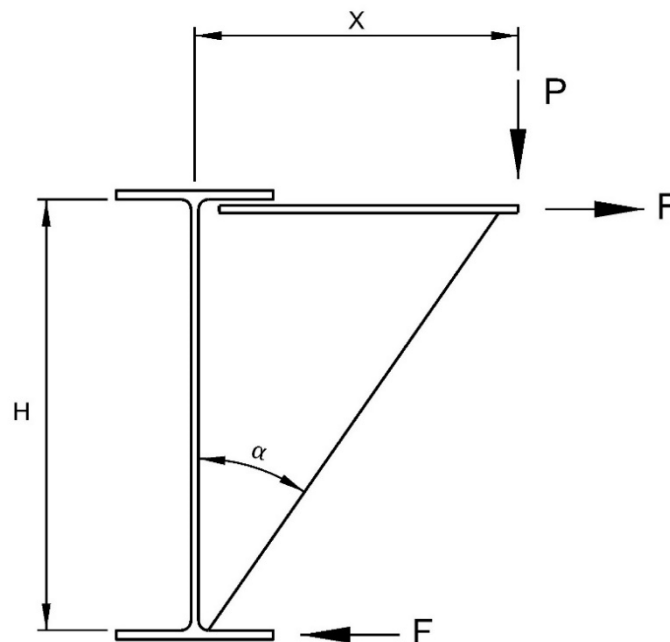


Figure 6-2: Deck Overhang Bracket

Although the brackets are typically spaced at 3 to 4 ft. along the exterior girder, all bracket loads except the finishing machine load are assumed to be applied uniformly. The Designer calculates the vertical load, P , acting at the edge of the overhang bracket. The bracket is assumed to extend near the edge of the deck overhang; therefore, half the deck overhang weight is placed on the exterior girder and half the weight is placed on the overhang brackets. Designers may conservatively include one-half the deck haunch weight in the total overhang weight.

Construction loads or dead loads and temporary loads that act on the overhang only during construction are assumed (as minimum) as follows:

Overhang deck forms: $P = 40$ lbs/ft.

Screed rail: $P = 85$ lbs/ft.

Railing: $P = 25$ lbs/ft.

Walkway: $P = 125$ lbs/ft.

Finishing machine: $P = 3,000$ lbs

The finishing machine load is estimated as one-half of the total finishing machine truss weight, plus additional load to account for the weight of the engine; drum and operator are assumed to be located on one side of the truss. Note: The above loads are estimates only. It is recommended that the Designer contact the Contractor, if known at the time of design, to obtain more accurate construction load values. Otherwise, the Designer shall validate the assumed loads during review of construction submittals regarding the deck forming system and finishing machine.

Falsework shall not be used for new construction and deck replacement construction. If falsework appears necessary during design, discuss with Staff Bridge.

6.6.5 Longitudinal Stiffeners and Cover Plates

CDOT does not allow longitudinal stiffeners and cover plates on new construction without Unit Leader approval in coordination with the State Bridge Engineer.

6.6.6 Simple Made Continuous

The Designer is encouraged to consider simple made continuous (SMC) bridges in the design of multi-span structures. SMC bridges reduce uplift in unbalanced spans, reduce negative moments at the piers, simplify fabrication, and eliminate the need for bolted field splices.

Critical to the functionality of SMC structures is the continuity connection at the piers. As industry best practices develop with new research, the Designer is encouraged to discuss with Staff Bridge to select the most appropriate connection details for design and construction.

AISC Engineering Journal, Second Quarter, 2014, provides commentary on several connection details and a design procedure for SMC bridges.

6.7 TUB/BOX-SECTION FLEXURAL MEMBERS

6.7.1 General

This section supplements AASHTO Section 6.11.

For reference, see the NSBA website for design examples for straight and curved box/tub girders (<https://www.aisc.org/nsba/nsba-publications/steel-bridge-design-handbook/>).

6.7.2 Bearings

Straight, not skewed, tub girders may use two bearings at supports. Curved or skewed tub girders should have one bearing at supports.

6.7.3 Cross-Section Proportion Limits

In 2006, the Transportation Research Board (TRB) published a paper "Practical Steel Tub Girder Design," providing guidance for preliminary design considerations, including preliminary girder sizing and spacing.

Bottom flange longitudinal stiffeners are permitted, but unstiffened bottom flanges are preferred. Using longitudinal stiffeners may result in undesirable fatigue details. The Designer should investigate thickening the bottom flange and/or reducing the bottom flange width in lieu of using longitudinal stiffeners. Bottom longitudinal stiffeners that terminate at the girder ends or at the end of the flange at splices do not present a fatigue problem.

Box girder segment widths greater than 12 ft. may present transportation issues and should be avoided where feasible. Be aware of girder curvature because it increases the overall out-to-out segment width. Consult with

fabricators on shipping when wider segments could eliminate an extra field splice.

Maximum allowable shipping lengths are highly variable from state to state, but 120 ft. is a common restriction. Splices may be used, but the Designer is recommended to consult potential fabricators of tub/box girders when laying out field splices.

Provisions for adequate draining and ventilation of the interior of the tub are essential. Appendix 6A provides a typical drain hole detail.

6.8 CONNECTIONS AND SPLICES

6.8.1 Bolted Connections

Bolted connections of primary members subjected to significant load reversals, heavy impact loads, severe vibration or where a joint slippage would be detrimental to the serviceability of the structure shall be designated and designed as slip-critical. Load cases during construction should be considered.

**AASHTO
6.13.2.1.1**

Typical slip-critical connections include, but are not limited to, the following connections:

- Girder splices
- Connections for primary member diaphragm that experience axial tension or combined axial tension and shear
- Any connection in shear with oversized or slotted holes

The most typical surface condition used in Colorado is Class A, unpainted clean mill scale, and blast-cleaned surfaces with Class A coatings. Where special consideration is necessary, Class B surfaces may be submitted to Staff Bridge for approval by Unit Leader in coordination with the Fabrication/Construction Unit. When Class B friction surfaces are used, the plans shall specify the connection surface conditions that must be present at the time of bolting.

6.8.2 Flange Splices

CDOT allows flange width and thickness transitions at splices. The Designer should ensure that enough material is saved for the flange transition to be cost effective.

6.8.3 Welded Connections

Other than welds between girders and bearing plates, CDOT does not permit any field welds or permanent tack welds.

Full penetration welds on webs and flanges made with backing should not be allowed. The following pre-qualified welds may be used: B-U3c-S, B-L1a-S, B-L2c-S, B-U6-S, C-U6-S, and B-U7-S.

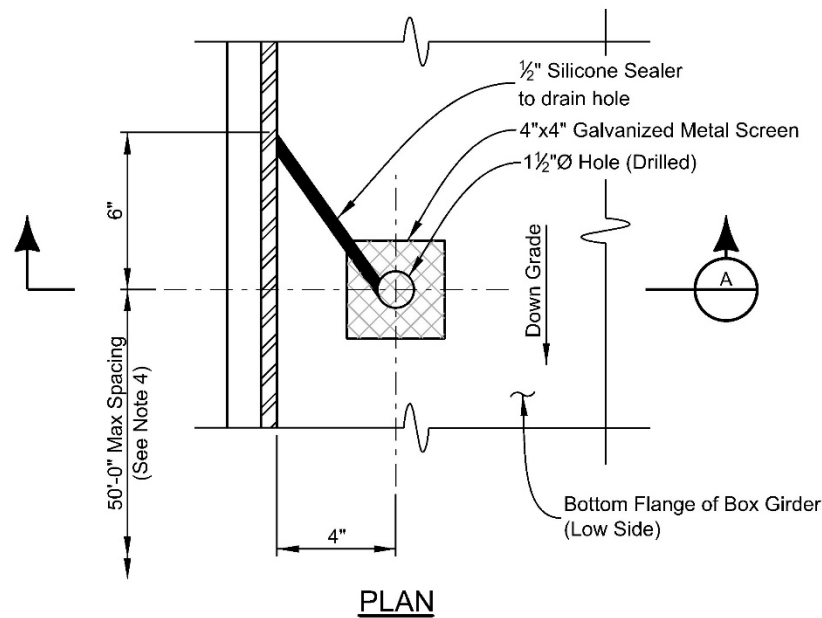
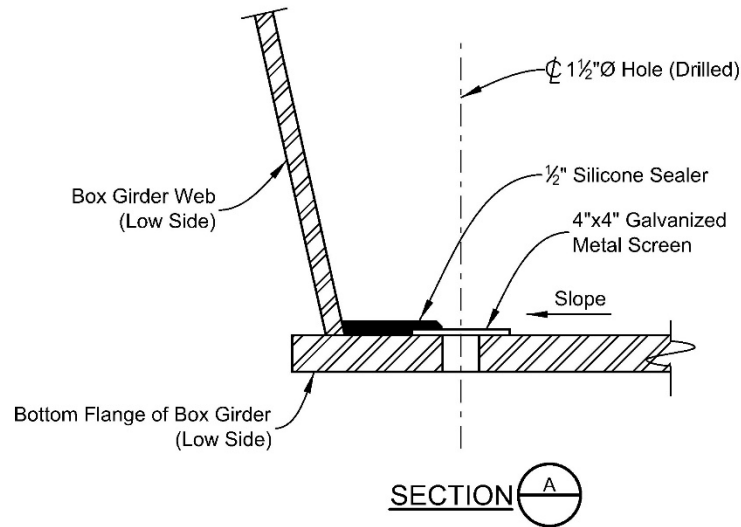
6.9 REFERENCES

Azizinamini, Atorod. "Simple for Dead Load-Continuous for Live Load Steel Bridge Systems." *Engineering Journal*. American Institute of Steel Construction, Vol. 51, Quarter 2 (2014): 59–82.

Coletti, Domenic; Zhanfei (Tom) Fan; John Holt; John Vogel. "Practical Steel Tub Girder Design." Transportation Research Board 85th Annual Meeting, 2006.

Robert I. Johnson, and Rebecca A. Atadero. "Simple-Made-Continuous Steel Bridges with Steel Diaphragms." *Engineering Journal*. American Institute of Steel Construction, Vol. 54, Quarter 1 (2017): 3–20.

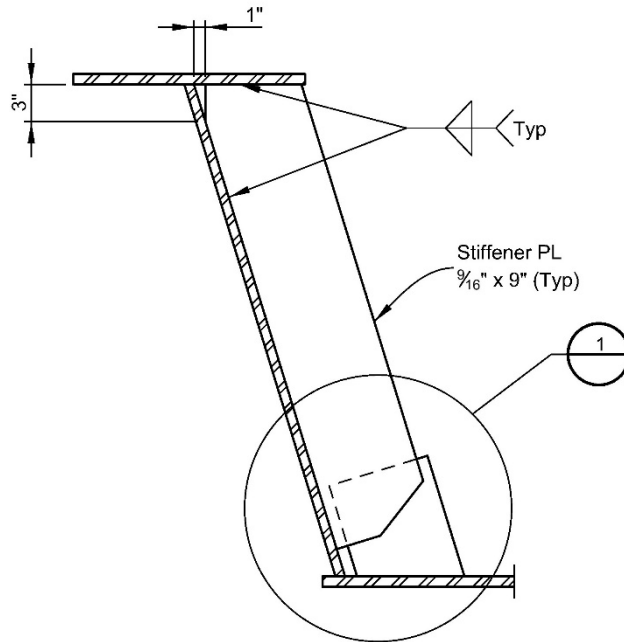
APPENDIX 6A - TYPICAL TUB/BOX GIRDER DETAILS



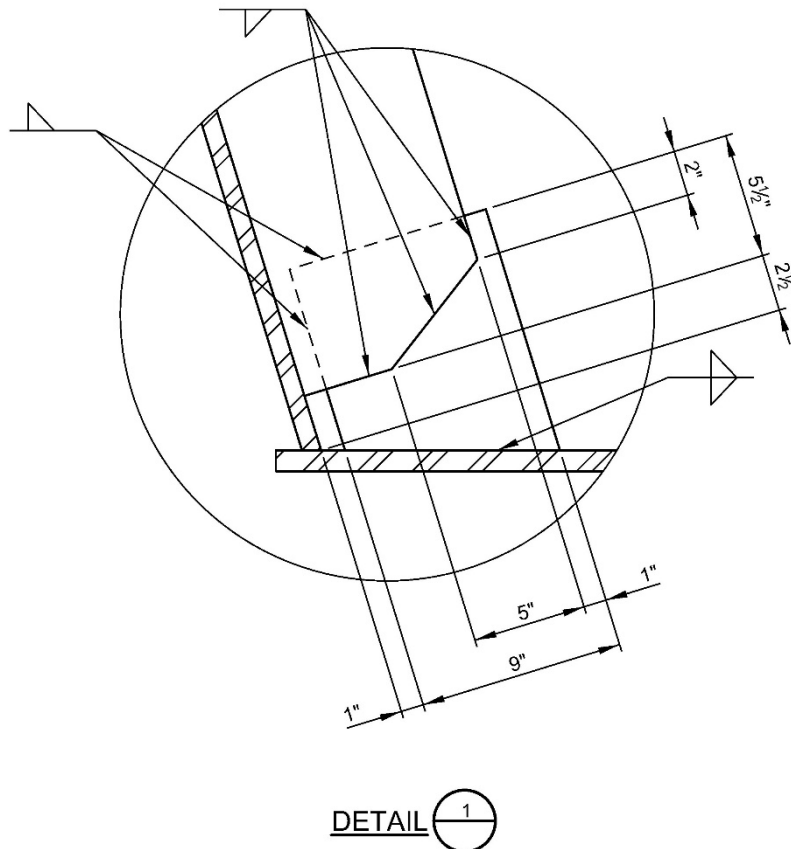
GIRDER DRAIN HOLE DETAILS

NOTES:

1. Clean steel before applying silicone sealer compound.
2. Compound is subject to approval of Engineer.
3. Silicone sealer will not be paid for separately, but included in the cost of the work.
4. In addition, provide drain holes at low points of box girder.
5. Galvanized metal screen shall be included in drain hole cost. Attach to top of bottom flange using Engineer approved adhesive.



INTERMEDIATE STIFFENER



DETAIL 1

APPENDIX 6B - GIRDER PRELIMINARY DESIGN AIDS

General

The following table and graphs are design aids to assist with the selection of girder types and spacing for preliminary design only.

Table 6B-1

A simple spreadsheet was developed by CSU in conjunction with a research project entitled “Development of Steel Design Details and Selection Criteria for Cost-Effective and Innovative Steel Bridges in Colorado”, Report No. CDOT-2008-12. The design tables below are a small sample of the design tables produced. The tables shown below are for a 44 foot width bridge and simple spans. Additional tables are available in the report and the spreadsheet is available on the CDOT website. These tables and the software is based on rolled beam shapes and simple for dead load and continuous for live load details. For continuous steel girders and longer span capabilities, additional design standards can be found at: <https://www.aisc.org/nsba/design-resources/continuous-span-standards/>

Some Design Assumptions:

- 8 - 9” slab depending on girder spacing 4.5 ksi concrete w/ 4” future wearing surface
- 2 – 2.5 ft Overhang
- C15 x 33.9 Diaphragms
- 18 ft interior and 12 ft exterior diaphragm spacing
- 3 rows of 5” x 7/8” Shear Studs spaced at 5.25” or 6” dia throughout length for conservative estimate
- 2 – 486 lbs/ft barriers with 1.5 ft width
- 2 design lanes when out to out width was 44 ft or less, 3 design lanes for widths greater than 44 ft
- Weight estimate per square foot includes: Lightest wide flange beam weight, shear studs, and diaphragm weight
- All design charts were designed using a HL-93 Design Truck

60 ft span

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	60	ft					
Full Width	w	44	ft	W40	X	167	\$17.17	19.05
Slab Thickness	Ts	9	in	W36	X	170	\$17.41	19.32
No. of girders	Nb	4		W36	X	182	\$18.36	20.42
Girder spacing	S	13	ft	W40	X	183	\$18.43	20.51
Overhang		2.5	ft	W30	X	191	\$19.06	21.23

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	60	ft					
Full Width	w	44	ft	W33	X	141	\$18.22	19.69
Slab Thickness	Ts	8.25	in	W40	X	149	\$19.04	20.60
No. of girders	Nb	5		W36	X	150	\$19.14	20.71
Girder spacing	S	9.75	ft	W33	X	152	\$19.34	20.94
Overhang		2.5	ft	W36	X	160	\$20.15	21.85

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	60	ft					
Full Width	w	44	ft	W33	X	130	\$20.09	21.33
Slab Thickness	Ts	8	in	W30	X	132	\$20.33	21.60
No. of girders	Nb	6		W36	X	135	\$20.71	22.01
Girder spacing	S	7.8	ft	W33	X	141	\$21.45	22.83
Overhang		2.5	ft	W27	X	146	\$22.06	23.51

70 ft span

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	70	ft					
Full Width	w	44	ft	W40	X	211	\$20.66	23.15
Slab Thickness	Ts	9	in	W40	X	215	\$20.97	23.51
No. of girders	Nb	4		W36	X	231	\$22.17	24.97
Girder spacing	S	13	ft	W36	X	232	\$22.25	25.06
Overhang		2.5	ft	W40	X	235	\$22.47	25.33

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	70	ft					
Full Width	w	44	ft	W40	X	167	\$20.92	22.75
Slab Thickness	Ts	8.25	in	W36	X	182	\$22.41	24.46
No. of girders	Nb	5		W40	X	183	\$22.51	24.57
Girder spacing	S	9.75	ft	W36	X	194	\$23.58	25.82
Overhang		2.5	ft	W40	X	199	\$24.07	26.39

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	70	ft					
Full Width	w	44	ft	W40	X	149	\$22.51	24.03
Slab Thickness	Ts	8	in	W36	X	150	\$22.63	24.17
No. of girders	Nb	6		W36	X	160	\$23.84	25.53
Girder spacing	S	7.8	ft	W40	X	167	\$24.68	26.49
Overhang		2.5	ft	W33	X	169	\$24.92	26.76

80 ft span

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	80	ft					
Full Width	w	44	ft	W40	X	277	\$25.40	28.97
Slab Thickness	Ts	9	in	W40	X	278	\$25.47	29.06
No. of girders	Nb	4		W36	X	282	\$25.75	29.42
Girder spacing	S	13	ft	W33	X	291	\$26.38	30.24
Overhang		2.5	ft	W40	X	294	\$26.59	30.51

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	80	ft					
Full Width	w	44	ft	W40	X	211	\$25.08	27.55
Slab Thickness	Ts	8.25	in	W40	X	215	\$25.46	28.01
No. of girders	Nb	5		W36	X	231	\$26.96	29.83
Girder spacing	S	9.75	ft	W36	X	232	\$27.05	29.94
Overhang		2.5	ft	W40	X	235	\$27.33	30.28

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	80	ft					
Full Width	w	44	ft	W40	X	183	\$26.43	28.46
Slab Thickness	Ts	8	in	W36	X	194	\$27.71	29.96
No. of girders	Nb	6		W40	X	199	\$28.29	30.64
Girder spacing	S	7.8	ft	W33	X	201	\$28.53	30.91
Overhang		2.5	ft	W36	X	210	\$29.56	32.14

90 ft span

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	90	ft					
Full Width	w	44	ft	W40	X	324	\$28.69	33.32
Slab Thickness	Ts	9	in	W36	X	361	\$31.12	36.69
No. of girders	Nb	4		W40	X	362	\$31.18	36.78
Girder spacing	S	13	ft	W40	X	372	\$31.81	37.69
Overhang		2.5	ft	W33	X	387	\$32.75	39.05

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	90	ft					
Full Width	w	44	ft	W40	X	249	\$28.69	31.96
Slab Thickness	Ts	8.25	in	W40	X	264	\$30.05	33.67
No. of girders	Nb	5		W40	X	277	\$31.21	35.15
Girder spacing	S	9.75	ft	W40	X	278	\$31.30	35.26
Overhang		2.5	ft	W36	X	282	\$31.65	35.71

				Nominal Depth		Weight per linear foot	Erected Cost per Square Foot (Steel)	Pounds per Square Foot (Steel)
Longest Span	L	90	ft					
Full Width	w	44	ft	W40	X	215	\$30.21	32.92
Slab Thickness	Ts	8	in	W36	X	231	\$32.01	35.10
No. of girders	Nb	6		W36	X	232	\$32.12	35.24
Girder spacing	S	7.8	ft	W40	X	235	\$32.46	35.65
Overhang		2.5	ft	W33	X	241	\$33.12	36.47

Figures 6B-2

This table shows the current span capabilities of the “Press Brake Tub Girders”. Based on a 2022 steel estimate of \$2.50/pound the per costs will vary from \$242 to \$277 per linear foot depending on girder type (U12 to U33). Actual cost estimates should reflect unit costs based on specific project constraints and current market conditions. In general, these tub girders are considered compact sections unless skew is involved. Additional design will be required.

U-BEAM™ Selection Chart Per AASHTO LRFD Requirements																
Spacing	Bridge Length (ft)															
	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
4' - 6"	U12	U12	U12	U12	U12	U18	U18	U18	U24	U24	U30	U30	U33	U33	S.D.	S.D.
5' - 0"	U12	U12	U12	U12	U12	U18	U18	U24	U24	U24	U30	U30	U33	S.D.	S.D.	S.D.
5' - 6"	U12	U12	U12	U12	U12	U18	U18	U24	U24	U30	U30	U33	S.D.	S.D.	S.D.	S.D.
6' - 0"	U12	U12	U12	U12	U18	U18	U18	U24	U24	U30	U30	U33	S.D.	S.D.	S.D.	S.D.
6' - 6"	U12	U12	U12	U12	U18	U18	U24	U24	U30	U30	U33	S.D.	S.D.	S.D.	S.D.	S.D.
7' - 0"	U12	U12	U12	U12	U18	U18	U24	U24	U30	U33	S.D.	S.D.	S.D.	S.D.	S.D.	S.D.
7' - 6"	U12	U12	U12	U12	U18	U24	U24	U30	U30	U33	S.D.	S.D.	S.D.	S.D.	S.D.	S.D.
8' - 0"	U12	U12	U12	U18	U18	U24	U24	U30	U33	S.D.	S.D.	S.D.	S.D.	S.D.	S.D.	S.D.

SECTION 7 ALUMINUM STRUCTURES

7.1 GENERAL REQUIREMENTS

This section will provide guidance to the design and construction requirements for aluminum structures. Unless specified in the latest edition of the CDOT Standard Specifications for Road and Bridge Construction, [M&S Standard Plans](#), Staff Bridge Worksheets, or this BDM, the use of aluminum as a bridge or structural component is not permitted.

Aluminum appurtenances to bridge structures may be used as shown in the [M&S Standard Plans](#) or with the approval of Unit Leader in coordination with the Concrete SMEs. Due to concerns with dissimilar metals in contact in a corrosive environment and the occurrence of accelerated galvanic corrosion, measures such as using plastic washers or bushings should be taken to separate dissimilar metals.

7.2 CODE REQUIREMENTS

The design of aluminum components shall be in accordance with AASHTO.

**AASHTO
Section 7**

SECTION 8

8.1 GENERAL REQUIREMENTS

This section will provide guidance to the design and construction requirements for wood structures. Unless specified in the latest edition of the [M&S Standard Plans](#), Staff Bridge Worksheets, or this BDM, the use of wood as a bridge, retaining wall, sound barrier, or structural component is not permitted for new on-system structures.

Wood appurtenances to bridge structures may be used as shown in the Staff Bridge Worksheets only with the approval of Unit Leader in coordination with the State Bridge Engineer. Review Section 508 of CDOT Standard Specifications for Road and Bridge Construction and AWPA standards for treated wood in bridge applications and coordinate with Unit Leader and State Bridge Engineer for best preservative treatment.

The use of timber piles for new construction is not allowed.

Temporary structures may be constructed of wood, with prior approval of Unit Leader in coordination with the State Bridge Engineer.

8.2 CODE REQUIREMENTS

The design of wood structures shall be in accordance with AASHTO and *USDA Forest Service Timber Bridges, Design, Construction, Inspection, and Maintenance*.

**AASHTO
Section 8**

SECTION 9 DECK AND DECK SYSTEMS

9.1 GENERAL REQUIREMENTS

The following section provides CDOT practice for bridge deck thickness, overhangs, transverse and longitudinal reinforcement, protection criteria, and supplemental deck components. The Designer shall coordinate with Staff Bridge regarding project-specific circumstances warranting deviations from standard practices referenced herein.

To improve service life, weather resistance, and ease of future maintenance procedures, all bridge decks shall be designed as continuous and without expansion devices when possible. Additionally, the Designer shall incorporate a deck protection strategy on all bridge decks in accordance with this BDM. When the top slabs of CBCs are intended to be used as a driving surface, they shall be protected with waterproofing membrane and asphalt or polymer concrete similar to bridge decks.

Use of alternative deck systems, including but not limited to, open, filled, and partially filled metal grid decks, orthotropic steel decks, aluminum decks, fiber reinforced polymer (FRP) decks, and sandwich deck panels, requires discussions with Staff Bridge during the preliminary design phase and approval by Unit Leader in coordination with the State Bridge Engineer and shall be documented in the Structure Selection Report.

Use of wood decks and bare concrete decks on new bridge construction is not permitted.

Bridges should be designed to allow future deck replacement. This is important for post-tensioned bridges for which detensioning may be required. See Section 5.8.6 of this BDM for design requirements.

9.2 CODE REQUIREMENTS

Unless otherwise modified by this section of the BDM, the minimum requirement for loading, limit states, design analysis, and detailing for bridge deck and deck systems shall be in accordance with Sections 3, 4, and 9 of the current AASHTO *LRFD Bridge Design Specifications* (AASHTO). This section is intended to supplement AASHTO code requirements. Any requests to vary from methodologies presented herein shall be discussed with Staff Bridge.

The Design Engineer is encouraged to review Design Example 6 – Deck Design located in Appendix A.

9.3 PERFORMANCE REQUIREMENTS

9.3.1 Service Life

To minimize corrosion and deterioration, newly constructed bridge decks shall implement practical designs, construction materials, and deck protection strategies as specified in this BDM for the purpose of achieving a minimum service life of 75 years.

A greater level of durability to attain a minimum service life of 100 years is required for qualified bridges funded through the Colorado Bridge Enterprise

(CBE) Program. Prior to final design, Staff Bridge will provide the Designer CBE's Strategies for Enhancing Bridge Service Life Memorandum for reference to approved deck protection methods of qualified bridges.

9.3.2 Maintenance Requirements

Bridge decks shall be designed and detailed to facilitate future maintenance and inspection. This includes the following:

- Providing continuous and joint free bridges, where feasible
- Minimizing construction joints when required
- Using corrosion resistant reinforcing with recommended clear cover
- Specifying deck protection
- Optimizing placement of bridge deck drains.

Additionally, the Designer will give consideration to future deck repairs and the inevitable replacement of bridge overlays during the initial design process.

Refer to BDM Section 5.4, Reinforced Concrete.

9.4 ANALYSIS METHOD

9.4.1 General

The approximate method of analysis specified in AASHTO shall be used for the design of concrete deck slabs that are within the limitations outlined for its use.

**AASHTO
4.6.2.1**

For atypical bridge decks not meeting the conditions explicit to the approximate method of analysis, refined methods of analysis, as identified in AASHTO, shall be used.

**AASHTO
4.6.3.2**

The Designer may propose the use of the AASHTO empirical design method for consideration by Staff Bridge during the preliminary design phase. Prior to CDOT consideration, the Designer will confirm that the design conditions satisfy those outlined in AASHTO. Upon approval by Unit Leader in coordination with the State Bridge Engineer, an explanation for the use of the empirical method will be documented in the Structure Selection Report.

AASHTO 9.7.2

Use of AASHTO exposure factor coefficient in deck design shall be as follows:

AASHTO 5.6.7

- Use Class 1 exposure factor when deck has a waterproofing membrane and overlay or polyester overlay installed.
- Use Class 2 in all other cases.

9.4.2 Deck Design Tables

To maintain consistency in detailing, this section provides deck design values, including recommended deck slab thicknesses, overhang widths, transverse and longitudinal reinforcing, for a variety of girder arrangements (see Tables 9-1 to 9-4 and Figure 9-1). These design tables are valid for old CDOT standard BT girders and rolled steel or steel plate girders with a 12 in. minimum top flange width. The designs may be conservative and can be used for preliminary estimates. The Designer is responsible for exercising design judgment when using these tables for final design, noting the following limitations in their development:

- LRFD approximate method using 32 kip axle AASHTO design truck with three or more girders.
- 3 in. wearing surface dead load = 36.67 psf.
- Deck skews less than 25°.
- Minimum concrete clear cover to bottom transverse reinforcement = 1 in.
- For economy, the maximum tension reinforcement ratio, ρ , is approximately half the balanced reinforcement ratio, ρ_{bal} . This assumes that controlling deck deflections is not critical to bridge performance.
- Top primary reinforcing extending into deck overhangs may not be adequate to resist rail impact loads and shall be designed accordingly for each project. Refer to BDM Section 9.7 for additional information.
- Use of precast deck panels is accommodated in the deck thicknesses listed; however, the Designer shall confirm that the deck thickness selected from the tables is adequate to accommodate both the deck panels, if used, and any necessary negative moment reinforcing while providing the minimum clearances. Refer to BDM Section 9.13.2 for additional information.
- Use exposure factor of 1.0, assuming a waterproofing membrane or polyester polymer overlay on the surface of the deck.
- Load modifier $\eta_i = 0.95$ is used in the design (LRFD 1.3.2)

Table 9-1: CDOT Standard CBT Girder Load and Resistance Factor Design ($f_y = 60$ ksi)

Girder spacing CL to CL (ft.)	Concrete deck thick. (w/o haunch) (in.)	* Maximum overhang (ft. - in.)	Transverse reinforcing		Longitudinal reinforcing	
			Top / bot. mat slab reinforcing.		** Top mat (#4 min.)	Bot. mat/ "D" bar (#5)
			Size	Max. Spacing (in.)	Max. Spacing (in.)	Max. Spacing (in.)
4.00	8.00	2' - 3.5"	#5	9	12	12
4.25	8.00	2' - 3.5"		9		12
4.50	8.00	2' - 3.5"		9		12
4.75	8.00	2' - 4.5"		9		12
5.00	8.00	2' - 6"		9		12
5.25	8.00	2' - 7.5"		9		12
5.50	8.00	2' - 9"		9		12
5.75	8.00	2' - 10.5"		9		12
6.00	8.00	3' - 0"		9		12
6.25	8.00	3' - 1.5"		9		12
6.50	8.00	3' - 3"		9		12
6.75	8.00	3' - 4.5"		9		12
7.00	8.00	3' - 6"		9		12
7.25	8.00	3' - 7.5"		9		12
7.50	8.00	3' - 9"		9		12
7.75	8.00	3' - 10.5"		9		12
8.00	8.00	4' - 0"		9		12
8.25	8.00	4' - 1.5"		8.5		12
8.50	8.00	4' - 3"		8.5		12
8.75	8.00	4' - 4.5"		8		11
9.00	8.00	4' - 6"		8		11
9.25	8.00	4' - 7.5"		8		11
9.50	8.00	4' - 9"		7.5		11
9.75	8.00	4' - 10.5"		7.5		11
10.00	8.00	5' - 0"		7.5		11
10.25	8.50	5' - 1.5"		7.5		11
10.50	8.50	5' - 3"		7.5		11
10.75	8.50	5' - 4.5"		7		10
11.00	8.50	5' - 6"		6.5		9
11.25	8.50	5' - 7.5"		6.5		9
11.50	9.00	5' - 9"		6.5		9
11.75	9.00	5' - 10.5"		6.5		9
12.00	9.00	6' - 0"		6		8

NOTES:

The design data does not apply to deck overhang that need to be designed according to AASHTO LRFD, Section A13.4.1.

* The deck overhang varies from 27.5" to 0.5 times the girder spacing that is measured from the center of the exterior girder.

** Negative moment reinforcing steel over the pier is not included.

Table 9-2: Rolled Steel Beams/Steel Plate Girders (12 in. [min.] wide top flange) Load and Resistance Factor Design ($f_y = 60$ ksi)

Girder spacing CL to CL (ft.)	Concrete deck thick. (w/o haunch) (in.)	* Maximum overhang (ft. - in.)	Transverse reinforcing		Longitudinal reinforcing	
			Top / bot. mat slab reinforcing.		** Top mat (#4 min.)	Bot. mat/ "D" bar (#5)
			Size	Max. Spacing (in.)	Max. Spacing (in.)	Max. Spacing (in.)
4.00	8.00	1' - 6"	#5	9	12	12
4.25	8.00	1' - 6"		9		12
4.50	8.00	1' - 6"		9		12
4.75	8.00	1' - 7"		9		12
5.00	8.00	1' - 8"		9		12
5.25	8.00	1' - 9"		9		12
5.50	8.00	1' - 10"		9		12
5.75	8.00	1' - 11"		9		12
6.00	8.00	2' - 0"		8.5		12
6.25	8.00	2' - 1"		8.5		12
6.50	8.00	2' - 2"		8		11
6.75	8.00	2' - 3"		8		11
7.00	8.00	2' - 4"		7.5		11
7.25	8.00	2' - 5"		7.5		11
7.50	8.00	2' - 6"		7.5		11
7.75	8.00	2' - 7"		7		10
8.00	8.00	2' - 8"		7		10
8.25	8.00	2' - 9"		7		10
8.50	8.00	2' - 10"		6.5		9
8.75	8.00	2' - 11"		6.5		9
9.00	8.00	3' - 0"		6.5		9
9.25	8.00	3' - 1"		6		8
9.50	8.00	3' - 2"		6		8
9.75	8.00	3' - 3"		5.5		8
10.00	8.00	3' - 4"		5.5		8
10.25	8.50	3' - 5"		5.5		8
10.50	8.50	3' - 6"		5.5		8
10.75	8.50	3' - 7"		5		7
11.00	8.50	3' - 8"		5		7
11.25	8.50	3' - 9"		5		7
11.50	9.00	3' - 10"		5		7
11.75	9.00	3' - 11"		4.5		6
12.00	9.00	4' - 0"		4.5		6

NOTES:

The design data does not apply to deck overhang that need to be designed according to AASHTO LRFD, Section A13.4.1.

* The deck overhang varies from 21" to 0.33 times the girder spacing that is measured from the center of the exterior girder.

** Negative moment reinforcing steel over the pier is not included.

Table 9-3: CDOT Standard CBT Girder Load and Resistance Factor Design ($f_y = 100$ ksi)

Girder spacing CL to CL (ft.)	Concrete deck thick. (w/o haunch) (in.)	* Maximum overhang (ft. - in.)	Transverse reinforcing		Longitudinal reinforcing	
			Top / bot. mat slab reinforcing.		** Top mat (#4 min.)	Bot. mat/ "D" bar (#4)
			Size	Max. Spacing (in.)	Max. Spacing (in.)	Max. Spacing (in.)
4.00	8.00	2' - 3.5"	#4	9	12	12
4.25	8.00	2' - 3.5"		9		12
4.50	8.00	2' - 3.5"		9		12
4.75	8.00	2' - 4.5"		9		12
5.00	8.00	2' - 6"		9		12
5.25	8.00	2' - 7.5"		9		12
5.50	8.00	2' - 9"		9		12
5.75	8.00	2' - 10.5"		9		12
6.00	8.00	3' - 0"		9		12
6.25	8.00	3' - 1.5"		9		12
6.50	8.00	3' - 3"		8.5		12
6.75	8.00	3' - 4.5"		8.5		12
7.00	8.00	3' - 6"		8		11
7.25	8.00	3' - 7.5"		8		11
7.50	8.00	3' - 9"		7.5		11
7.75	8.00	3' - 10.5"		7		10
8.00	8.00	4' - 0"		7		10
8.25	8.00	4' - 1.5"		7		10
8.50	8.00	4' - 3"		6.5		9
8.75	8.00	4' - 4.5"		6.5		9
9.00	8.00	4' - 6"		6.5		9
9.25	8.00	4' - 7.5"		6		8
9.50	8.00	4' - 9"		6		8
9.75	8.00	4' - 10.5"		5.5		8
10.00	8.00	5' - 0"		5.5		8
10.25	8.50	5' - 1.5"		5.5		8
10.50	8.50	5' - 3"		5.5		8
10.75	8.50	5' - 4.5"		5.5		8
11.00	8.50	5' - 6"		5.5		8
11.25	8.50	5' - 7.5"		5		7
11.50	9.00	5' - 9"		5		7
11.75	9.00	5' - 10.5"		5		7
12.00	9.00	6' - 0"	↓	5	↓	7

NOTES:

The design data does not apply to deck overhang that need to be designed according to AASHTO LRFD, Section A13.4.1.

* The deck overhang varies from 27.5" to 0.5 times the girder spacing that is measured from the center of the exterior girder.

** Negative moment reinforcing steel over the pier is not included.

Table 9-4: Rolled Steel Beams/Steel Plate Girders (12 in. [min.] wide top flange) Load and Resistance Factor Design ($f_y = 100$ ksi)

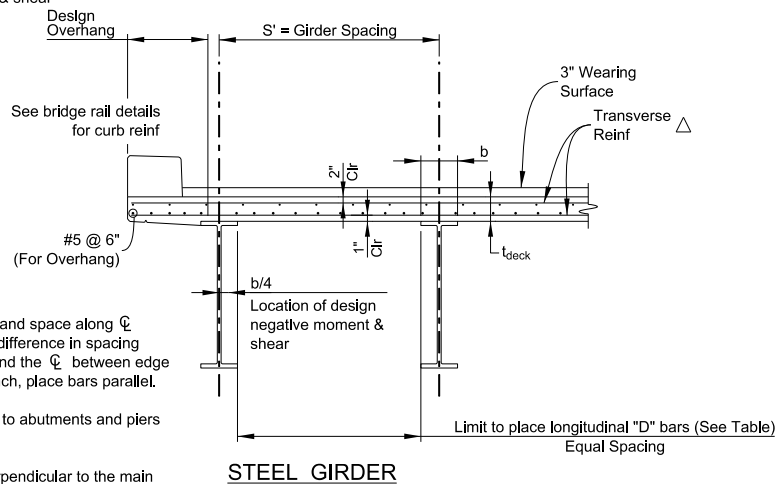
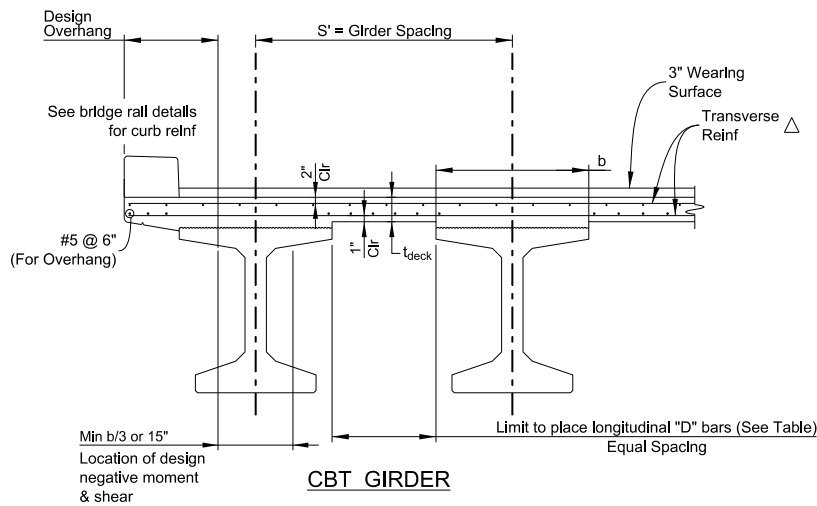
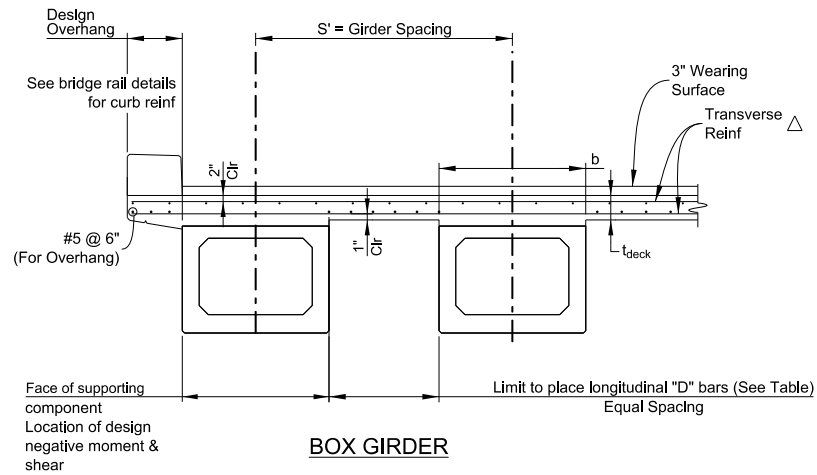
Girder spacing CL to CL (ft.)	Concrete deck thick. (w/o haunch) (in.)	* Maximum overhang (ft. - in.)	Transverse reinforcing		Longitudinal reinforcing	
			Top / bot. mat slab reinforcing.		** Top mat (#4 min.)	Bot. mat/ "D" bar (#4)
			Size	Max. Spacing (in.)	Max. Spacing (in.)	Max. Spacing (in.)
4.00	8.00	1' - 6"	#4	9	12	12
4.25	8.00	1' - 6"		9		12
4.50	8.00	1' - 6"		9		12
4.75	8.00	1' - 7"		8.5		12
5.00	8.00	1' - 8"		8		11
5.25	8.00	1' - 9"		7.5		11
5.50	8.00	1' - 10"		7.5		11
5.75	8.00	1' - 11"		7		10
6.00	8.00	2' - 0"		7		10
6.25	8.00	2' - 1"		6.5		9
6.50	8.00	2' - 2"		6.5		9
6.75	8.00	2' - 3"		6.5		9
7.00	8.00	2' - 4"		6		8
7.25	8.00	2' - 5"		6		8
7.50	8.00	2' - 6"		5.5		8
7.75	8.00	2' - 7"		5.5		8
8.00	8.00	2' - 8"		5.5		8
8.25	8.00	2' - 9"		5.5		8
8.50	8.00	2' - 10"		5.5		8
8.75	8.00	2' - 11"		5		7
9.00	8.00	3' - 0"		5		7
9.25	8.00	3' - 1"		5		7
9.50	8.00	3' - 2"		4.5		6
9.75	8.00	3' - 3"		4.5		6
10.00	8.00	3' - 4"		4.5		6
10.25	8.50	3' - 5"		4.5		6
10.50	8.50	3' - 6"		4.5		6
10.75	8.50	3' - 7"		4.5		6
11.00	8.50	3' - 8"		4.5		6
11.25	8.50	3' - 9"		4.5		6
11.50	9.00	3' - 10"		4.5		6
11.75	9.00	3' - 11"		4.5		6
12.00	9.00	4' - 0"	↓	4.5	↓	6

NOTES:

The design data does not apply to deck overhang that need to be designed according to AASHTO LRFD, Section A13.4.1.

* The deck overhang varies from 18" to 0.33 times the girder spacing that is measured from the center of the exterior girder.

** Negative moment reinforcing steel over the pier is not included.



Δ For curved structures place radially and space along ϕ between edges of deck. When the difference in spacing between the outside edge of deck and the ϕ between edge of deck becomes greater than 1/4 inch, place bars parallel.

For skews 25° or less place parallel to abutments and piers and space along ϕ of structure.

For skew greater than 25° place perpendicular to the main supporting members

Figure 9-1: Deck Design Table Detail

9.5 DECK THICKNESS

The minimum deck thickness, not including allowances for haunch depth or the wearing surface thickness (asphalt or PPC overlay), shall be as specified:

- Decks with overlays: 8 in.
- Adjacent box girders/T-beams/CBT girders: 5 in.

The flange thickness of precast box girders and T-beams shall be as determined by design per AASHTO, but the combined composite thickness of the cast-in-place deck slab and top flange shall not be less than 8 in.

**AASHTO
9.7.1.1**

9.6 LONGITUDINAL REINFORCEMENT

9.6.1 Minimum Required Reinforcing

The minimum longitudinal reinforcing steel in the top of concrete bridge decks shall be #4 at 12 in. spacing. This new spacing is assumed adequate for crack control due to the use of Class DF with macrofibers as well as the required use of deck protection such as waterproofing membrane and asphalt or Polyester Polymer concrete. The designer will need to verify that all AASHTO requirements are still met including distribution steel per 9.7.3.2 of negative moment over girders.

The spacing of #4 at 6 in. was chosen in the early 1990s from an original #5 at 18" to improve crack control when bare deck bridges were still allowed. Cracking was seen in bridge rail with a spacing of #4 at 18" and the cracking was eliminated when changing to #6 at 18" (.41% of the section).

Longitudinal reinforcement in the bottom of the deck slab (D bars) shall be as indicated in Tables 9-1 to 9-4 in Section 9.4.2. For girder arrangements or specific circumstances not meeting the design table requirements, the longitudinal reinforcement shall be distributed as a percentage of the primary reinforcement in accordance with AASHTO.

**AASHTO
9.7.3.2**

To control transverse cracking at the bottom of deck overhangs, D bars shown in Tables 9-1 to 9-4 in Section 9.4.2 for various overhang widths is adequate reinforcing. When the project requires a larger overhang, the Designer shall design the longitudinal reinforcing steel in accordance with AASHTO.

**AASHTO
9.7.3.2**

9.6.2 Negative Moment Reinforcing

For simple span bridges made continuous, the negative moment at the pier may be taken at the face of the concrete diaphragm. Negative moment reinforcing shall be designed for composite load moments at the strength limit state. Negative moment reinforcing shall terminate beyond the inflection point per AASHTO.

**AASHTO
5.10.8.1.2c**

To accommodate the longitudinal reinforcement required for negative moment regions, small size bars bundled together or bars placed in two layers is permitted. Use the smallest bar size allowed by design to meet clearance requirements and avoid overcrowding bars when precast deck panels are permitted.

Unless stay-in-place deck forms are prohibited by the contract documents, bridge deck designs shall consider only the top longitudinal deck reinforcing when determining the continuity reinforcing capacity.

9.7 DECK OVERHANG DESIGN

9.7.1 Overhang Requirements

Deck overhang shoring subject to screed rail loads and construction loads has resulted in excessive deflections and torsional rotation of the exterior girders. To eliminate potential construction problems from deflections and rotation, the limits for deck overhangs shall be as described herein. The maximum deck overhang for various beam types, measured from the centerline of girder web to edge of deck, is presented as follows, where S' (ft.) is the center to center spacing of the webs for I girders or web of adjacent boxes or U girders, and b (in.) is the top flange width:

- CBT girders, steel box, and concrete box girders: $(S/2) \leq 6$ ft
- Steel I girders: Maximum overhangs shall not exceed the larger value:
 - $(S/3) \leq 6$ ft
 - $(b/2) + 12$ in. ≤ 6 ft

Tables 9-1 and 9-2 in Section 9.4.2 calculate and show the maximum overhang widths for both CBT and steel I girders.

On curved decks where the overhang varies along the bridge length, the maximum overhang width should not exceed the average overhang width by more than 1 ft.

A $\frac{3}{4}$ in. V-drip groove shall be located 6.00 in. from back face of barrier on the underside of the deck overhang for all bridges. Where 6 in. cannot be provided due to a minimal overhang width, 3 in. shall be used.

Deck overhangs shall extend beyond the edge of the top flange or box girder web a minimum of 6.00 in. to prevent water from dripping onto the girder. Additionally, cantilever shall extend 1 in. below the top of girder flange, and the bottom flange or web shall not extend beyond the formed drip groove of the deck.

Any exceptions to the above criteria shall be addressed on a project-specific basis and must be approved by Unit Leader.

9.7.2 Deck Overhang Loading and Design

To balance exterior girder designs with interior girder designs, overhangs should generally be limited to less than half the interior girder spacing or less as described in Section 9.7.1 of this BDM. Deck overhangs shall be designed for bridge rail and self-weight dead loads, HL-93 live loads, and barrier impact loads in accordance with AASHTO. The area of top transverse reinforcing provided in Tables 9-1 and 9-2 in Section 9.4.2 may be counted toward the area of steel required to resist moments caused by all overhang loads (see Deck Design Example 6). Deck reinforcing required to resist overhang loads shall be developed per AASHTO 5.11; larger reinforcing bars may require hooks at the edge of deck to meet development length requirements. As mentioned in Section 3.8 of this BDM, overhangs shall be evaluated for

**AASHTO
3.6.1.3.4**

construction loads as well. Some of these loads are discussed in Section 6.6.4.2 of this BDM and are generally applicable to all girder types.

Refer to BDM Section 9.6 for recommended longitudinal reinforcing in the bridge deck overhang.

9.8 SKEWED DECK LIMITS

The Designer is encouraged to reduce the skew of new bridges during the preliminary bridge layout process. Highly skewed bridges are associated with unwanted shear stresses at the deck corners and promote maintenance concerns for expansion joints and bearings. Refer to BDM Section 4.6, Skew Effects on Bridges, for additional information.

9.8.1 Transverse Reinforcement

When the skew angle of the deck does not exceed 25° , the primary reinforcement may be placed in the direction of the skew. As an alternative, the primary reinforcing may be placed in a splayed arrangement as shown in Figure 9-2, with reinforcing gradually adjusting along the bridge length from its placement parallel to the skew near the bearings to perpendicular to the main supporting members.

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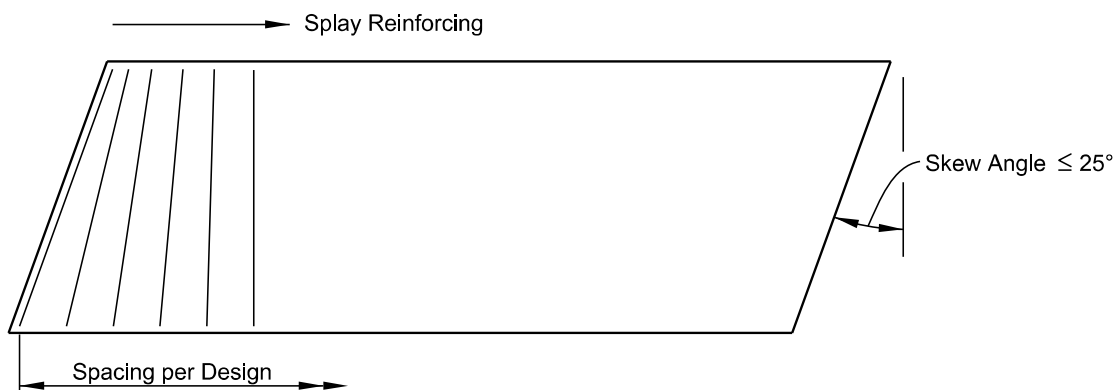


Figure 9-2: Splayed Deck Reinforcing

For skew angles exceeding 25° , the primary reinforcement shall be placed perpendicular to the main supporting members. The Designer shall consider performing a refined method of analysis as referenced in BDM Section 9.4.1 for the design of decks with extreme skews to limit cracking in the acute corners. The design span length is taken parallel to the primary reinforcement, as shown in Figure 9-3.

9.8.2 Reinforced End Zones

If the bridge skew exceeds 25° , additional reinforcement should be evaluated to be placed below the main top mat (transverse and longitudinal) reinforcing steel in the acute corners of the deck slab to control cracking and spalling of the concrete. A finite element model may be helpful in determining additional reinforcement requirements.

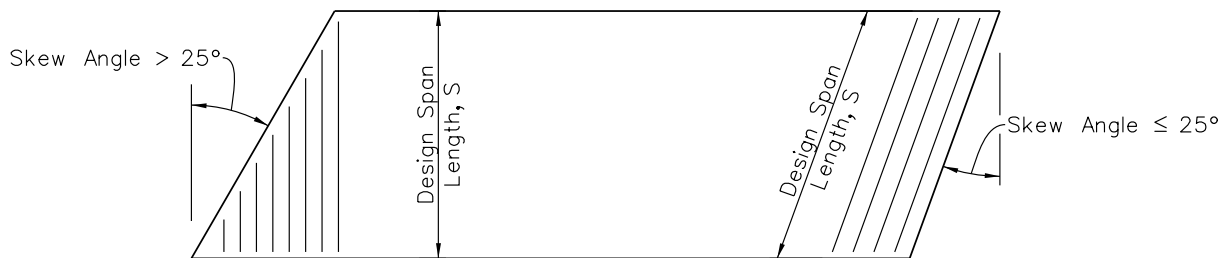


Figure 9-3: Skewed Deck Reinforcing Placement

The reinforcing shall be extended one development length past the centerline of the exterior girder in accordance with AASHTO (see Figure 9-4).

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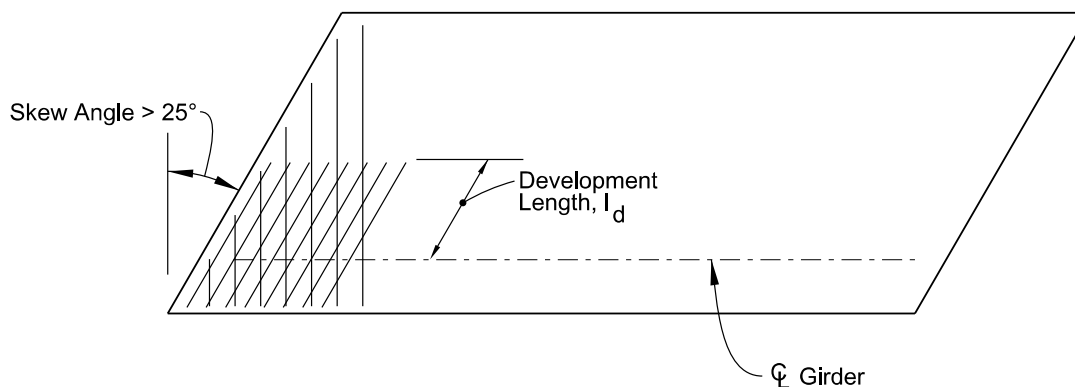


Figure 9-4: Acute Corner Reinforcing

9.9 OVERLAYS

New bridge construction shall use one of the following deck protection strategies:

1. 3 in. asphalt wearing surface over a waterproofing membrane applied over the concrete bridge deck and approach slabs to allow future mill and fill without damaging the waterproofing membrane. Asphalt overlays may not be desirable where concrete roadway is adjacent to the bridge and will be used as warranted by roadway requirements.
2. Polyester Polymer Concrete (PPC) overlay applied over the concrete bridge deck and approach slabs. PPC overlays shall have a minimum thickness of $\frac{3}{4}$ in. Requests to revise the thickness shall be at the approval of Unit Leader in coordination with the Overlay SMEs. The layer of PPC shall be omitted from the deck section properties.

New concrete deck slabs shall be designed to include 3 in. of asphalt overlay of 36.67 psf applied as a superimposed dead load over the bridge deck area.

Concrete decks with a PPC overlay shall consider the asphalt overlay load as a future load applied without the PPC in place. Construction notes shall include a note stating that the PPC must be removed before placing an asphalt wearing surface.

The Designer may discuss the use of alternative bridge deck overlays (e.g., Silica Fume modified concrete and Epoxy-polymer concrete) with Staff Bridge during the preliminary design phase. Discussions shall be documented in the Structure Selection Report.

9.10 WATERPROOFING

9.10.1 Membranes

All bridge decks using asphalt pavement as a deck and approach slab protection measure shall require a waterproofing membrane between the concrete deck and the asphalt overlay to serve as a deck surface sealant.

9.10.2 Sealer

Due to their low tolerance to abrasion and minimal service life, application of concrete sealers on bridge decks is not permitted.

Sidewalks placed on bridges do not require a protective concrete sealer.

9.11 DECK POURING SEQUENCE

9.11.1 Rate of Pour and Direction

The rate of placing concrete shall equal or exceed half the span length per hour but need not exceed 100 cy/hour for bridges designed as continuous. Concrete pumps can reasonably be expected to provide 100 cy/hour without significant malfunctions.

In general, the deck pour should progress uninterrupted from one end of the bridge to the other, in the direction of increasing grade along the longitudinal length of the bridge. If the bridge deck cannot be completed as a single pour, the Designer shall follow the direction presented in Section 9.11.2 and Section 9.12.

9.11.2 Deck Pour Sequence Details

All bridges with decks containing more than 300 cy of concrete shall have the pouring sequence shown on the plans, including sections to be placed first and last, pouring direction, and locations of transverse construction joints as specified in Section 9.12. If the Designer elects not to detail on the plans, the Designer shall add a note stating that the Contractor will submit a pouring sequence for approval by the Engineer of Record in coordination with the Fabrication/Construction Unit and Unit Leader.

As an alternative to starting at the ends of longer bridges, the deck pour sequence may begin at any location along the bridge, completing positive moment regions first and ending with negative moment regions over the piers.

Uplift at supports, girder deflections and stresses, in span hinges, and cut-off points for continuity reinforcing shall be considered when designing and detailing the deck pour sequence.

9.11.3 Diaphragms

For bridge abutment diaphragms and pier diaphragms designed integral with the deck slab, the deck pour shall include the diaphragm and deck as one continuous pour, with optional construction joint locations specified in Section 9.12.

9.12 DECK JOINTS

9.12.1 Transverse Joints

Optional transverse construction joints are permitted on continuous concrete deck structures to limit the concrete volume in a single pour. If used, locate transverse construction joints near the $\frac{3}{4}$ point of the span being poured in the direction of pour to minimize cracking in the negative moment region.

The General Notes drawing in the project plans shall include a note stating that the Contractor shall notify the Engineer of Record for approval of emergency construction joints.

For skewed bridges, transverse construction joints shall be parallel to the transverse reinforcement.

9.12.2 Longitudinal Joints

Longitudinal joints are generally discouraged, even for wide bridges since CDOT has not seen any durability issues without them. Longitudinal construction joints may be necessary due to phasing and finishing machine limitations for wide bridges. Construction joints are generally located near girder edges to allow for deck forming but should be located under barriers for additional protection when possible.

Use of closure pours shall be project specific and based on considerations such as:

- Excessive dead load deflection that may prevent transverse reinforcing bars from lining up properly prior to the closure pour.
- Excessive live load deflection during construction that may cause poor concrete bond to the reinforcing. Lane closures adjacent to closure pours should be used where possible.
- Construction phasing.
- Maintenance of traffic impacts.

Reinforcement through the construction joint shall be designed to limit deflections and shall be detailed in the project plans. Refer to BDM Section 5, Concrete Structures, for detailed reinforcing splice lengths.

9.13 STAY-IN-PLACE FORMS

9.13.1 General

The use of metal stay-in-place (SIP) deck forms is optional unless requested by the Region or Staff Bridge. A note stating whether metal SIP deck forms are required, prohibited, or optional shall be included on the General Notes

drawing in the final bridge plans. Metal SIP deck forms are encouraged for the following conditions:

- Structures crossing over heavy traffic, interstate highways, or railroads
- Where form removal is difficult or hazardous
- As requested by the Region or Staff Bridge

Transparent or Precast concrete panel deck forms are preferred to metal SIP deck forms.

Stay-in-place deck forms shall not be permitted for cantilevered portions of decks or where architectural constraints prohibit their use.

Refer to BDM Section 5 for special requirements concerning SIP forms for the regions of deck over U girders.

9.13.2 Concrete Stay-in-Place Forms

When partial depth precast concrete deck panels are used, one layer each of both transverse and longitudinal reinforcing is required over the panels with a minimum 3/8 in. clear distance between the top of deck panel and bottom of longitudinal reinforcing.

Placing deck reinforcement with no clearance to the top of precast concrete deck panels is not permitted. The Designer shall confirm that the deck thickness selected from the deck design tables in Section 9.4.2 is adequate to provide the required clearance when detailing longitudinal reinforcing in the negative moment regions over piers.

Refer to CDOT Bridge Structural Drawings for additional information.

9.13.3 Metal Stay-in-Place Forms

All form flutes shall be kept free of concrete either by filling them with polystyrene or by topping them with sheet metal covers.

The ability to perform comprehensive deck inspections and future deck maintenance is restricted when using metal deck forms. Consideration for their use should be acknowledged on a project-specific basis. The Contractor can remove regions of metal deck forms to provide discrete location for inspecting the deck subject to Unit Leader in coordination with the Fabrication/Construction Unit approval. When not permitted, the final project plans shall include a note disallowing their use.

9.13.4 Transparent Stay-in-Place Forms

For full depth CIP bridge decks, transparent SIP forms may be used. Transparent forms allow for inspection during and after construction. Routine inspection and maintenance can be performed with over 70% visibility to the underside of the deck.

9.14 FULL DEPTH PRECAST CONCRETE DECK PANELS

Full depth precast concrete deck panels are an acceptable design solution for bridges qualified under accelerated bridge construction techniques. Use of full depth panels shall be discussed with Staff Bridge during the preliminary design process, with discussions and approval by Unit Leader documented in the Structure Selection Report.

9.15 DECK DRAINS

The Designer shall follow the deck drain procedures and details outlined in AASHTO. Additionally, the Hydraulic Engineer shall use FHWA publications Design of Bridge Deck Drainage, Hydraulic Engineering Circular 21 and 22 (HEC-21 and HEC-22) to determine the type and size of bridge deck drains appropriate for the bridge geometry and design storm.

AASHTO 9.4.2

Due to the high maintenance requirements associated with deck drainage structures, it is preferred that the Designer minimize the number of bridge deck drains by carrying the water to approach drainage grates off the bridge. Deck drains shall be placed as necessary to intercept water away from expansion joints and bearing devices and shall discharge water away from all girders, pier and abutment caps, roadways, railroad properties, and pedestrian trails. Openings in deck slabs due to drainage components shall include additional reinforcing to account for changes in structural capacity.

Refer to BDM Section 2.11.3, Deck Drainage Requirements, and the CDOT Drainage Design Manual for additional deck drain requirements.

9.16 LIGHTS AND SIGNS ON DECK

Bridge mounted lighting and signs should be avoided when possible to avoid additional load and to avoid vibrations that may increase maintenance. Where project circumstances require that a light or sign be located on the bridge, it shall be located directly over the pier. The structure, including the anchor bolt connection to the deck, shall be designed in accordance with the current *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*.

9.17 CONDUIT IN DECK

Conduit used by CDOT for bridge deck lighting, traffic signals, or anti-icing systems may be embedded in the concrete deck as an alternative to embedding in the bridge rail if approved by Unit Leader. The conduit shall be rigid and placed between the top and bottom reinforcing mats with consideration for providing adequate concrete cover and reinforcement spacing.

Conduit pipes for private utilities are not permitted in concrete decks and must otherwise be attached externally to the structure in accordance with agreements between CDOT and the private utility company. For aesthetic and safety reasons, conduits are not permitted under deck overhangs or on bridge railings.

9.18 ANTI-ICING SYSTEMS

Anti-icing systems involve treating the bridge deck before inclement weather to prevent snow and ice accumulation, thus reducing traffic accidents and snow removal efforts. Use of Fixed Automated Spray Technology (FAST) is a recent development and is best suited for bridges with a higher level of service due to the cost, attention, and commitment necessary for installation and future maintenance. Installation of automatic anti-icing systems in new bridge decks shall be discussed with Staff Bridge on a project-specific basis and shall be approved by CDOT Maintenance in coordination with the Unit Leader and Anti-icing SMEs, for qualified bridges. When implemented, the manufacturer shall provide the locations of anti-icing nozzles in the bridge deck.

Anti-icing systems are often necessary when the super-elevation reverses on a bridge due to the zero cross-slope and ponding issues. Efforts should be made to eliminate or shift super-elevation reversals on a bridge.

Refer to CDOT Bridge Structural Drawings B-614-1 through B-614-4 and Anti-Icing Project Special Provisions for additional guidance.

SECTION 10 FOUNDATIONS

10.1 GENERAL SCOPE

Design of structure foundations shall be in accordance with AASHTO, project contract documents, and CDOT Geotechnical Design Manual, unless otherwise specified in this Section of the BDM.

10.2 GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations shall be conducted in accordance with AASHTO and the guidance provided in the CDOT Geotechnical Design Manual.

10.2.1 Ring-Lined Split Barrel Sampler

The 2.5-in. outside diameter, ring-lined split barrel sampler, often referred to as the Modified California (MC) sampler, is routinely used in Colorado to obtain disturbed samples of cohesive soil/rock for swell testing.

If penetration resistance values (blow counts) obtained using an MC sampler are used in conjunction with correlations based on standard penetration test (SPT) resistance values (N-values), the penetration resistance values should be corrected to account for the size of the MC sampler (see Fang, 1991), as appropriate based on the judgment of the Geotechnical Engineer.

In general, it is preferable to use SPT resistance values in SPT-based correlations rather than to correct MC penetration resistance values.

10.2.2 Energy Measurements for Sampling Hammers

The energy delivered to drill rods when conducting SPT and MC sampling can vary significantly depending on factors, including the type of sampling hammer, the general condition of the hammer, and the operator. Therefore, CDOT requires the use of sampling hammers that have been tested to determine the actual energy transfer to the drill rods.

All sampling hammers used to complete field explorations for CDOT projects shall be tested to determine the energy transfer ratio (the measured energy transferred to the drill rods divided by the theoretical potential energy of the sampling hammer) in accordance with ASTM D4633. The testing shall be completed no more than two years before the date of sampling.

The project geotechnical report or the individual boring logs shall indicate the energy transfer ratio. The energy transfer ratio shall also be reported on the geology sheet. In addition, the geology sheet shall indicate whether the reported penetration resistance values are raw values or values that have been corrected for hammer efficiency.

As appropriate for use in geotechnical evaluations, the Geotechnical Engineer should correct penetration resistance values to an equivalent hammer efficiency of 60 percent (N_{60} values).

10.3 LIMIT STATES AND RESISTANCE FACTORS

10.3.1 Service Limit State

Foundation design at the service limit state shall be in accordance with AASHTO.

10.3.2 Strength Limit State

Resistance factors at the strength limit state for foundation design shall be in accordance with AASHTO, unless otherwise indicated in this Section of the BDM.

10.3.3 Extreme Event Limit State

As specified by AASHTO, resistance factors for the extreme event limit state, including earthquake, ice, vehicle, or vessel impact loads, shall be taken as 1.00. For uplift resistance of piles and shafts at the extreme event limit state, the resistance factor shall be taken as 0.80 or less.

**AASHTO
10.5.5.3.3**

10.4 SPREAD FOOTINGS

10.4.1 General

The Designer shall evaluate the suitability and applicability of spread footing foundations on a case-by-case basis.

10.4.2 Footing Embedment

The base of spread footings on soil shall be embedded below the local or regional frost depth, with a minimum embedment of 3 ft. The minimum embedment of spread footings on bedrock may be reduced to less than 3 ft. based on the recommendation of the Geotechnical Engineer.

**AASHTO
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For establishing spread footing embedment into stream banks based on scour considerations, see Section 2.11.2 of this BDM.

The requirements of this section do not apply to MSE wall footers. Refer to current Staff Bridge Worksheets_for MSE Walls for MSE wall requirements.

10.4.3 Tolerable Movements

Tolerable foundation movements shall be in accordance with AASHTO. As noted by AASHTO, angular distortions between adjacent foundations should not exceed 0.008 radians in simple spans and 0.004 radians in continuous spans.

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Consistent with AASHTO, transient loads may be omitted from time-dependent settlement analyses at the Service I Load Combination.

10.5 DRIVEN PILES

10.5.1 General

10.5.1.1 Pile Types

Driven H-piles are frequently used to support structures in Colorado. In most applications, H-piles are driven to practical refusal on bedrock. H-pile sections are supplied standard as Grade 50 steel ($f_y = 50$ ksi).

For bridges, the most readily available H-pile sections include:

- HP 14x89
- HP 12x74
- HP 12x53

Other H-pile sizes may be used when availability is verified with local suppliers and when any delays due to custom pile orders do not negatively affect the project schedule.

Although less frequently used in Colorado, other pile types may be feasible and preferable to H-piles depending on project requirements. For instance, closed-end pipe piles may be advantageous at sites with relatively deep bedrock, where a closed-end pipe pile may develop greater axial resistance at shallower depths compared to a comparable H-pile section. Sheet piles may be used for foundation support, especially for projects where such use may benefit the construction schedule or cost.

When using a less common pile type, the Designer shall confirm that the selected pile section is available from local suppliers.

10.5.1.2 Battered Piles

Battered piles may be used to increase lateral resistance of driven piles. The Designer should consider that battered piles will provide a stiffer lateral response than that of vertical piles.

Where used, the preferred pile batter is 1 horizontal to 6 vertical (1H:6V). The maximum batter of driven piles shall not exceed 1H:4V due to constructability considerations.

Piles less than 15 ft. in length and driven to refusal on bedrock shall not be battered.

10.5.1.3 Embedment

The Designer should consider the potential for piles to encounter refusal on bedrock or obstructions, such as boulders, before reaching the depth required for stability under axial and lateral loading. The Designer may specify a minimum tip elevation on the plans to address this issue. Pre-boring may be used in cases where refusal is anticipated to occur above the required minimum tip elevation, although the Designer should consider using other foundation types that may be preferable in terms of design or constructability.

AASHTO
10.7.1.4

10.5.1.4 Corrosion of Piles in Soil/Rock

In aggressive soil/rock, the Designer shall incorporate appropriate corrosion mitigation measures. Acceptable corrosion mitigation measures for driven piles include the use of sacrificial steel, concrete encasement, and factory-applied coatings in combination with a reduced thickness of sacrificial steel. Field-applied coatings shall not be used, except as repairs to factory-applied coatings. Weathering steel is not considered a mitigation measure for corrosion.

**AASHTO
10.7.5**

In general, corrosion of steel piles is greatest in soils that have been disturbed, that is, where earthwork activities have occurred. Compared to undisturbed soils, disturbed soils have increased oxygen content, which supports corrosion. In undisturbed soils, corrosion may occur in the zone of unsaturated soil above the groundwater table. Corrosion may be exacerbated in the zone of fluctuation of the groundwater table. Significant corrosion does not generally occur in undisturbed soil/rock below the groundwater table.

In soil/rock above the groundwater table, the Geotechnical Engineer shall conduct corrosion testing of representative soil/rock samples. If any of the following conditions exist, the soil/rock shall be classified as aggressive:

- Resistivity is less than 2,000 ohm-cm.
- pH is less than 5.5.
- pH is between 5.5 and 8.5 in soils with high organic content.
- Sulfate concentration is greater than 1,000 parts per million (ppm).
- Chloride concentration is greater than 500 ppm.

Where corrosion testing indicates aggressive soil/rock, the Geotechnical Engineer shall indicate the elevation range(s) where the aggressive soil/rock is anticipated based on test results.

Where aggressive soil/rock is present, the thickness of sacrificial steel shall be calculated based on a minimum corrosion rate of 0.001 in. per year. Published corrosion rates vary widely. The specified minimum corrosion rate is based on criteria established by the California Department of Transportation (2013), the US Army Corps of Engineers (2012), and the Florida Department of Transportation (2016).

The Designer shall assume that corrosion occurs over all steel surfaces in contact with the aggressive soil/rock. Corrosion rates greater than the minimum value specified herein may be appropriate, particularly where piles are installed in landfill materials, cinder fills, organic soils, or mine waste/drainage. Corrosion mitigation is not required in soil/rock below the groundwater table.

If factory-applied coal-tar epoxy coating is used for corrosion mitigation, the coating shall be assumed to be effective for 30 years. In calculating the sacrificial steel thickness, the Designer shall assume corrosion begins after the first 30 years and continues through the remaining design life, as appropriate. If protective coatings are used, the Geotechnical Engineer shall provide

appropriate axial design parameters accounting for a potential reduction in side resistance.

Sacrificial steel is not necessary where concrete encasement is used for corrosion mitigation. Piles protected by concrete encasement should be coated with a dielectric coating near the base of the concrete jacket.

**AASHTO
C10.7.5**

10.5.1.5 Corrosion of Piles Exposed to Atmospheric Conditions

The following provisions apply only to situations where piles are extended above the ground, such as sheet pile abutments or H-pile/pipe pile piers.

For non-weathering steel piles, aggressive conditions shall be assumed for the first 5 ft. of pile below grade and for the entire portion of the pile exposed to atmospheric conditions.

Corrosion mitigation is not required for weathering steel piles exposed to atmospheric conditions and not located within the splash zone or underneath a bridge expansion joint.

Corrosion mitigation for the remaining portion of piles embedded in soil/rock shall be as required in Section 10.5.1.4, for both non-weathering and weathering steel piles.

10.5.1.6 Pile Cap Embedment

For establishing pile cap footing embedment into stream banks based on scour considerations, see Section 2.11.2 of this BDM.

10.5.2 Geotechnical Design and Analysis

**AASHTO
2.6.4.4.2**

10.5.2.1 Point Bearing Piles on Rock

Piles that will penetrate the bedrock 3 ft. or more shall be designed in accordance with the requirements specified by AASHTO for “Piles Driven to Soft Rock.” Piles that will penetrate the bedrock less than 3 ft. shall be designed in accordance with the requirements specified by AASHTO for “Piles Driven to Hard Rock.”

**AASHTO
10.7.3.2.2 and
10.7.3.2.3**

In general, it is anticipated that piles driven into the relatively weak sedimentary bedrock typically encountered along the Front Range would classify as “Soft Rock,” while piles driven to higher strength bedrock where significant bedrock penetration is not typically achieved would classify as “Hard Rock.”

Pile protection (tips, points, or shoes) shall be included for all piles driven to bedrock.

10.5.2.2 Small Groups of Piles

At the strength limit state, the resistance factor for geotechnical axial resistance shall be reduced by 20 percent for groups of piles containing three or fewer piles, unless otherwise approved by Unit Leader in coordination with the Foundation SMEs.

**AASHTO
C10.5.5.2.3**

10.5.2.3 Drivability Analysis

CDOT Standard Specification 502 provides requirements for pile drivability analyses (wave equation analysis of pile driving [WEAP]). The Contractor typically completes WEAP.

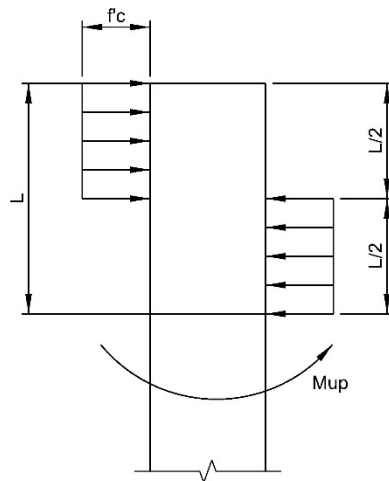
**AASHTO
10.7.8**

The Geotechnical Engineer should consider completing WEAP during the design phase when:

- A pile type, section, or driving procedure not routinely used in local practice (see Section 10.5.1.1) is proposed.
- A pile with an axial resistance greater than what is typically used in local practice or which may require the use of a pile driving hammer larger than typically used in Colorado (nominal resistance greater than approximately 500 kip) is proposed.
- A pile will be driven into a relatively deep bearing layer such that the driving resistance is likely to exceed the required geotechnical axial resistance (over-driving).

10.5.3 Top of Pile Fixity

The following simplified method may be used to calculate the minimum pile embedment required to classify the connection at the top of the pile as fixed.



$$M_{up} = \phi f'_c b_f \left(\frac{L}{2} * \frac{3L}{4} - \frac{L}{2} * \frac{L}{4} \right)$$

$$M_{up} = \phi f'_c b_f L^2 \left(\frac{3}{8} - \frac{1}{8} \right)$$

$$4M_{up} = \phi f'_c b_f L^2$$

$$L = \sqrt{\frac{4M_{up}}{\phi f'_c b_f}}$$

Figure 10-1: Pile Fixity

Where:

L = Required pile embedment into cap (in.)

ϕ = Strength reduction factor for concrete bearing = 0.7 (AASHTO 5.5.4.2)

f'_c = 28-day compressive strength of concrete (ksi)

M_{up} = Plastic moment capacity of pile about strong axis (kip-in.)

b_f = Pile flange width (in.)

Table 10-1 presents the calculated embedments for the most common HP shapes, based on a ϕ of 0.7 and f'_c of 4.5 ksi.

Table 10-1: Calculated Embedments

HP Pile Section	Minimum Embedment (in.)
12x53	20
12x74	24
14x89	26

For specific criteria regarding pile embedment at integral abutments, see BDM Section 11.

10.5.4 Field Splice

The Designer shall note on the plans the elevation above which complete joint penetration (CJP) welds are required for the flanges of all H-pile field splices. The Designer shall also note on the plans that below this elevation, partial joint penetration (PJP) flange welds or other commercially available splices using mechanical connections may be permitted upon review by the Engineer. The elevation shall be taken as the lowest primary moment inflection point in the pile obtained from all load combinations producing bending moment in the pile, including scour and extreme event load cases (see Figure 10-2). At the Designer's discretion, piles that are not subjected to significant bending moment may be exempt from this provision.

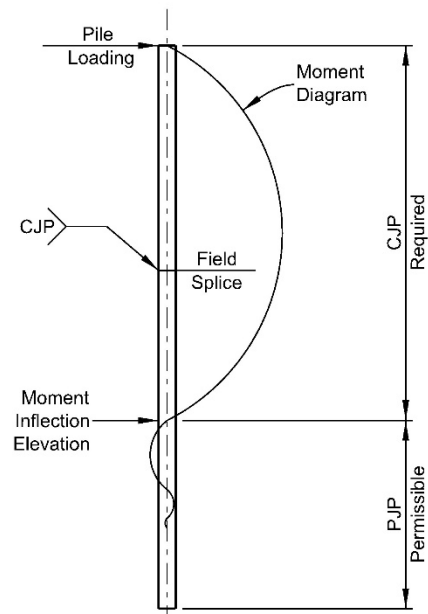


Figure 10-2: Moment Inflection Point and H-Pile Field Splices

10.5.5 Dynamic Testing

As required by AASHTO and CDOT Standard Specification 502, dynamic testing shall be completed during pile installation to monitor potential pile damage, to determine axial resistance, and to establish driving criteria.

**AASHTO
10.7.3.8.3**

In accordance with AASHTO, higher resistance factors for geotechnical axial resistance may be used if dynamic testing is completed during pile installation. The Designer should note that for bridges with more than 100 piles, the test frequency required by AASHTO to use a resistance factor of 0.65 is more stringent than the test frequency required by CDOT Standard Specification 502. Therefore, if a resistance factor of 0.65 is used for a bridge with more than 100 piles, a Project Special Provision is required to modify the dynamic testing frequency indicated in the Standard Specification to maintain compliance with AASHTO.

**AASHTO
Table
10.5.5.2.3-1**

10.5.6 Load Testing

Load testing (axial or lateral) may be conducted to justify the use of increased resistance factors and to reduce uncertainty in the performance of driven piles. During the structure selection process, the Designer shall review and evaluate the need, benefits, and feasibility of conducting load testing.

When load testing is completed, the entity completing the load test shall prepare a report sealed by a professional engineer licensed in the State of Colorado summarizing test results.

10.6 DRILLED SHAFTS

The term “drilled shaft” as used herein is interchangeable with drilled pier, drilled caisson, and other similar terms.

10.6.1 General

10.6.1.1 Geometry and Dimensions

Drilled shafts used to support bridges and retaining walls shall have a minimum diameter of 24 in. Drilled shafts used to support sound walls shall have a minimum diameter of 18 in. Length to diameter ratios, L/D, are typically less than 25.

Where a drilled shaft supports a single column, the top of shaft shall be embedded a minimum of 2 ft. below ground surface, unless the Geotechnical Engineer recommends deeper embedment.

In contrast to AASHTO, CDOT allows the use of drilled shafts that are smaller in diameter than the columns they support. This allows constructability advantages, such as eliminating the need for separate column dowels embedded into the caisson.

**AASHTO
10.8.1.3**

10.6.1.2 Tip Elevation

The Designer shall add a note on the plans requiring drilled shafts to be advanced to the estimated tip elevation or to the minimum penetration into bedrock, whichever produces the lower tip elevation. No allowance will be made to terminate the drilled shafts above the estimated tip elevation on

account of encountering bedrock above the anticipated elevation or any other circumstances.

10.6.2 Geotechnical Design and Analysis

10.6.2.1 Axial Resistance in Weak Rock

Rock-socketed drilled shafts are frequently used in Colorado. SPT-based methods are often used to estimate the axial resistance of sedimentary bedrock encountered along the Front Range. For sites with bedrock N-values typically between 20 and 100 blows per foot, the “soil-like claystone” design procedure described by Abu-Hejleh et al. (2003) may be used to determine nominal unit side resistance and end bearing values.

The resistance factor of 0.75 recommended by Abu-Hejleh et al. (2003) for the “soil-like claystone” method shall not be used because this value exceeds typical resistance factors specified by AASHTO, including the maximum resistance factor of 0.70, which assumes load testing is completed.

**AASHTO
Table
10.5.5.2.4-1**

A resistance factor of 0.60 shall be used with the “soil-like claystone” method (Abu-Hejleh et al., 2003). The resistance factor was calculated by fitting to allowable stress design (ASD) assuming the following:

- Ratio between permanent and live loads of 3:1
- Permanent Load Factor of 1.25
- Live Load Factor of 1.75
- Factor of Safety of 2.25

For sites with bedrock N-values typically greater than 100 and where rock coring produces suitable core recovery (i.e., samples can be recovered for strength testing and the rock mass can be characterized to an appropriate degree), it is preferable to evaluate axial resistance using design methods based on the unconfined compressive strength, as described in AASHTO and FHWA Report No. FHWA-NHI-10-016 (Brown et al., 2010).

**AASHTO
10.8.3.5**

10.6.2.2 Roughening and Shear Rings

Roughening may be completed to remove smeared or disturbed materials from the sides of drilled shaft excavations. The Geotechnical Engineer shall indicate when roughening is required. Roughening is difficult to inspect and should be used only when approved by Unit Leader in coordination with the Foundations SMEs.

Because shear rings are difficult to inspect, they shall not be used unless approved by Unit Leader in coordination with the Foundations SMEs.. As an alternative to using shear rings to increase axial resistance, the drilled shaft could be lengthened or increased in diameter.

10.6.3 Non-destructive Integrity Testing

10.6.3.1 Test Methods

Cross-hole sonic logging (CSL) is an acceptable non-destructive method to evaluate the integrity of completed drilled shafts. Thermal Integrity Profiling (TIP) may be used with approval from Unit Leader in coordination with the

Foundations SMEs. If TIP is specified, the designer shall prepare an appropriate Project Special Provision.

Methods based on the analysis of stress waves, such as sonic echo and impulse response, shall not be used as the primary test method unless access tubes are unavailable.

All testing shall be completed in accordance with the applicable ASTM standards.

10.6.3.2 Test Frequency

The requirements presented in this section are only applicable to drilled shafts used as bridge foundations. The frequency of integrity testing for drilled shafts used in other applications (retaining structures, landslide stabilization, etc.) shall be at the discretion of the Designer, as approved by Unit Leader in coordination with the Foundations SMEs. As necessary for non-bridge applications, the Designer should prepare a Project Special Provision to specify the desired test frequency.

CSL access tubes shall be installed in all non-redundant drilled shafts. With respect to CSL testing requirements, a non-redundant drilled shaft is defined as any drilled shaft at an abutment or a pier supported by two or fewer drilled shafts. CSL access tubes shall also be installed in all drilled shafts to be constructed in a water crossing and in all drilled shafts that will be constructed in soil/rock requiring the use of temporary excavation support (i.e. casing or drilling fluid). At the discretion of the Designer, other drilled shafts on the project may be selected to require CSL testing, such as largely spaced shafts.

CSL testing shall be completed on all non-redundant drilled shafts. CSL testing shall be completed on a minimum of 50 percent of drilled shafts equipped with CSL access tubes. Testing locations shall be at the discretion of the Engineer. If CSL testing indicates anomalies, the remaining drilled shafts at the pier/abutment shall also be tested.

Installation of CSL access tubes and integrity testing are not required for drilled shafts with permanent casing socketed into bedrock, regardless of redundancy or shaft location.

Other agencies, such as railroads, may have more stringent testing requirements. The designer shall determine if any non-CDOT entities have applicable testing requirements.

The Designer shall indicate in the plans the minimum number of drilled shafts to be tested.

10.6.3.3 Addressing Anomalies

Anomalies indicated by CSL testing shall be addressed in accordance with Standard Specification 503.

Guidance on repairing drilled shaft anomalies is described in FHWA Report No. FHWA-NHI-10-016 (Brown et al., 2010). Additional information is provided in the ADSC – IAFD Standard Drilled Shaft Anomaly Mitigation Plan

(Association of Drilled Shaft Contractors – International Association of Foundation Drilling, 2014).

If test methods other than CSL are proposed, the Designer shall specify criteria for the evaluation and acceptance of test results in a Project Special Provision.

10.6.4 Load Testing

Load testing (axial or lateral) may be conducted to justify the use of increased resistance factors and to reduce uncertainty in the performance of drilled shafts. During the structure selection process, the Designer shall review and evaluate the need, benefits, and feasibility of conducting load testing.

When load testing is completed, the entity completing the load test shall prepare a report sealed by a professional engineer licensed in the State of Colorado summarizing test results. The report shall include all necessary information and data to enter the test into the DSHAFT load test database (see Garder et al., 2012).

10.7 REFERENCES

CDOT Research, 2003, Improvement of the Geotechnical Axial Design Methodology for Colorado's Drilled Shafts Socketed in Weak Rocks, Report No. CDOT-DTD-R-2003-6.

Federal Highway Administration, September 2018, Drilled Shafts: Construction Procedures and Design Methods, Publication No. FHWA-NHI 18-024.

Colorado Department of Transportation, 2021, Geotechnical Design Manual.

SECTION 11 ABUTMENT, PIERS, AND RETAINING WALLS

11.1 GENERAL REQUIREMENTS

This section provides design guidance and construction requirements for abutments, piers, and retaining walls. Abutments and piers support bridge superstructures, whereas retaining walls function primarily as earth retaining structures but can serve a dual purpose as an abutment.

11.2 CODE REQUIREMENTS

The design of abutments, piers, and retaining walls shall be in accordance with AASHTO, this BDM, the Geotechnical Design Manual, and current Staff Bridge Worksheets.

**AASHTO
Section 11**

11.3 ABUTMENTS

CDOT permits the following abutment types:

- Integral
- Semi-integral
- Tall Wall
- Seat Type
- Geosynthetic Reinforced Soil (GRS)
- Other, (i.e., semi-deep, exposed multi-column in front of a retaining wall, integral on sheet piling) with approval from Unit Leader in coordination with Foundations SMEs.

Abutments shall be designed for all applicable AASHTO load combinations. Loads from the girders shall be applied at the centerline of bearing and can be assumed continuous over the centerline of foundation elements. Dynamic load allowance shall be included in the design of the bearing cap and diaphragm but not the foundation elements. The Designer need only apply one-half of the approach slab dead load to the bearing cap. Live loading on the approach slab may be ignored. If no approach slab is provided, equivalent soil heights for live load surcharge of varying abutment heights shall be as provided in AASHTO.

**AASHTO
Table
3.11.6.4-1,
3.11.6.5**

If the height of the bearing cap varies more than 18 in. from each end, the Designer should slope the bottom of the cap.

When Strut & Tie Models are used for the design, they must be shared with the design checker to obtain concurrence on the models. Refer to Section 37.5 of this BDM for more details.

Pile and drilled shaft spacing and minimum clearances shall be per AASHTO. The minimum foundation element length shall be 10 ft. below bottom of bearing cap.

**AASHTO
10.7.1.2,
10.7.1.3, &
10.8.1.2**

The Structure Selection Report shall document the recommended type of abutment selected for the project.

11.3.1 Integral Abutments

Integral abutments are preferred for most bridges due to the elimination of expansion joints and bearings at supports, simplified construction, and reduced maintenance costs. Integral abutments rigidly attach both superstructure and supporting foundation elements so that the thermal translation and girder end rotations are transferred from the superstructure through the abutment to the foundation elements. The superstructure and substructure act as a single structural unit by distributing system flexibilities throughout the soil.

Use integral abutments where continuous structure units are shorter than the lengths shown in Table 11-1 (from FHWA Evaluation of Integral Abutments Final Report, 2006). A bridge unit includes one or more spans and can be separated at a pier from an adjacent unit by an expansion device or a fixed gap.

Table 11-1: Limiting Structure Lengths for Integral Abutments

Girder Material	Maximum Unit Length
Steel	460 ft.
Cast-in-Place Concrete	460 ft.
Precast and Post Tensioned Concrete	As calculated (460 ft. max.)

Assumptions:

- Point of zero movement is located at the midpoint of the bridge unit.
- Maximum unit lengths shown are per current research recommendations.

In addition to meeting the maximum unit length restrictions in Table 11-1, the total factored movement in one direction, expanding or contracting, at the integral abutment from the point of zero movement shall be 2 in. or less. The total factored movement shall include temperature, creep, shrinkage, and elastic shortening. The temperature range used to determine the movement shall be per Section 14 of this BDM and AASHTO. Assume a base uniform temperature of 60° in calculating the directional movement toward each abutment.

**AASHTO
3.12.2**

With Unit Leader approval, greater unit lengths may be used if analysis shows that abutment, foundation, and superstructure design limits are not exceeded, and that the expansion joint can accommodate movement at the end of the approach slab. Include an analysis backing up the decision with the design calculations for the structure. The Structure Selection Report shall include a discussion of this approach. CDOT has successfully used longer unit lengths on integral bridges of 1,000 ft. (for the Vasquez over Colorado Blvd bridge) by using a finger plate expansion device. Unit lengths when using a 0-4 in. strip seal shall be limited to 800 ft.

Do not use integral abutments when a straight-line grade between ends of a unit exceeds 5 percent. Research shows that the presence of high grades tends to lock up one end, thereby causing higher movements on the other.

During design, a pinned connection is assumed to develop between the pile cap and foundation element to allow the transfer of vertical and shear loads into the foundation element. If a pin does not develop, a fixed or partially fixed

condition will be present, which can cause cracking in the deck and girders due to the developed moment from lack of girder rotation.

The preferred pile orientation is to align the weak axis of the pile with the centerline of abutment. The Designer should use the detail shown on Figure 11-1. Weak axis bending generates less resisting force in the piles from unintended frame-action with the superstructure and better accommodates bridge displacements, when compared with strong axis bending. A single row of piles shall be used with integral abutments.

To increase pile flexibility, the Designer may use the details shown on Figure 11-1 and shall determine the pile depth to establish stability. If oversized holes are used, the length shall be determined by the design and the hole shall have a minimum diameter of pile $d + 1$ ft., where "d" is pile depth. This detail increases the depth to point of fixity, thereby decreasing pile stiffness. Assume the point of fixity for laterally loaded piling as either the location of zero movement or location of maximum moment. The pile should extend a minimum length of 10 ft. beyond the prebore/pipe and through the overburden until stability is achieved. Design the single row of piles as an axial loaded beam-column interaction. Check steel H-piles for lateral stability and buckling capacities. Ignore soil confinement to the full depth of estimated scour or limits of pea gravel fill when not in a scour situation. However, the soil confinement of pea gravel may be considered when the designer needs the extra lateral stability that it provides, either to reduce the pile length or to avoid upsizing to a larger pile size. If the soil confinement of pea gravel is considered and if project specific geotechnical information is not available, the designer may use the following parameters: $k = 300$ pci, $\Phi = 40^\circ$, and $\gamma = 95$ pcf. Consider a semi-integral abutment configuration or seat type abutment if there is uncertainty about the development of a pin, insufficient flexibility, or if integral abutment design criteria cannot be met.

Drilled shafts may be used for integral abutments provided a pin detail such as that shown on Figure 11-2 is specified at the top of caisson. Extending fully developed drilled shaft reinforcing around the perimeter into the bearing cap prevents a pin from forming and is not permitted. Design dowels connecting the drilled shaft to the bearing seat for seismic loading.

To ensure that girder ends will rotate during the deck pour, the Designer shall add a note to the plans requiring the Contractor to pour the deck within two hours of the integral diaphragms.

The depth of the integral abutment, measured from top of deck to bottom of pile cap, shall typically be less than or equal to 13 ft. The maximum pile cap depth shall be less than or equal to 6 ft. and the minimum shall be 3.5 ft. These maximum limits prevent framing action on an integral abutment from occurring and ensure it acts like the intended pin by controlling bending and torsional forces. Designs that require greater abutment depth will need a special design with considerations for torsional and passive earth pressure bending forces.

The bottom of the bearing cap shall be embedded 1.5 ft. minimum into the embankment and provide 2 ft. minimum from the top of the embankment to the bottom of the girder. If the bridge is curved, the maximum degree of curvature shall be less than or equal to 5° .

Skewed bridges induce biaxial bending into the foundation elements from passive soil pressure. Unless otherwise approved by Unit Leader, limit skew angles to 30° or less. The Designer shall also include in the analysis all forces that rotate the structure.

On skewed bridges, the Designer shall provide 3 in. minimum clearance from the girder flanges to the back face of abutment. If sufficient clearance is not provided, the flange shall be coped or the abutment width increased. The coping shall parallel the centerline of abutment and not extend across the girder web.

For pre-tensioned or post-tensioned concrete bridges, use methods to increase foundation flexibility when the girder contraction due to elastic shortening, creep, shrinkage and temperature fall exceeds 1 in. Methods include temporarily sliding elements between the diaphragm and bearing cap, details that increase the foundation flexibility, or other details approved by the Unit Leader. Take steps to ensure that the movement capability at the end of the approach slab is not exceeded.

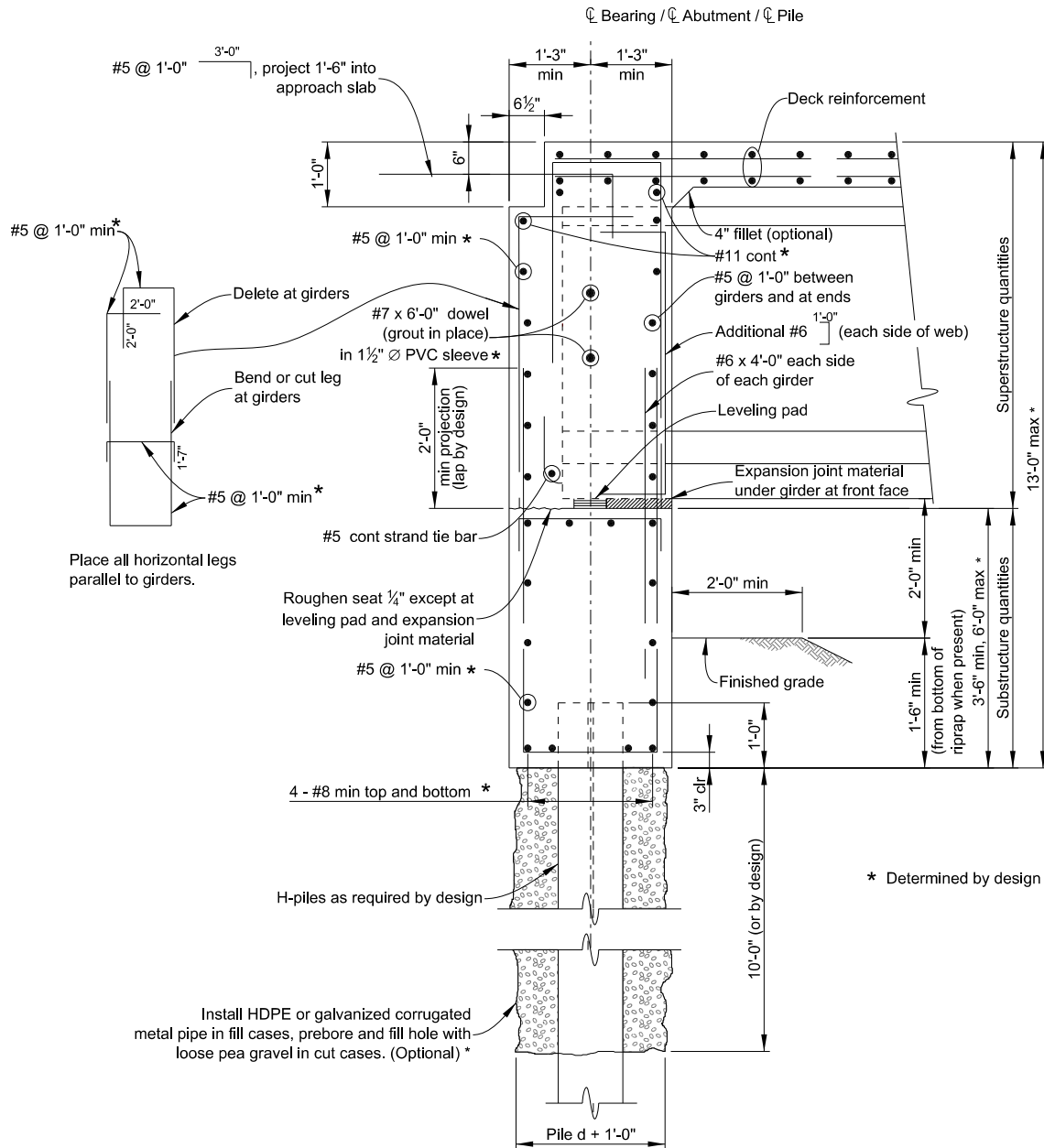


Figure 11-1: Integral Abutment on H-Piles

Notes:

1. All abutment and wingwall concrete shall be Class DF or D (Bridge).
2. Extend strands, per design, from the bottom of precast sections into the abutment. See Staff Bridge Worksheets.
3. Anchor the bottom of steel girder sections to the abutment with studs, bearing stiffeners, anchor bolts, or diaphragm gussets.
4. Pour the deck and portion above the bearing seat within 2 hours of each other.

5. Reinforcing steel shall be determined by design.
6. All reinforcing shall be epoxy coated or corrosion resistant.
7. Place all horizontal reinforcement legs above the bearing seat parallel to girders.
8. For integral abutments on drilled shafts, height of gap between top of caisson and bottom of diaphragm shall be verified to ensure that girder rotation will not cause the gap to close.
9. Use a leveling pad designed per Section 14.5.7 of this BDM on integral type abutments.
10. For thermal stress relief, H-Pile should have the weak axis aligned with centerline of abutment. Strong pile axis alignment is allowed provided thermal modeling with a refined method of pile-soil interaction analysis to determine actual movement is used and full thermal movement is accommodated.
11. Include the cost of pipe (CMP/HDPE), prebore, and fill material inside pipe (pea gravel or alternative approved by Unit Leader) in the work.
12. The field splice weld zones defined in Section 10.5.4 of this BDM shall be noted in the plans.
13. Grout #7 Bars into the PVC sleeve prior to the diaphragm pour. The girder worksheet should show the cast in PVC sleeve instead of a coil tie.

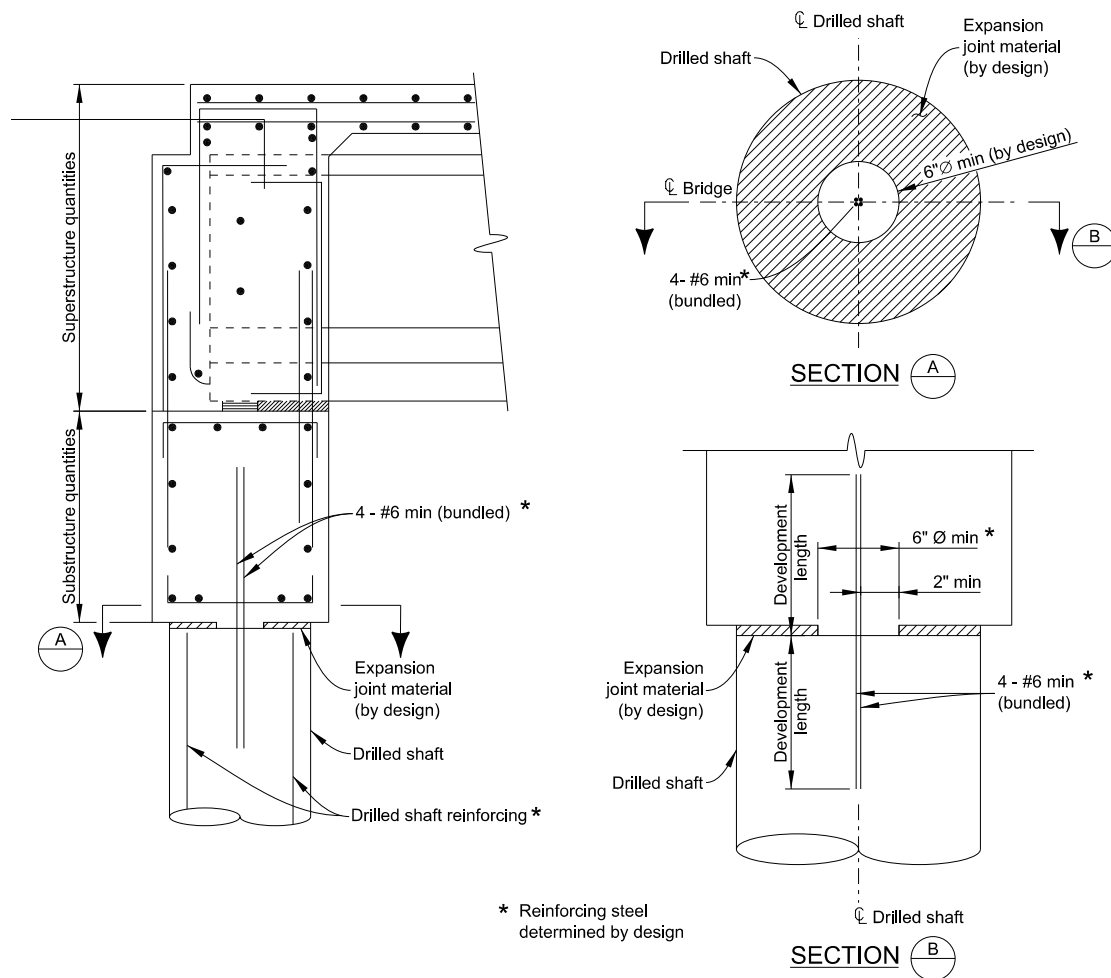


Figure 11-2: Integral Abutment on Drilled Shafts

(For details of reinforcement, refer to Figure 11-1. See Notes 1–13 with Figure 11-1.)

11.3.2 Semi-integral Abutments

Semi-integral abutments are like integral abutments because both eliminate the expansion joints at supports and encase the girder ends in concrete. The difference is that the pin for a semi-integral abutment is located at the top of bearing seat via a bearing device and the foundation element connection at the bottom of bearing cap is fixed. The bearings accommodate the rotational and horizontal movements. Using spread footings, footings on piles or drilled shafts, multiple rows of piles, or drilled shafts can establish abutment fixity.

When semi-integral abutments are used, intermediate shear blocks between girders or end blocks beyond the edge of deck shall allow a means for lateral load distribution to the substructure. If a shear block is not practical, use anchor bolts with a sole plate. The Designer shall provide an area to allow for jacking the superstructure and bearing replacement per Section 14.5.6 of this BDM.

Figure 11-3 and Figure 11-4 show semi-integral abutments on drilled shafts.

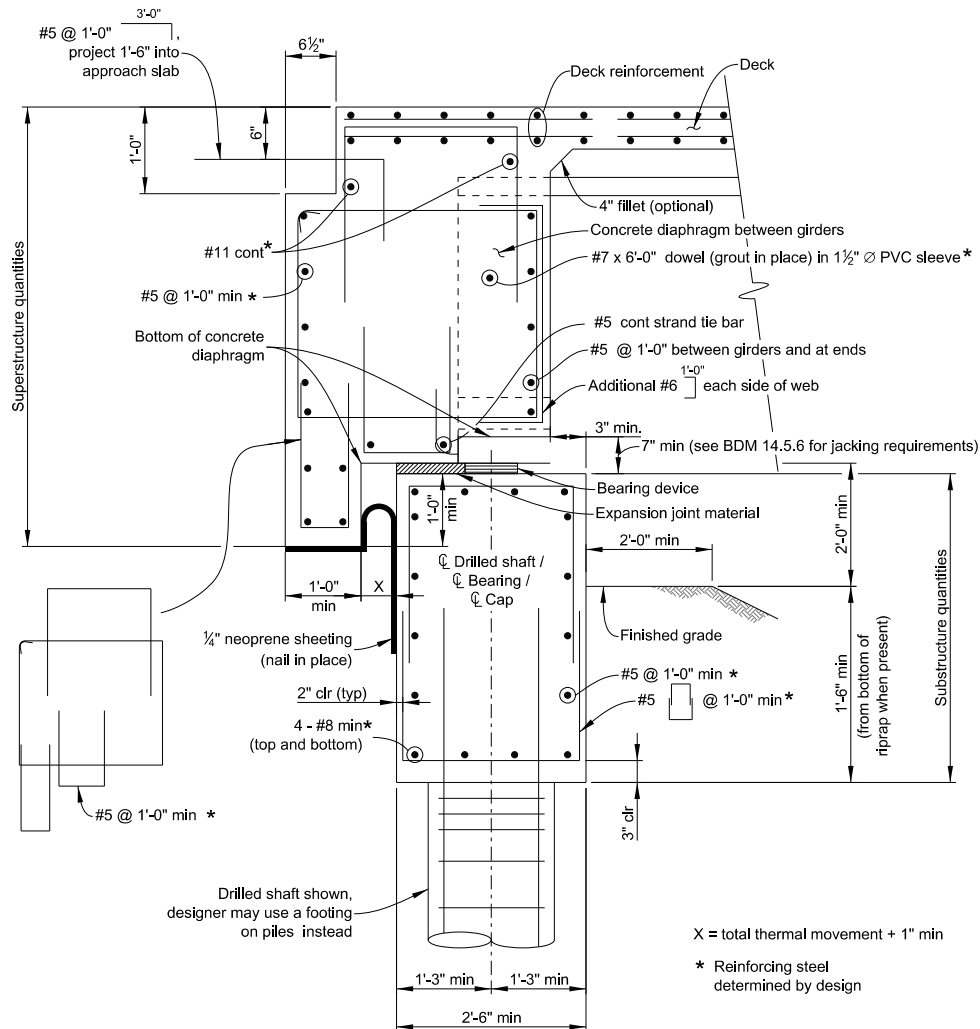
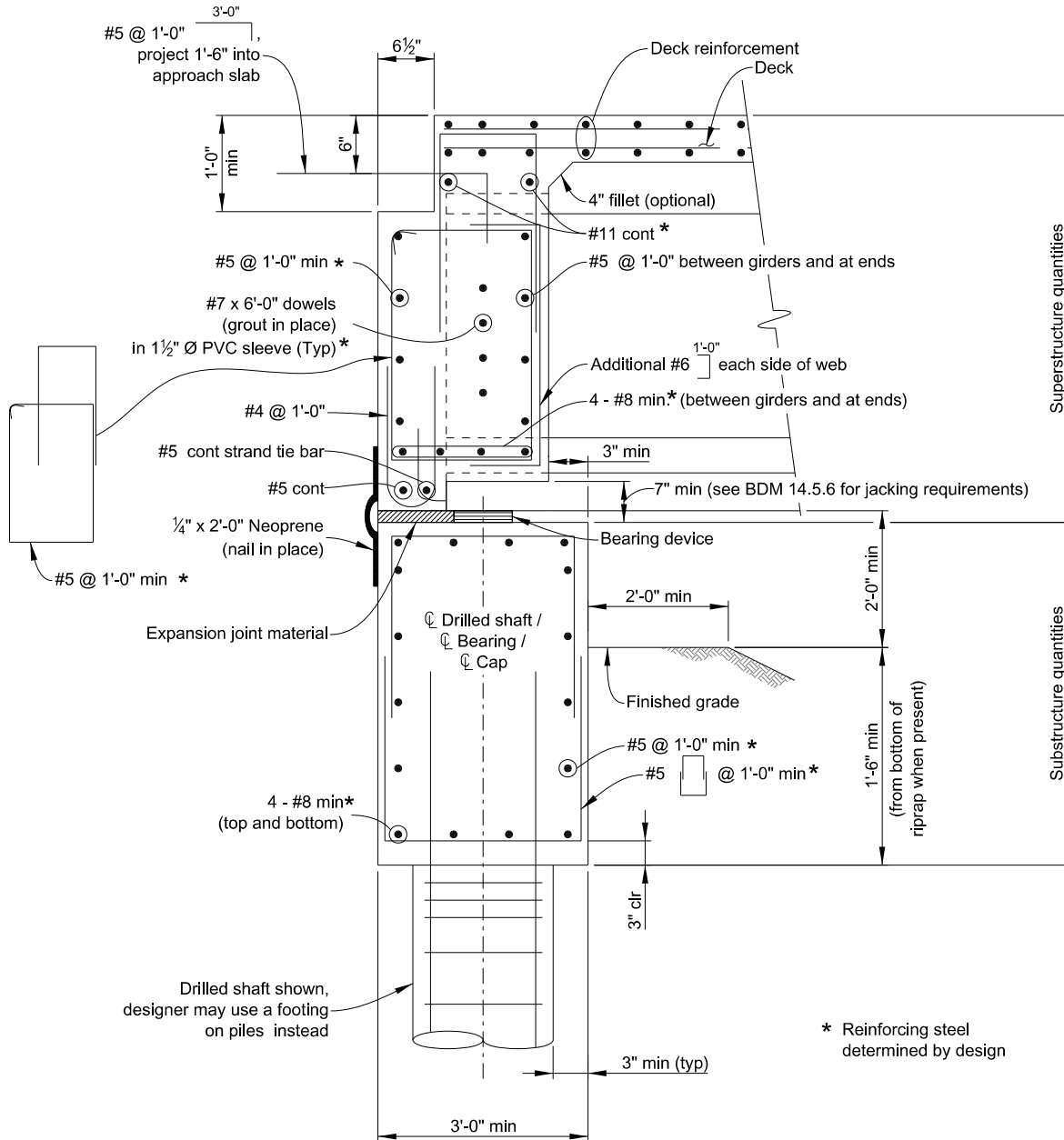


Figure 11-3: Semi-Integral Abutment (Alternative 1)

Notes (For Figures 11-3 & 11-4):

1. All abutment and wingwall concrete shall be Class DF or D (Bridge).
2. Extend strands, per design, from the bottom of precast sections into the abutment. See Staff Bridge Worksheets.
3. Anchor the bottom of steel girder sections to the abutment with studs, bearing stiffeners, anchor bolts, or diaphragm gussets.
4. Pour the deck and portion above the bearing seat within 2 hours of each other.
5. Reinforcing steel shall be determined by design.
6. All reinforcing shall be epoxy coated or corrosion resistant.
7. Place all horizontal reinforcement legs parallel to girders.
8. Provide lateral restraint with anchor bolts and/or intermediate or end shear blocks.

9. Bearings pads designed per Section 14.5 of this BDM are required for semi-integral abutment types. Leveling pads are not allowed.
10. Grout #7 Bars into the PVC sleeve prior to the diaphragm pour. The girder worksheet should show the cast in PVC sleeve instead of a coil tie.



11.3.3 Seat Type Abutments

Seat type abutments have an expansion gap between the backwall and end of girders, as shown on Figure 11-5, and are typically used when large movements require a modular expansion device rather than a strip seal placed at the end of the approach slab.

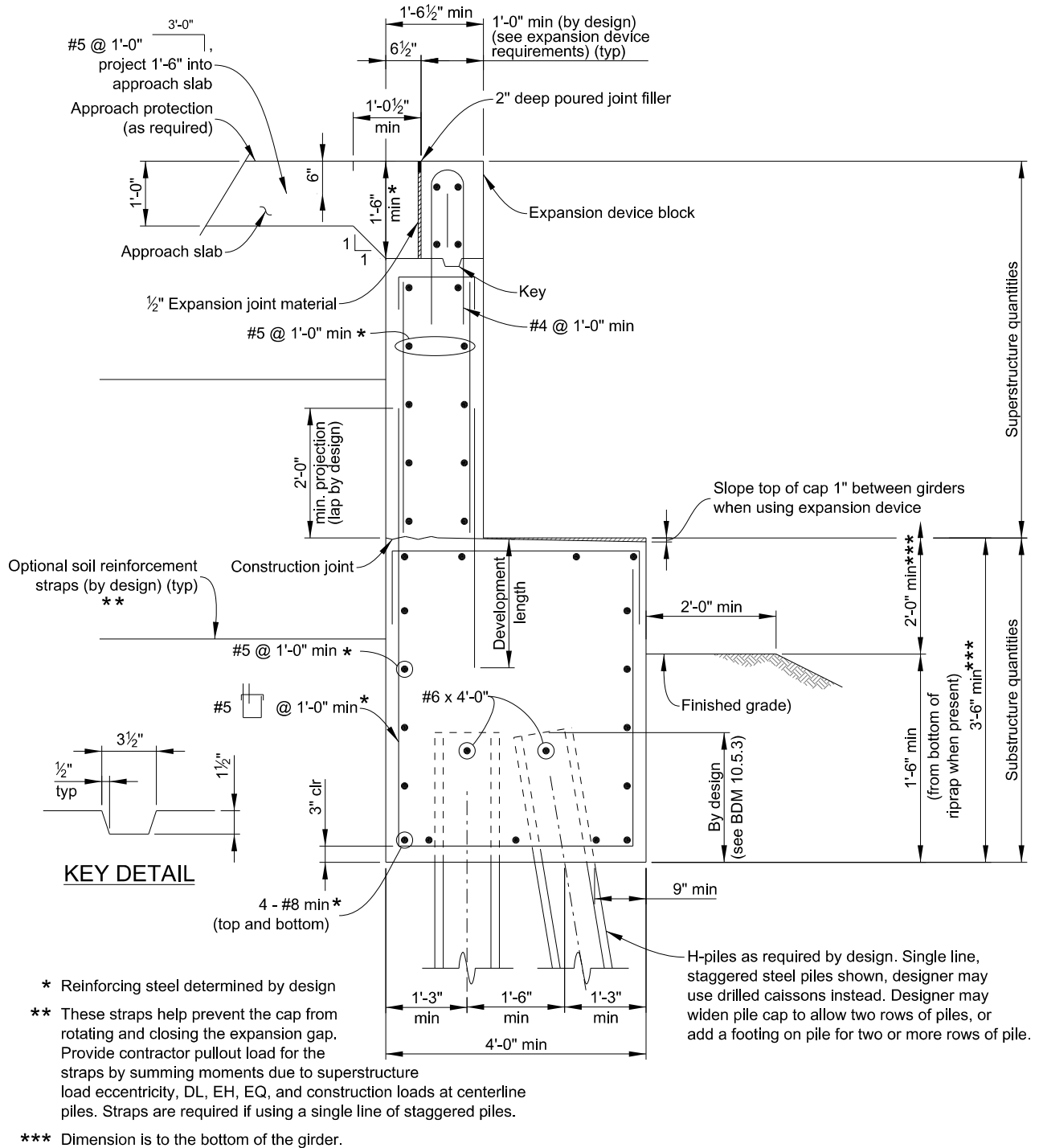


Figure 11-5: Seat Type Abutment

To provide a pinned connection between the superstructure and substructure, place the girders on bearing devices, thereby allowing rotational and horizontal movements. Using seat type abutments is discouraged due to the high maintenance costs associated with leaking expansion joints, substandard expansion device performance, and being prone to rotation and closing the expansion device.

Notes:

1. All abutment and wingwall concrete shall be Class DF or D (Bridge).
2. Reinforcing steel shall be determined by design.
3. All reinforcing shall be epoxy coated or corrosion resistant.
4. Apply an epoxy protective coating to the exposed portion of backwall, top of bearing seat, and front face of bearing cap.
5. Bearings pads designed per Section 14.5 of this BDM are required for seat type abutments. Leveling pads are not allowed.
6. To decrease a lateral load pressure on backface of abutment, a woven fabric soil reinforcing straps with 12 in. typical spacing with 3 in. low density polystyrene board or collapsible cardboard isolator may be used.

11.3.4 Tall Wall Abutments

Tall wall abutments, as shown on Figure 11-6, are used to shorten span lengths and are typically located at the approximate front toe of approach embankment. Depending on the required height, they can be founded on a single row of drilled shafts, footing on piles, or footing on drilled shafts. Due to the high cost of concrete, careful cost comparisons should be done before using this type of abutment instead of lengthening the bridge span. Architectural requirements can drive the use of this type of abutment rather than cost. The details shown in the semi-integral or seat type abutment sections can be used to connect the superstructure to the substructure.

11.3.5 Geosynthetic Reinforced Soil Abutments

Geosynthetic Reinforced Soil (GRS) is a type of retaining structure that consists of closely spaced (12 in. or less) geosynthetic reinforcement installed in granular backfill, along with a facing system approved by the Unit Leader in coordination with the Wall SMEs. GRS can be used at bridge abutments to directly support the bridge superstructure without the use of deep foundations. Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS) is a unique application of GRS bridge abutments. Compared to a conventional GRS abutment, which combines GRS with traditional elements of bridge design, GRS-IBS integrates the bridge approach, abutment, and superstructure to create a joint-free bridge system, without deep foundations or approach slabs.

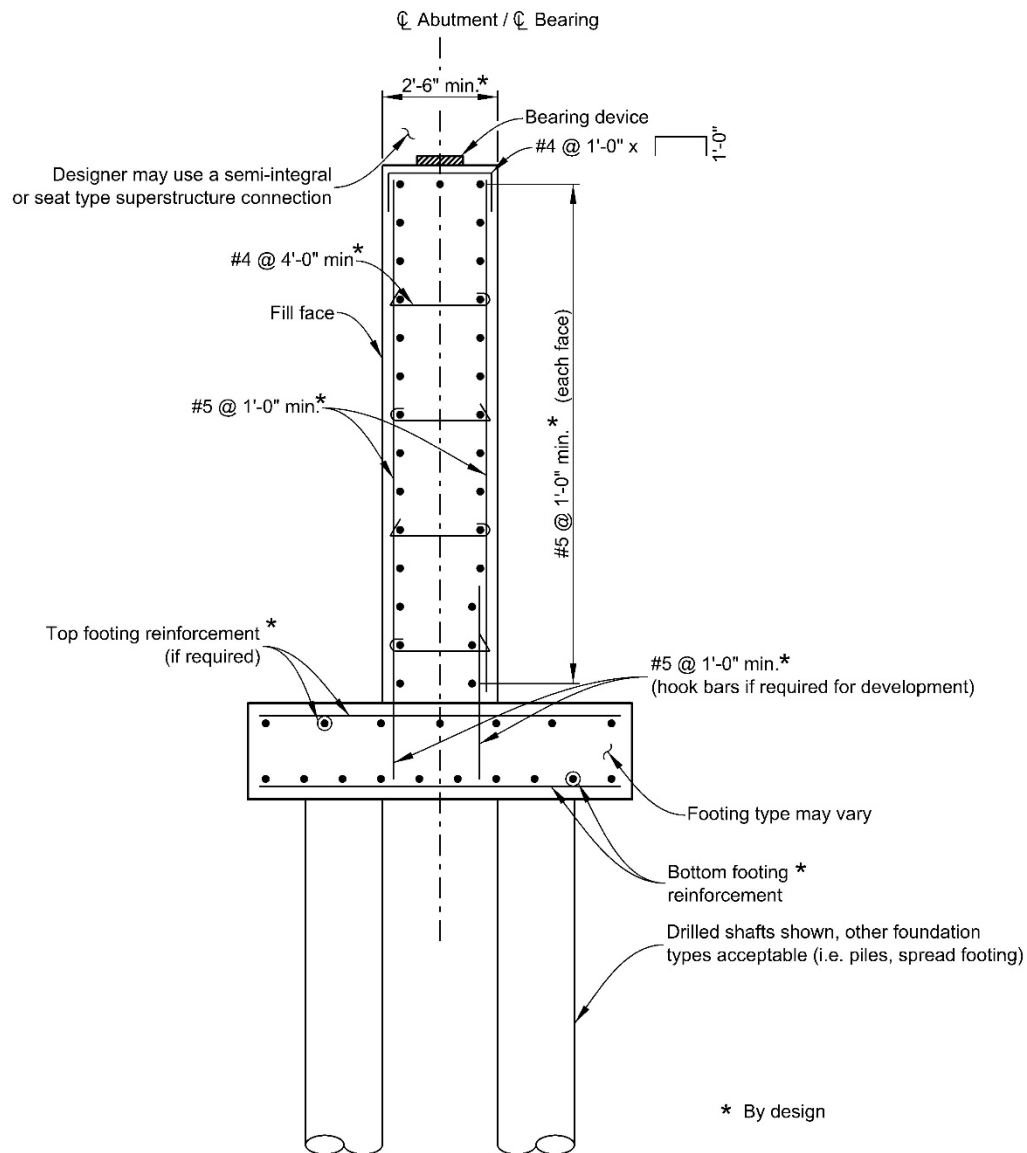


Figure 11-6: Tall Wall Abutment

The primary advantage of GRS abutments is that differential settlement between the approach fill and the bridge is minimized. The abutment fill supports the bridge, decreasing the severity of the “bump at the end of the bridge.”

Other potential advantages of GRS compared to conventional bridges supported on deep foundations include, but are not limited to:

- Decreased cost
- Accelerated construction

- Decreased reliance on specialized equipment and skilled labor for construction
- Flexible design that can be adjusted easily in the field to fit actual conditions
- Decreased maintenance due to the lack of expansion devices

GRS has been used most widely to support single-span bridges. However, the use of GRS to support continuous-span bridges is also feasible.

As discussed in the following subsections, GRS is not appropriate for sites where significant post-construction settlement or scour is expected.

11.3.5.1 Structure Selection Requirements

For bridges meeting one or more of the following structural, geotechnical, and hydraulic criteria, GRS shall be considered during the structure selection process:

- a. Single or continuous span bridges where long-term foundation settlement is anticipated to be less than 1 in.
- b. Single-span bridges where bearing seat elevations can be adjusted during construction to provide the required vertical clearance, accounting for the anticipated short- and long-term foundation settlement.
- c. Bridges where scour is negligible or can be mitigated to a negligible level by features such as a cut-off apron wall, riprap, a reinforced soil foundation (see FHWA-HRT-11-026), or a combination thereof.

11.3.5.2 Design Criteria

GRS shall be designed in accordance with this BDM, the CDOT Geotechnical Design Manual, the FHWA publication *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide, FHWA-HRT-11-026* (FHWA, 2012), and AASHTO. The design shall be completed using LRFD methodology (see Appendix C of FHWA-HRT-11-026).

Additional geotechnical borings may be required to adequately characterize settlement of GRS abutments, particularly the settlement of the integration zone (i.e., the reinforced transition zone immediately behind the abutment). The geotechnical exploration shall be sufficient to characterize short- and long-term settlement of the GRS abutments. As appropriate, obtain relatively undisturbed thin-wall tube samples during the field investigation for consolidation testing to support the evaluation of post-construction settlement behavior.

The design of GRS abutments is an iterative procedure 11-13 requiring coordination among the structural, geotechnical, and hydraulics engineers, e.g., the Geotechnical Engineer must know footing dimensions and bearing pressures to estimate settlement values. Therefore, the design disciplines should coordinate as necessary for the evaluation and design of GRS abutments.

11.3.5.3 Settlement

The tolerable settlement is defined in terms of angular distortion between supports. Without a refined superstructure and substructure interaction analysis, use the angular distortion requirements stipulated in AASHTO as a guide.

**AASHTO
LRFD
10.5.2.2,
C10.5.2.2**

The primary factor in the design of a GRS abutment is tolerable settlement, which is closely related to superstructure continuity (simple or continuous). Achieving and maintaining vertical clearance requirements must also be considered.

Settlement of GRS abutments includes short-term settlement (occurring during construction) due to the elastic compression of foundation materials and long-term (post-construction) settlement, which can occur due to time-dependent consolidation of clay soils. Settlement also includes compression of the GRS itself.

Consider the estimated short- and long-term settlement when establishing abutment girder seat elevations. Evaluate actual loads and loading sequences before and after girder placement. For phased construction, evaluate the settlement between abutment phases to determine if a closure pour is needed. Surcharging and/or subgrade improvement measures can also be used to limit the differential settlements between phases.

During construction, monitor and record settlements before and after placement of girders and deck. Provide these settlements to the Bridge Designer and Geotechnical Engineer for their information. Due to the variability in methods available for settlement monitoring, write a Project Special Provision to indicate the method to use, minimum number of points to monitor, preservation of points, reporting frequency, and measurement and payment criteria.

Uncertainty in the calculation and estimation of settlement values can contribute to the risk of unsatisfactory long-term performance of a structure. However, the risk can be managed by considering the likelihood and consequences of settlement that are greater than the estimated values. For example, a single-span bridge can tolerate more angular distortion than a continuous-span bridge. Similarly, settlement of granular soils occurs relatively quickly and could be compensated for during construction. Post-construction settlement could also be corrected by adding an asphalt overlay, but the weight of the additional overlay should be considered in the design. The risk of long-term settlement can also be reduced by surcharging or pre-loading.

11.3.5.4 Approach Slabs and Pre-camber

For single-span bridges less than 150 ft. long and continuous-span bridges with a total length less than 250 ft., CDOT prefers to use asphalt-paved approaches and no expansion joints. See Figure 11-12.

To compensate for long-term differential settlement of the abutment and the adjacent roadway, a pre-camber (increase in proposed profile to account for settlement) of 1/100 longitudinal grade is allowed at either the expansion joint

at the end of the approach slab or, for bridges without an approach slab, at the back face of abutment, as shown on Figure 11-11 and Figure 11-12, respectively. The asphalt pavement camber can be accomplished with added asphalt during construction or post-construction resurfacing if the actual settlement is greater than that estimated.

The amount of pre-camber should be sufficient to compensate for long-term differential settlement and to eliminate ponding near the expansion joint, if used. Depending on the abutment height, a $\frac{1}{2}$ in. to $\frac{3}{4}$ in. pre-camber has typically been specified over the approach slab length. In addition to the pre-camber, a 4 in. PVC trough (a PVC pipe cut in half and daylighted at the edge of roadway), matching the roadway cross slope, should be used under the expansion joint to capture surface run-off and reduce infiltration into the GRS.

11.3.5.5 Design and Detailing Requirements

Figure 11-7 through Figure 11-15 provide example details for GRS abutment design. The following represent additional requirements and considerations:

- a. Connect the soil reinforcement directly under the girder seat spread footing to the facing with either a frictional or a mechanical connection.
- b. Limit the nominal soil bearing resistance beneath the spread footing to 14,000 pounds per square foot or as stated in the project geotechnical report. Higher bearing pressures may be feasible depending on the maximum grain size of the backfill and the spacing and properties of the reinforcement.
- c. Require a setback equal to $H/3$, with a minimum value of 3 ft., from the back of the facing to the centerline of the Service I resultant, where H is the height from the bottom of the spread footing to the roadway. See Figure 11-9 and Figure 11-10.
- d. Use reinforced concrete for the girder seat and back wall.
- e. Provide a GRS slope face with the reinforcement wrapped up and around the face of the individual soil layers and anchored (burrito wrap) behind the abutment and wingwalls.
- f. Require a minimum vertical clearance of 2 ft. from the top of wall facing to the bottom of girder (see Figure 11-7 through Figure 11-10 and Chapter 11 in the Bridge Detail Manual).
- g. Use concrete for the leveling pad at the base of the GRS abutment.
- h. Provide drainage measures to reduce the likelihood of water accumulating in the GRS backfill. Appropriate drainage features could include encapsulating the top of the reinforced soil zone with dual-track seamed thermal welded geomembrane or providing an internal drainage system.
- i. Provide a 3 in. minimum thick low-density polystyrene, collapsible cardboard void, or a void space with burrito wrap geosynthetic reinforcement behind the abutment back wall to isolate the back wall from the GRS backfill and to allow thermal expansion of the bridge.

- j. Provide a 6 in. wide polystyrene spacer or 3 in. minimum clear space between the back of wall facing to the toe of abutment spread footing to accommodate thermal movement.
- k. Extend the length of abutment soil reinforcement as a stiffness transition zone into the roadway embankment with a 1H(min):1V slope for cut or 2H(min):1V slope for fill to mitigate differential settlement caused by dissimilar foundations.
- l. Use GRS abutments with a truncated base (minimum reinforcement length of $0.35DH$, where DH is the design height measured from the top of the leveling pad to the roadway) and cut benches with a maximum height of 4 ft. if the global stability requirements are met (see Figure 11-7). GRS abutments with a truncated base are more likely to meet global stability requirements in cut conditions rather than fill conditions.
- m. For bridges with a non-yielding foundation at the pier(s) and a semi-yielding reinforced soil/foundation at abutment(s), there is a possibility that cracks will appear in the top of the deck over the first pier near the abutment. Cover these cracks with waterproofing membrane and asphalt overlay; however, with bare concrete decks, check the crack size and rigorously control or mitigate with FRP top reinforcement in the deck.

11.3.6 Wingwalls

11.3.6.1 Wingwall Design Length

The wingwalls, as shown in the Bridge Detail Manual, shall be laid out from a working point defined as the intersection of abutment back face and wingwall fill face to 4 ft. minimum beyond the point of intersection of the embankment slope with the finished roadway grade. In most situations, using the working point provides the Contractor economy of design by having the same wingwall length at opposite corners. It is preferred that the wingwall be constructed parallel to girders to minimize the soil pressure against the wingwalls. The maximum integral wingwall length from the working point shall be 20 ft. If a longer wingwall is required, as shown in the Bridge Detail Manual, the Designer should use a maximum of a 10 ft. long integral wingwall in conjunction with an independent wingwall to achieve the required design length. It is not desirable to add a footing or support at the end of wingwalls for integral abutments unless provision for movement and rotation are provided. It is acceptable to support the wingwall ends on seat type abutments, on semi-integral abutments if the wingwall is not attached to the superstructure, or where no abutment rotation is expected.

The Designer needs to be aware of the various effects of soil on wingwalls and design for the anticipated loading due to the downdrag from fill settlement or uplift due to expansive soils. These forces can cause cracking of the wingwalls and abutment if they are not accounted for. If significant movement is predicted, the Geotechnical Report shall provide design recommendations and coordinate with the Designer on possible solutions. The Designer should analyze the torsional effects from the soil on the wingwall abutment connection and determine if 135° hooked stirrups are required. For wingwalls on box culverts, see Section 12.

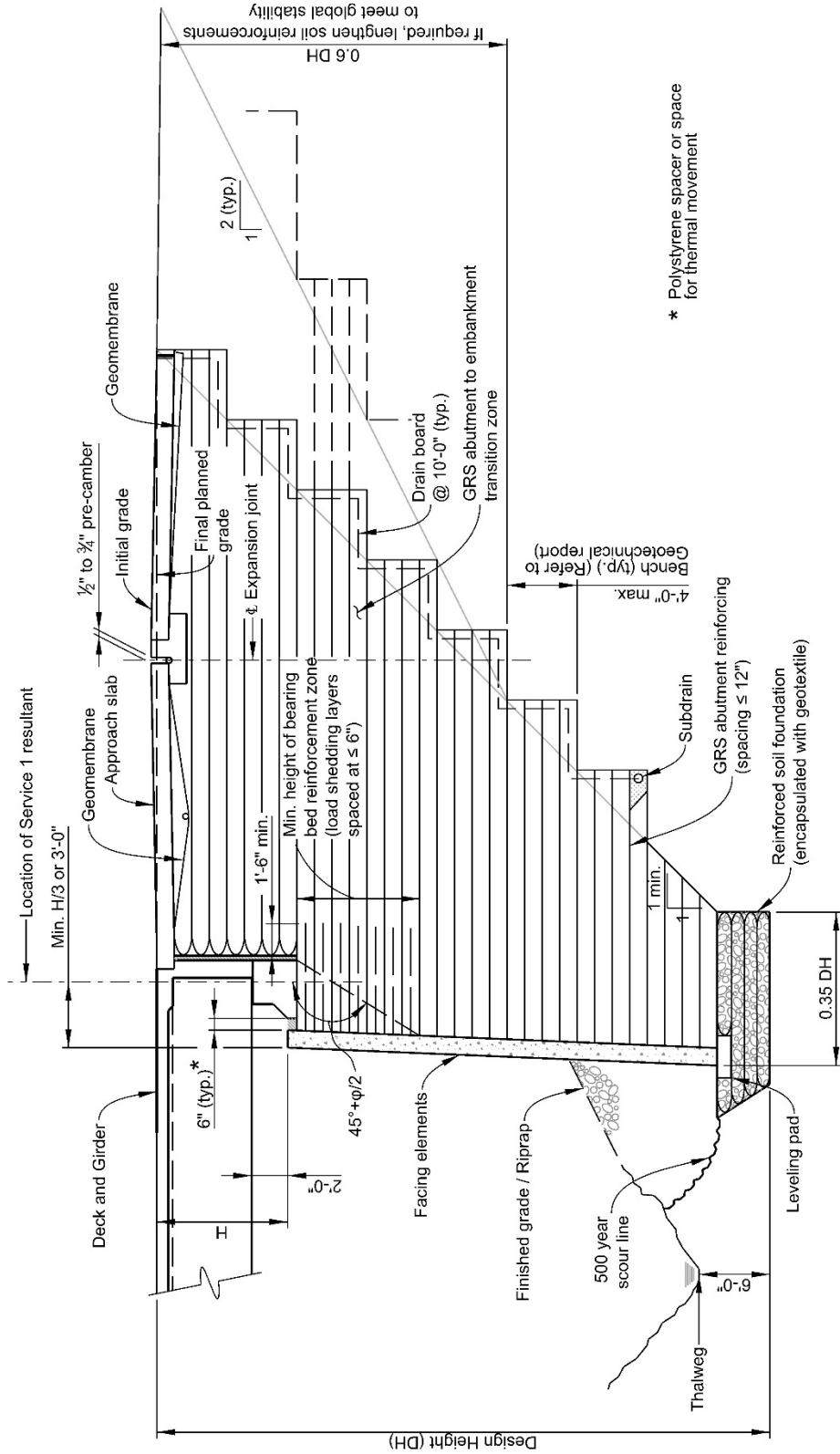


Figure 11-7: GRS Abutment (Cut Case)

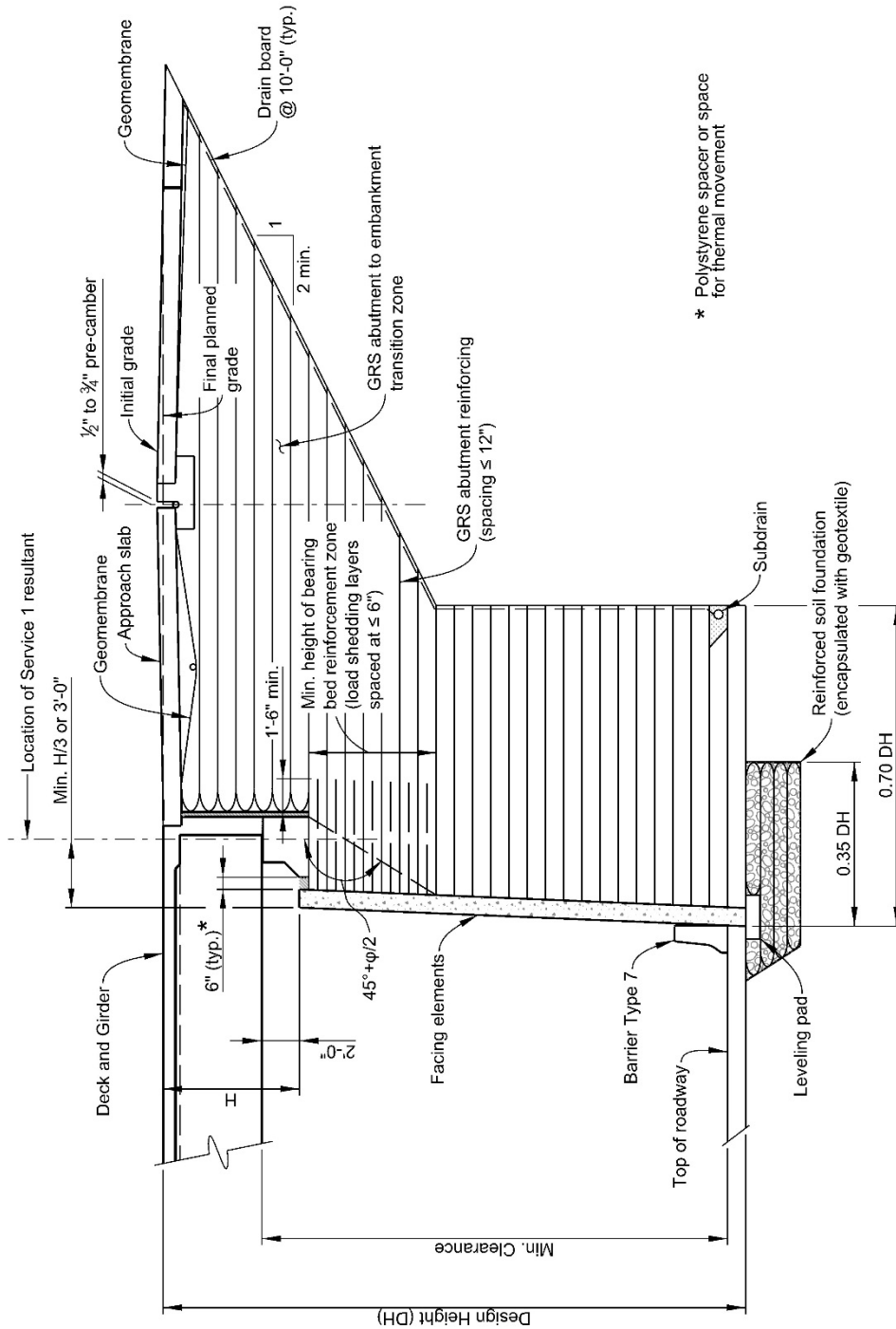


Figure 11-8: GRS Abutment (Fill Case)

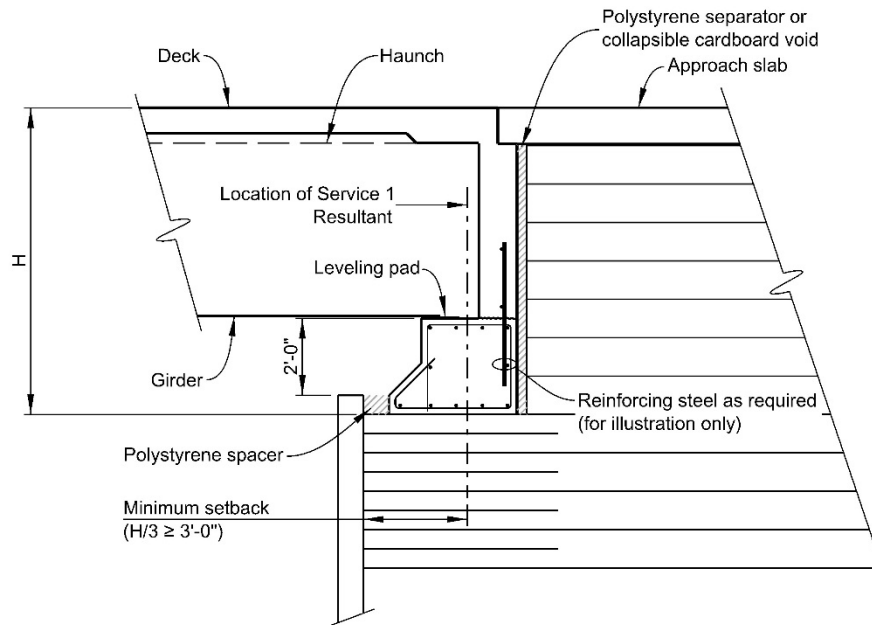


Figure 11-9: Integrated Girder Seat with Footer

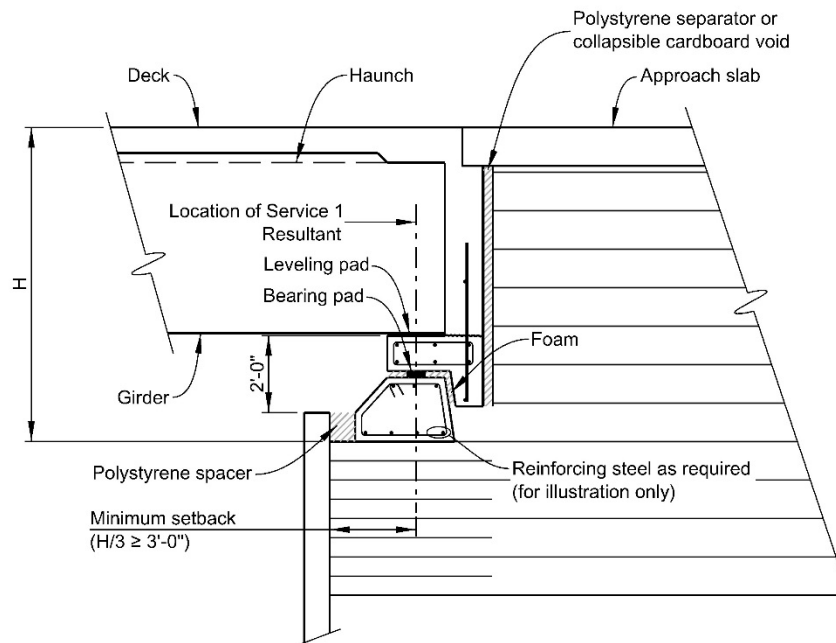


Figure 11-10: Separated Girder Seat with Footer

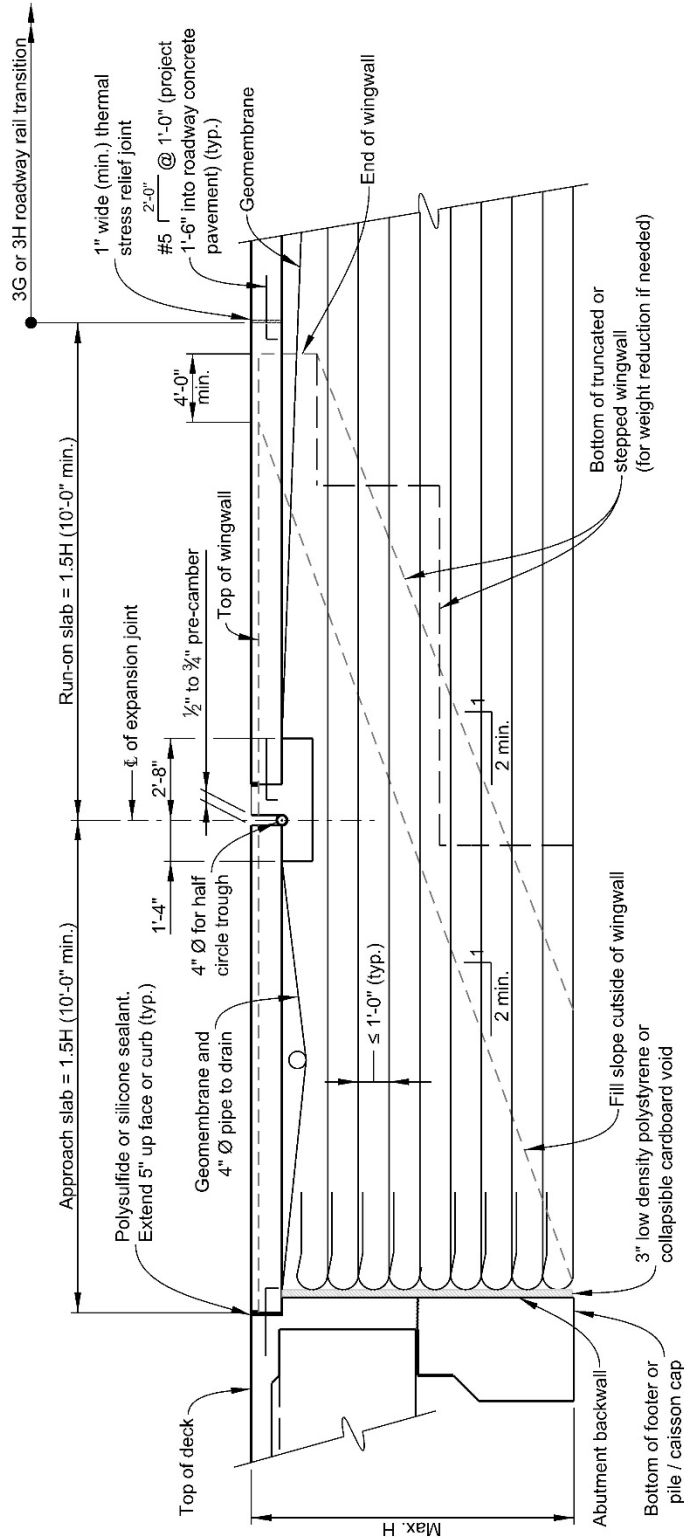


Figure 11-11: Transition Zone Behind Abutment Backwall (With Expansion Joint, Concrete Slab, and Roadway Pavement)

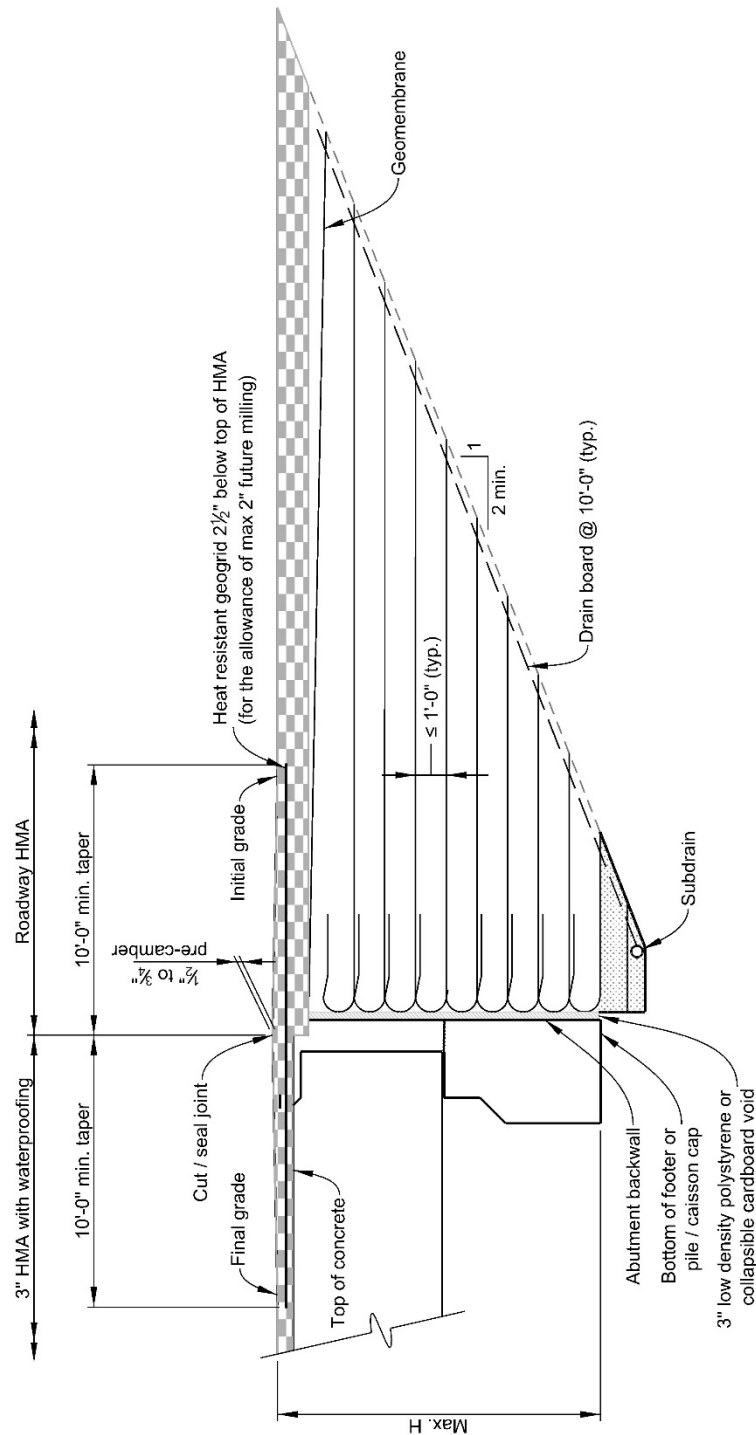


Figure 11-12: Transition Zone Behind Abutment Backwall (With Asphalt Pavement, No Approach Slab and No Expansion Joint)

11.3.6.2 Wingwall Design Loads

Design cantilevered wingwalls for tangent, non-skewed bridges for an active equivalent fluid pressure as recommended in the Geotechnical Report but not less than 36 psf. Design all other wingwalls for an at-rest equivalent fluid pressure recommended in the Geotechnical Report but not less than 57 psf. At-rest pressure is recommended for design in most cases because wingwalls on non-square bridges may undergo a transverse deflection into the backfill during longitudinal bridge movements, which could increase the pressure above active level.

The wingwall analysis shall include a live load surcharge load per AASHTO 3.11.6.4, regardless of the presence of an approach slab. Do not include vehicular collision unless the barrier is attached to the top of the wingwall.

Due to equilibrium of fill pressures on each side of the wingwall, the Designer may ignore the earth pressure below a line that extends from a point 3 ft. below the top of the wingwall at the end of the wingwall to another point at the bottom of the wingwall at the back face of the abutment. For erosion along the outside of the wingwall, 3 ft. is an assumed depth. This trapezoidal loading condition applies to wingwall design only and is not to be used for foundation stability analyses. Refer to Example 8: Cantilever Wingwall Design Loads for sample calculations and equations.

11.3.7 Approach Slabs

Construct approach slabs to match the required roadway width and sidewalk approaches. When a guardrail transition is required, the Designer shall provide 6 in. between the outside face of the bridge rail and the inside face of the wingwall, refer to the Bridge Detail Manual. This clearance may be eliminated when no guardrail transition is required or when rail anchor slab is used.

If not using the details shown on the Staff Bridge Worksheets, design the approach slab per AASHTO. Limit post-construction settlement at the free end of the slab to 1 in. If the Geotechnical Engineer anticipates settlement greater than 1 in., the Designer shall incorporate plan details to mitigate the amount of settlement to 1 in. or less. One possible mitigation detail would be to raise the end of approach slab by the anticipated long-term settlement. For additional information on approach slabs, see Section 2.13 of this BDM and Staff Bridge Worksheets.

**AASHTO
Sections 3
& 5**

11.4 PIERS

Bridge piers provide intermediate support to the superstructure and a load path to the foundation. Suitable types of piers include, but are not limited to, the following:

- Solid Wall Piers
- Multi-Column (Frame) Piers
- Single Column (Hammerhead) Piers
- Straddle Bent Piers

**AASHTO
11.7.1**

Forces acting on the pier in the vertical, longitudinal, and transverse direction shall be per AASHTO. The connection between the superstructure and pier should be pinned by use of bearings or a key detail, allowing rotation in the longitudinal direction of the superstructure and eliminating longitudinal moment transfer to the substructure. Fixed or integral connections between the superstructure and substructure are not desirable. If the bridge is being designed with staged construction, each stage shall meet AASHTO.

The bearing cap should be a sufficient width and length to support the superstructure, meet support length requirements, and provide adequate bearings edge distances. A recommended pier width to depth ratio is less than or equal to 1.25. If the depth of the cap varies more than 18 in. from each end, slope the bottom of the cap. For precast prestressed concrete girder superstructure types, place the bearing lines a minimum of 12 in. normal to the centerline of cap. The minimum cap size shall be 3 ft. by 3 ft. and should increase thereafter by 3 in. increments. In section, the cap should overhang the column by 3 in. minimum. The length of the cap should not extend past the drip groove and should be rounded down to the nearest inch.

When designing the pier cap for negative moment, the preferred design plane is located at the face of the column or equivalent square for a round column.

To properly model the column / pier cap connection, provide a rigid link from the centerline column to the face of the column. If a rigid link is not provided, use the maximum moment at the centerline of column. See Section 5.4.11 of this BDM for pier cap reinforcing details.

When Strut & Tie Models are used for the design, they must be shared with the design checker to obtain concurrence on the models. Refer to Section 37.5 of this BDM for more details.

To ensure that the girder ends will rotate during the deck pour, the Designer shall add a note to the plans requiring the Contractor to pour the deck within two hours of the integral diaphragms.

Coordinate the selection of column type with the architect and CDOT. Possible column types include, but are not limited to, round, square, rectangular, tapered, and oblong. Standard forms should be used whenever possible and shall be 2 ft-6 in. minimum. To match standard form sizes, round, rectangular, and square columns should have length and width dimensions in 3 in. increments. When the columns are tall, place construction joints at approximately 30 ft. spacing. The preferred method of analysis for columns is moment magnification.

In lieu of moment magnification analysis, a second-order analysis is required. If magnification factors computed using AASHTO exceed about 1.4, then a second-order analysis will likely show significant benefits. The second-order analysis of the frame can be modeled using nonlinear finite element analysis software. AASHTO Seismic 4.11.5 discusses P- Δ effects and when they should be considered in the design.

Unless in a seismic zone as defined in Section 5.4.9 of this BDM or requested otherwise, tied hoops are preferred for transverse reinforcement, rather than spirals. The column spacing on framed piers should balance the dead load moments in the cap.

When setting the foundation location, the Designer shall provide 2 ft. minimum cover on top of the foundation element. To protect from frost heave, place the bottom of any footing below the frost depth indicated in the Geotechnical Report and no less than 3 ft. minimum below finished grade. The minimum depth of a footing on pile/drilled shafts and spread footings is 2 ft.-6 in. See Section 10.4.2 of this BDM for additional details.

When placing a pier in the floodplain, the Designer should align the pier with the 100-year flood flow. The preferred pier location is outside the floodplain whenever possible. To prevent drift buildup and when recommended by the Hydraulics Engineer, provide web walls between columns. The Designer shall consider the effects of uplift due to buoyancy forces when designing piers located in floodplains. Final pier locations should be coordinated with the Hydraulics Engineer.

When checking cracking, all caps and columns shall use Class 1 exposure condition. Foundation elements shall use Class 2 exposure condition.

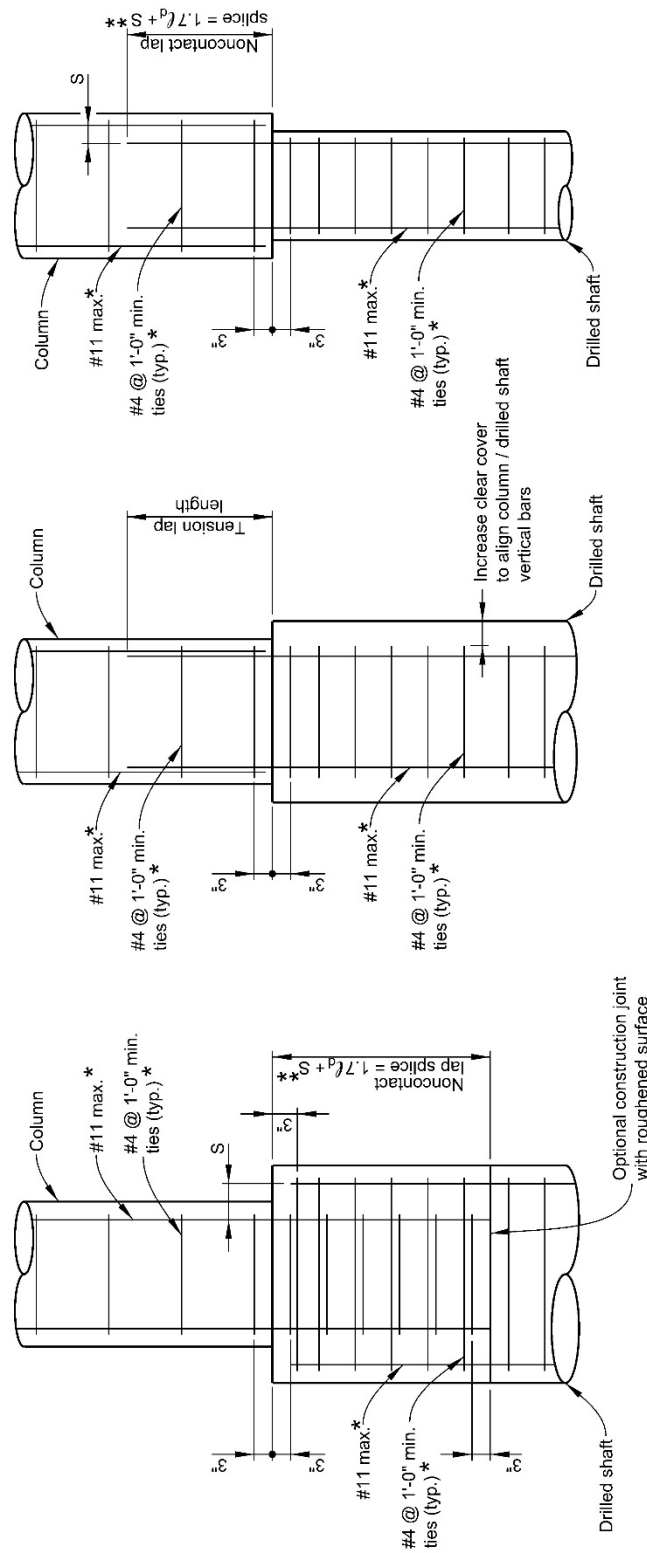
The Structure Selection Report shall document the selected pier type and its location for the project.

If the pier has bearings that may need future maintenance or replacement, the Designer should show jacking locations and loads on the drawings. CDOT Standard Specification 503.20 provides the following horizontal tolerances for drilled shaft construction:

- 3 in. for shafts with diameters less than or equal to 2 ft.
- 4 in. for shafts with diameters greater than 2 ft. and less than 5 ft.
- 6 in. for shafts with diameters 5 ft, or larger.

These construction tolerances must be accommodated in the pier cap design to prevent a need to adjust the pier cap location during construction. In situations where the column steel has a contact lap splice with projected drilled shaft reinforcing, the column is required to follow the drilled shaft if the drilled shaft is misaligned. Therefore, provide pier cap overhang (distance from the column to the face of the cap) equal to or greater than the construction tolerance above to allow column location adjustment while the pier cap remains in place.

Also provide adequate dimensional tolerance between the column and drilled shaft via a non-contact lap splice, either by oversizing the drilled shaft or by oversizing the column. The inside cage should be able to move laterally by the amount of specified allowed construction tolerance without compromising the design or details of the members.



UNDERSIZED DRILLED SHAFT

SLIGHTLY OVERSIZED DRILLED SHAFT

OVERSIZED DRILLED SHAFT

* Reinforcing steel determined by design

** Refer to AASHTO C5.11.5.2.1 for information.

Maximum bar spacing in the splice zone shall

meet AASHTO 5.11.5.2.1-1.

S - distance between caisson and column reinforcing.

Figure 11-13: Column-Drilled Shaft Connection Details

11.4.1 Multi-Column Piers

Multi-column piers, the most commonly used pier type, consist of two or more transversely spaced columns. This type of pier is designed as a frame about the transverse direction (strong axis of the pier). The columns are usually fixed at the base and supported by one of the following foundation types: spread footing, footing on pile/drilled shafts, or drilled shafts.

11.4.2 Single Column (Hammerhead) Piers

Single column (Hammerhead, Tee) piers are usually supported at the base by a drilled shaft, spread footing, or footing on pile/ drilled shafts. Either the pier cap can be pinned in the longitudinal direction to the pier diaphragm and the diaphragm poured monolithically with the superstructure or the pier cap can be poured integrally with the superstructure. The column cross section can be various shapes and can be either prismatic or flared to form to the pier cap.

It is recommended that hammerhead style piers be modeled using the strut-and-tie method. This method creates an internal truss system that transfers the load from the bearings through the cap to the columns. The truss uses a series of compressive concrete struts and tensile steel ties to transfer the loads. Place nodes at each loading and support point. The angle between truss members should be between 25° minimum and 65° maximum with a preferred angle of 45°. If a wide column is used, place two or more nodes at points along the column.

11.4.3 Solid Wall Piers

Design solid wall piers per AASHTO. Assume the top of pier wall to be pinned or free at the top. Support the bottom of wall on either a spread footing or footing on piles/drilled shafts.

AASHTO
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11.4.4 Straddle Bent Piers

Use straddle bent piers where there is a geometrical constraint in placing the piers. Such geometrical restrictions can be one or more of the following:

- Spanning a wide roadway
- Right-of-way (ROW) issues not permitting placing columns under the bridge
- Presence of railroad tracks to span over
- Presence of underground utilities where relocating them can be cost prohibitive
- Other

Straddle bent piers are non-redundant structures that can be conventionally reinforced, pre-tensioned or post-tensioned. Consider constructability, cost, span, and construction schedule when selecting the type of bent style.

Steel straddle bent caps are not permitted due to corrosion issues, inspection access concerns, fracture critical designation, high cost, and maintenance issues.

11.4.5 Aesthetics

Special corridor projects and signature bridges can have variations of the standard pier types or entirely unique pier designs. Coordination with Staff Bridge is essential at the preliminary phase of the project to determine the aesthetic requirements. The Structure Selection Report should document all aesthetic treatments required by the project.

11.4.6 Details

When a footing on pile is used, refer to Figure 11-14.

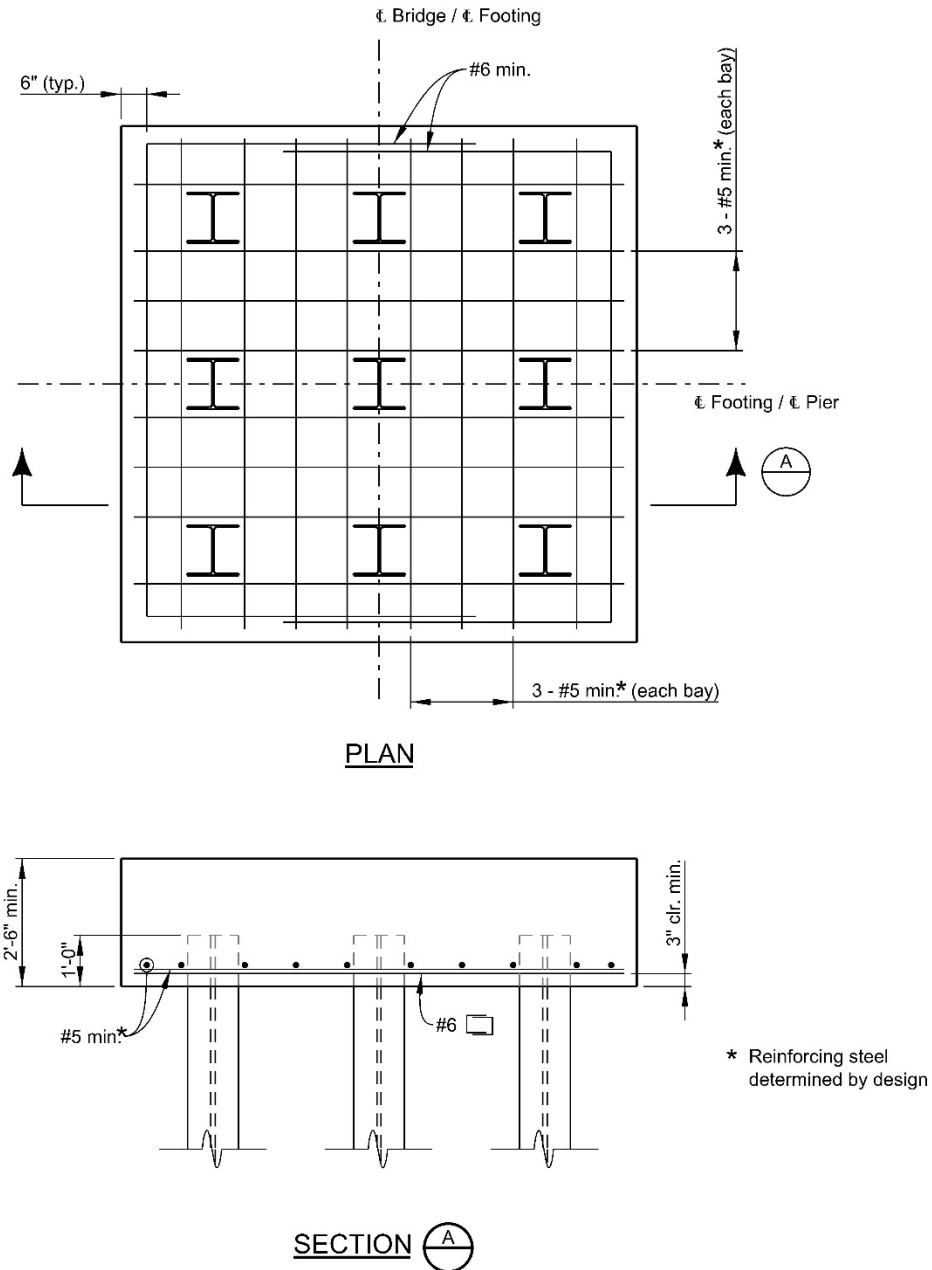


Figure 11-14: Footing on Pile

11.5 RETAINING WALLS

Design permanent retaining walls for a service life based on AASHTO. Design retaining walls for temporary applications for a service life of 3 years.

**AASHTO
Section 11**

Retaining walls can be classified into three categories according to their basic mechanisms of soil retention and source of support. Externally stabilized systems use a physical structure to retain the soil. Internally stabilized systems involve reinforcement (e.g., soil nails and geosynthetics) to support loads. The third system is a hybrid that combines elements of both externally and internally stabilized systems.

Calculate earth pressures in accordance with AASHTO. The Designer shall use Coulomb's earth pressure theory to determine the active coefficient of lateral earth pressure. The minimum equivalent fluid due to soil pressure shall be 36 pcf. If the wall design height is less than 4 ft. and a geotechnical report is not required or has not been provided, the Designer may assume a nominal soil bearing capacity of 6 ksf.

**AASHTO
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Settlement criteria will depend on the wall type and project constraints, such as nearby structures and the project schedule. The structural and geotechnical engineers should coordinate to select and design an appropriate wall system capable of meeting project requirements. For instance, the bearing resistance of wall footings will depend on the footing size.

Most walls that support vertical loads, unlike columns, do not require the 1% minimum longitudinal steel. When the vertical load becomes so great that buckling is a concern, walls should be treated like columns and meet compressive member requirements. A ratio of the clear height to the maximum plan dimensions of 2.5 may be used per AASHTO to differentiate between walls and columns (C5.11.4.1), but it should primarily be behaviorally based. Some references use b/d ratios of 3 to 6 to differentiate between walls and columns. See Section 11.4.3 of this BDM for more information on solid pier walls.

**AASHTO
5.11.4.1**

Provide weep holes or a drainage system behind the wall stem to prevent water accumulation. The Designer should reference Staff Bridge Worksheets for required size and spacing of weep holes or provide drainage system details in the project plans. The final drainage system selected will depend on the amount of water anticipated to infiltrate into the backfill and shall consider groundwater conditions.

Runoff shall not be permitted to pass freely over the wall; rather, a wall coping, drain system, or a properly designed ditch shall be used to carry runoff water along the wall to be properly deposited. Where this is not feasible, such as soil nail walls in steep terrain, the Designer shall coordinate with Staff Bridge to develop a solution that has concurrence from Region Maintenance and Bridge Asset Management.

When laying out walls, if possible, provide a 10 ft. inspection zone in front of the wall. The Designer must consider ROW limits for placement of the footings and if temporary easements are needed for excavation. Any wall footings, straps, soil anchors, or other wall elements shall be contained within the established ROW limits unless a permanent easement is obtained. The

Designer shall coordinate with the Roadway Engineer to determine final wall layouts and grading requirements.

The Wall Structure Selection Report shall be provided per Section 2.10.4 of this BDM. Appendix 11A contains worksheets to assist in developing wall selection options.

The following are the most common retaining walls used in Colorado:

11.5.1 Cantilever Retaining Wall

Cast-in-place and precast cantilever retaining wall systems are considered semi-gravity walls. Conventional cantilever walls consist of a concrete stem and a concrete footing, both of which are relatively thin and fully reinforced to resist the moment and shear to which they are subject. A cantilever wall foundation can be either a spread footing or a footing on deep foundations. Document the recommendation of the soil parameters and preferred foundation type in the Geotechnical Report and include in the plan set.

For retaining walls without concrete curb or barrier attached to the top of the wall, top of the wall shall be a minimum of 6 in. above the ground at the back face.

If a shear key is required to provide adequate sliding resistance, place it approximately one-third of the footing width from the heel to the centerline of the key. If additional depth for development length of the reinforcing is needed, it may be shifted to under the stem in lieu of increasing the footing thickness. Passive resistance shall be neglected in stability calculations and shall not be counted on for sliding resistance unless a shear key below frost depth is provided. Soil that may be removed due to future construction, erosion, or scour shall not be included in determining passive sliding resistance. The Designer shall, at a minimum, ignore the top 1 ft. of front face fill when determining sliding resistance. See Figure 11-15 for the passive resistance loading due to the shear key.

Protect retaining wall spread footings from frost heave by placing the bottom of the footing a minimum of 3 ft. below finished grade at front face. Top of footings shall have a minimum of 1.50 ft. of cover.

Sloped footings are permitted with a maximum slope of 10 percent.

Stepped footings may be used with maximum step of 4 ft.

Reinforcement should be as shown on Figure 11-16.

11.5.2 Counterfort Retaining Wall

Counterfort retaining walls, another type of semi-gravity wall, are an economical option for wall heights 25 ft. and taller. They are designed to carry loads in two directions. The horizontal earth pressure is carried laterally to the counterfort through the stem. The counterfort is a thickened portion that extends normal to the stem and is used to transfer the overturning loads directly to the foundation.

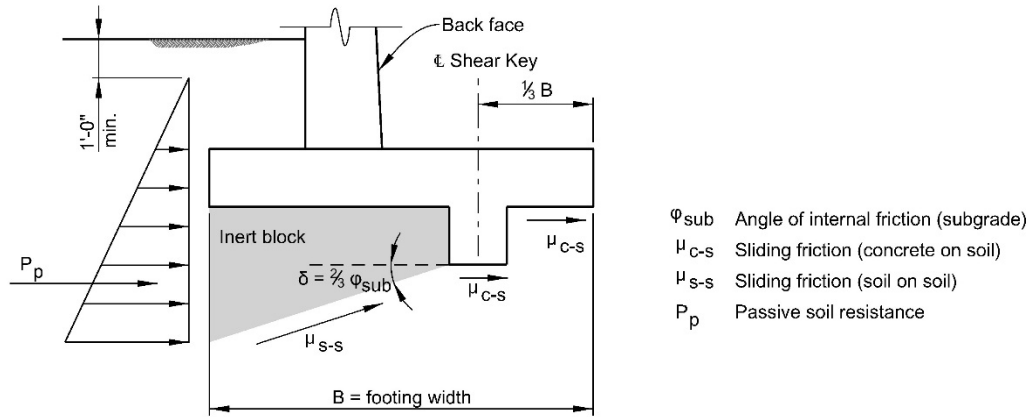


Figure 11-15: Shear Key

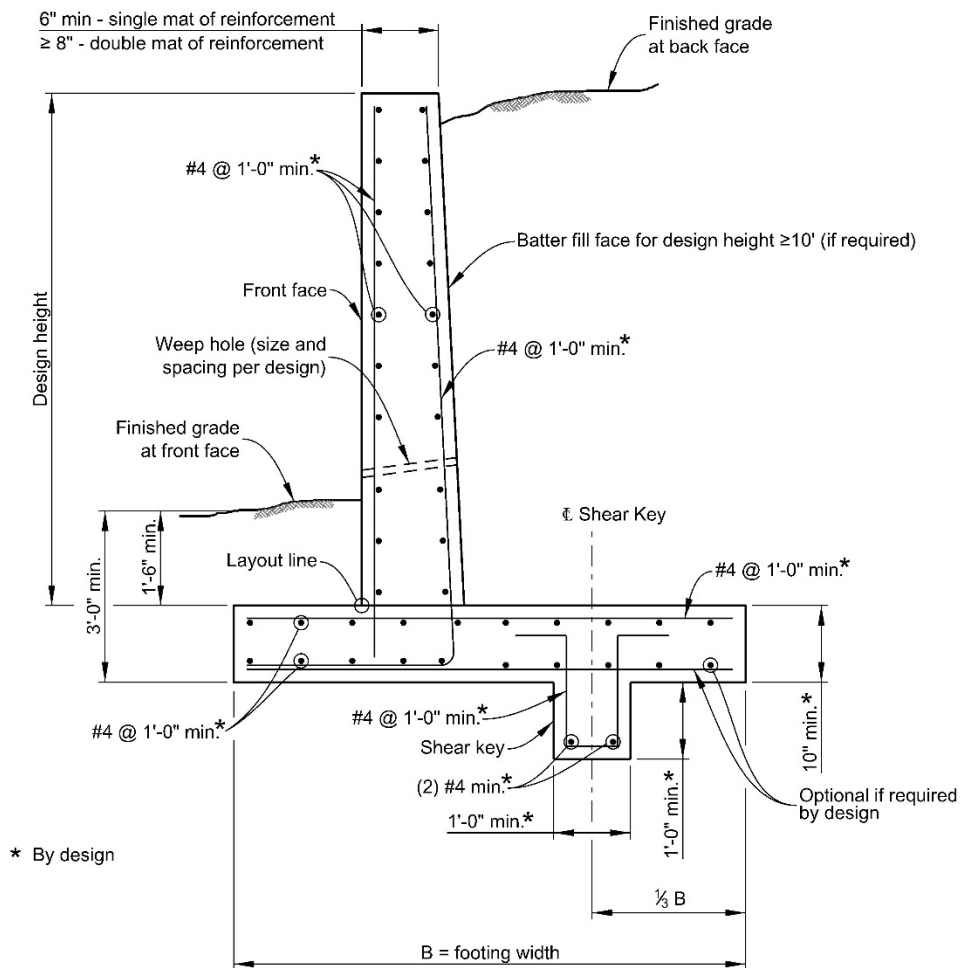


Figure 11-16: Cantilever Retaining Wall Reinforcement

11.5.3 Mechanically Stabilized Earth Wall

MSE walls, as detailed in the Staff Bridge Worksheets, are reinforced soil retaining wall systems that consist of vertical or near vertical facing panels or blocks, metallic or polymeric tensile soil reinforcement, and granular backfill. MSE walls are typically classified into one-stage and two-stage, where two-stage are used for large long-term settlements as outlined in Section 11.5.3.1 of this BDM. The strength and stability of MSE walls derive from the composite response due to the frictional interaction between the reinforcement and the granular fill. MSE systems can be classified according to the reinforcement geometry, stress transfer mechanism, reinforcement material, extensibility of the reinforcement material, and type of facing.

Sufficient ROW is required to install the reinforcing strips that extend into the backfill area 8 ft. minimum, 70 percent of the wall height or as per design requirements, whichever is greater. Truncated base or linearly varied reinforced zone per Staff Bridge Worksheets is allowed in cut conditions; they can be used when space constraint is a concern. Barrier curbs constructed over or in line with the front face of the wall shall have adequate room provided laterally between the back of the curb or slab and wall facing so that load is not directly transmitted to the top wall facing units. For more details, refer to Staff Bridge Worksheets B-504-V1.

**AASHTO
11.10.2.1**

For block walls and partial height panel facing walls, set the leveling pad a minimum of 18 in. from finished grade at front face to top of pad. When using full height panels, set them a minimum of 3 ft. below finished grade at front face to top of pad. If the front face fill is sloped in either direction, the Designer shall provide a 4 ft. minimum horizontal bench measured from the front face of facing. MSE structures are considered earth structures and are not subject to the minimum depth requirements for frost heave. The concrete leveling pad shall be reinforced along its entire length per the worksheet details.

For a retaining wall with a rail anchor slab placed at the top of the wall, allow a minimum 8 ft. wide (including rail), 20 ft. long monolithically constructed reinforced concrete barrier and slab system to carry and spread loads. See Example 12, Rail Anchor Slab Design, for additional information on the design of a rail anchor slab.

Attach a minimum 12 in. wide geotextile to the back face of all joints in facing panels to reduce the loss of backfill through the joints.

The Designer must be aware of the possibility of the presence of an abutment or other additional loads near the MSE wall affecting the design of the wall. It is the Designer's responsibility to determine if an MSE wall is in the influence zone of an abutment, thus adding surcharge loads per AASHTO 3.11.6.3, and to adjust the design accordingly per AASHTO 11.10 as required.

The Designer shall reference the Standard Special Provisions, Standard Specifications for Road and Bridge Construction, and Staff Bridge Worksheets for the most current design requirements and material properties required for design. The Staff Bridge Worksheets were created based on the AASHTO Simplified Method, which is CDOT's preferred method of design. Any other design method requires approval by the Unit Leader.

11.5.3.1 Two-Stage MSE Walls

One-stage MSE wall detail shown on Staff Bridge MSE Wall Worksheets can accommodate up to 1 in. of differential settlement between soil mass and the panels. If this limit is exceeded, the wall shall be evaluated for use of modified details or a two-stage MSE Wall. Geotechnical Engineer shall provide wall type recommendations for every project.

Two-stage MSE walls are constructed in two stages. During the first stage, the reinforced soil mass is constructed and left to settle until the remaining settlement is within the tolerances of the permanent facing. Settlement could be accelerated by installing wick drains, if necessary. The second stage is the installation of the permanent wall facing.

Other options to mitigate the long-term settlement, such as excavation and replacement of soil, deep foundations, and ground improvement, may be more expensive than a two-stage wall. In the Structure Selection Report, all alternatives should compare settlement mitigation, schedule, constructability, and cost.

11.5.3.2 Precast Concrete Panel Wall

MSE walls often use a fascia consisting of precast concrete panels. Full height or segmental panels based on the corridor architectural requirements are allowed.

Full height panel width is limited to 10 ft. and the height to 30 ft. The use of larger panel dimensions will require the approval of Unit Leader in coordination with the Wall SMEs and must be documented in the Structure Selection Report.

The segmental panel area is limited to a maximum of 50 sf. with a minimum panel height of 2.5 ft.

The segmental panel will tolerate more differential settlement than the full height panel.

11.5.3.3 Modular Block Wall

Block wall facing is made of various shapes and colors of concrete block units that will fit many architectural needs and has been specifically designed and manufactured for retaining wall application. Two types of blocks are available for use: dry cast and wet cast. Dry cast blocks have shown a propensity to degrade with age and exposure to weather and salts and can be difficult and expensive to repair. Wet cast blocks have been shown not to have many of these issues.

This type of retaining wall will tolerate greater differential settlement between the blocks than a segmental panel or full height panel.

Use of dry cast blocks in a wall is not a preferred option adjacent to a roadway due to challenges of repair in the event of vehicular collision, water intrusion, and deterioration from de-icing chemicals and therefore their use requires Unit

Leader approval. Dry cast blocks are an acceptable facing solution for landscape walls and around detention basins.

CDOT has experienced wall failures when using blocks in front of soil nail walls with inadequate block anchoring. To prevent future failures, the Designer shall apply the full earth pressure to the block anchorage connection.

11.5.3.4 Cast-in-Place and/or Shotcrete Facing

MSE walls can also have a cast-in-place (CIP) facing in front of the reinforced soil mass. The CIP facing can be either CIP and/or shotcrete concrete.

11.5.3.5 GRS Walls

This type of wall is generic (non-proprietary) and has a single grade of woven geotextile spaced at 8 in., including 4 ft. of tail soil reinforcement. Every modular block facing in a GRS wall is connected with a layer of soil reinforcement, Reinforcement-to-block connection mechanism is primarily based on friction and clamping action. Soil reinforcement-to-block pullout test is waived for this type of MSE wall; thus no soil reinforcement schedule or shop drawing submittal is required.

The design Engineer of Record shall thoroughly check internal, external, and global stability. The geotechnical report shall address temporary cut slope stability.

11.5.3.6 Truncated Base Walls

For a MSE wall within a cut condition, a truncated base soil reinforcing zone can provide an economical space constrained solution. The truncated base of trapezoidal soil reinforcing zone shall be 45 percent of design wall height or 4 ft., whichever is greater. The linearly varied soil reinforcement length and its maximum length at top depend on temporary cut slope stability. Use of this type of MSE wall is determined by geotechnical stability.

11.5.3.7 Collision on MSE Walls

MSE wall panels are considered sacrificial and do not require design for the vehicular collision force (CT), unless directed otherwise.

Current interpretation of AASHTO and federal design guidance for collision loads is shown on Staff Bridge Worksheets (B504H or B504O series). Depending on selected barrier type or moment slab use, the application of force may vary. Standard design practice will use a TL4 transverse design force of 80 kips. This force shall be distributed horizontally along the barrier and vertically into the resisting elements as appropriate. For rail slab applications, the 80 kip force is distributed along a 10.83 ft. length and linearly from a maximum to zero at a depth of 15.1 ft. When using a moment slab a factored impact load of 900 lbs is applied to each of the top two reinforcing layers. See Staff Bridge MSE Wall Worksheets for additional information. NCHRP 663 may be used as a reference for collision design.

11.5.4 Drilled Shaft Walls

Drilled shaft walls, also known as secant or tangent pile walls, consist of drilled shafts spaced along the wall alignment with an attached precast or CIP facing. They are typically used in areas where excavation limits are restricted due to ROW or there is an obstruction such as a building or utility. Micropiles can also be used when access is limited for drill rigs. The micropiles can be a single row or two rows with one battered to form an A-frame configuration.

11.5.5 Anchored Walls

Anchored walls (externally stabilized), although not routinely used in Colorado, may be appropriate for relatively high cuts or sites with stringent deformation criteria, particularly in situations where top-down construction is required. Anchored wall systems use ground anchors (e.g., tiebacks bonded into the ground, deadman anchors) to resist earth pressures acting on the wall. Anchored systems may include soldier pile and lagging, sheet pile, and drilled shaft walls.

The design of anchored walls should follow AASHTO.

**AASHTO
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11.5.6 Soil Nail Walls

Soil nail walls (internally stabilized) are frequently used as top-down, permanent retaining structures in Colorado. Soil nail walls are best suited to sites with adequate “stand-up” time, i.e., the ability of the soil to stand unsupported during wall construction.

**AASHTO
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The FHWA publication *Soil Nail Walls Reference Manual* (FHWA-NHI-14-007) provides guidance for the design of soil nail walls and is the recommended design manual for soil walls used on CDOT projects.

The Geotechnical Engineer shall be responsible for the entirety of the wall design, except for structural components such as the permanent facing, or as otherwise identified by the Geotechnical Engineer and shown in the Structure Selection Report.

When soil nail walls extend past the existing bridge abutment, future widenings need to be considered. To allow room for future pile installation, diamond patterns shall not be used within the ultimate configuration of the bridge (Figure 11-17).

Soil nail walls are typically designed with the assumption of dry soil conditions. For dry conditions, the typical soil nail bond strength is 10 to 15 psi with a maximum of 30 psi. However, for a high ground water table, spring water seepage, or heavy storm water runoff conditions, bond strength is reduced significantly. Without rigorous temporary drainage measures required during construction, wet condition bond strength must be considered and designed for by the Contractor's design Engineer of Record.

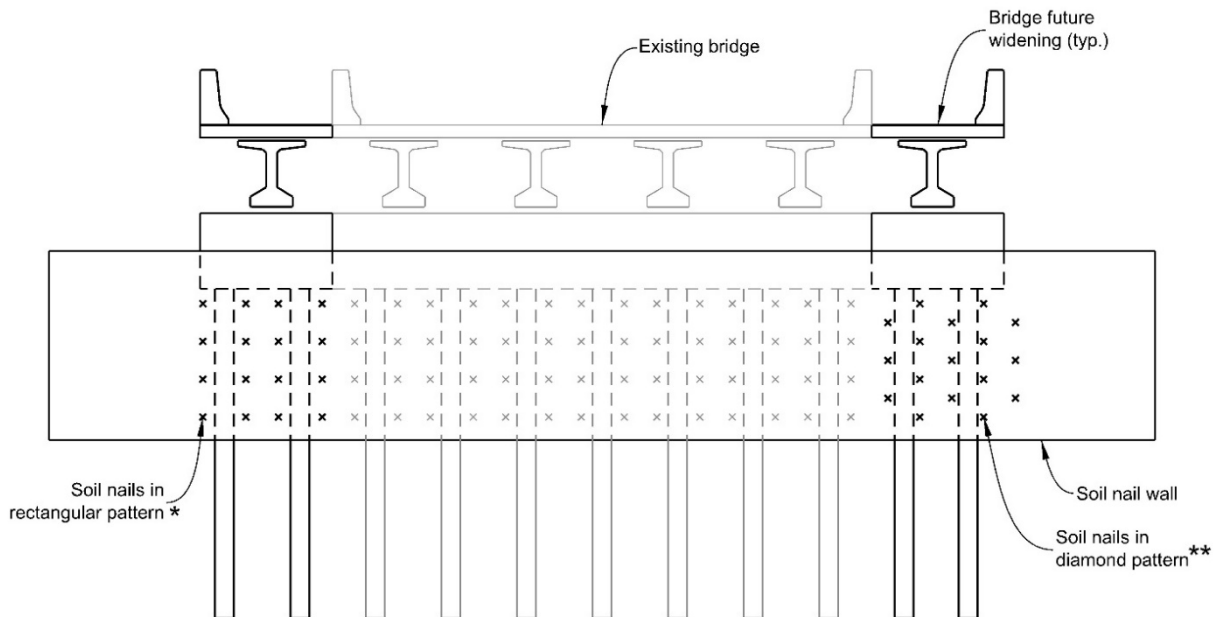
11.5.7 Gravity Walls

Rigid retaining walls of concrete or masonry stone that derive their capacity through the dead weight of their mass may be used for earth retention. Due to increases in material costs, conventional types of these walls made from

concrete or stone are expensive. More affordable gravity walls, such as gabion baskets, have become more prevalent and are easily constructible.

11.5.8 Landscape Walls

Landscape walls retain soil less than 4 ft. in height from the finished grade to the top of the wall at any point along the length of the wall.



* Rectangular pattern is preferred in areas of future widenings. Other patterns that provide sufficient clearance for future piles may be allowed.

** Do not use diamond pattern in areas of future widenings to avoid potential pile interference.

Figure 11-17: Soil Nail Wall in Future Bridge Widening Area

11.5.9 Load Combinations

Table 11-2 summarizes the load combinations used for wall design. Use Strength Ia and Extreme Event II to check sliding and overturning and to minimize resisting loads and maximize overturning loads. Use Strength Ib and Extreme Event II to check bearing and maximize loads for both overturning and resisting.

Note that live load surcharge (LS) and horizontal earth load (EH) are not included in the Extreme Event load case for vehicle collision load (CT). It can be assumed that the horizontal earth pressure is not activated due to the force of the collision deflecting the wall away from the soil mass at the instant of collision.

Use the service limit state for the crack control check.

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Table 11-2: Load Factors for Retaining Wall Design

Combination	γ_{DC}	γ_{EV}	γ_{LS_V}	γ_{LS_H}	γ_{EH}	γ_{CT}	Application
Strength Ia	0.90	1.00	–	1.75	1.50	–	Sliding, Eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	–	Bearing, Strength Design
Strength IV	1.50	1.35	–	–	1.50		Bearing
Extreme II	1.00	1.00	–	–	–	1.00	Sliding, Eccentricity, Bearing
Service I	1.00	1.00	1.00	1.00	1.00	–	Wall Crack Control

11.5.10 Resistance Factors

Resistance factors shall be per AASHTO or as given in the Geotechnical Report. Resistance factors for sliding and bearing are given in AASHTO Table 11.5.7-1. Resistance factors for passive pressure resistance are given in AASHTO Table 10.5.5.2.2-1. If an extreme event affects the wall, the resistance factors shall be per AASHTO 11.5.8.

**AASHTO
10.5, 11.5****11.5.11 Collision with a Wall**

AASHTO does not explicitly address how to design for collision load (CT) with a wall or how the load is distributed. Conservatively, CT shall be applied at the end of the wall unless the barrier does not extend to the end of the wall. Figure 11-18 provides an example of the distribution. Assume that the horizontal earth pressure is not activated due to the force of the collision deflecting the wall away from the soil mass at the instant of collision.

For a Type 9 barrier, assume that the total lateral distribution will extend horizontally for 3.5 ft. and then downward at 45° from the point of collision. The length of distribution from impact force, $L_t = 3.5$ ft., for a TL4 rated barrier is taken from AASHTO LRFD Table A13.2-1.

For collision with a Type 10 barrier (post and rail), distribute CT horizontally between posts (3 maximum) and down from top of curb/wall to bottom of footing at 45°. At the end of a wall, assume a horizontal distribution distance from the edge distance to the first post plus one bay and then down at

45 percent.

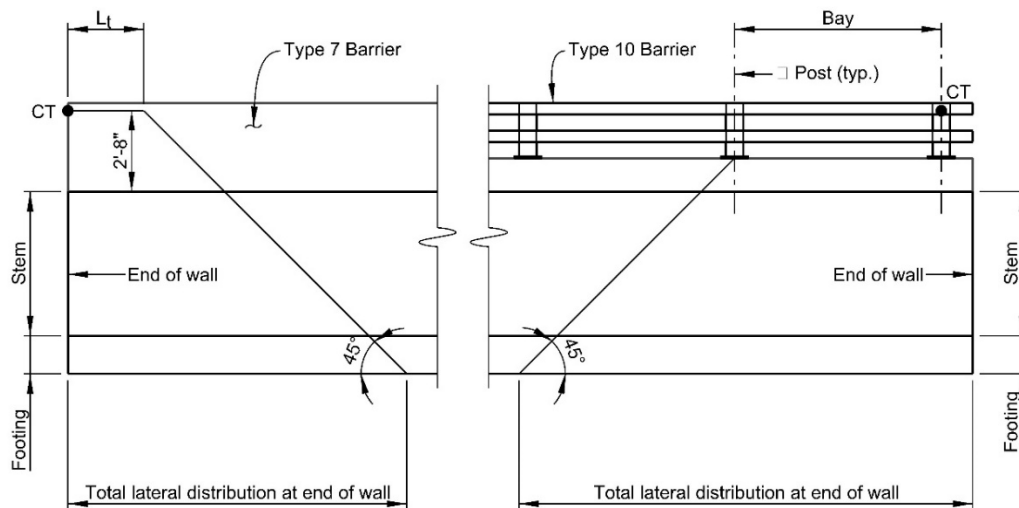


Figure 11-18: Lateral Collision Distribution

The previously described method is fairly conservative and does not always correlate with reality well since it assumes that reinforcing is similar vertically and horizontally. Walls with barrier on top should generally be designed using Chapter 13 of the AASHTO code as a very tall parapet which makes L_u a function of the relative strength vertically and transversely. For barrier with steel posts, the transferred load should be based on the capacity of the post as the impact is typically shared between 3 and 6 posts.

The findings of NCHRP 22-20(2) and report 663 may be used to determine equivalent static forces for sliding and overturning stability on MSE walls. For CIP walls the load may be reduced as the section of interest goes below the riding surface due to the increased mass and reaction time as more of the wall is involved. These values can be tentatively used as 100% at the ground line, 33% at 6' below the load application and 0% at 9' below the load application. These values are extrapolated from the data in 22-20(2) with the "6' below" percentage reflecting the results for sleeper significant movement at the back of the sleeper for TL3 and 4 crashes. The value of the 0% at 9' reflects the depth at no movement for TL-5 crashes.

To mitigate the effects of live load collision with CIP or precast face panels, for all walls that use face panels (e.g., caisson walls, soil nail walls, MSE Walls), a void between soil mass (or caissons) and back face of the panel shall be filled with granular material to the minimum height of 5 ft. above the roadway surface. Wall panels shall be required to support their own weight in case of impact damage that would allow the panel to slip below precast copings or clip angles. Reinforcing spacing should be minimized on panels to limit projectile size when impacted (6 in. max spacing). Welded wired fabric may be used in addition to reinforcing to minimize projectile size.

11.5.12 Global and Compound Stability

The global stability and compound stability shall be per AASHTO and the Geotechnical Design Manual. Global stability of the wall depends on the footing width and embedment.

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The project Geotechnical Engineer shall evaluate global stability. Minimum factors of safety for global stability shall meet the requirements of the Geotechnical Design Manual and AASHTO. The Geotechnical Engineer shall specify the minimum requirements to achieve the specified factors of safety (e.g., minimum reinforced zone length for MSE walls, minimum soil nail length, and configuration for soil nail walls).

Compound stability of MSE and soil nail walls will depend on the reinforcement type, length, and spacing. Therefore, the vendor is responsible for checking compound stability based on their submittal (see Figure 11-19).

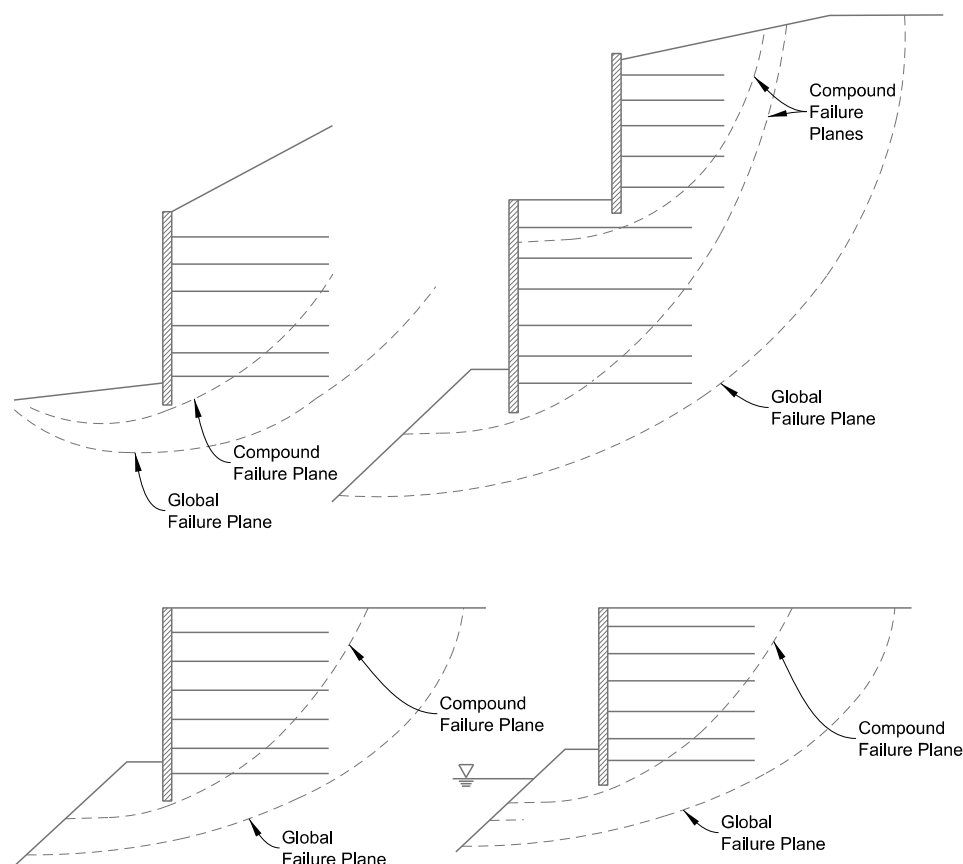


Figure 11-19: Global and Compound Failure Planes

11.5.13 Designer Responsibility for Walls

External stability addresses concerns with the stability of sliding masses defined by slip surfaces that pass outside the reinforced soil zone. The checks required include global stability of the structure, determination of eccentricity limits, sliding analysis, bearing capacity analysis of the foundation/supporting soils, and settlement analysis. These checks shall be performed by the

**AASHTO
11.6.2.3**

Engineer of Record responsible for the design, whether that be the owner's representative, Geotechnical Engineer, Structural Engineer, or Vendor. If the wall is a vendor design, the vendor's Independent Design Engineer is responsible for submitting stamped calculations showing the external stability check for review. All walls are to be designed and built according to Standard Specifications Subsection 504.

Internal stability typically includes both pullout and rupture of the reinforcement. Responsibility for this check includes wall system components, including facing units, soil reinforcements, structural attachments, reinforcement connections to the facing units, bearing pads, and joint covering filter fabrics. Design responsibility shall fall on the engineer responsible for the design, whether that be the owner's representative, Geotechnical Engineer, Structural Engineer, or Vendor.

Global stability, compound stability, and deep seated failure conditions are closely related to external stability checks. It can be defined as the overall stability of the wall and surrounding slopes and structures. It requires the analysis of the surrounding circular slip surfaces. See Section 11.5.12 of this BDM for global stability requirements.

The Project Engineer of Record is responsible for collecting and reviewing wall submittals, which can include, but are not limited to, stamped calculations, shop drawings, etc. During the shop review process, bearing pressure, strap length and other minimum requirements from the worksheets shall be reviewed. Separate contractor designs are required when not meeting minimum requirements of the worksheets.

11.5.14 Designer Responsibility for Using MSE Wall Worksheets

CDOT MSE wall worksheets contain details such as vertical slip joints, coping, leveling pad, end of wall treatment, waterproofing membrane with drainage, and damage avoidance measures for improving wall and seismic performance. These worksheet details are CDOT minimum requirements consistent with MSE/GRS wall design criteria and policies. The Designer may provide alternatives for approval by Unit Leader to some of these details as identified in the worksheets and in Section 2.16.

Internal, external, and compounded stability are checked based on assumed soil reinforcement shown on the wall worksheets. In addition to using the worksheets, the Project Structural Engineer shall be responsible for site geometry, soil conditions, slope stability checks, and construction sequencing. For a GRS wall with only one specific grade of geotextile with a fixed spacing, the Contractor's selected supplier is only required to meet material certification and shop drawings are not required.

Alternate contracting methods may alter Designer responsibility on a project-specific basis. For example, if a project requires a complete MSE design proposal by a Contractor or appropriate Subcontractor, the Contractor is responsible for all elements of design, including reinforcement grade and placing schedule, and will provide in stamped shop drawings. The aforementioned damage avoidance details still apply.

11.5.15 Designer Responsibility for Using Soil Nail Wall Worksheets

Soil nail walls can either be designed by an in-house or consultant designer in a Design-Bid-Build situation or provided using more of a "Design Build" approach where the Contractor will design the wall based on project requirements. The soil nail worksheets provide generic details for construction as well a project example set. CDOT Geotechnical will typically design in-house soil nail walls in coordination with Staff Bridge. At a minimum, the Designer will provide the required wall alignment and determine the required project requirements. Designer shall show proposed locations of verification tests. Where geotechnical report shows varying strata or for very long walls, more than the minimum of 2 tests should be shown.

11.5.16 Seismic Design Requirements

Seismic analysis for retaining walls is not required unless they are supporting a bridge abutment or liquefaction that will affect the foundation performance is anticipated. Section 3.13 of this BDM provides additional information on seismic design requirements. Current Staff Bridge Worksheets for MSE walls use details for improved seismic performance, thus, if the worksheets are used, AASHTO 11.10.7.4 can be waived.

AASHTO
3.10.9.2,
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11.6 DRAINAGE REQUIREMENTS

Backfill material behind abutments and retaining walls shall be well drained and not allow water to collect. If this cannot be accomplished, the abutment and retaining walls should be designed for loads due to earth pressure plus hydraulic pressure due to water in the backfill. Class 1 backfill can have up to 20 percent fines and thus may not be classified as free draining. Design a drain system if using a Class 1 backfill.

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11.9.9,
11.10.8

If the wall or abutment includes conditions or areas that promote the trapping or intrusion of water, such as low point on a sag curve or a drainage inlet, the Designer shall create details to address the issues that may occur. The approach slab drain details used shall allow movement of the abutment while noting that the approach slab drain does not move. Add water sealers, waterproofing membranes, and protection details to the plans.

11.7 SHORING

Shoring is generally not designed by the EOR, but shall be designated in the plans and indicate which shoring areas will require an independent review. Areas that typically need review are those areas that support the roadway or could cause a safety issue.

11.8 REFERENCES

FHWA, 2012, Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide, Publication No. FHWA-HRT-11-026.

FHWA, 2015, Soil Nail Walls Reference Manual, Publication No. FHWA-NHI-14-007.

NCHRP, Report 663: Design of Roadside Barrier Systems Placed on MSE Retaining Walls.

APPENDIX 11A - WORKSHEETS FOR EARTH RETAINING WALL TYPE SELECTION

NOTES ON USING WORKSHEETS

1. Factors that can be evaluated in percentage of wall height:
 - Base dimension of spread footing.
 - Embedded depth of wall element into firm ground.

2. Factors that can be described as 'large (high)', 'medium (average)', or 'small (low)':

Quantitative Measurement
 - amount of excavation behind wall.
 - required working space during construction.
 - quantity of backfill material.
 - effort of compaction and control.
 - length of construction time.
 - cost of maintenance.
 - cost of increasing durability.
 - labor usage.
 - lateral movement of retained soil.
Sensitive Measurement:
 - bearing capacity.
 - differential settlement.

3. Factors that can be appraised with 'yes', 'no' or 'question (insufficient information)'
 - Front face battering.
 - Trapezoidal wall back.
 - Using marginal backfill material.
 - Unstable slope.
 - High water table/seepage.
 - Facing as load carrying element.
 - Active (minimal) lateral earth pressure condition.
 - Construction dependant loads.
 - Project scale.
 - Noise/water pollution.
 - Available standard designs.
 - Facing cost.
 - Durability.

4. Factors that can be approximated from recorded height:
 - Maximum wall height.
 - Economical wall height

SECTION 12 BURIED STRUCTURES AND TUNNEL LINERS

12.1 GENERAL REQUIREMENTS

This section covers the design of buried structures, including, but not limited to, precast and cast-in-place concrete box culverts, wildlife crossings, tunnels, and pipes.

12.2 CODE REQUIREMENTS

Design shall be in accordance with AASHTO, unless modified herein.

Chapter 9 of the Drainage Design Manual shall be referenced for buried structures that convey water.

12.3 GEOTECHNICAL REQUIREMENTS

All major structures, as defined in Part D of the Policies & Procedures section of this BDM, require a geotechnical analysis. Minor structures may require a geotechnical investigation when issues such as thrust blocks, large settlement, and deep foundations affect the design.

12.4 CONCRETE BOX CULVERTS

12.4.1 Design Criteria

CIP and precast concrete box culverts (CBCs) and wingwalls shall be designed according to the applicable M-Standard drawings and design criteria. Designs not meeting the standard sizes, loadings, or conditions provided in the M-Standard drawings require a site-specific design. Site-specific design is generally required for non-standard box culvert spans or heights, CBCs with top and/or bottom corner chamfers, live load surcharge greater than 2 ft., fill heights or wearing surface thickness greater than those listed in the M-Standards, wingwalls subject to live load surcharge, and headwalls subject to live load impact, including transfer of live load impact into the top slab. A site-specific design may provide significant cost savings, especially in the case of long CBCs due to the conservative nature of the assumptions used to develop the design tables.

Site-specific designs shall follow AASHTO standards and design criteria listed in this section. Wingwalls shall be monolithic and rigidly connected to concrete headwalls to reduce the possibility of differential movement. The design of M standard M-601-20 is based on this assumption of 2 way action. In the event that a non-rigid connection is allowed by the unit leader, an independent wingwall design and check is required.

The limits of a CBC should be kept within CDOT right-of-way (ROW). If the end of a culvert extends beyond the ROW, the Engineer shall provide inspection access from within the ROW.

12.4.2 Loading

When designing non-standard CBCs, live load is applied as follows:

- For design of culvert walls and bottom slabs, only the design lane load is applied.
- For design of culvert top slabs, only axle loads of the design truck or design tandem are applied.

Apply live loads to both earth pressure cases shown in the M-Standard and as described in AASHTO 3.11.7 and AASHTO C3.11.7. Note that, due to the 50 percent reduction in earth pressure, the minimum load factor need not be applied to the 30 lb/ft³ horizontal earth pressure load case. Live load distribution for various earth fills shall be per AASHTO 12.11.2.1. For CBC designs using either M-Standard or site-specific method, the controlling fill heights shall be shown on the plans. Designer shall consider live load on travel lanes and shoulders and any future roadway expansion.

It is preferred that bottom slabs for non-standard designs be modeled as rigid, not using soil springs, unless significant benefits can be demonstrated.

Thrust (axial compression) shall be assumed to be zero for design of CIP and precast culvert top and bottom slabs. Applying thrust forces is inadvisable when designing non-standard CBCs unless significant benefits can be demonstrated. This criterion is consistent with CBC M-Standard and AASHTO BrR rating software design methodology, and is conservative due to unpredictable on-site foundation conditions and preparations. The Engineer may consider the benefits of thrust forces in non-standard designs but shall discuss its use in the Structure Selection Report and obtain approval from Unit Leader in coordination with the Staff Bridge Manager of Policy and Standards.

12.4.3 Replacement

Existing culverts under consideration for replacement, extension, or other modifications shall be assessed as part of the Structure Selection Report. A culvert that shows no visible distress but yields an operating rating factor less than 1.0 when rated in accordance with the Bridge Rating Manual is not necessarily a candidate for replacement; refer to Section 33 – Rehabilitation of Structures of this BDM for additional information. Considerations for keeping an existing culvert include the age and condition of the existing culvert compared with the constructability and economy of a proposed replacement.

12.4.4 Stream Crossing

When designing non-standard CBCs, the Engineer shall consider both the presence and absence of water to determine controlling force effects acting on a CBC. Design water levels shall be in accordance with the maximum headwater to depth ratios provided in Table 9.3 in the Drainage Design Manual, unless otherwise directed by a Hydraulic Engineer. While required for design, water loads may be excluded when performing load ratings.

If requested or recommended by CDOT Environmental, water slowing devices may be required to assist upstream fish passage through culverts. Concrete aprons shall be provided as recommended by a Hydraulic Engineer.

12.4.5 Pedestrian Crossing

Pedestrian underpasses shall be designed to remain dry by providing waterproofing etc. and provide a clear line of sight through the underpass. Precast and steel arch structures shall use seals between joints to prevent water leaks. The Engineer shall reference the CDOT Lighting Design Guide for lighting requirements. CDOT shall approve lighting plans designed by a qualified lighting designer. The minimum opening provided for pedestrian crossings and equestrian paths shall be 10 ft. high by 10 ft. wide.

12.5 WILDLIFE CROSSINGS

Open-span bridges and overpasses are CDOT's preferred structure types for wildlife crossings, followed by arch structures and, lastly, CBCs. For guidance, refer to Wildlife Crossing Structure Handbook - Design and Evaluation in North America (FHWA-CFL/TD-11-003, March 2011) and Safe Passage: Developing Effective Highway Crossings for Carnivores and Other Wildlife by Bill Ruediger (USDA Forest Service, 2007).

The Engineer shall coordinate with CDOT Environmental for guidance on sizing Wildlife crossings. Underpasses designed for deer and elk should be a minimum of 13 ft. high by 23 ft. wide (see Figure 12-1), but preferred dimensions may be greater than those minimums. Coordinate with Environmental group for project specific requirements. If wildlife underpass openings do not meet the minimum dimensions shown in Figure 12-1, a variance is required unless approved by the Environmental group and coordination is documented. All crossings shall provide a line of sight through the structure, and the structure invert elevation shall be below existing grade to maintain the natural path.

For overpasses, the Engineer shall coordinate with CDOT Environmental for guidance on determining the loads for which the overpass has to be designed. The Engineer shall define all overpass design loads, such as animal live load, snow loads, soil, landscaping materials, maintenance vehicles, impact loads on side walls, etc. This design information shall be included in the drawings. All wildlife overpasses going over state-owned highways or roads shall be rated for an H10 vehicle.

Wingwalls for arch structures shall be monolithic and rigidly connected to concrete headwalls. Steel arch structures shall be protected by waterproofing membrane.

The design and layout of wildlife crossings shall include 8 ft. high game fencing and escape ramps at a 3H:1V slope. Game fencing shall be installed between the structure and roadway, rather than terminated at the wingwalls. Note that nonstructural items, such as fencing, are typically the responsibility of the Roadway design team and are included in Roadway bid items.

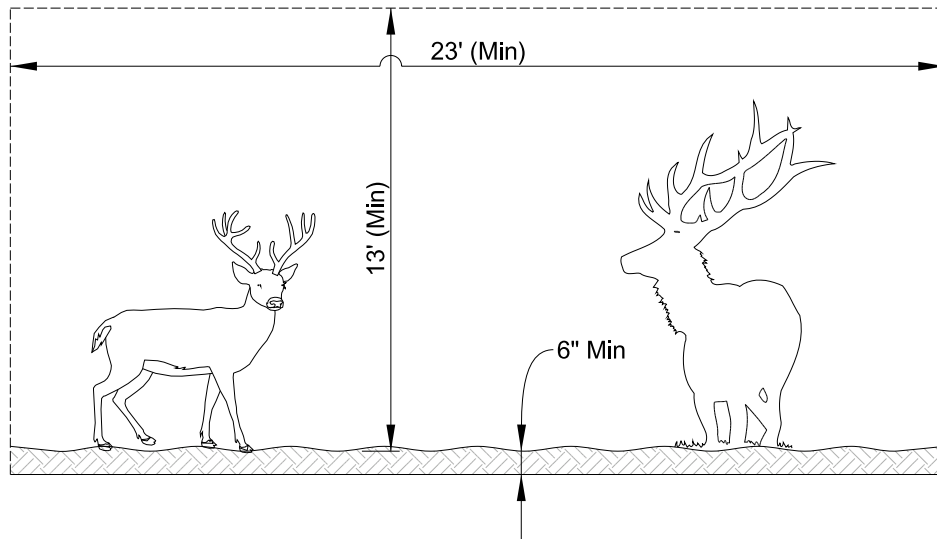


Figure 12-1: Minimum Deer and Elk Underpass Design Dimensions

12.6 TUNNELS

For tunnel design criteria, refer to AASHTO, Technical Manual for Design and Construction of Roadway Tunnels – Civil Elements (FHWA-NHI-10-034), NFPA 502: Standard for Road Tunnels, Bridges and Other Limited Access Highway and AASHTO LRFD Road Tunnel Design and Construction (LRFDTUN).

12.7 PIPES

For design of metal pipe, reinforced concrete pipe, corrugated polyethylene pipe, PVC pipe, metal pipe arches, pipe headwalls and outlet paving, and concrete and metal end sections, refer to the M-Standards.

SECTION 13 RAILINGS

13.1 GENERAL REQUIREMENTS

This section will provide guidance on the selection, design, and construction requirements for bridge railing. For pedestrian, bicycle, and safety railing requirements, refer to Section 2.4 of this BDM and to AASHTO.

Traffic railings provide protection at the edges of traffic and pedestrian structures and in median areas to prevent crossover collisions. In achieving this function, the railing must have the strength to withstand the vehicular impact and safely contain and redirect vehicles without snagging or overturning.

CDOT Bridge Rail Type 9 and Type 10 MASH are provided by CDOT to meet MASH 2016 Test Level 4 (TL-4) and in some cases TL-5 requirements. They are to be used on all new and widened bridges, box culverts, and retaining walls. M-Standard options such as inside mounted MGS rail on CBCs or MGS rail a minimum of 3 feet away from wall faces are acceptable options but only provide TL-3 protection. Other available or retired Colorado railing systems shall be used only with approval from the State Bridge Engineer in coordination with the bridge rail SMEs.

Any other proposed railing system shall be documented to be MASH compliant by meeting the full-scale crash test criteria established in the most current *AASHTO Manual for Assessing Safety Hardware* (MASH), professional evaluation by a crash test facility, comparative analysis to other crash test rails, non-linear time dependent FEM analysis and/or analysis through AASHTO LRFD Appendix A13 with the latest loads from crash testing research and experience. The previous list is in order of preference. Due to the complicated nature of crash and impact mechanics, the best evaluation of a bridge rail system is a physical crash test. Transitions should be evaluated using the same criteria.

AASHTO defines TL-4 as “taken to be generally acceptable for the majority of applications on high speed highways, freeways, expressways and Interstate highways with mixture of trucks and heavy vehicles.” For local agency projects a test level lower than TL-4 may be accepted by CDOT Staff Bridge based on design speed, ADT and other factors but any rail systems and transitions shall still be MASH compliant for the selected test level. A variance/risk recognition letter will be required from the local agency as well as attesting data supporting the bridge rail selected (evaluation factors, test level etc.).

MASH testing involves utilizing vehicles with characteristics similar to, or more critical than, 85% to 95% of vehicles of the type and speeds and angles of incidence similar to, or more critical than, 85% to 95% of road departure incidences. Actual crash conditions are at least partially random and chaotic in behavior so design and testing does not assure benign behavior during all crash incidents. Bridge rail design and evaluation should:

- Use best available methods (MASH criteria, current research, similarity with Crash tested rails, professional evaluations, non-linear time dependent FEM analysis, AASHTO Chapter 13).
- Use redundancy to protect from the unknown or occurrences beyond assumptions. Prevent progressive collapse under realistic but rare loads.
- Promote continuity in rail system
- Avoid obvious snag points or spearing issues
- Avoid transitions with markedly different stiffness

CDOT will monitor in-service behavior of rail types to identify flaws in design or operation. Most fatalities associated with rail are due to rollovers or “bouncing” out into thru traffic for another collision (vehicle interaction). Deflection of a rail system reduces occupant injury potential and the tendency for the vehicle to be thrown back into traffic, if without penetration or “pocketing”. Looking at old bridge rails, abutments, and piers that previously lacked motorist protection you will occasionally come across little crosses painted or scratched into them. There is a reason that bridge rail ends were called tombstones. CDOT will make any crash test results and evaluations of current bridge rails available upon request.

13.2 CODE REQUIREMENTS

The design of the railings shall be in accordance with AASHTO and MASH criteria and follow current Staff Bridge Worksheets, when applicable.

13.2.1 AASHTO LRFD

Bridge railing test levels and crash criteria shall be in accordance with AASHTO and MASH. The minimum test level shall be TL-4 for all new bridges, culverts, and retaining walls except as described in Section 13.1.

AASHTO
13.7.2,
Table 13.7.2-1

Railing design, including, but not limited to, height of traffic barrier or railing, bicycle railing, pedestrian railing, and design live loads for pedestrian railings, shall adhere to AASHTO and MASH evaluation criteria.

AASHTO
Section 13

Railing geometry and anchorages shall be in accordance with AASHTO and MASH.

AASHTO
Appendix A13

Traffic railing design forces for concrete railing and post and beam railing shall follow AASHTO and MASH criteria.

AASHTO
A13.2,
A13.3.1,
A13.3.2

Design calculations are not required to be performed for Type 9 and Type 10 MASH bridge railings, provided they are not modified to affect performance from the worksheet details.

13.2.2 AASHTO Manual for Assessing Safety Hardware (MASH)

MASH is the state of practice for crash testing of safety hardware devices for use on the NHS. It updates and replaces NCHRP Report 350 *Recommended Procedures for the Safety Performance Evaluation of Highway Features*.

MASH presents uniform guidelines for crash testing permanent and temporary highway safety features and recommends evaluation criteria to assess test results.

- All new testing will follow MASH evaluation techniques.
- Guardrail hardware shall meet MASH requirements for replacement and new installation.
- All new products must be tested using MASH crash test criteria for use on the NHS.

MASH loads and evaluation are based on the most recent research and crash testing and is typically more up to date than the AASHTO Chapter 13 analysis. Below are the loads that should be used in any evaluation or analysis of existing or new rails. New rails designed for TL-2 should have a minimum height of 24". Although there are a few crash tested TL-2 bridge rails with 18" height, 24" minimum is required. These will require a variance.

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F _t (kip)	F _L (kip)	F _v (kip)	L _t /L _L (ft)	L _v (ft)	H _e (in)	H _{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

13.2.3 FHWA Bridge Rail Requirements

FHWA mandated all new bridges carrying traffic on the NHS to have crash tested railing in accordance with MASH 2016

All projects on the NHS after December 31, 2019, shall be at least TL4 MASH compliant bridge rail systems per CDOT requirements.

Existing bridge rails not meeting the above FHWA mandate should be evaluated based on site and traffic conditions and the condition of the existing railing. Rails that are too short or too weak for the appropriate TL level should

be replaced. For additional information about evaluation and rehabilitation of existing bridge rail, refer to Section 2.4.1.1 of this BDM.

13.3 CDOT BRIDGE RAILS

The region typically selects the rail type, which shall be documented in the Structure Selection Report. Corridor requirements, aesthetics, hydraulics, environmental concerns, maintenance, snow removal, and railroad crossings shall be used in the selection. The use of weathering steel is not allowed for bridge railing. Galvanizing of steel portions of a bridge rail is the minimum standard required. In cases where the steel portion of the bridge rail is to be painted for aesthetic or other reasons, it must be done in addition to galvanizing using a duplex coating system. Details for Rail Type 9 and 10MASH can be found in the Staff Bridge Worksheets and Section 2.4 of this BDM. Bridge Rail Types 3, 4, 7, 8 and 10 have been retired but are prevalent on CDOT's roadways and should be used only for rehabilitation of the existing railing. Details can be obtained from Staff Bridge if not on the website. The following railings are available for use.

13.3.1 Type 3 (Retired)

Bridge Rail Type 3 is composed of continuous steel W shape attached to steel posts. The posts can be mounted on a bridge deck, a concrete box culvert top slab or headwall, or the top of a retaining wall. It should be used only for a railing repair of an existing bridge that has Type 3 on it. This railing shall not be used on CDOT structures without prior approval from State Bridge Engineer in coordination with the Bridge Rail SMEs. Due to primarily height issues, Type 3 bridge rail is most likely TL-2 or below and may need to be replaced or modified as safety funding allows.

13.3.2 Type 4 (Retired)

Bridge Rail Type 4 is a reinforced concrete barrier with a sloped front face. This type of barrier is not allowed for use on new bridges or as part of rail replacement rehabilitation projects. This rail, however, remains in service on several existing bridges and may require repair if damaged. Details for Type 4 barrier are not in the Staff Bridge Worksheets but can be obtained from Staff Bridge upon request. Due to primarily height issues, Type 4 bridge rail is most likely TL-3 or below and may need to be replaced or modified as safety funding allows.

13.3.3 Type 7 (Retired)

Bridge Rail Type 7 (F-shape) is a reinforced concrete barrier with a sloped front face. This type of barrier is no longer allowed for use on new bridges or as part of rail replacement rehabilitation projects. This rail, however, remains in service on several existing bridges and may require repair if damaged. Details for Type 7 barrier are not in the Staff Bridge Worksheets but can be obtained from Staff Bridge upon request. Due to primarily height issues, Type 7 bridge rail is TL-3 or below and may need to be replaced or modified as safety funding allows.

13.3.4 Type 8 (Retired)

Bridge Rail Type 8 is composed of a continuous horizontal steel tube attached to steel tube posts. The posts are mounted on a reinforced concrete curb anchored to the bridge deck. Use of this railing originated during the construction of the I-70 corridor through Glenwood Canyon. For aesthetic reasons, use of this rail may be allowed for repairs. This railing shall not be used on new CDOT structures without prior approval from State Bridge Engineer in coordination with the Bridge Rail SMEs. It is classified as a TL-2 railing by NCHRP 350 although a recent crash test article passed a TL-3 crash test. Due to primarily height issues, Type 8 bridge rail is TL-3 or below and may need to be replaced or modified as safety funding allows.

13.3.5 Type 8R MASH

Bridge Rail Type 8R MASH is composed of a continuous horizontal steel tube attached to steel tube posts. The posts are mounted on an existing reinforced concrete curb anchored to the bridge deck. This rail is intended for retrofits or rehabilitation of existing Type 8 rail (primarily in Glenwood Canyon) and is crash tested to a TL-3 level.

13.3.6 Type 10 (Retired)

Bridge Rail Type 10 is composed of two continuous horizontal steel tubes attached to steel W shape posts on top of a concrete curb. This type of barrier is no longer allowed for use on new bridges or as part of rail replacement rehabilitation projects. This rail, however, remains in service on several existing bridges and may require repair if damaged. Details for Type 10 barrier are not in the Staff Bridge Worksheets but can be obtained from Staff Bridge upon request. Due to primarily height issues, Type 10 bridge rail is TL-3 or below and may need to be replaced or modified as safety funding allows. The Type 10 bridge rail with 12'-6" spacing is weaker than the 10'-0" spacing.

13.3.7 Type 9

Bridge Rail Type 9 (Single Slope) is a reinforced concrete barrier with a sloped front face. This bridge rail meets MASH requirements for TL-4 and in some cases TL-5. This bridge rail can be mounted to a bridge deck, to a moment/gravity slab, or on top of cast-in-place retaining walls. CDOT requires the use of Bridge Rail Type 9 or Type 10 MASH on all new and rehabilitated bridges, concrete box culverts, and retaining walls. To maximize splash protection and allow easier installation of protection panels, this is generally the required railing for bridges over railroads.

13.3.8 Type 10 MASH

Bridge Rail Type 10 MASH is composed of two continuous horizontal steel tubes attached to steel W shape posts on top of a concrete curb. Type 10 MASH rails meet AASHTO and MASH TL-4 requirements and in some cases TL-5. CDOT requires the use of Bridge Rail Type 10 MASH or 9 on all new and rehabilitated bridges, concrete box culverts, and retaining walls. Because of splash and railroad requirements, this rail may not be allowed for bridges over railroads without additional features. See Section 13.6 for conduit restriction in the railing.

13.4 COMBINATION VEHICULAR PEDESTRIAN RAILS

Combination vehicular pedestrian railings shall be used at the edge of deck when the sidewalk is not protected from traffic. If the sidewalk is protected from traffic, the edge of deck shall protect the pedestrians with a fence or another combination railing. Combination vehicular and pedestrian railing shall meet AASHTO requirements. Galvanizing of steel portions of rail is the minimum standard required. In cases where the steel portion of the bridge rail is to be painted for aesthetic or other reasons, it must be done in addition to galvanizing using a duplex coating system.

**AASHTO
13.10**

13.5 PIER AND RETAINING WALL PROTECTION

13.5.1 Pier Protection

Piers or abutments located inside the clear zone, as defined by AASHTO *Roadway Design Guidelines*, and not designed to resist the vehicular collision force (CT) shall be protected with a TL-5 rated barrier, approved by the State Bridge Engineer in coordination with the Bridge Rail SMEs, that meets AASHTO and MASH crash test requirements. Because CDOT does not have an approved TL-5 barrier, the Designer may submit a crash tested TL-5 barrier from another state to State Bridge Engineer and Bridge Rail SMEs for review and possible acceptance. The submittal shall include all documentation showing conformance to current criteria outlined in AASHTO, MASH, this BDM, and FHWA acceptance. If a TL-5 barrier is not used as pier protection, the pier shall be designed to resist the CT load in accordance with AASHTO.

**AASHTO
3.6.5.1,
Section 13**

For piers located inside the clear zone and designed for the CT force, the Designer shall consult the CDOT Project Manager to determine if safety protection is still desired.

Clear zone to the pier shall be determined at the ultimate configuration of the roadway adjacent to the pier. It shall consider all anticipated widenings.

13.5.2 Retaining Wall Protection

When a retaining wall front face is located within the clear zone or when requested by the region, it shall be protected by a barrier. See Section 11.5.11 of this BDM for details.

13.5.3 Sound Barriers

Sound barriers within the clear zone shall meet AASHTO collision requirements. The Designer shall coordinate with the region and roadway engineer to determine the type of protection and setback. If the sound barrier is outside the clear zone, it does not need to be designed for collision.

**AASHTO
15.8.4**

13.5.4 Rail Anchor Slabs

Bridge rails are often required on retaining walls, culverts, and or other structural systems. Due to the significant loads associated with vehicular impact, railings can be connected to an independent structural foundation called a rail anchor slab. The Designer shall evaluate the cost difference between mounting the barrier directly to the structure or using a rail anchor slab.

To avoid excessive damage from an impact, expansion joint material or other type of separator shall be installed between the nose of the anchor slab and the wall facing below. The Designer shall evaluate vertical and lateral loads that may be transferred from the anchor slab to the wall element below during a vehicular impact.

When a rail anchor slab is required to be designed, the Designer shall use the recommended design procedures from NCHRP Report 663 and outlined in the BDM Example 12, Moment Slab Design. For anchor slab details on MSE walls, the Designer should reference the Staff Bridge Worksheets for MSE walls.

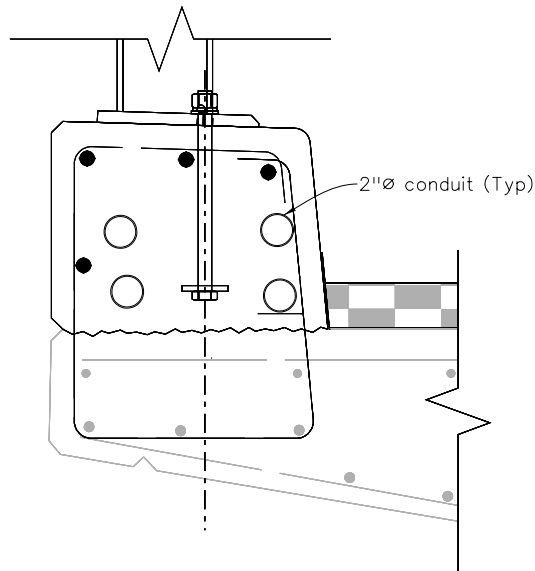
13.6 ATTACHMENTS TO AND CONDUITS IN BRIDGE RAIL SYSTEMS

During collisions with barrier systems, it has been shown that vehicles slide along the top of the barrier and that parts of the vehicle extend over the barrier a considerable distance. This envelope of the vehicle encroaching beyond the barrier is known as the zone of intrusion. Attachments to barrier systems within the zone of intrusion, such as fencing, signs, and light poles, should address safety concerns such as snagging, spearing, and debris falling into traffic below. The amount of intrusion is related to the height and profile of the barrier, as well as the vehicle size, speed, and angle of impact. See NCHRP Research Report 1018: Zone of Intrusion Envelopes Under MASH Impact Conditions for Rigid Barrier Attachments as a reference. The Designer should minimize any attachments to the railing system within this zone whenever possible. Attachments within this zone will affect the safety of the bridge rail. Whenever possible, light poles should be located behind the back face of the barrier.

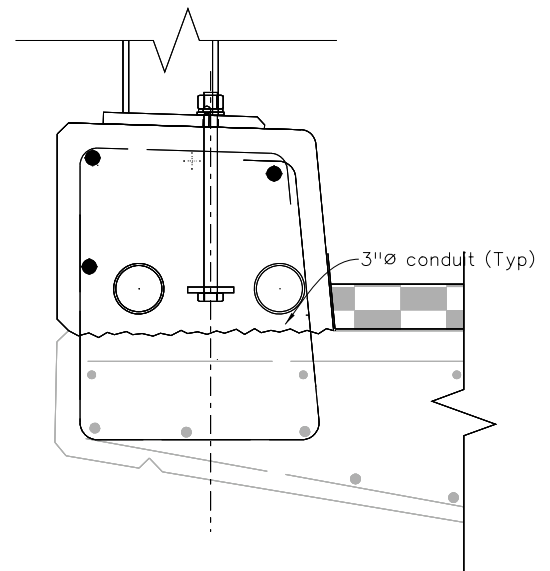
The curb of 10 MASH bridge rails limits the number and size of conduits to assure ease of placement and proper consolidation of concrete. Requirements for location of conduits inside the curb are:

- 1" min. clear from the construction joint between bottom of the curb and the deck
- 1.5" clear spacing between rebar and conduit and between each conduit

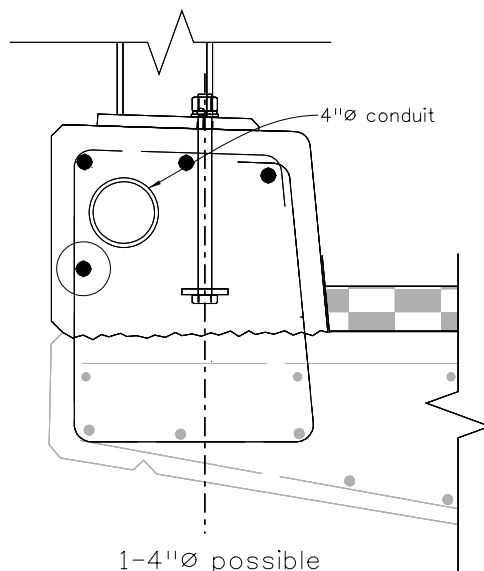
A maximum number of four 2"Ø, two 3"Ø with 1- 2"Ø or 1-4"Ø with 1-3"Ø conduit are allowed. Conduits placed in excess of this are at a greater risk of damage due to vehicle collisions. Using galvanized rigid conduit will minimize damage as well. See below figure of 10 MASH curb about conduit configuration.



4-2"Ø possible
conduits configuration



2-3"Ø possible
conduits configuration



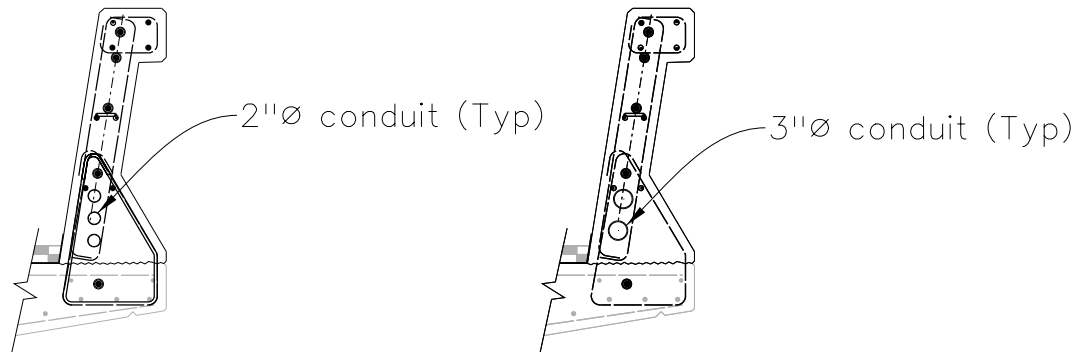
1-4"Ø possible
conduits configuration

According to the crash test result at the location of the steel post for 10 MASH bridge rails, spalled concrete was observed on the back of the curb as well as cracks on the front face. Based on this observation, placing conduits close to the front face of the curb should have less potential damage risk than on the back. For higher risk systems such as Intelligent Transportation Systems (ITS) locations should be limited to 2-2"Ø conduit placed close to the front face of the curb with an additional requirement of change to "5.5" minimum from top of curb. When using higher risk systems such as Intelligent Transportation Systems (ITS) conduit, the designer should consider Type 9 bridge rail to reduce damage potential. ITS conduits may still have a certain degree of damage after collision regardless of their locations inside the curb.

The Type 9 bridge rail also limits the number and size of conduits to assure ease of placement and proper consolidation of concrete but provides more flexibility. Requirements for location of conduits inside the rail are:

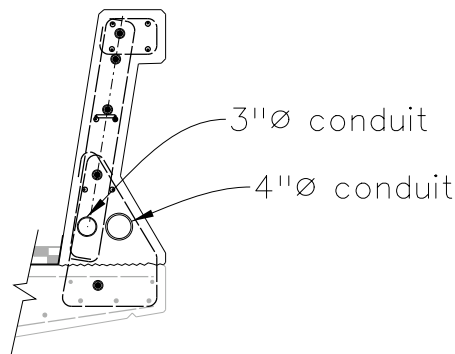
- 1" min. clear and 15" max from the construction joint between curb and deck
- 1.5" clear spacing between rebar and conduit and between each conduit

The maximum number of 3-2"Ø, 2-3"Ø or 1-4"Ø with 1-3"Ø conduit are allowed. See below Figure of Bridge Rail Type 9 about conduit configuration. Conduits placed in excess of this are at a greater risk of damage due to vehicle collisions. Using galvanized rigid conduit will minimize damage as well.



3-2"Ø possible
conduits configuration

2-3"Ø possible
conduits configuration



1-4"Ø and 1-3"Ø possible
conduits configuration

13.7 AESTHETIC TREATMENTS TO BRIDGE RAIL SYSTEMS

Except for color treatments, aesthetic enhancements shall not be applied to the traffic face of the barrier systems. Applying aesthetic enhancements to this face increases the likelihood of vehicle snagging and damage caused by snowplows, thereby increasing maintenance costs, and decreasing traveler safety.

13.8 RAILING ATTACHMENT TO HEADWALLS

If a railing is attached to a headwall on a culvert, the Designer shall analyze the structure for collision loading. Headwall mounted barriers are required only if they are within the clear zone and not protected with a roadway barrier.

SECTION 14 JOINTS AND BEARINGS

14.1 GENERAL REQUIREMENTS

Joint and bearing systems shall be designed to accommodate all calculated movements and loading expected throughout the life of the bridge. Joints and bearings shall also be designed to accommodate regular maintenance activities that will prolong the life of these devices.

14.2 CODE REQUIREMENTS

Unless otherwise noted, the design of joints and bearings shall be in accordance with the latest AASHTO, as supplemented by the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, where applicable.

Long-term concrete properties, including creep and shrinkage strains, shall be determined in accordance with AASHTO.

**AASHTO
5.4.2.3.1**

14.3 UNIFORM TEMPERATURE MOVEMENT

Bridges are subject to heat transfer from the ambient air temperature and radiant heat from direct sunlight. Bridges of different structure types react at different rates, with concrete structures reacting more slowly due to a larger thermal mass than that of steel structures, making them less susceptible to large temperature swings over a short amount of time.

Variations in the average temperature of the bridge superstructure result in thermal expansion and contraction. Maximum and minimum anticipated temperatures over the life of the structure shall be used for design.

Temperature ranges for either Procedure A or B (preferred) may be used for structures designed in accordance with AASHTO 3.12.2, along with the appropriate load factors provided in AASHTO Table 3.4.1-1. Temperature gradient may be considered where appropriate in accordance with AASHTO 3.12.3.

14.4 EXPANSION JOINTS

14.4.1 General

Bridges shall be capable of accommodating movements, rotations, and deformations imposed on the structure through temperature changes, concrete creep and shrinkage, and shortening due to applied loading. Expansion joints shall also accommodate both bridge skew and curvature and have adequate maintenance access.

Other possible sources of joint movement and rotation include, but are not limited to, live load (such as braking), wind, seismic loads, and settlement. Movements from these force effects vary based on code requirements, bridge configuration, and the complexity of the bridge and shall be considered as appropriate.

Expansion joint devices shall prevent water, deicing chemicals, and debris infiltration to the substructure elements below. Expansion joints shall also provide

a relatively smooth riding surface between approach pavements and the structure, or adjacent structural elements.

The Designer is responsible for giving adequate thought to the type, size, and performance of the selected expansion joint system to ensure that the appropriate system is used on the structure.

When the skew angle is greater than or equal to 30°, the Designer shall consider placing the expansion joint normal to the roadway alignment to prevent snowplow damage.

Due to maintenance concerns with expansion joints, it is preferred to implement jointless construction wherever possible. Jointless construction uses integral or semi-integral abutments and piers to eliminate expansion joints on the bridge superstructure. A joint at the end of the approach slab shall be used to accommodate movement and to prevent damage to the roadway pavement as defined in 14.4.2.

Refer to BDM Section 11.3 for additional information on integral abutments and approach slab requirements.

14.4.2 Design Guidelines and Selection

The need for an expansion joint will be determined based on the amount of bridge movement (determined from design) and roadway approach type. Bridges with total temperature movement (expansion plus contraction) of $\frac{3}{4}$ in. (which typically corresponds to a bridge length of about 150 ft.) or less are not required to have expansion joints at substructure locations or at the ends of approach slabs, unless the roadway approach is concrete pavement. When the roadway approach is concrete pavement, an expansion device shall be required between the end of the approach slab and the roadway approach despite the amount of bridge movement in order to deal with the concrete pavement growth, unless approved by Unit Leader in coordination with the Expansion Joint SMEs. In lieu of a strip seal joint, a silicone seal joint or compression seal joint may be used between the end of the approach slab and the roadway.

For non-complex straight bridges with no skew, the total movement shall be determined by using AASHTO 3.12.2 and 14.5.3.2. For complex bridges, movement calculation shall include consideration for superstructure type, contributing length, structure curvature, construction phasing, fixity condition between superstructure and substructure, superstructure rotations, and substructure stiffness. Skews, horizontal and vertical alignment, grade, and cross slopes shall be considered when selecting and designing a joint system. This can be accomplished by finite element analysis, modeling soil as springs, calculating depth of fixity based on soil/structure interaction analysis, etc.

Wherever practical, expansion devices shall be installed in preformed concrete block-outs after completion of the bridge deck. The installed expansion gap shall correspond to the ambient temperature at the time headers are placed. The plan sheets shall include installation gaps sizes and corresponding temperatures for the range recommended by the manufacturer.

14.4.3 Small Movement Joints

Small movement joints are not recommended when total movement is greater than 2 in. The total movement shall be determined in accordance with AASHTO 14.5.3.2. These joint systems shall not be used for new construction on Interstate Highways or State Highways without Unit Leader in coordination with Expansion Joint SMEs approval.

14.4.3.1 Asphaltic Plug

Asphaltic plug joints consist of modified asphalt installed in a preformed block-out over a steel plate and backer bar. These joints provide a smooth riding surface that is built to match the adjacent roadway.

Due to observed creep and poor expansion performance of these joints, CDOT does not recommend asphaltic plug joints on Interstate Highways, State Highways with high traffic counts, or roadways with heavy trucks. Therefore, use of asphaltic plug joints requires approval from Unit Leader in coordination with Expansion Joint SMEs approval.

14.4.3.2 Silicone Seals

Silicone seals are flexible, poured sealants designed to provide a watertight expansion joint seal in both new and rehabilitation projects. Silicone sealants allow good elastic performance over a range of temperatures; provide self-leveling installations; can be installed against non-parallel surfaces; and bond without the use of additional adhesives.

Silicone seals shall be considered for rehabilitation projects where long-term closures are not acceptable or where rehabilitation on the joint header is not possible, thereby eliminating compression seals as a viable option.

Silicone seals shall be installed such that the maximum tension movement is no more than 100 percent of the install width and the compression movement does not exceed 50 percent of the install width. Silicone seals shall be installed a minimum of $\frac{1}{4}$ in. below the pavement surface to minimize contact with crossing tires.

Installation gaps shall not be less than 1 in. at 60° F.

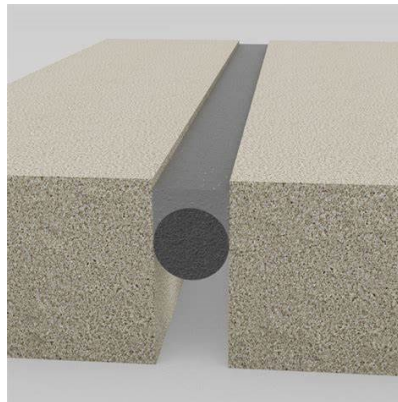


Figure 14-1: Silicone Seal

14.4.3.3 Compression Seals – Elastomeric or Foam

Compression seals are continuous manufactured elastomeric or foam elements, typically extruded with an internal grid system. These joints shall be installed against prepared concrete or steel faces with adhesive material and may or may not be armored.

Foam elements shall be comprise of a precompressed, silicone and foam hybrid system. Generally, foam joint systems come in stick lengths between 6 and 7 feet and are installed into field-applied epoxy adhesive on the joint faces. The sticks are easily joined in the field with silicone to accommodate the total gap length. Additional silicone sealant bands are applied to the joint faces and are tooled to a cove-bead to provide a watertight seal.

Elastomeric compression seals shall be furnished and installed as a single continuous piece across the full width of the bridge deck. Field splices are not allowed. Termination in median barriers is recommended on wide bridges.

The maximum gap shall not exceed 2 in. at -30° F to prevent damage from debris and wheel loads.

Compression seals are not allowed on bridges with skew angles exceeding 15° . This is due to past performance and improper joint sizing to accommodate the transverse movement component.

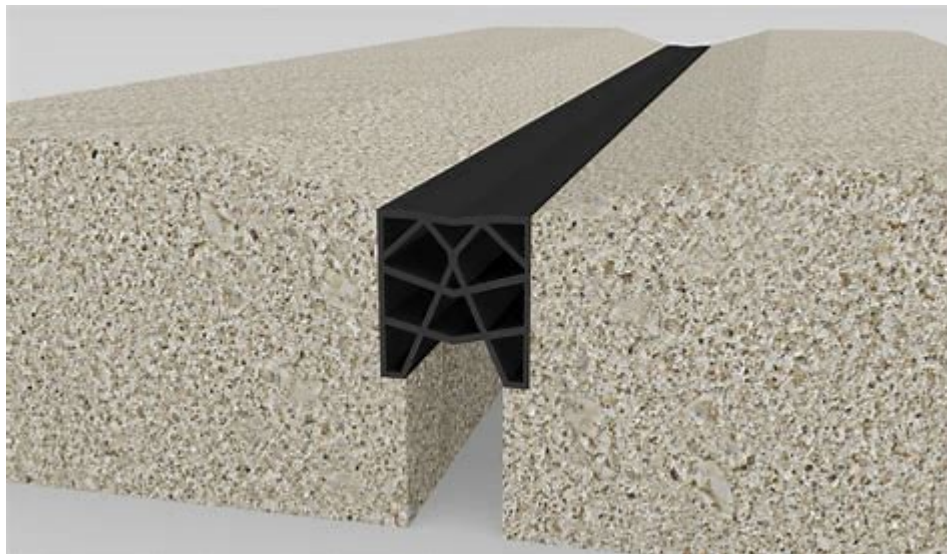


Figure 14-2: Compression Seal - Elastomeric

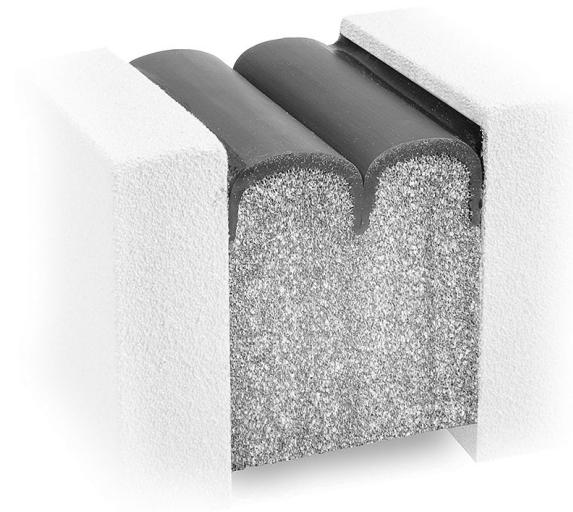


Figure 14-3: Compression Seal - Foam

14.4.3.4 Saw-Seal Joint

A saw-seal joint shall be placed in the top of asphalt and polyester polymer concrete (PPC) overlays when expansion joints are not used at the following locations:

- Interface between the bridge deck and approach slab
- Interface between the approach slab and roadway approach pavement

Saw-seal joints control cracking in the overlays and reduce potholes, which increase the likelihood of water intrusion in the deck.

14.4.4 Strip Seals

Strip seal systems consist of a preformed neoprene gland mechanically locked into steel edge rails embedded into concrete on both sides of an expansion gap. Strip seal joints provide a cost-effective joint system that allows easy neoprene gland replacement when needed.

The use of epoxy bonded strip seal joint systems is not allowed on new construction.

Strip seal steel rail components shall be installed as one continuous length where possible due to maintenance concerns. It is preferred to have the steel rail component of the strip seal be supplied in as long of pieces as possible based on phasing and slope changes to minimize the number of field splices. Horizontal angle changes in the expansion joint exceeding 35° shall be avoided so that the factory requirement of vulcanizing the strip seal corners is not necessary.

Strip seal neoprene glands shall be installed as one continuous length to provide a watertight joint sealing system.

Strip seals are the preferred joint alternative for bridge lengths greater than 250 ft. because they have proven to provide the best long-term performance. Strip seals shall be used for all new construction where the total joint movements are expected to be 4 in. or less and the skew is less than or equal to 25°. For joints between ¾ to 2" of movement, small movement joints may be used with Unit Leader's approval in coordination with Expansion joint SMEs. If the skew is greater than 25°, oversized glands shall be considered subject to the conditions below.

Unit Leader in coordination with Expansion Joint SMEs will approve the use of oversized glands, but oversized glands may be considered under the following conditions:

- Total factored joint movement does not exceed 5 in.
- Factored cyclical (Thermal) joint movement does not exceed 3.5 in.
- Modular joints are not practical due to joint lead time during construction.
- Use of oversized glands allows the bridge to require joints at the ends of approach slabs only.

Due to life-cycle maintenance costs with oversized glands in comparison to modular joints, the use of oversized glands shall not be made based solely on initial construction cost alone but also consider the durability of the joints. Evaluate on a case-by-case basis.

Appendix A contains a design example for a strip seal expansion.

14.4.5 Modular Joints

Modular joints are complex structural assemblies that consist of multiple pre-molded neoprene strip seals held into place by separate extruded steel beams. These joints are designed for movements greater than 4 in. The use of modular joints should be avoided by designing multiple strip seals if possible. For new bridges, use of modular joints needs Unit Leader approval. Modular joints shall not be placed at either end of approach slabs due to maintenance and inspection concerns.

Modular joints shall be designed by the manufacturer to the latest AASHTO requirements for fatigue and fracture. The Contractor shall submit to the Engineer of Record calculations signed and sealed by a Colorado Professional Engineer, along with the shop drawing, for review and acceptance prior to fabrication. The Designer shall be responsible for ensuring this requirement is in the project specifications.

Modular joints shall be specified in 3 in. increments, with 6 in. being the minimum. In addition to thermal movements determining the size of joints, manufacturers have gap requirements that may increase the size of the required joint. For example, a 0 in. to 9 in. joint may be required where movement indicates that a 0 in. to 6 in. joint is feasible. The Designer shall check manufacturer's requirements before final sizing.

Modular expansion joints shall be installed as one continuous unit due to maintenance and manufacturer concerns. Field splicing of modular joints is not allowed without Unit Leader in coordination with Expansion Joint SMEs approval. Where field splicing is required, all splices shall be fully welded or hybrid welded/bolted splices. Fully bolted splices are not allowed.

14.4.6 Finger Joints

Finger joints can be used to accommodate moderate to high movement ranges. Finger joints can also accommodate minor rotations and vertical displacements across the joint. Finger joints are fabricated from steel plate, with the fingers sized to maintain minimum spacing and to minimize live load deflections. Fabricated sections shall be less than or equal to 6 ft. to allow maintenance access. A taper shall be fabricated on each finger to ease the transition between plates and to minimize the potential for snowplow damage.

To provide a watertight seal, finger joints require the installation of an elastomeric or metal trough to capture water and convey it away from the substructure. Without proper and routine maintenance, these trough systems clog and lead to water damage to the joint and substructure below. For this reason, finger joints shall be limited to replacement of existing finger joint expansion devices only.

14.4.7 Cover Plates

14.4.7.1 Sidewalk Cover Plates

Expansion joints shall be extended across all sidewalks and into the bridge rail. Accessible sidewalks shall have expansion joints covered with Americans with Disabilities Act (ADA) compliant cover plates. Cover plates may be fabricated or proprietary but shall comply with the latest ADA requirements. ADA compliant expansion joints installed at the top of the sidewalk shall not have cover plates.

Cover plates shall not protrude above the walking surface by more than $\frac{1}{2}$ in. and shall be installed flush with the walking surface whenever possible. Where cover plates protrude more than $\frac{1}{4}$ in. above the walking surface, a 2:1 edge taper shall be provided.

Cover plates shall have an anti-slip surface treatment such as treads and roughened surfaces. These surfaces shall be galvanized.

14.4.7.2 Bridge Rail Cover Plates

Bridge Rail and bridge rail curbs shall have removable steel cover plates to provide continuity of the bridge rail over the expansion joint and to protect the expansion joint embedded in the bridge rail. See the CDOT Staff Bridge Structural Worksheets for bridge rail for cover plate details.

14.4.8 Joint Headers

Expansion joint headers shall be the same material as the bridge deck or better products approved by the Unit Leader in coordination with the Expansion Joint SMEs. They shall be installed $\frac{1}{4}$ in. above the top of the expansion system and even with the final roadway surface.

When using modular joints or replacing finger joints, the Engineer of Record shall be responsible for ensuring that the provided block-out can accommodate the specified joint system, regardless of manufacturer.

The use of accelerated mix designs and bagged mixes is allowed per the requirements of Concrete Class DR.

14.4.9 Expansion Joint Details

CDOT Staff Bridge provides Structural Worksheets for 0 to 4 in. expansion joints, modular expansion joints, and asphaltic plug joints.

14.5 BEARINGS

14.5.1 General

Bridge bearings transfer permanent and transient loads from the bridge superstructure to the substructure. These loads can be vertical (e.g., dead load or live load) and horizontal (e.g., wind, braking, or seismic). Bearings shall also accommodate anticipated movements (e.g., thermal/creep/shrinkage) and rotations. When bearings and expansion joints are collocated, movements allowed by bearings shall be accommodated by adjacent expansion joint systems, which requires that bearings and expansion joints be designed interdependently and in conjunction with the anticipated behavior of the overall structure.

Several bearing types are available that can achieve the above requirements, including elastomeric bearings (plain and reinforced); polytetrafluoroethylene (PTFE) sliding bearings; and High-Load Multi-Rotational (HLMR) bearings (pot, spherical, and disc bearings). Each bearing type differs in regard to vertical and horizontal load carrying capacity, displacement capacity, and rotational capacity. Understanding the properties of each bearing system is critical for economical selection of bearing systems or the elimination of bearings in favor of integral connections of the superstructure to the substructure.

14.5.2 Design Guidelines and Selection

Where bearings are required, the following bearings shall be used unless otherwise approved by Unit Leader in coordination with the Bearing SMEs through the Structure Selection Report process:

- CDOT Type I (plain or steel reinforced elastomeric bearing pads)
- CDOT Type II (PTFE sliding elastomeric bearings)
- CDOT Type III (pot or disc bearings)
- CDOT Type IV (rocker plate with elastomeric pad)
- CDOT Type V (rocker plate with PTFE)

All bearings shall be the same size and type at each substructure unit. This is due to potential damage from differing deflection and rotational characteristics. Bridge superstructure units (e.g., superstructure limits between expansion joints) requiring Type III bearings shall use Type III bearings for the entire superstructure

unit except where the superstructure is integrally connected to the substructure (e.g., integral abutments and fixed piers with integral pier diaphragms).

14.5.3 Thermal Movement

All bridges with bearings shall be designed for a thermal movement range determined in accordance with AASHTO 3.12.2 and factored using AASHTO Table 3.4.1-1, plus the effects of creep, shrinkage, and post-tensioning, if applicable. When designing the elastomer for Type I and Type II bearings, the 65 percent reduction of the design thermal movement range shall not be used. This allows the bridge to be constructed on the hottest day of the year without having to reset the bearings after construction is complete.

When the erection temperature of the bridge is known or if a special provision to verify/adjust the position of the bearings after the completion of the bridge is included in the construction specifications, the application of the 65 percent reduction in the design thermal movement may be used.

**AASHTO
14.7.5.3.2**

14.5.4 Additional Rotation Requirements

CDOT follows the AASHTO requirement that adds a tolerance of 0.005 rad. to the calculated rotations of the structure to account for uncertainties in the fabrication and placement of the bearings. Section 512.11 of CDOT's Standard Specifications for Road and Bridge Construction provides a flatness tolerance for the bearing seat location, which is included in this tolerance.

**AASHTO
14.4.2**

14.5.5 Design Coefficient of Friction Requirements

PTFE sliding surfaces can be effective in reducing the friction coefficient between the bearing and the sliding surface. When the temperature is cold, the coefficient of friction can increase dramatically. CDOT uses a range of friction values in the design of bridges to cover the variations in the coefficient of friction that the structure may see during its life. A minimum coefficient of friction shall be 0.02, and the maximum coefficient of friction shall be taken from AASHTO. The maximum coefficient of friction shall be based on the Dead Load only case for determining the compressive stress on the PTFE.

**AASHTO
Table
14.7.2.5-1**

14.5.6 Bearing Inspection and Removal

All bridges shall be designed such that the bearings can be inspected, and if necessary, the bearings can be removed without special tools. Normal girder construction typically provides access to the bearings from both the front and the sides of the bearings. These access locations shall be kept clear whenever possible. Cast-in-place concrete box girder bridges are the hardest to inspect and replace the bearings. Pedestals for bearings shall be used whenever practical.

The bridge plans shall provide all structural elements necessary to jack and support the bridge for bearing replacement. This may consist of a block-out in the superstructure diaphragm, corbels, or steel jacking brackets bolted to the substructure. The design of the jacking system shall be based on using either 50-ton or 100-ton jacks, which are commonly used in Colorado. The minimum size of 50-ton jacks is 6 in. high by 8 in. in diameter. The minimum size of 100-ton jacks is 8 in. high by 10 in. in diameter. Designing for these sizes ensures that most jacks that differ from these sizes will still fit the designed structural

element supporting the jack. Only one size of jack shall be used at each substructure location. If multiple jacks are required or a jacking block-out in the diaphragm is used, an additional 3 in. horizontally shall be provided for the hydraulic jack hoses. Bearings shall be designed to be removed with a jacking height of $\frac{1}{4}$ in. or less. Other commonly used and available jacks with reduced height requirements may be used with Unit Leader in coordination with the Fabrication/Construction Unit approval.

Jacking the bridge under live load is not permitted without Unit Leader in coordination with the State Bridge Engineer and Fabrication/Construction Unit approval. Live load may be placed on the bridge provided that temporary blocking is in place or the jacks are securely locked out. The substructure plans shall state this policy and show the Service Loads for Dead Load, Live Load, and Live Load plus Dynamic Load Allowance.

14.5.7 Leveling Pads

Leveling pads are plain elastomeric pads used for locked-in-girders at integral substructures and shall be thick enough to prevent girder-to-support contact due to anticipated girder rotations up through and including the deck pour. Leveling pads shall be designed for dead loads only using AASHTO Design Method A. Rotation restrictions other than preventing girder-to-support contact shall not be considered. Compressive stress and stability during construction shall be checked in accordance with BDM Section 5.5.1.2. A Shore A durometer hardness of 60 shall be used in the design. Normally these pads are $\frac{1}{2}$ in. thick and may be up to 1 in. thick.

**AASHTO
14.7.6**

Appendix A includes a leveling pad design example.

14.5.8 Type I Bearings

Type I bearings that are plain pads may be designed using AASHTO Design Method A. The minimum Shore A hardness shall be 60 durometer.

Type I bearings that are steel reinforced elastomeric pads shall be designed using AASHTO Design Method B. If approved by Unit Leader in coordination with the Bearing SMEs, AASHTO Design Method A may be used for light to moderately loaded steel reinforced elastomeric bearings if determined to be more economical based on eliminating the testing and quality control costs required for AASHTO Design Method B. The minimum low-temperature grade of elastomer shall be Grade 3.

**AASHTO
14.7.5**

The minimum bearing height shall be 2 in. to facilitate inspection and removal of the bearing. The bearing height shall be limited to 6 in. based on constructability and cost-effectiveness.

Appendix A includes reinforced Type I Bearing design examples.

14.5.9 Type II Bearings

A Type II bearing is a Type I bearing with a bonded PTFE surface with a stainless steel mating surface to provide the necessary horizontal displacement capacity for the bridge. The elastomeric portion of the bearing shall meet the requirements of a Type I bearing. The sliding surfaces shall meet AASHTO requirements.

**AASHTO
14.7.2**

The Structural Design Engineer shall verify that the stiffness of the elastomeric pad is sufficient to enable the sliding surface to engage without excessive pad deflection.

**AASHTO
14.7.5.3.2**

Appendix A includes a Type II bearing design example.

14.5.10 Type III Bearings

Type III bearings shall consist of HLMR bearings and are a special design for each bridge. These bearings shall follow the AASHTO specifications for pot bearings and disc bearings. Disc bearings are preferred to pot bearings.

**AASHTO
14.7.4 &
14.7.8**

The minimum bearing height shall accommodate minimum jacking spacing required for the readily available jacks. This requirement is applicable to any other bearing type that is desired to be replaceable or repairable.

14.5.11 Type IV & V Bearings

Type IV and V Bearings consist of rocker plates on top of elastomeric pads with and without sliding surfaces. These bearings are not typically used but may be an option.

14.5.12 Bearing Details

CDOT Staff Bridge provides Structural Worksheets for Type I, II, III, IV, and V bearings.

All bearings shall be installed on a level concrete surface. In the direction of movement, the minimum length of the concrete surface (beam seat) shall be the maximum of the following:

- The dimension of the bearing in the direction of consideration, plus 50 percent of the maximum horizontal displacement (Δ_o) on each side, or 50 percent of the minimum longitudinal plan dimension of the bearing, whichever is greater
- The minimum support length for the seismic design requirements of AASHTO 4.7.4.4

The size of the level concrete surface is to provide the ability to adjust the position of the bearing in the future and to provide adequate beam seat width for seismic displacement. Unit Leader shall review all deviations from the aforementioned seat width requirements, such as a narrower beam seat with a recessed bearing.

The plans shall clearly show the orientation of guided bearings along the bent line.

Sole plates and masonry plates shall be a minimum of $\frac{3}{4}$ in. thick at the edges of the plate.

Sole plates and bearing top plates shall be oversized 2 in. longitudinally (1 in. in each direction) to accommodate construction tolerances.

Because Type III bearings are dependent on the manufacturer of the bearing, they are generally shown schematically on design drawings. The Structural Design Engineer shall be responsible for coordinating with bearing suppliers and/or manufacturers when Type III bearings are required.

If slotted holes are needed in bearing top plates for anchor bolts in the direction of structure movement, they shall be sized for the maximum horizontal displacement (Δ_o). Slots shall be oversized a minimum of 1 in. ($\frac{1}{2}$ in. in each direction) or 1 anchor bolt diameter, whichever is greater.

Anchor bolts in sole plates may be omitted if an alternate transverse restraint is provided. Sole plates without anchor bolts shall be a minimum of 2 in. wider than the bearing device or the girder to accommodate construction tolerances.

14.6 SHOP DRAWINGS

The Structural Design Engineer shall review shop drawings for all fabricated bearing and joint elements. Particular attention shall be paid to Type II and Type III bearings and modular expansion joints. The Contractor performing the work shall submit modular joint calculations. Working drawings for 0 in. to 4 in. expansion joints shall be reviewed as time allows to avoid possible construction issues. In addition, compatibility between the bearings and the joint elements shall be checked. The Structural Design Engineer shall be responsible for reviewing calculations submitted with the shop drawings. The review verifies that calculations, shop drawings, and design drawings are compatible and in compliance with AASHTO and the BDM.

SECTION 15 DESIGN OF SOUND BARRIERS

15.1 GENERAL REQUIREMENTS

This section provides guidance for the design of sound or noise barriers.

AASHTO and CDOT pay items refer to sound barriers while FHWA typically refers to noise barriers. The terms should be considered interchangeable although noise wall is preferred and used throughout this Design Manual, inspection reports, and APL lists.

15.2 CODE REQUIREMENTS

This section of the BDM supplements AASHTO Section 15.

AASHTO generally concerns itself with the structural design requirements for noise barriers while FHWA Noise Barrier Design Handbook deals with the noise reduction properties of the wall.

15.3 AESTHETICS

A typical CDOT noise barrier consists of a concrete panel mounted on concrete or steel posts. Refer to Section 2.3.3 of this BDM for acceptable concrete aesthetic treatments.

Wood is not allowed because of past experience with durability issues. Staff Bridge will consider other materials and designs if design criteria are met.

15.4 LOADS

Wind loads shall be in accordance with AASHTO. When a noise barrier is in Colorado's special wind region, use the Partial Special Wind Region Map in Section 32 of this BDM to determine wind speed.

**AASHTO 3.8
& Figure
3.8.1.1.2-1**

Vehicle collision forces need not be considered for the following cases:

**AASHTO
15.8.4**

- Noise barriers located beyond the acceptable clear zone.
- Noise barrier/rail systems within the clear zone that have been successfully crash tested.
- Noise barriers behind a crashworthy traffic railing with a setback greater than 4 ft. The Designer should make every effort to achieve a minimum setback greater than 4 ft.
- Noise barriers or portions thereof at locations where the collapse of a wall has minimal safety consequences, as determined by Staff Bridge.

When the above requirements cannot be met, the railing test levels and crash criteria shall be in accordance with AASHTO.

**AASHTO
Sections 13,
A13.3, & 15**

Noise barrier materials shall be selected to limit shattering of the noise barrier during a vehicle collision. When reinforced concrete panels are used, AASHTO recommends the use of two mats of steel to limit the concrete shattering during a vehicle collision.

**AASHTO
15.8.4**

New noise barriers shall meet AASHTO Test Level 3 (TL-3) requirements.

**AASHTO
Table A13.2-1**

SECTION 16 THROUGH 30

Reserved for Future Use

SECTION 31 PEDESTRIAN STRUCTURES

31.1 GENERAL REQUIREMENTS

This section addresses design and performance requirements for typical pedestrian bridges intended to carry pedestrians, bicyclists, equestrian riders, and light maintenance vehicles.

31.2 CODE REQUIREMENTS

Design shall be in accordance with the AASHTO *Standard Specifications for Highway Bridges*, except as modified by the AASHTO *LRFD Guide Specifications for the Design of Pedestrian Bridges* and this BDM.

31.3 PERFORMANCE REQUIREMENTS

31.3.1 Service Life

Pedestrian bridges must be designed to achieve a minimum service life of 75 years.

31.3.2 Maintenance Requirements

Pedestrian bridges should be designed to allow ease of inspection and maintenance. Periodic preventive maintenance and inspections will be performed on all pedestrian bridges to extend the service life of the structure. Preventive maintenance may include cleaning, removing debris, painting, sealing deck joints, etc.

31.3.3 Aesthetic Goals

Refer to Section 2.3 of this BDM for information about aesthetic requirements.

31.4 GEOMETRY AND CLEARANCES

31.4.1 Geometry

31.4.1.1 Width

Bridge deck width should be based on the type of anticipated local usage and corresponding current ADA Standards for Accessible Design guidelines. Clear width should be measured from face to face of rail.

Wider bridges are preferred for two-directional pedestrian traffic rather than narrow decks with passing spaces due to the difficulty in design and constructability of the landings. However, when passing spaces are used, they should conform to ADA requirements and be located at reasonable intervals, not to exceed 200 ft.

Coordinate with the local agency to determine the final section on a pedestrian or bicycle bridge.

Refer to Chapter 14 of CDOT Roadway Design Guide for additional pedestrian facilities geometry requirements.

31.4.1.2 Profile and Grade

Refer to current ADA Standards for Accessible Design for maximum grade allowed on pedestrian bridges.

Pedestrian bridges over waterways shall satisfy all requirements set for vehicle bridges for freeboard, scour, and overtopping.

31.4.1.3 Ramps

Pedestrian overpass structures, if practical, may be provided with both ramps and stairways, but under no condition should a structure be built with stairs only.

Maximum grades on approach ramps shall conform to ADA requirements. Whenever existing structures or other local constraints prevent design of the structure that satisfies maximum grade requirement, landings shall be provided to accommodate a maximum rise of 2.5 ft. Landings shall be level, the full width of the bridge, and a minimum of 5 ft. in length. Landings shall also be provided whenever the direction of the ramp changes. However, straight grades or vertical curves are preferred instead of landings whenever possible.

The deck and ramps shall have a non-skid surface, such as a transverse fiber broom finish for concrete. Concrete bridge decks must have transverse joints to minimize map cracking. The Designer shall specify the spacing of the joints.

31.4.1.4 Physical Requirements

The Structure Selection Report should evaluate all feasible structure types.

The deck of the bridge should maintain the cross-slope of the approach trail. Cover plates should be provided at all expansion joints to minimize tripping hazards. Approach slabs are not required on pedestrian bridges unless requested by the Owner.

Section 2.4 of this BDM outlines the requirements for the pedestrian and bicycle railing.

31.4.2 Vertical Clearances

The minimum vertical clearance from an under-passing roadway surface to a pedestrian bridge shall be 17.5 ft. as outlined in Section 2.2.2 of this BDM. The minimum vertical clearance from a pedestrian or bicycle path to an overhead obstruction shall be 8 ft.-4 in., or 10 ft. for an equestrian path, measured at 1 ft. from the face of curb, parapet, or rail.

31.4.3 Horizontal Clearances

Horizontal clearances shall conform to AASHTO.

AASHTO 2.3.3

31.5 LOADS AND DEFLECTIONS

31.5.1 Live Loads

31.5.1.1 Pedestrians

Refer to the current edition of *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges* for the design value of the pedestrian live load.

31.5.1.2 Maintenance Vehicles

Whenever vehicle access is not prevented by permanent physical methods, pedestrian/bicycle bridges shall be designed for vehicle live load. In most cases, maintenance vehicle H5 or H10 will be used (refer to *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges* for maintenance vehicle configurations).

However, in some locations pedestrian bridges are expected to carry emergency vehicles and construction live loads, such as firetrucks in rural areas where no other route is available. In such instances, pedestrian structures must be designed to carry CDOT Legal Load Type 3 (shown on Figure 31-1). The Designer must coordinate with Staff Bridge and the local authority to determine the type of live load required on each pedestrian bridge. The Structure Selection Report and bridge project special specification must discuss live load selection. No vehicle live load is required for bridges with clear widths equal to or less than 7 ft.

All pedestrian bridges designed to carry vehicle load must be rated, with the rating factor specified on the plans or shop drawings. Either the truss manufacturer or the Engineer of Record is expected to perform the rating. Rating requirements should be coordinated with Staff Bridge to determine the appropriate vehicles and load case assumptions.

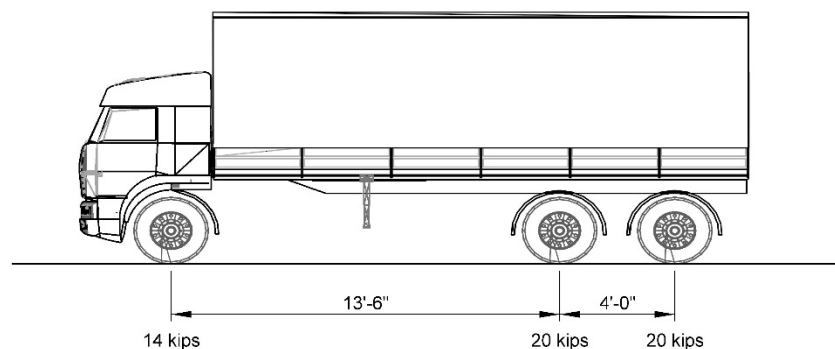


Figure 31-1: CDOT Legal Load Type 3

31.5.2 Collision

Vehicular collision load will not be considered in the structural design of the pedestrian bridge superstructure. However, all pedestrian bridges must be provided with the means to prevent the superstructure from sliding off the supports and onto the highway in case of collision. These means can include shear keys, keeper blocks, and anchor bolts at piers and abutments.

Design of the sliding prevention mechanisms can be done based on a concentrated 54 kips collision load applied at the support. Note that this load value is taken directly from AASHTO Table A13.2-1, as transverse collision load on traffic barrier at Test Level-4. No additional research or case studies were performed prior to publication to improve the accuracy of this value. The Designer must exercise engineering judgment when using this design method.

31.5.3 Deflection Limits

AASHTO *LRFD Guide Specifications for the Design of Pedestrian Bridges* outlines requirements for deflection limits of pedestrian bridges.

31.5.4 Vibration Limits

AASHTO *LRFD Guide Specifications for the Design of Pedestrian Bridges* outlines requirements for vibration limits of pedestrian bridges. However, in rare cases that experience unusually high pedestrian traffic loads, setting lower vibration limits is advised, such as bridges next to sport stadiums. The Designer is expected to exercise engineering judgment and consult similar projects.

31.6 FRACTURE CRITICAL DESIGNATION

Fracture critical members and welds shall satisfy provisions of AASHTO *LRFD Guide Specifications for the Design of Pedestrian Bridges* and be clearly identified on both the structural plans and the shop drawings. The reviewing engineer is responsible for identifying missing fracture critical designations while checking vendor shop drawings.

31.7 RAILING AND FENCING REQUIREMENTS

Pedestrian railings shall be designed in accordance with AASHTO *LRFD Guide Specifications for the Design of Pedestrian Bridges*. Handrails shall be provided for all stairs and ramps with grades greater than 5%. Refer to current ADA guidelines and Section 2.4 of this BDM for pedestrian and bicycle railing and fencing requirements.

31.8 COVERED/ENCLOSED STRUCTURES

Staff Bridge does not regulate the use of enclosed bridge structures. Local Agencies or the Landscape Architect can decide when to use them. However, whenever covered bridges are used, the roof of the enclosure should be designed to all applicable Local Agencies' loads and load cases, including the uplift wind forces. Because this topic is not covered by AASHTO, the Designer can use other applicable codes, such as IBC and ASCE 7 – Minimum Design Loads for Buildings and Other Structures.

31.9 DECK

Any available deck types, except steel grid, are allowed on pedestrian bridge structures. The Designer should consider the use of protection systems on all pedestrian bridge decks to extend the service life of the structure. Use of innovative materials is encouraged but must be discussed with Staff Bridge. All pedestrian bridge decks shall have non-skid surfaces.

31.10 LIGHTING

For pedestrian bridge lighting requirements, refer to Section 2.3.2 of this BDM.

31.11 DRAINAGE

Curbs shall be provided on both sides of pedestrian bridges that cross roads and highways to prevent water running over the sides. Drainage systems must be installed at bridge ends in combination with the curbs. Positive deck cross-slope may be used to facilitate drainage.

SECTION 32 SIGNS, LUMINAIRES, AND TRAFFIC SIGNALS

32.1 GENERAL REQUIREMENTS

This section provides guidance for the design and construction of signs, luminaires, and traffic signals. Such structures include but are not limited to:

- Bridge mounted signs.
- Ground mounted signs, including overhead sign bridges (that is, single span, multi-span), and cantilevered sign structures (that is, single sided, two sided/butterfly).
- Pole and wire systems for signs and traffic signals; and
- Poles for traffic lighting, luminaires, and traffic cameras.

Existing caissons are not to be reused for new sign structures unless they match the current standards or can be analyzed to verify they match any new loads from the current code.

32.2 CODE REQUIREMENTS

Unless modified herein, design of highway signs, luminaires, and traffic signals shall be in accordance with the most current edition of *AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (AASHTO LTS) or current M&S Standards. The M&S Standards take precedence, even though the current M&S Standards have not been updated to most current AASHTO LTS. If a design falls within the parameters of the M&S Standards, then the M&S Standards shall be used. A project specific or special design is only required when a project specific design does not meet the limits of the M&S Standard. When a project specific design is required, it shall follow the most current edition of AASHTO LRFD LTS. Where sign panels are changed or updated at existing sign structure locations, the original design assumptions may be used to evaluate the additional or revised signage.

When M&S Standard requires that the Contractor submit a design for the item, for example M-613-1 Roadway Lighting, then the design shall follow the most current edition of AASHTO LRFD LTS. A summary of Contractor submittals can be found in Table 105-1 of CDOT's Standard Specifications for Road and Bridge Construction.

Designs falling outside the limits of the S-Standards will require a special design. Sign structures are to follow the Major Structures requirements in the Structures Process outlined in Part E of the Policies and Procedures section of this BDM.

Due to concerns with fatigue, CDOT limits pole shapes to those that are round or have greater than or equal to the minimum number of sides defined in AASHTO LTS.

**AASHTO
LTS 5.6.2**

32.3 DESIGN CRITERIA

Designs shall follow AASHTO LTS for all design elements and include the following design clarifications.

32.3.1 Loads

32.3.1.1 Live Load

Live load shall be applied at the most critical locations to determine the design envelope.

**AASHTO
LTS 3.6**

32.3.1.2 Ice Load

Unless requested by the region, ice loading is not required. If ice load is to be applied due to the special icing requirement, consult the most recent edition of ASCE/SEI 07 for guidance.

**AASHTO
LTS 3.7**

32.3.1.3 Wind Load

AASHTO basic wind speeds cannot be directly compared to wind speeds used for design in the current M&S Standards. The code design factors, probability of exceedance, and/or averaging times associated with the wind velocities in the M&S Standards vary. If the member selection tables in the M&S Standards do not accommodate the given sign panel and span dimensions, the structure design shall use the wind loads described herein.

All CDOT on-system sign structures and signals should be considered high risk and can cross travel ways if they fail. Thus, the basic wind speed, V , used to determine the design wind pressure shall be 120 mph per the 1700 year Mean Recurrence Interval (MRI) drawing shown in AASHTO LTS Figure 3.8-2a and discussed in C3.8. The basic wind speed shall be used except in the following circumstances:

**AASHTO
LTS 3.8**

- Colorado's Special Wind Region (SWR) requires that the basic wind speed be calculated case by case. The western edge of the SWR follows the Continental Divide extending from the Colorado/Wyoming border south to the Colorado/New Mexico border. The eastern edge of the SWR is defined as a line extending from 5 miles west of I-25 at the Colorado/Wyoming border to 5 miles west of I-25 at the Colorado/New Mexico border, including all of Boulder County. A 300-year MRI shall be used to determine the basic wind speed and design wind pressure for all structures within this region.
- Figure 32-1 was developed from a partial SWR map for the northern section of the state. The southern portion of the map is a projection of the wind contours south to the border. An electronic Google Earth © version of this map is available and can be accessed by following this link: Colorado Gust Map.kmz. All data south of the "assumed data demarcation line" (39.39 degrees North) are assumed wind gust routes. Data are to be updated pending the completion of a wind gust study project.
- For special structures not noted previously, such as span wire signal structures, contact Unit Leader in coordination with the Signal SMEs for basic wind speed design values.

The alternate method for fatigue design per AASHTO LTS Appendix C shall not be used to determine alternative wind loads.

The effects of torsional load on caisson-soil interaction must be evaluated for all structures where torsional load is present due to wind load and structure geometry. This check is shown in Example 10 of this BDM.

Dampeners or other mitigation for galloping will be considered on a project basis since galloping is challenging to predict.

Table 32-1: Wind Speed Data at Other Mean Recurrence Intervals

Line Color	Mean Recurrence Interval						
	10 years	25 years	50 years	100 years	300 years	700 years	1700 years
Green	75 mph	85 mph	90 mph	95 mph	105* mph	115 mph	120 mph
Blue	85 mph	95 mph	100 mph	105 mph	120 mph	125 mph	135 mph
Yellow	90 mph	100 mph	110 mph	120 mph	130 mph	140 mph	150 mph
Peach	100 mph	110 mph	120 mph	130 mph	140 mph	150 mph	160 mph
Orange	110 mph	120 mph	130 mph	140 mph	155 mph	165 mph	175 mph
Red	115 mph	130 mph	140 mph	150 mph	165 mph	175 mph	190 mph
Pink	150 mph	165 mph	180 mph	190 mph	210 mph	225 mph	245 mph

* min value of 120 mph must be used in design

32.3.1.4 Fatigue Load

**AASHTO LTS
Section 11**

An infinite life fatigue design approach shall be applied for overhead sign structures, luminaire supports, and traffic signal structures.

32.4 BRIDGE MOUNTED STRUCTURES

**AASHTO LTS
Section 2**

Bridge mounted sign panels, signal systems, and luminaires are not permitted unless otherwise approved by Unit Leader.

If a bridge mounted sign, luminaire, or signal is approved by Unit Leader, it shall be positioned such that the bottom of the component is located a minimum of 2 in. above the bottom of the bridge girder to allow for sag and construction tolerances. For aesthetics, it is preferred that the sign structure not extend above the top of the bridge rail.

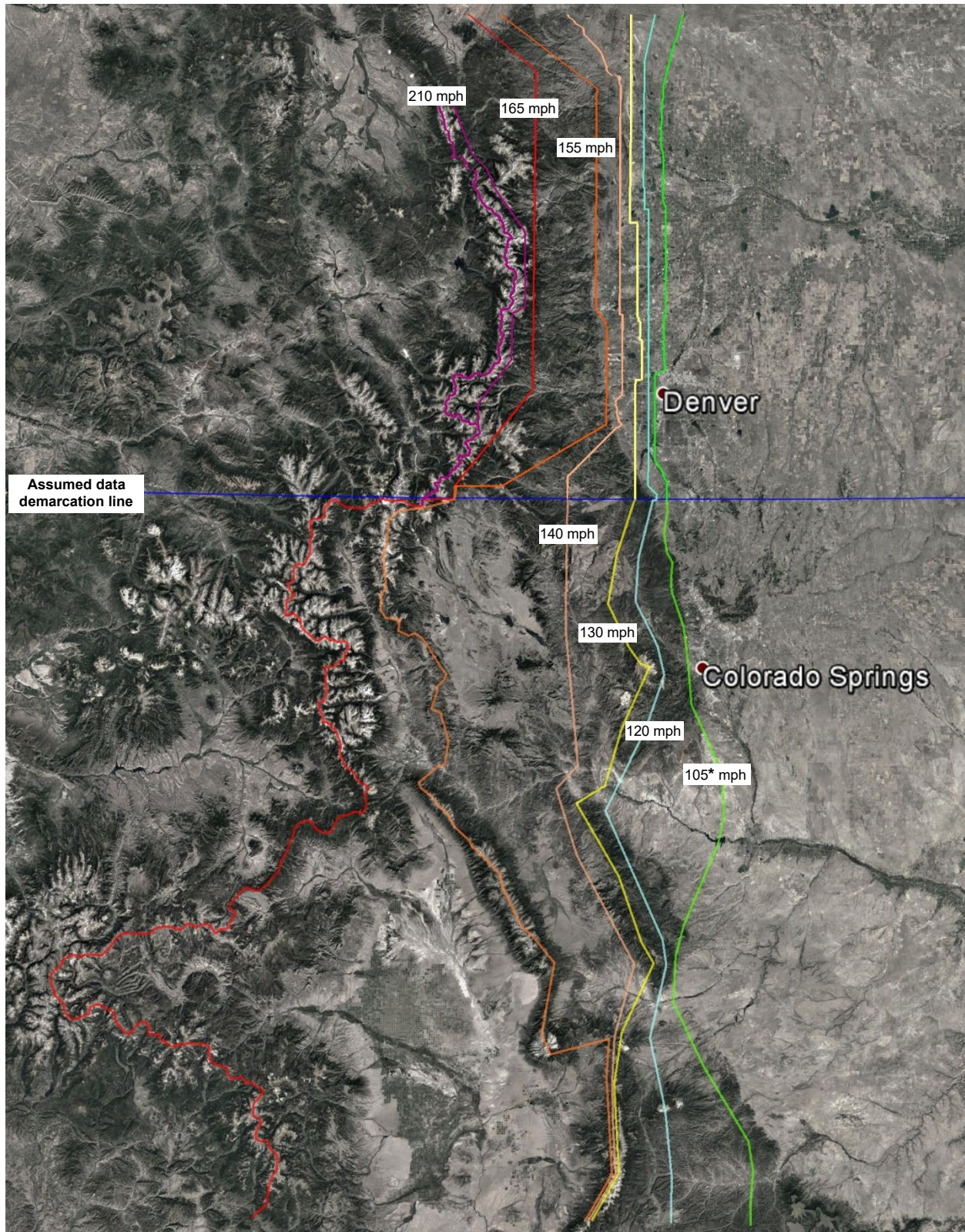


Figure 32-1: Partial Special Wind Region Map (300 year MRI)

(* min value of 120 mph must be used in design)

Unless the Traffic Engineer directs otherwise, place bridge mounted sign structures normal to an approaching vehicle's line of sight. For horizontally curved roadways below bridges, place bridge mounted sign structures normal to a 500-ft.-long chord that extends from the intersection of the centerline of travel lanes and the back face of the bridge barrier to a point on the centerline of travel way (see Figure 32-2).

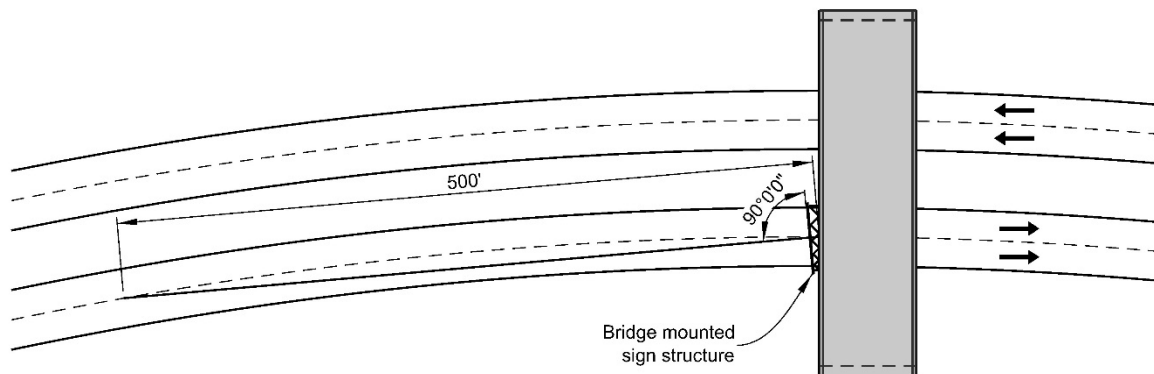


Figure 32-2: Sign Alignment for Curved Roadways

Expansion type concrete anchors are undesirable for attaching sign support brackets to the supporting structure because of vibration and pullout concerns. Instead, A307 or A325 bolts shall be used as through bolts or A307 all-thread rod may be used to make drilled-in-place anchor bolts bonded to the supporting concrete with an approved two-part epoxy system. If the anchor is in continuous tension, the Designer shall use only an epoxy system if it is approved for use in continual tension loading. Many epoxy systems are not allowed if the anchor is in continuous tension. Refer to ACI 318 and ACI 355.4 for more information on using post-installed adhesive anchors. Through and drilled-in-place anchor bolts can be used to resist direct tension and shear loads. Unless a refined analysis permits shallower anchorage, a minimum depth and diameter of drilled holes for bonded anchor bolts shall be 9 bolt diameters plus 2 in. and one bolt diameter plus 1/8 in. respectively. Bonded anchors bolts are 100 percent effective if the spacing and edge distance is equal to or greater than 9 bolt diameters and are considered to be 50 percent effective when the edge distance or spacing is reduced to 4.5 bolt diameters. Edge distances and spacings less than 4.5 bolt diameters are not allowed.

Use cast-in-place A307 J-bolts for new concrete work.

When an approved proprietary bolting system is specified, add the following note to the plans:

“The bolting system is to be installed using the manufacturer’s recommendations.”

When an approved two-part epoxy system is specified, add the following note to the plans:

“The two-part epoxy system shall be installed using the manufacturer’s recommendations.”

For torque limits for all through bolts and tension limits due to permanent service dead load for bonded anchor bolts, see Table 32-2. Use interpolation for values not shown in the table.

Table 32-2: Torque and Tension Limits

Bolt Type	Bolt Diameter	Torque (ft-lbs)		Tension Limit (lbs)
		Dry	Lubed	
A307*	0.50	25	20	1400
	0.75	85	60	3300
	1.00	200	150	6000
A325*	0.50	70	50	–
	0.75	240	180	–
	1.00	350	265	–

* A36 may be substituted for A307; A449 may be substituted for A325.

SECTION 33

PRESERVATION AND REHABILITATION OF STRUCTURES

33.1 GENERAL REQUIREMENTS

The provisions of this section apply to structure preservation and rehabilitation projects, as defined herein.

33.1.1 Definitions of Preservation and Rehabilitation

Preservation and rehabilitation projects can be categorized into two primary groups based on the general scope of the work performed and the expected improvement to structure condition and structure lifecycle.

33.1.1.1 Bridge Preservation

The FHWA defines bridge preservation as “actions or strategies that prevent, delay or reduce deterioration of bridges or bridge elements, restore the function of existing bridges, keep bridges in good condition and extend their life” (2011). Preservation includes bridge maintenance activities (both preventive and reactive), as well as major preservation work.

Bridge maintenance projects are typically narrow in scope and restore the structure to its original condition by addressing existing deficiencies. These projects have minor costs and require minimal new design work. Example work types are crack sealing, concrete patching, debris clearing, and joint repair.

Preservation involves the repair and protection of a bridge element against future deterioration, thereby extending the service life of a bridge without significantly increasing load-carrying capacity or improving geometrics.

Preservation projects typically cost less than 30 percent of the cost of a new replacement bridge.

33.1.1.2 Bridge Rehabilitation

Bridge rehabilitation involves a significant investment in a bridge to improve its condition, geometrics, or load-carrying capacity to a minimum standard. This work is expected to provide a long-term benefit and reduce the need for additional investments. Projects that cost more than 30 percent of the cost of a new bridge are generally considered rehabilitation projects. Deck replacements, bridge widenings and superstructure replacement projects are considered rehabilitation projects regardless of estimated costs.

Bridge replacement should be considered if the cost of rehabilitation approaches or exceeds 70 percent of the cost of a new replacement bridge. The final determination on rehabilitation vs. replacement should be based on many factors, as discussed in the following sections.

33.1.2 Rehabilitation vs. Replacement Selection Guidelines

The following factors should be considered when deciding between rehabilitation and replacement for a structure. It should be noted that these are not absolute criteria for investment decisions. Because each project is unique, all circumstances and constraints should be considered during evaluation.

33.1.2.1 Cost

In conjunction with the CDOT Project Manager, the Designer shall coordinate the development of an appropriate comparison of the total project cost estimates for both rehabilitation and replacement options. Comparison of total project costs (including any anticipated costs associated with phasing, realignment, detours, environmental concerns, right-of-way acquisition, etc.) is necessary to determine the most cost-effective alternative. Rehabilitation and replacement costs should be estimated after all other factors have been investigated because the other factors may affect or determine the scope of the rehabilitation or replacement project.

As the estimated cost of the rehabilitation project approaches 70 percent of the cost of the replacement project, replacement becomes the more cost-effective choice in terms of life-cycle costs. This threshold is based on life-cycle cost models of rehabilitation and replacement for various bridges and is consistent with thresholds adopted by other state agencies.

As an alternative to using the above threshold, a refined life-cycle cost analysis may be performed. In this case, estimated life-cycle costs for rehabilitation and replacement options should be compared directly; applying the 70 percent factor when dealing with life-cycle costs is not appropriate. For more information about bridge life-cycle cost analysis, see NCHRP Report 483, "Bridge Life-Cycle Cost Analysis."

33.1.2.2 Safety

Accident history should be considered for the existing structure. Accident potential should be considered for both existing and potential replacement structures. If the accident history or potential of the existing structure is determined to be unacceptable, the safety problem must be addressed either through rehabilitation or replacement. Rehabilitation costs associated with safety improvements shall be included in the rehabilitation estimate for comparison to replacement cost.

33.1.2.3 Structure Type

Certain bridges will be inherently predisposed to either rehabilitation or replacement based on their type and location. Structure types that are difficult or costly to rehabilitate may be stronger candidates for replacement. Special consideration should be given to the replacement of non-redundant bridges because they present increased maintenance costs and risk.

Historical significance may be a factor in favor of rehabilitation. For historical bridges that will be kept in the system, the Secretary of the Interior's Standards for Rehabilitation shall be consulted and close coordination with the Environmental group will be required throughout the project.

33.1.2.4 Bridge Standards

Existing vertical clearance, horizontal clearance, lane width, and shoulder width should be considered. If the existing features are nonstandard, consideration should be given to improving them through rehabilitation or

replacement. Substandard geometry that cannot be reasonably addressed through rehabilitation is a factor in favor of replacement.

33.1.2.5 Hydraulic Performance

The hydraulic history of the bridge should also be reviewed. If the existing features are nonstandard, consideration should be given to improving them through rehabilitation or by replacing the bridge. Up and downstream impacts should be considered because the hydraulic implications of rehabilitation or replacement can push the decision in either direction.

Scour critical bridges for which there are no feasible countermeasures to mitigate the scour problems are stronger candidates for replacement.

33.1.2.6 Traffic Control

In some cases, practical solutions for temporary traffic control may drive the rehabilitation vs. replacement decision. For example, if project specifics prohibit temporary traffic configurations that could accommodate bridge replacement, rehabilitation may be the reasonable decision.

33.1.2.7 Environmental

Environmental impacts should be estimated for rehabilitation and replacement option and considered in the rehabilitation vs replacement decision.

33.1.3 Required Inspection and Testing

During the project scoping phase and before developing preliminary cost estimates, the Designer should conduct a field visit after general drawings have been developed to verify the deficiencies noted on the Structure Inspection and Inventory Report (SIA) and to document any additional issues that should be addressed or might require further testing and analysis. The Designer shall review the recommended maintenance activities and expand on them, if necessary. The Designer should verify and address the fundamental issue that caused the structure to be targeted for rehabilitation or replacement.

Chloride testing is required during the scoping phase for any project with new full-depth overlay replacements, deck widening or deck rehabilitation/repair. A minimum of 5 cores, but not less than 1 per 3,000 square feet of bridge deck, are required to be taken and tested. The cores shall be evenly distributed over the travel lanes. At a minimum, the chloride content at the level of the top mat of reinforcing must be determined. This requirement can be waived for bridge decks less than 20 years old that have been continuously protected throughout the life of the bridge deck by a thin bonded epoxy overlay, a polyester concrete overlay, or a functioning waterproofing membrane and asphalt wearing surface. This exception is granted under the assumption that these decks have not been critically contaminated with chlorides.

On partial-depth resurfacing projects, the thickness of the existing asphalt mat should be verified to prevent damage to the waterproofing membrane if applicable and to prevent damage to the bridge deck from the milling operations. One method of verifying the existing asphalt thickness is by drilling or coring into the asphalt mat down to the deck surface and measuring the asphalt thickness with a probe. The plan-view location of each measurement

and the asphalt thickness should be recorded in an organized format and submitted to the Engineer prior to milling. Figure 33-1 shows an example asphalt thickness verification detail prior to milling and after the resurfacing.

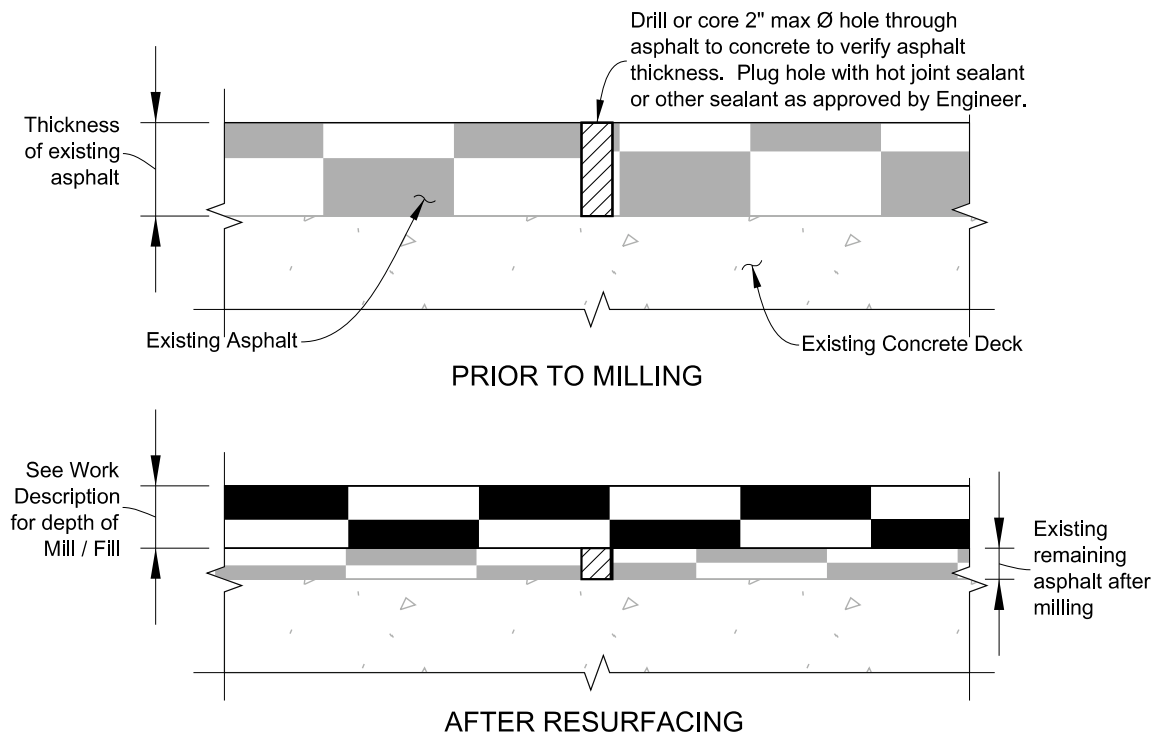


Figure 33-1: Asphalt Thickness Verification Detail

33.2 CODE AND PERFORMANCE REQUIREMENTS

The following provisions apply to all preservation and rehabilitation projects.

33.2.1 Existing Structure Evaluation and Preservation Projects

This section defines the acceptable design methodologies, codes, and minimum performance requirements to be used for both preservation projects (as defined in Section 33.1.1.1) and when evaluating an existing structure to determine if repair or rehabilitation measures are necessary. This includes existing structures that are being evaluated for scour criticality or increased dead load and structures with measured corrosion, section loss, or other damage in superstructure or substructure elements. Permanent load increases of 3 percent or less over what the bridge was originally designed for may not require analysis or rating, at the Designer’s discretion.

33.2.1.1 Code Requirements

Structures designed per AASHTO LRFD shall be evaluated using AASHTO LRFD.

Structures designed by LFD or ASD methods may be evaluated with either the AASHTO Standard Specifications or AASHTO LRFD.

It is appropriate and acceptable to analyze older structures with the AASHTO Standard Specifications. However, in some cases, an LRFD analysis may yield

more favorable results due to more refined methods of live load distribution or structural capacity. The intent of this provision is to not preclude the use of LRFD in these situations. A structure found to meet the minimum performance criteria when checked with either code should be considered acceptable.

When projects in this category require the design of a new element or retrofit, it is preferred to use AASHTO LRFD, when practical.

If existing caissons meet the current S-Standard requirements and the anchor bolts are in good condition, i.e. minimal corrosion, no loss of capacity, they may be reused for new sign structures.

33.2.1.2 Required Documentation and Minimum Performance Criteria

For existing structure evaluations, a rating summary sheet shall be completed for the element(s) under investigation using the applicable design code. Super- and substructure ratings shall be completed and documented in accordance with the CDOT Bridge Rating Manual and the Technical Rating Memorandum dated February 10, 2017. Additionally, for applicable substructure load combinations beyond the standard rating equations, performance ratios shall be reported separately.

Acceptable performance objectives for existing structure evaluations are as follows:

- Operating rating factor ≥ 1.0
- White color code
- Performance ratios for other load combinations ≥ 1.0

If all the above criteria are met, generally, no action needs to be taken or scour critical designation applied. When scour is involved, the operating rating factors and performance ratios typically refer to substructure elements affected by the scour, e.g. pile or caisson capacity.

If any of the above criteria are not met, it is not necessarily cause for action. The ratings of the element(s) under investigation shall be compared to the overall load rating of the bridge. In some cases, the overall bridge rating will not be controlled by the elements that required special investigation. If the overall rating is controlled by a substructure element, repairs are typically desired before making posting decisions. In all cases, existing structure evaluation results that do not meet the above criteria shall be discussed with Staff Bridge to determine the appropriate course of action. If a substructure is determined to be scour critical, refer to Section 33.13 for more information.

33.2.2 Rehabilitation Projects

This section defines the acceptable design methodologies, codes, and minimum performance requirements to be used for rehabilitation projects, as defined in Section 33.1.1.2. Because rehabilitation projects represent a substantial investment in an existing structure, they are subject to more stringent performance criteria to help ensure that they meet service life extension goals commensurate with their level of investment.

33.2.2.1 Code Requirements

All structures for rehabilitation projects shall be evaluated and/or designed using AASHTO LRFD regardless of original design code unless previous documentation in the Structure Selection Report and approval by Unit Leader (per Section 1.3 of this BDM).

33.2.2.2 Required Documentation

A Load and Resistance Factor Rating (LRFR) summary sheet shall be completed for the super- and substructure, as required. Super- and substructure ratings shall be completed and documented in accordance with the CDOT Bridge Rating Manual and the Technical Rating Memorandum dated 02/10/2017. For applicable substructure load combinations beyond the standard rating equations, performance ratios shall be reported.

For rehabilitation projects where no additional load is transferred to the substructure, and the substructure is otherwise performing adequately and has an NBI rating of 6 or greater, no analysis or rating of the existing substructure is required. Permanent load increases of 3 percent or less over what the original bridge was designed for may not require analysis or rating, at the Designer's discretion.

Note that changes in superstructure continuity or boundary conditions can alter the distribution of forces and impose additional load on some substructure units. Such changes in load distribution shall be considered when determining if a substructure rating is required for a rehabilitation project.

33.2.2.3 Minimum Performance Criteria – Excluding Deck Replacements and Existing Portions of Bridge Widening

For rehabilitation projects, excluding deck replacements, the inventory rating factor and all performance ratios shall be 1.0 or greater.

33.2.2.4 Minimum Performance Criteria for Deck Replacements

For deck replacement projects, the inventory rating factor shall be 0.9 or greater. The reduced minimum inventory rating accounts for the fact that some of the service life of the structure has already been realized. The new deck shall meet all AASHTO requirements.

For deck replacement projects where additional load is transferred to the substructure, the inventory rating of the substructure shall be 0.9 or greater. For load combinations not including live load, the performance ratio shall be 1.0 or greater.

33.2.2.5 Minimum Performance Criteria for Existing Portions of Bridge Widening

Acceptable performance objectives for the existing portion of a widened structure are as follows:

- Operating rating factor ≥ 1.0
- No required posting
- White color code

- Performance ratios for other load combinations ≥ 1.0
- Superstructure, substructure, and deck condition ratings of 6 or greater

If the existing portion does not meet these performance objectives, the structure should be evaluated for strengthening and/or repair to the same load-carrying capacity as the widened portion. For the evaluation, the following should be considered, as appropriate:

- Cost of strengthening or repairing the existing structure
- Physical condition, operating characteristics, and remaining service life of the structure
- Other site-specific conditions
- Width of widening
- Traffic accommodation during construction

The final decision on whether the existing portion requires rehabilitation, and what it should include, shall be coordinated with the Region and Unit Leader.

33.3 REHABILITATION

33.3.1 General Requirements

The rehabilitated structure shall have a fair or good NBI condition rating after rehabilitation.

Rehabilitation projects should seek to eliminate functional obsolescence if reasonable. For example, if widening a bridge, the width should be increased enough to accommodate standard roadway geometry, where feasible.

If a structure is functionally obsolete for reasons that cannot be easily addressed through rehabilitation, structure replacement should be considered rather than making further investments in a functionally obsolete structure through a rehabilitation project. The ability to address functional obsolescence during structural rehabilitation is highly project specific.

33.3.2 Added Service Life

The following are target service life extensions for various types of rehabilitation and preservation:

- Estimated deck service life
 - Terminal decks (condition rating 3 or less) with minor patching and bituminous overlay: 2 to 5 years
 - Deck to remain in place with protective measures: 20 years for deck
- Membrane waterproofing and bituminous overlay. The life of the bituminous overlay may be 10 to 12 years. The membrane may need to be replaced each time the overlay is replaced if it has been damaged or is otherwise performing poorly.

- Polyester concrete overlays, cathodic protection, and rehabilitation of other deck types: 15 to 25 years depending on traffic volume and prior condition of deck
- New concrete deck with epoxy-coated reinforcement: 50 years
- Expansion joint end dams
 - Same as deck – periodic replacement of glands or trough should be expected
- Beam end repairs and/or rehabilitation
 - Minimum: Same as deck
 - Desirable: 50 years
- Repair and/or rehabilitation of other superstructure types and their elements
 - Minimum: Same as deck
 - Desirable: 50 years
- Bearings
 - Same as the existing girders
- New superstructure
 - Minimum: 50 years
 - Desirable: 75 years
- Substructure rehabilitation
 - Same as superstructure
- Retaining walls
 - Minimum: 25 years
 - Desirable: 50 years
- Culverts
 - Minimum: 15 years
 - Desirable: 50 years
- Bridge widening
 - Minimum: 50 years
 - Desirable: 75 years
- Sign structures
 - Minimum: 25 years
 - Desirable: 50 years
- Ground-mounted sound barriers
 - Minimum: 15 years
 - Desirable: 40 years
- Structure-mounted sound barriers
 - Same as deck

- Temporary bridges
 - 3 to 5 years

33.3.3 Acceptable Methods

Many systems and products can be effectively used for rehabilitation, including, but not limited to, the following.

33.3.3.1 Micropiles

Micropiles are commonly used for a range of retrofit or rehabilitation purposes, including:

- Arresting or preventing structure movement
- Increasing load-bearing capacity of existing foundations
- Repairing or replacing deteriorating or inadequate foundations
- Adding scour protection to existing structures

Micropiles are well suited to projects with the following constraints:

- Restrictions on footing enlargements
- Low overhead clearances
- Difficult access

33.3.3.2 External Post-tensioning

External post-tensioning (PT) may be considered for retrofit of all girder or other structural elements, including concrete and steel. Active strengthening systems, such as external PT, introduce external forces to the structural elements that would offset part or all the effects of external loads. Active systems are usually engaged in load sharing immediately after installation and can provide increased strength and instantaneously improve the service performance, such as reducing tensile stresses (or cracking) and deflections.

An advantage of external PT is that it needs to engage the structure only at end anchorages and at points of tendon deviation. For this reason, external PT can be added to existing structures with relative ease. Both steel and concrete box girders can usually accommodate the necessary anchorages and tendon deviations from inside the box. Monostrands require relatively small anchorage forces on a per tendon basis, thereby allowing simplified anchorage and deviation details on the retrofitted structure.

33.3.3.3 Carbon-fiber Reinforced Polymer

Passive strengthening systems, such as Carbon-fiber Reinforced Polymer (CFRP), do not introduce forces to the structure or its components. Passive systems contribute to load sharing and the overall resistance of the member when it deforms under external loads. As such, the effectiveness and load sharing of passive systems significantly affect their axial and bending stiffness.

CFRP features include a slim profile, high strength to weight ratio, chemical resistance, and ease of application. These attributes can lead to long-lasting, inexpensive, and rapid restorations that can be implemented in the field with

minimal disturbance to traffic flow. Lastly, the structure's original configuration, including vertical and horizontal clearances, is maintained.

ACI 440.2R-08, "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures," provides guidance for the design and construction requirements of CFRP retrofits.

33.3.3.4 Ultra-high Performance Concrete

Ultra-high performance concrete (UHPC) exhibits high early strength, develops a strong bond to existing concrete surfaces, and has enhanced durability. These characteristics make it an acceptable candidate for repair and rehabilitation work such as concrete patching, closure pours, and toppings.

33.3.4 Timber Structures

Due to the difficulty of finding new girders, durability and challenges of crash worthiness, widening of timber structures is generally not recommended.

33.3.5 Concrete for Repairs

Concrete Class DT shall be used only for complete toppings.

The current CDOT Project Special Provision – Revision of Section 601 Concrete Class DR – allows the use of either pre-packaged concrete patching material (bagged mix) or plant batched mix, giving the Contractor the ability to select the most economical and practical choice for the project. However, the Designer should be aware that certain circumstances may necessitate the use of a bagged mix only.

Patch repairs on bridge decks present logistical complications. Because of traffic control implications, deck repairs are often performed at night when batch plants are not operating. In this case, a bagged patching mix must be used. Additionally, total patch volume is commonly much less than the smallest volume able to be batched (2 cubic yards), resulting in waste.

Projects involving night-time lane closures may also benefit from the use of a bagged patching mix because of the reduced cure time compared to Class DR. A bagged mix can accommodate traffic loading in as little as 3 hours, where Class DR requires 6 hours. This time constraint is especially restrictive when replacing expansion devices and end dams because these projects require the completion of time intensive tasks during the closure, thereby limiting the time available for concrete curing. If a night-time closure cannot accommodate the required cure time before reopening to traffic, temporary bridge decks must be used. Temporary bridge decks may require the placement of extensive asphalt ramps and have experienced other difficulties in the field. For these reasons, a bagged mix is typically preferable for deck patching and placement of new expansion joint end dams.

The current policy of allowing either a bagged mix or Class DR may be revised in the future if either option proves to have superior durability.

33.4 BRIDGE WIDENING

Bridge widening represents a substantial investment in an existing structure and presents many unique challenges and opportunities for improvement. See Section 33.2.2 for required design code and performance objectives for the new and existing portions of widened bridges.

33.4.1 General Widening Requirements

The new portion of a widened structure shall comply with the following requirements:

- The bridge should be widened sufficiently to accommodate standard lane and shoulder widths, where feasible.
- Longitudinal deck joints are not permitted due to durability concerns.
- Fatigue-prone details should not be perpetuated.
- Mixing steel and concrete girders in the same span should be avoided due to thermal movement incompatibility.

33.4.2 Design Considerations

33.4.2.1 Differential Superstructure Stiffness

Live load distribution factors given in AASHTO 4.6.2.2 for beam-slab bridges are conditional upon the beams having approximately the same stiffness. Widening a bridge with a girder shape different from the existing girders may require a more refined analysis to determine accurate live load distribution and to verify the design loads for the deck between the new and existing girders.

Generally, the Designer should attempt to limit the amount of differential deflection between the widened and original portions of the superstructure, where feasible. The Designer shall account for additional forces and stresses due to any differential deflection anticipated along the widening interface.

33.4.2.2 Differential Superstructure Creep and Shrinkage

Newly placed prestressed concrete will shorten due to long-term creep and shrinkage. When connected to an existing concrete structure that has already experienced most of its creep and shrinkage, the existing structure will restrain the shortening of the new structure to some degree. This restraint causes forces along the widening interface that shall be considered in design.

Similarly, differential strains of the superstructures can result in force effects at the interface between the existing and new substructures. Isolating the existing and new substructures is a potential strategy to mitigate this issue.

33.4.2.3 Differential Foundation Stiffness

When a structure widening includes widening the substructure and foundation elements, the compatibility of the new and existing foundation systems should be considered. If the new and existing foundations have substantially different stiffness, a differential deflection or settlement can be expected. This effect should be considered and minimized, particularly as it relates to imposed deformation and stresses on the superstructure.

The effect of initial settlement of the new foundation elements relative to the existing foundation should also be considered. This phenomenon can be expected even where the widened foundation is of similar type and stiffness to the existing foundation. Isolating the existing and new substructures is a potential strategy to mitigate this issue.

33.4.2.4 Closure Pours

Closure pours shall be used between the existing and new portions of deck when the dead load deflection due to deck placement is greater than 0.25 in.

The width of closure pours should be a function of the amount of differential deflection expected and a minimum of 24 in. for conventional concrete. The width of the closure pour may be less than 24 in. if UHPC is used in conjunction with a wearing surface to smooth out any abrupt differences in elevation on either side of the closure.

33.4.2.5 Galvanic Anodes

When a bridge widening includes exposing and lapping onto existing uncoated reinforcing steel in the deck or any other element that may be contaminated with chlorides, consideration shall be given to the use of galvanic anodes along the widening interface.

If the concrete of the existing bridge deck is sufficiently contaminated with chlorides and galvanic anodes are not used, corrosion along the existing-new concrete boundary can initiate or accelerate. See Section 33.5.1 for more information.

33.5 BRIDGE DECK REPAIR AND REHABILITATION

33.5.1 Chloride Induced Corrosion

Infiltration of chloride ions into concrete is the most common cause of corrosion initiation in reinforcing steel. Bridge decks in Colorado are primarily exposed to chloride ions through the application of deicing salts, such as magnesium chloride.

Once the concentration of chloride ions at the level of reinforcing reaches a critical threshold, the protective passive film surrounding the reinforcing breaks down and corrosion initiates. While the subsequent rate of corrosion depends on many parameters, including several environmental factors, some level of corrosion will be observed until the concentration of chloride ions is reduced to below the threshold through remedial measures.

Several options are available for repair and rehabilitation of chloride contaminated concrete structures, including, but not limited to:

1. Do nothing.
2. Remove spalled and delaminated concrete and replace with patching material.
3. Remove all chloride contaminated concrete and replace with patching material (this includes sound but chloride contaminated concrete).

4. Use electrochemical chloride extraction (ECE) to remove chloride from the surface of the reinforcing bars.
5. Install a barrier system.
6. Install cathodic protection to protect the steel from further corrosion.

Repair and rehabilitation options involving concrete patching introduce additional complications. The process of patching unsound and/or chloride contaminated areas of existing decks requires placing new chloride-free concrete adjacent to existing concrete. If the existing concrete has a sufficiently high chloride concentration level, the patching process will lead to the formation of incipient anodes just outside the patched area. The difference in electric potential between the steel in the chloride-free and chloride contaminated sections drives corrosion at the incipient anodes, accelerating deterioration of the adjacent concrete. Rapid deterioration of the concrete surrounding the patch necessitates future repairs, creating a compounding maintenance and service issue. This phenomenon is commonly referred to as the halo effect.

Installing a barrier system (i.e., waterproofing membrane and wearing surface) on a deck that is chloride contaminated but not yet showing signs of distress may be ineffective. If the chloride concentration is at or near the threshold, corrosion of reinforcing will continue, resulting in deck deterioration. The damage occurring in the deck may become apparent only after significant damage has occurred under the overlay. In this scenario, the expected service life of the barrier system will likely not be realized.

For these reasons, projects that will include deck repair, patching, or installation of new waterproofing membrane and overlay should first identify the chloride contamination of the deck before determining viable rehabilitation methods. See Section 33.1.3 for requirements on coring and chloride testing of existing bridge decks.

33.5.2 Susceptibility Index

The first step in selecting a corrosion control system is to identify if local systems will suffice. If not, appropriate global systems must be identified. To determine the appropriateness of a local or global system, the distribution of chloride ions needs to be determined. NCHRP Report 558, "Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements," proposed a quantitative method for determining viable corrosion control alternatives that includes calculating a Susceptibility Index (SI) for the structure.

Chloride testing results are required to calculate the SI of the structure.

The distribution of chloride ions at the steel depth should be used to quantify both the susceptibility of the concrete element to corrosion in areas that are not currently damaged and the future susceptibility to corrosion-induced damage. If sufficient chloride ions are present to initiate corrosion, then corrosion-induced damage in the near future is expected, and only aggressive corrosion mitigation techniques, such as cathodic protection and

electrochemical chloride extraction, can be used to control the corrosion process. However, if the chloride ion concentration distribution at the steel depth is low and future corrosion is not expected to initiate, less expensive corrosion control systems—such as sealers, membranes, and/or corrosion inhibitors—can be used to either control or stop the rate of corrosion. Therefore, an index that provides a good representation of the distribution of chloride ions at the steel depth is useful in selecting a corrosion control system.

The SI shall be calculated as follows:

$$SI = \left[\left(\sum_{1}^n (Cl_{th} - X_i) \right) / (n \times Cl_{th}) \times 10 \right]$$

Where

Cl_{th} = Chloride concentration threshold

X_i = Chloride concentration at the i th location at the depth of reinforcing

n = number of locations where measurements were made

The chloride concentration threshold depends on many factors but may be assumed to be 1.2 lbs/CY of concrete (or 0.03 percent chloride by weight), for uncoated reinforcing, if no better information is available.

The SI is a scaled ratio of the average moment from the threshold normalized by the threshold. An SI of 10 means that no chloride ions exist at reinforcing depth for any test location. The SI is 0 if the chloride concentration at every location is equal to the threshold. A negative SI indicates that corrosion has initiated at most tested locations and that deterioration of the deck, even in currently sound areas, is expected.

33.5.3 Selection of Corrosion Control Alternatives

Once the SI of a structure has been calculated, corrosion control alternatives can be evaluated and selected. A lower SI, which corresponds to higher levels of chloride contamination, requires a more aggressive corrosion control system.

Most corrosion control systems, including those described in the following sections, are intended for use with uncoated (black) reinforcing. For concrete elements with epoxy-coated reinforcing, the Designer shall select a compatible corrosion control system. Any damage to the epoxy coating in the repair area should be repaired. NCHRP Web Document 50, "Repair and Rehabilitation of Bridge Components Containing Epoxy-Coated Reinforcement," provides guidance on the repair and rehabilitation of concrete with epoxy-coated reinforcing.

Selection of corrosion control systems must also consider the desired service life of the rehabilitated element to avoid unnecessary expenditures. For example, structures programed for replacement within the next 10 years may not be good candidates for a cathodic protection system that could be expected to last up to 25 years.

Figure 33-2 shows the optimal corrosion control systems for a given SI. See Section 33.5.3.1 through Section 33.5.3.7 for more information.

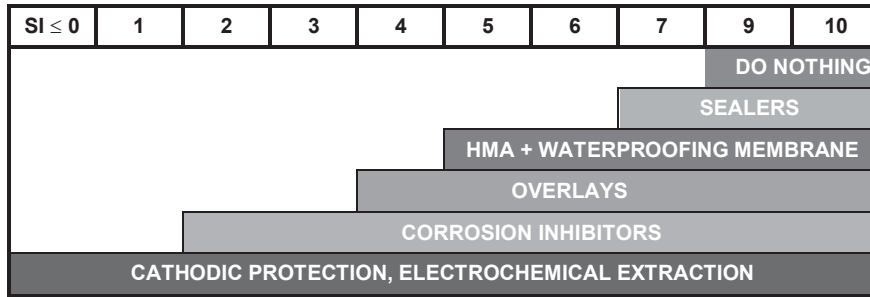


Figure 33-2: Optimal Corrosion Control Based on Susceptibility Index

The control systems shown in Figure 33-2 are intended to be used in conjunction with the removal and patching of spalled and delaminated concrete. Consideration should also be given to removing and patching all chloride contaminated concrete, in addition to spalled and delaminated concrete. There is a risk of corrosion initiating or continuing in the original concrete if contaminated concrete is left in place. For example, if a polyester concrete overlay is installed over sound but chloride contaminated concrete, corrosion may still occur, resulting in deterioration of the original concrete. This may compromise the newly placed overlay, resulting in a reduced effective service life and necessitating future repairs.

33.5.3.1 Do Nothing

SI values greater than or equal to 8.0 indicate that a corrosion control system is not necessary.

33.5.3.2 Sealers

For the purposes of Figure 33-2, a sealer is defined as any coating that is “breathable,” that is, capable of limiting the flow of moisture into the concrete but still allowing the flow of moisture out of the concrete. CDOT commonly uses an alkyl-alkoxy silane sealer. Sealers are an acceptable form of corrosion control for decks with SI values of 7.0 or greater.

33.5.3.3 Membranes

Membranes are differentiated from sealers in that they restrict the movement of moisture in either direction and do not allow chloride intrusion. The membrane category includes asphalt wearing surfaces over a waterproofing membrane and thin-bonded epoxy overlays. As shown in Figure 33-2, membrane type corrosion control systems can be used as the primary form of protection when the SI is 5.0 or greater. For decks with an SI less than 5.0, a membrane may be used in conjunction with more aggressive corrosion control systems.

33.5.3.4 Overlays

Overlays include both cementitious and non-cementitious wearing surfaces installed on the deck surface. Polyester concrete overlays fall into this category. Asphalt wearing surfaces are not considered overlays (in terms of

corrosion protection) because they do not serve as barriers to moisture and chloride ions. Overlays limit corrosion by reducing the rate of chloride and water diffusion into the deck and by increasing the depth to which chlorides must diffuse to reach the reinforcing. The result is an increased time to initiation of corrosion. Overlays also serve as a wearing surface.

As shown in Figure 33-2, overlays can be considered the primary form of corrosion protection when the SI is 4.0 or greater or can be used in conjunction with more aggressive corrosion control systems for lower SI values.

33.5.3.5 Corrosion Inhibitors

Corrosion inhibitors include any material that chemically slows or stops the corrosion process. Inhibitor systems can be surface applied or admixed with repair concrete. Deck repairs on structures with an SI less than 4.0 should include corrosion inhibitors or a more aggressive corrosion control system. Corrosion inhibitors are not commonly used in Colorado.

33.5.3.6 Electrochemical Chloride Extraction

Electrochemical chloride extraction (ECE) is a short-term treatment of the bridge deck that lowers the chloride levels in the bridge deck to an acceptable level. Removing chloride ions increases the alkalinity at the surface of the reinforcing, which re-passivates the reinforcing and prevents future corrosion from initiating. ECE is not commonly used in Colorado.

33.5.3.7 Cathodic Protection

Cathodic protection systems include galvanic systems and impressed current systems and can be used in conjunction with other corrosion control systems.

Cathodic protection is a rehabilitation technique that has been proven to stop corrosion in chloride contaminated bridge decks (Sohanghpurwala, 2006). However, it is appropriate for use only on structures with SI values less than 2.0 and is most cost-effective for structures where a service life extension of greater than 15 years is desired.

One acceptable form of cathodic protection is the application of galvanic anodes in the patch area. The galvanic anodes corrode sacrificially themselves, reducing the corrosion in the reinforcing itself. The size and spacing of anodes should be selected to provide the desired service life of the repair. When no better information is available, CDOT has commonly specified 100 gram anodes at 18 in. to 24 in. spacing along the interface.

33.5.3.8 Complete Topping Replacement

Rehabilitation options that involve removing and replacing the top layer of concrete in its entirety may be more cost-effective than patching each damaged area individually. This type of repair can be performed using hydrodemolition or standard methods of concrete removal. When the depth of replacement is selected such that all chloride contaminated concrete is removed, this type of repair also serves as a method of corrosion control. The cause of the corrosion (chlorides) has been removed and, therefore, no other corrosion control system is necessary. However, because of the relatively high

cost of this type of repair, it is discouraged for decks that require minimal patching or have an SI of 5.0 or greater.

33.6 CONCRETE REHABILITATION – EXCLUDING BRIDGE DECKS

Other concrete elements besides bridge decks can be exposed to chlorides throughout their service life. This includes abutments, piers, and walls within the splash zone, as well as elements exposed to chlorides due to leaking expansion joints.

Concrete repairs required on elements within the splash zone or due to damage caused by leaking expansion joints should include galvanic anodes at the patch interface to mitigate the halo effect and protect the surrounding concrete from accelerated corrosion. Depending on the element and its risk to continued exposure to chlorides, the addition of a membrane or sealer may also be appropriate.

33.7 DECK REPLACEMENT

Deck replacement projects can be a cost-effective means of extending the service life of a bridge when a deck has deteriorated beyond what can be reasonably repaired but the remainder of the structure is otherwise performing well and has no underlying deficiencies. They also present opportunities to strengthen the superstructure, upgrade bridge rail, and move or eliminate expansion joints. However, due to their cost, these projects should be considered carefully to ensure that completed structures do not result in the continuation of substandard conditions (such as insufficient clearances or roadway geometry) that would need to be addressed during the anticipated life of the new deck.

Deck replacement projects should implement the following improvements, where feasible:

- Make the new deck composite with the girders to increase capacity.
- Eliminate any existing longitudinal deck joints.
- Provide a deck with 8 in. minimum thickness.
- Eliminate expansion joints at abutments and/or throughout the structure. See Section 33.8 for more information on expansion joint removal.

If the weight of the proposed deck and attachments causes the load rating of the girders or substructure to fall below the minimum acceptable rating as defined in Section 33.2.2.4, the following measures may be considered to reduce dead load:

- Specify a lighter wearing surface (either a $\frac{3}{4}$ in. minimum polyester concrete or $\frac{3}{8}$ in. thin-bonded epoxy overlay) in combination with waiving the minimum rating requirement for a future 3 in. overlay.
- Use a lighter bridge rail (e.g., use a Type 10 MASH instead of Type 9).
- Use a voided sidewalk.
- Reduce deck thickness, with the approval of Unit Leader in coordination with the State Bridge Engineer. CDOT allows a minimum deck thickness

of 7.0 in. Reducing the deck thickness should be considered only after all other strategies for reducing weight have been exhausted. Deck thickness should be reduced only by the minimum amount needed to meet the minimum rating requirement.

33.8 EXPANSION JOINT ELIMINATION

Preservation and rehabilitation projects present opportunities to either eliminate or relocate existing expansion joints. Removing existing expansion joints reduces future inspection and maintenance needs, eliminates the possibility of future joint failure, and can improve ride quality.

Expansion device elimination should be considered for all preservation and rehabilitation projects. Changes in the structural behavior of the structure must also be considered, which may result in necessary modifications to other elements.

33.8.1 Expansion Joints at Abutments

For expansion joints at abutments, moving the joint to the end of the approach slab should be considered. This solution may require modification or replacement of the approach slab to resist the imposed forces and movements.

Figure 33-3 depicts one option for moving an expansion joint at a seat-type abutment to the end of a new approach slab.

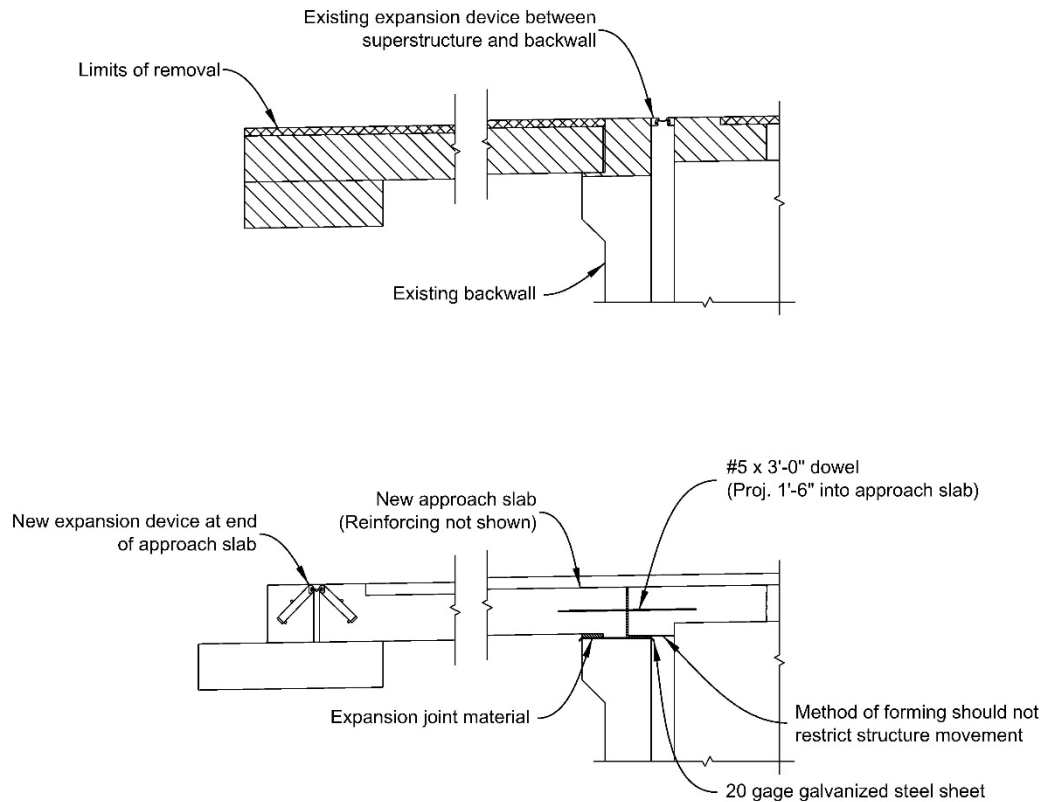


Figure 33-3: Expansion Joint Relocation

33.8.2 Expansion Joints at Piers

Eliminating an expansion joint at an interior pier requires that some degree of continuity be established, either complete continuity of the deck and girders or continuity of the deck only. Establishing continuity can alter the structural behavior of the bridge, including thermal movement demands from a new bridge center of stiffness location. External longitudinal force distribution may also be affected. As a result, the bridge may require modification or replacement of bearings to mitigate the behavior change. All structural consequences related to the elimination of expansion joints at piers must be carefully considered and resolved.

This type of joint replacement should be considered for existing multi-simple span bridges. In some cases, it will be possible to eliminate some but not all expansion joints. This is still considered an improvement over not eliminating any joints.

CDOT has accomplished this type of joint removal successfully in the past. Details for any proposed joint elimination shall be coordinated with Staff Bridge.

33.9 BEARING REPLACEMENT

Bearing replacements should address the root cause of the existing bearing deficiency. Fixing the root cause of an issue may not be possible given the cost of necessary modifications and funding constraints.

For neoprene pads, replacement bearings shall meet current design standards, or as close to current standards as practical, without requiring excessive modifications to bearing seats or other structural elements. Current seismic connection force requirements should be met, where practical.

The contract plans shall show:

- Jacking locations and design forces
- Any structural modifications required prior to jacking
- Phasing or traffic restrictions
- Extents of required removals
- Details for the new bearing devices
- Other special requirements

33.9.1 Structure Jacking Requirements

The Designer is responsible for determining suitable jacking locations for the structure. Structures are typically jacked from the diaphragms between girders at supports or from the girders directly in front of the bearing device either from the support seat or next to the support seat. See Section 14.5.6 of this BDM for typical jack clearance requirements.

The Designer shall verify that the structure can be jacked to the necessary height without overloading any structural components, including, but not limited to, girders, diaphragms, deck, and substructure.

To avoid overloading structure components, modifications may be required prior to jacking, such as adding bearing stiffeners to steel I-girders if jacking under the girder in front of the bearing device.

For situations where a jacking height of $\frac{1}{4}$ in. or less is required and all girders at a support will be jacked simultaneously, 1.3 times the permanent load reaction at the adjacent bearing may be assumed as the design jacking force. Otherwise, a refined jacking analysis is required to determine the design jacking force. The unfactored jacking force resulting from a refined analysis shall be increased by a minimum load factor of 1.3 to obtain the design jacking force.

**AASHTO
3.4.3.1**

Refined jacking analyses shall account for the stiffness contributions of the deck, diaphragms, and other structural elements, as appropriate.

Overload traffic shall not be permitted on the structure during jacking operations. Normal traffic shall not be permitted on the bridge during jacking operations unless:

- Overnight closures are not permitted, and
- Prior approval is obtained from Unit Leader in coordination with Fabrication/Construction Unit).

If traffic is permitted on the structure during jacking operations:

- Traffic should be shifted away from the jacking locations, where possible,
- Locking jacks should be used as a fail-safe in the event of jack failure, and
- The jacking load shall include factored and service dead and live load reactions, including impact, consistent with the permitted traffic positioning during jacking operations.

33.10 BRIDGE RAIL REPLACEMENT

Substandard bridge rail and guardrail transitions should be replaced when feasible with TL-4 MASH compliant rail and TL-3 transitions respectively. The approximate test level of the existing bridge rail should be provided to the Region for their safety and replacement considerations. Retrofit or rehabilitation options should be provided as well. If overhang strength is a concern, replacement with Type 9 bridge rail will spread out loads greater than Type 10 MASH bridge rail. A 6" bridge deck with #4s at 6" was tested to NCHRP 350 TL-3 levels with no damage.

See Sections 2.4.1 and 13 of this BDM for additional information.

33.11 FATIGUE

33.11.1 Load Induced Fatigue

For rehabilitation projects involving steel superstructures, all superstructure components shall be checked for the remaining fatigue life. When feasible, the remaining fatigue life shall be at least the desired service life of the type of rehabilitation being considered.

33.11.2 Distortion Induced Fatigue

Unlike load induced fatigue, distortion induced fatigue is not equilibrium based but instead arises from stiffness incompatibility and differential deflection of adjacent members.

Distortion induced fatigue cracking is prevalent in steel bridges built before 1985. Bridges built during this period commonly did not connect diaphragm connection plates to the girder flange out of perceived fatigue concerns. This practice results in a length of unbraced web from the girder flange to the termination of the connection plate, known as the web gap. When adjacent girders undergo differential deflection due to live load, forces are induced in the connecting diaphragms, producing distortion and potentially large stresses in the web gap. Because these stresses are cyclical, fatigue cracking can occur. Figure 33-4 and Figure 33-5 depict this behavior.

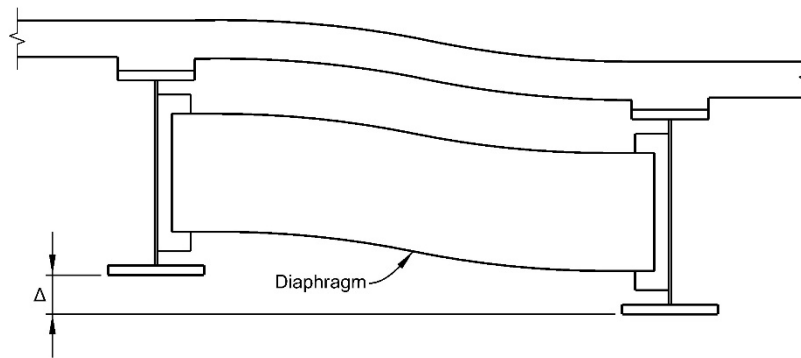


Figure 33-4: Differential Deflection

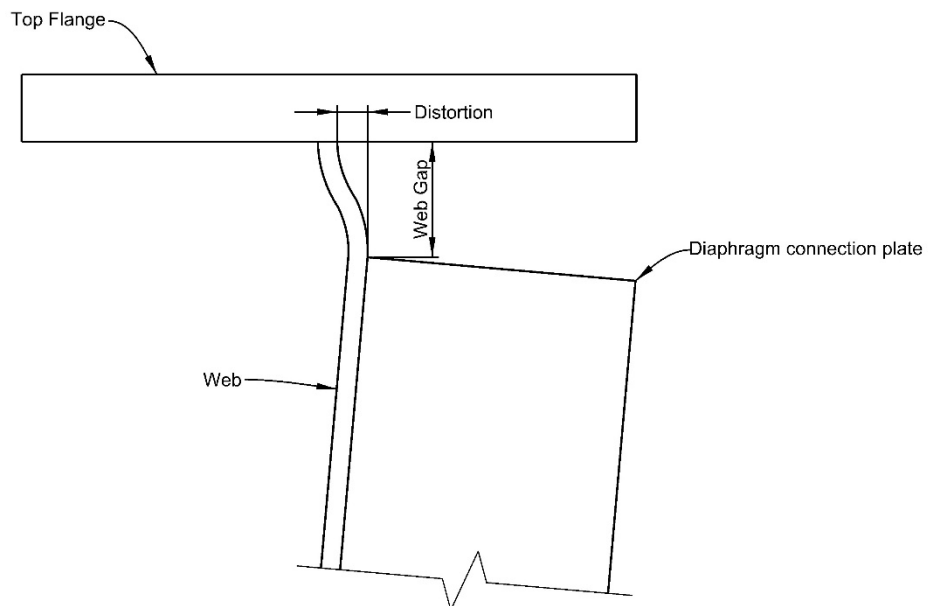


Figure 33-5: Web Gap Distortion

The magnitude of web gap distortion is proportional to the degree of differential deflection between the adjacent girders. For this reason, bridges with skewed supports and perpendicular diaphragms are particularly susceptible to distortion induced fatigue cracking.

The length of the web gap has a significant impact on the magnitude of fatigue stresses in the web gap. A longer web gap is more flexible and may be able to distort without resulting in large stresses, while a shorter web gap may be sufficiently rigid to reduce web gap distortion, which can also reduce fatigue stress magnitudes. Web gaps of approximately 2 to 4 in. in length generally produce the largest magnitude fatigue stresses.

Steel girder bridges built before 1985 and detailed with unstiffened web gaps are considered high risk for development of fatigue cracks. This includes bridges where girder connection plates attach to floor beams, diaphragms, or crossframes. Any preservation or rehabilitation project on a high-risk bridge shall determine if distortion induced fatigue cracking has occurred and develop a repair and retrofit plan to address any discovered deficiencies.

Superstructures that exhibit distortion induced fatigue cracking should be repaired and retrofitted according to the guidance in the FHWA *Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges*. A stiffening type retrofit is preferred because it produces similar behavior to that resulting from current design and detail methodologies.

The complexities of distortion induced fatigue may require refined structural models if accurate out-of-plane stress ranges in the web gap region need to be determined.

33.12 CULVERTS

For roadway widening projects that require extending an existing box culvert, consideration should be given to replacing the existing culvert in lieu of extending it if the existing portion is in poor condition and/or would require extensive repair during the predicted service life of the extended portion.

33.13 SCOUR CRITICAL STRUCTURES

33.13.1 Evaluation of Existing Structures for Criticality

Refer to Section 33.2.1 for code requirements and minimum performance criteria when determining if a structure is scour critical.

33.13.2 Rehabilitation of Scour Critical Structures

Once a structure has been assessed as scour critical, the processes and procedures outlined in FHWA Hydraulic Engineering Circular (HEC) numbers 18, 20, and 23 shall be followed, including development of a Plan of Action.

Depending on project specifics, the ideal corrective actions may be structural, hydraulic, or biotechnical countermeasures, a monitoring program, or a combination thereof. Acceptable scour countermeasures are shown in Table 2.1 of HEC 23. Scour countermeasures that are not acceptable for new structures may be acceptable for existing structures.

33.13.3 Structural Countermeasure Requirements

Structural scour countermeasures shall be designed to meet all requirements of AASHTO LRFD, where practical. Example structural scour countermeasures include foundation and substructure strengthening and independent structures that reduce or eliminate scour of the bridge.

33.14 PAINTING OF STEEL STRUCTURES

The corrosion of structural steel bridge members is an ongoing concern that must be addressed to prolong service life. Not only does corrosion change bridge aesthetics, it can seriously jeopardize the structural integrity of the entire structure. Painting is an efficient and economical method to provide corrosion protection to existing steel bridge members.

Maintenance painting is important for all bridges but is of particular concern for bridges more than 100 ft. long. For smaller bridges (less than 100 ft.), the proportionally higher cost of environmental controls for cleaning may outweigh the benefits of painting. Packaging multiple bridges into one contract for structures less than 100 ft. may be appropriate. For larger bridges (longer than 500 ft.) or complex bridges, paint preservation should be prioritized due to the high replacement cost of the bridge.

Bridge painting is weather sensitive. The air temperature must be warm and the humidity must be low. Therefore, work/letting needs to be scheduled when there is low probability of unsuitable weather conditions. Typically, May through September is the ideal time to accomplish bridge painting. If a painting project occurs outside this range, a controlled environment is required.

When possible, painting projects should be coordinated with roadway projects. The necessary time for a Professional Engineer to design and analyze a containment system should be included in the project schedule between the notice to proceed and the physical start of work. Also, consider the necessary time required for the industrial hygienist/certified professional to develop/review the lead safety plan and other submittals.

When repainting existing bridges over high ADT roadways where roadway restrictions must be minimized, use of a rapid deployment strategy should be considered. Rapid deployment is a viable option primarily designed for use on highway overpasses where the structural steel is easily accessible from the roadway below using a mobile work platform. This mobile work unit includes a containment device, dust collector, and blast equipment. Rapid deployment methodologies may be specified using Special Provisions. For field painting activities, use a two-coat system with an organic primer.

33.14.1 Zinc Rich Paint Systems

For a properly shop-installed zinc rich paint system, Table 33-1 identifies typical painting activities and frequencies to establish painting guidelines to maintain and preserve the life of steel bridges. Widespread use of these zinc rich paint systems began in the 1980s. Environmental factors (e.g., under a leaking deck joint, within “splash zone”) will have a detrimental effect on the life of the paint system, which will require an increased frequency of painting activities.

Leaking deck joints and other bridge deficiencies that may affect paint system performance should be corrected before completing any new painting activities.

Consideration must also be given to bridges that are on a program to be improved, rehabilitated, or replaced. Bridges on a program must be evaluated to determine if a painting activity is still warranted. The high cost of containment and mobilization require that a cost/feasibility estimate be completed to determine the most economic work scope for any given structure. For example, use of spot/zone painting vs. a full re-paint for any given structure or entire component replacement must be evaluated. This work scope should include aesthetic considerations for the visible portions of the bridge, such as fascia beams.

While a study of preliminary costs will likely conclude that an overcoat system is the most economical alternative, a life-cycle cost analysis will often show full paint removal and application of a high durability coating system to be more cost-effective than an overcoat option, particularly for bridges exposed to significant deicing salt application.

Table 33-1: Maintenance Painting Frequencies

Painting Activity	Frequency
Spot/Zone Painting	10–18 years
Full Re-paint	30–40 years

33.15 BRIDGE PREVENTATIVE MAINTENANCE

33.15.1 Program Objectives

Bridge Preventative Maintenance (BPM) seeks to extend the service life of structures through targeted improvements. Structures in good condition are the top priority of BPM funds because these bridges are near the top of their deterioration curve, and, therefore, see the greatest extension in service life per dollar spent. BPM projects typically cost less than 30 percent of the cost of a new bridge.

See Section 33.2 for code requirements and minimum performance criteria when design is required for a BPM project.

The primary BPM goals are to:

- Seal bare concrete decks.
- Add a waterproofing membrane to bridge decks that currently have an asphalt overlay but no membrane.
- Replace membranes on bridges where the existing membrane is nearing the end of its service life (approximately 30 years) or otherwise shows signs of deterioration.
- Replace leaking or otherwise non-functioning expansion joints.

- Replace functioning expansion joints at the end of their predicted service life, when convenient.

Examples of BPM actions include but are not limited to:

- Bridge rinsing
- Sealing deck joints
- Facilitating drainage
- Sealing concrete
- Painting steel
- Removing channel debris
- Protecting against scour
- Lubricating bearings

BPM projects also present an opportunity to perform other miscellaneous repair activities, such as bridge rail and substructure repair. The Designer should coordinate with CDOT to determine what additional activities to include in the project.

33.15.2 Bridge Preventative Maintenance Resources

33.15.2.1 Staff Bridge Worksheets for BPM

As of this writing, CDOT is in the process of developing standard worksheets for BPM work, including:

- General Information
- Summary of Quantities
- Deck Repair Details – HMA Overlay
- Deck Repair Details – Polyester Concrete Overlay
- Bridge Expansion Device (0–4 Inch) at Approach Slabs
- Taper Details for Polyester Overlay at Beginning/End of Structure and Bridge Drains

These worksheets can be obtained from Staff Bridge upon request.

33.15.2.2 Expansion Joint Replacement

The preferred type of replacement expansion device depends on the type of joint that is being replaced. A 0 to 4 in. joint is the preferred replacement joint type, when feasible. Table 33-2, the BPM joint replacement matrix, shows preferred and acceptable replacement types based on existing joint type.

Expansion joint elimination should be considered for all bridges requiring joint replacements. See Section 33.8 for more information.

Table 33-2: BPM Joint Replacement Matrix

BPM Joint Replacement Matrix												
Replacement Joint Type	Replacement Item Number	Existing Joint Types										
		Modular Expansion Device	New Style Strip Seal/Bridge Expansion Device (0-4 Inch)	Old Style Strip Seal	Bridge Expansion Joint (Asphaltic Plug)	Compression Joint Sealer	Pre-compressed Foam Joint	Sliding Plate/Finger Joint	Premolded Rubber Transflex Joint	Asphalt Over Non-expansion Joint	Pourable Joint Seal at Abutment or Pier	Roadway Pressure Relief Joint
Bridge Expansion Device (0-___ Inch)	518-010XX	X										
Bridge Expansion Device (0-4 Inch) ¹	518-01004	X ⁶	X	X	X	X	X	X	X		X	
Bridge Expansion Device (Gland) (0-4 Inches) ²	518-01060		X	X								
Bridge Expansion Joint (Asphaltic Plug) ³	518-01001				X	X	X			X		
Bridge Compression Joint Sealer	518-00000				X	X	X					
Joint Sealant ⁴	408-01100										X	
Sawing and Sealing Bridge Joint	518-03000									X	X	
Roadway Compression Joint Sealer ⁵	518-00010											X
None										X		

These are general recommendations, final determination of replacement joint type shall be discussed with Staff Bridge unit leader.

- X = Preferred joint type
- X = Acceptable joint type

¹This is CDOT's default joint. It has the longest service life and should be considered strongly for any location where there is potential leaking onto pier caps or abutment seats.

²The gland manufacturer must be the same as the manufacturer of the rails.

³To be used for rotational movement only. Translational movement of joint should be limited to ½". Proper seating of the bridging plate is critical to ensure it doesn't rock.

⁴To be used for rotational movement only. Translational movement of joint should be limited to ½".

⁵Parallel saw-cuts are critical on both sides of joint for proper placement.

⁶Some modular joints can be replaced with 0-4 Inch joints with an oversized gland.

33.15.2.3 Overlay and Wearing Surface Guidance

See Section 33.1.3 for bridge deck chloride testing requirements for projects that include installation of a new overlay. Chloride testing results may impact the selection of the wearing surface type or necessitate deck corrosion mitigation measures before installing the new wearing surface. See Section 33.5 for more information.

The following types of deck protection systems are permissible for use on preservation and rehabilitation projects:

- 3 in. HMA/SMA wearing surface over a waterproofing membrane
- ¾ in. polyester concrete overlay
- ¾ in. thin-bonded epoxy overlay

BPM projects with an asphalt approach roadway can be combined with roadway surface treatment projects to realize a substantially lower unit cost for

asphalt. For this reason, the preferred deck protection system for these bridges is a waterproofing membrane with a 3 in. asphalt wearing surface.

A $\frac{3}{4}$ in. polyester concrete overlay should be considered for BPM projects where the approach roadway is concrete or where other factors prevent reasonable inclusion in a surface treatment project. However, the additional height of the overlay requires that a taper detail be implemented to avoid modifying the existing expansion devices and end dams.

A $\frac{3}{8}$ in thin-bonded epoxy overlay is not a preferred option for long-term structure use (≥ 10 years) due to a high life cycle cost. However, if modification of expansion devices and end dams cannot be avoided or if it is cost prohibitive to do so, a $\frac{3}{8}$ in. thin-bonded epoxy overlay should be considered. Thin-bonded overlays are placed directly on the existing bridge deck without requiring modification of expansion joints and end dams.

When changing asphalt thickness, the maximum permanent grade changes shall be in accordance with the CDOT Roadway Design Manual.

33.15.3 Bridge Preventative Maintenance Project Delivery

33.15.3.1 Standalone Projects

Standalone BPM projects are maintenance projects that are independent of any other project work and specifically scoped for preventative maintenance work. Examples of these projects are Expansion Joint projects, Polymer Concrete Overlay projects and critical culvert projects. They are generally run by a regional RE or project engineer.

33.15.3.2 Overlay Projects

Bridge Maintenance work is often added on to CDOT overlay projects to take advantage of project mobilization and lane closures. For any overlay project, a letter shall be provided to the RE outlining all structures within the overlay limits and any restrictions for milling work and limitations on overlay thicknesses. Overhead limitations such as sign structures or bridges over the highway are also good to list with their clearance limitations. Vertical clearances should be verified after any overlay projects. The letter shall also include a listing of any essential repairs on the structures and all funded preventative maintenance work. Lastly, the letter shall include a listing of the current bridge rail or guardrails associated with the structures along with their assumed MASH compliance levels and recommendations for replacement or rehabilitation. Bridge funding is typically not available for rail replacement except for essential repair findings. This letter shall be delivered prior to FIR and preferably at the Scoping meeting.

33.15.3.3 Maintenance Projects

A third type of project delivery for Bridge Maintenance work would be maintenance projects going to advertisement similar to standalone projects or work done directly by maintenance forces. Plan and specification requirements would be similar to other project delivery methods.

33.15.3.4 Process Flow Charts

The process flow charts for rehabilitation and preservation projects are very similar to new bridge projects but are usually simplified due to the removal of some of the reports and survey requirements. Widening projects will be almost identical to new structures. Structure selection reports are still required for widenings but will be shorter. Rehabilitation work is simplified since survey, hydraulics and other specialty information is generally not required. Overlay recommendation/funded work letters are not required for standalone or M-projects.

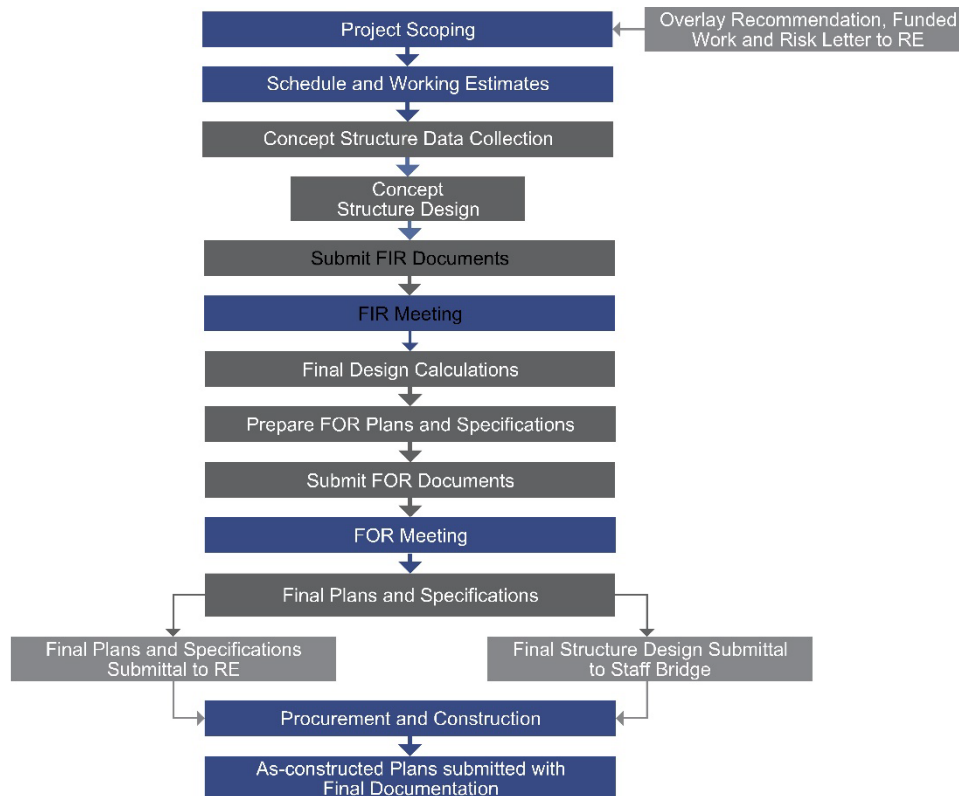


Figure 33-6: Structure Process Diagram (Overlay)

33.16 REFERENCES

The following references may be considered for further guidance:

ACI 222R-01: Protection of Metals in Concrete Against Corrosion.

ACI 440.2R-08: Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures.

Dexter, R.J. and J.M. Ocel. 2013. Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges, Report No. FHWA-IF-13-020. March.

Federal Highway Administration (FHWA). 2018. Bridge Preservation Guide. FHWA Publication Number: FHWA-HIF-18-022.

FHWA. 2009. Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance-Third Edition. Hydraulic Engineering Circular No. 23. FHWA Publication No. FHWA-NHI-09-111. September.

FHWA. 2001. Long-Term Effectiveness of Cathodic Protection Systems on Highway Structures. Publication No. FHWA-RD-01-096.

Hawk, H. 2003. Bridge Life-Cycle Cost Analysis. Washington, DC: Transportation Research Board. National Research Council, NCHRP Report 483.

Sohanghpurwala, A.A. 2006. Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements. Washington, DC: Transportation Research Board. NCHRP Report 558. doi:10.17226/13934

Sohanghpurwala, A.A., W.T. Scannell, and W.H. Hartt. 2002. Repair and Rehabilitation of Bridge Components Containing Epoxy-Coated Reinforcement. NCHRP Web Document 50.

SECTION 34 PLANS

34.1 GENERAL REQUIREMENTS

The Designer shall refer to the latest Bridge Detail Manual for guidance in structure plan preparation and generally accepted detailing notes, standards, and procedures. The Designer is responsible for becoming knowledgeable about the Bridge Detail Manual and subsequent updates to its contents.

SECTION 35 COST ESTIMATING AND QUANTITIES

35.1 GENERAL REQUIREMENTS

Quantities of the various materials involved in project construction are essential for determining the estimated project cost and for establishing a basis for the Contractor's bid and payment.

Prepare quantity calculations and project cost estimates at the conceptual, and preliminary stages of project development. Square Foot (SF) cost may be used at conceptual or planning stages. Cost estimate at preliminary stage is only required for structure selection report. The best available cost data and project information at the respective design stage shall be used.

Cost estimates at the FOR and Final will be completed by Engineering Estimate and Market Analysis (EEMA). For local agency project requirements see the Local Agency Manual.

35.2 BID ITEMS

Bid items shown in the Summary of Quantities Table located in the plans and in the Structure Selection Report shall be listed sequentially according to the most current CDOT Cost Data Book. The eight-digit cost code, item description, and unit of payment shall be used in the tabulations.

A Project Special Provision shall be written if an accurate description of the work and the method of measurement for each bid item is not adequately described in the drawings or outlined in the Standard Specifications for Road and Bridge Construction or in the Standard Special Provisions.

The Engineering Estimate and Market Analysis (EEMA) Unit tracks and tabulates bid items and costs for all projects awarded for construction. Data are published on CDOT's website. The Item Code Book provides individual bid items listed sequentially by eight-digit code, item description, and unit. If a new item is required, the Engineer shall coordinate a request through EEMA. CDOT's Construction Cost Data Book provides unit costs for each awarded project. The cost data summarize the final Engineer's estimate (completed by EEMA), the average project bid, and the awarded bid. Engineers and technicians should use these resources when developing project cost estimates.

35.3 PLANNING / CONCEPTUAL

During the early planning and conceptual phase of a project, estimated quantities may be required to evaluate viable and economical structure alternatives. If square foot costs for the structures cannot be determined, the structure should be broken down into individual cost items. At this stage, quantity accuracy between the design and check should be within 10 percent. Unless determined otherwise, the cost estimate at this project stage should include a 50 percent contingency.

35.4 SCOPING

The scoping phase follows the planning/conceptual phase. Scoping is the phase of the project to determine the objectives and requirements necessary to complete a project. Properly defining the scope allows the team to effectively estimate cost and schedule.

35.5 PRELIMINARY / FIELD INSPECTION REVIEW (FIR)

For the FIR submittal, the Designer and Independent Checker shall calculate estimated quantities and a preliminary cost estimate to include in the Structure Selection Report. Quantities at this stage may be estimated using quantity per cubic yard, per square yard, or as percentages of individual structure components. For example, reinforcing steel quantities may be estimated using average reinforcing weight per concrete volume (lb/cy). At this stage, quantity accuracy between the design and check should be within 5 percent. If the Region requests a comprehensive project estimate, these quantities and cost items should be coordinated with the design team for submittal. Unless determined otherwise, the cost estimate at this project stage should include a 15 percent contingency.

35.6 FINAL / FINAL OFFICE REVIEW (FOR)

For the FOR submittal final estimates will be completed by EEMA based on the final quantities calculated and independently checked as outlined in Section 35.7 of this BDM. FOR level quantities shall also be updated to address any comments made during the FOR review and submitted with the Ad document package. EEMA is responsible for the FOR and final Engineer's Project Cost Estimate.

35.7 QUANTITY CALCULATIONS

Use the available plan set to compute and check quantities independently at each design stage. If sufficient information is not on the plan set to determine the quantity, revise the plans to show the missing information. The Designer and Checker shall separately summarize their calculated quantities, compare their values, and resolve any differences in accordance with Section 35.8 of this BDM. The record quantity set shall typically be the Designer's, shown in the Summary of Quantities Table, and included in the field package as requested. Each set of quantity calculations shall include a summary showing the percent differences.

Each set of calculations shall compare and meet the required percent difference per Table 35-1 for each item in the element breakdowns as outlined in the *Bridge Detail Manual*, i.e., Superstructure, Abutment 1, Pier 2, etc. For example, the Designer's values for excavation for Pier 2 and Pier 3 shall be compared separately against the corresponding values determined by the checker. The quality process shall follow the QA/QC procedure outlined in Section 37 of this BDM.

Use logical breaks between the superstructure and substructure quantities for the calculations. Such breaks may be construction joints, bearing seats, expansion devices, abutment front face, abutment back face, or breaks indicated on the plans. Except for precast prestressed and post tensioned

members, all bridge concrete shall be Class D. Class DF shall be used to replace Class D in splash zones, see Section 5 for details.

The following recommended logical breaks for bridge quantities should be followed on all plan sets:

- Include all concrete and rebar below the top of bearing seats at abutments, wingwalls, and piers in the substructure quantities.
- Include all projecting rebar embedded into the concrete designated as substructure in the substructure quantities.
- Include a column in the Summary of Quantities Table for approach slab. Calculate approach slab from the back of approach notch. Include the anchorage bar into the abutment in the superstructure quantities.
- Except as noted below, include all concrete and rebar **above** the top of bearing seats at abutments, wingwalls, and piers in the superstructure quantities.
- Precast girder members, bridge railings, and caissons have designated pay items and do not require concrete and rebar quantities.
- Precast panel deck forms required by the plans will be paid for at the contract unit price for the area shown on the plans. The quantities shall be in the superstructure quantities.

The following will be included as roadway quantities only and will not be shown on the bridge Summary of Quantities sheet:

- All revetment such as slope mattress or riprap and associated excavation. When information for revetment is shown within bridge plan set, the quantities should be shown. The quantities and checking are the responsibility of the hydraulic designer.
- Excavation and backfill relating to revetment installation
- All excavation and embankment for spur dikes, channel improvements, or bike paths
- Common backfill not associated with the construction of the structure or not shown in the backfill quantities figure
- Unclassified excavation

35.8 ACCURACY AND FORMAT

Required quantity calculation accuracy between originator and checker for each design phase shall be as shown in Table 35-1. The cost estimate contingency shown is the preferred value of Staff Bridge but can be adjusted on a project basis to match the other disciplines involved with the project. For Information Only quantities shall be held to the same level of accuracy and checking.

Table 35-1: Contingency and Quantity Accuracy Percentage

Design Phase	Contingency	Quantity Percent Difference
Planning/Conceptual	50%	± 10%
Scoping	40%	± 10%
Preliminary/FIR	15%	± 5%
Final/FOR/Ad	0%	± 1% (unless noted otherwise)

$$\%Difference = \frac{Design - Check}{Design} \%$$

- Calculate the quantity percent difference for each structural element (i.e., abutment, pier), not the final total.
- For all design phases, excavation and backfill quantities may be within 10 percent difference.
- For Final, use actual reinforcing bar lengths, including calculated lap lengths, in calculating reinforcement weight.
- For Final, when calculating concrete haunch quantity, use the average haunch as shown in BDM Section 5.5.2.1G.
- For all design phases, use a unit weight of 146.7 pcf or 110 lb/sy per inch thickness when calculating quantities for Hot Mix Asphalt (HMA) and Stone Matrix Asphalt (SMA).
- Do not use preliminary quantities based on volume (#/cy), area (lb/sf), or percentages for final quantities.
- For all design phases, do not average quantities from the two independent sets.
- For Final, include a summary showing percentage differences in the calculations.
- Refer unresolved quantity differences to the Unit Leader or Project Manager for resolution.

Use the Tabulation of Bridge spreadsheet to create and populate a Summary of Quantities Table containing item codes and quantities for a bridge project. Format the Summary of Quantities Table so that it can be embedded in or linked to a MicroStation drawing file and included in the General Information section of the project plan set. The spreadsheet is located on CDOT's Bridge Design Manual download page. Manual tabulations are allowed.

Round totals shown in the Summary of Quantities Table as shown in Table 35-2.

Table 35-2: Summary of Quantities Table Rounding

Item	Rounding Criteria*
All, except as noted below	1
Concrete	0.1 ▲

* For exception items, e.g. timber, ton, etc, see Figure 100-19 in the Construction Manual for rounding criteria

▲ For quantities, over 25 (based on the 2% specification error limit), round to the nearest whole number

The project construction cost estimate should include a line item to cover overhead and indirect costs such as Construction Engineering, Owner Controlled Insurance Program (OCIP) and Design Services During Construction. The general percentage for construction overhead and indirects is typically 26 percent but can vary from project to project and should be coordinated with the Region Business Office.

SECTION 36 CONSTRUCTION

36.1 GENERAL REQUIREMENTS

The following section addresses the role of the Project Structural Engineer during project advertisement and construction. The Project Structural Engineer shall determine scope, hours, and fee for post-design services using items defined in this section and through conversations with the Project Engineer. For consultant designed projects, the CDOT Structural Reviewer shall also designate hours for assistance as defined herein.

The Request for Proposal and other related Contract Documents define the role of the Project Structural Engineer for projects contracted under the design-build delivery method. Consideration of construction methods and tolerances for specific design elements can be found in their respective sections within this BDM.

36.2 CONSTRUCTION SUPPORT

The Project Structural Engineer shall be available to the construction Project Engineer to assist in interpreting the structure plans and specifications and to resolve structure-related construction issues. Refer to Section 101.103.8.2 of the CDOT *Construction Manual* for additional information. Field personnel shall alert the Project Structural Engineer of any changes or additions to the structure as defined in the Contract Documents. For consultant designed projects, the CDOT Structural Reviewer shall be available to resolve construction-related problems requiring the decision of CDOT from the Owner's perspective.

The Bridge Fabrication and Construction Unit within Staff Bridge acts as a liaison between the field and design engineers and provides fabrication inspection for the Bridge Program. The Project Structural Engineer may consult this unit for advice when responding to questions from the field or during girder fabrication. It should not be assumed that this unit will handle all construction-related inquiries independently.

36.3 INQUIRIES DURING ADVERTISEMENT

If the Project Engineer requests, the Project Structural Engineer shall attend the pre-bid conference and assist with questions that arise during the advertisement period. Such questions may result in structural plan or specification revisions to provide clarification or correction.

During project advertisement, the Project Engineer shall respond to all inquiries from contractors, suppliers, or the media regarding the structural plans and specifications, unless the Project Engineer directs otherwise. This applies to CDOT employees and participating design consultants. All questions and responses will be archived and made available to all bidders during the advertisement phase.

36.4 CONTRACTOR DRAWING SUBMITTALS

There are two types of Contractor drawing submittals: shop drawings and working drawings. Subsection 105.02 of the CDOT *Standard Specifications for Road and Bridge Construction* (Standard Specifications) provides guidance on which type of drawing should be submitted for different structural works and which drawings should be sealed by the Contractor's Engineer). Structural engineers should become familiar with this subsection and verify project applicability. When project requirements differ from the Standard Specifications, the Project Structural Engineer shall include a Project Special Provision Revision of Section 105 to carefully specify the required submittal and type.

The Project Structural Engineer, or assigned designee, shall review submitted drawings for a given structure in accordance with Standard Specifications Subsection 105.02, except as noted in this section. Electronic submittals are acceptable. The preferred electronic format is Portable Document Format (PDF).

36.4.1 Shop Drawings

The Project Engineer will transmit shop drawings to the Project Structural Engineer for review. A high priority must be given to the review, keeping in mind the time necessary for resubmittal and subsequent reviews. A guide for reviewing structural shop drawings is offered below. For additional guidance on the review of structural steel and prestressed components, see Standard Specifications Subsections 509.15 and 618.04, respectively.

1. On the office copy, mark in red any errors or corrections; highlight in yellow all verified information; and mark all other notations in pencil, blue pen, or black pen. Note: Only red marks shall be transferred to the copies returned to the Project Engineer and Contractor. The Project Structural Engineer shall alert the Project Engineer if deviations from the Contract Plans are allowed. The Contractor should clearly mark any proposed deviations on the shop drawings as such. The Project Structural Engineer may suggest a new or revised detail provided that the detail is clearly noted: "Suggested Correction – Otherwise Revise and Resubmit."
2. The Project Structural Engineer shall, in addition to Standard Specifications Subsection 105.02(c), check the following items for compliance with Contract Plans, Special Provisions, and Standard Specifications. Note: Manufacturers' details may deviate from Contract Plans but may still conform to design requirements.
 - a. Material specifications
 - b. Size of member and fasteners
 - c. Dimensions when also shown in Contract Plans
 - d. Finish (surface finish, galvanizing, anodizing, painting, priming, etc.)
 - e. Weld size and type and welding procedure, if required
 - f. Fabrication – reaming, drilling, and assembly procedures
 - g. Adequacy of details
 - h. Erection procedure when required by Contract Plans or Specifications

For MSE walls, the bearing pressure, strap lengths and other design elements shall be checked against the project worksheets. Values not meeting worksheet requirements will require stamped calculations from the Contractor. Values exceeding the values in the worksheet may not be cause for Change Orders. The Contractor needs to be made aware that quantity overruns from changes to the project worksheets will not be paid for.

3. The following items need not be checked; however, they should be corrected, if necessary, for consistency with other corrections:
 - a. Quantities in bill of materials
 - b. Dimensions not shown in Contract Plans
4. If issues arise causing delays in the checking process, the Project Structural Engineer shall notify their supervisor and the Project Engineer. In the case of consultant designed projects, the Project Structural Engineer shall notify the CDOT Structural Reviewer and the Project Engineer.
5. When the review is complete, the Project Structural Engineer will sign, date, and mark the shop drawings in accordance with Standard Specifications Subsection 105.02(c). A shop drawing review stamp indicating the review action is required on each sheet of the shop drawings.
6. The Project Structural Engineer shall retain, in addition to the office copy, one set of reviewed and marked shop drawings, forward one set to the CDOT Structural Reviewer (consultant designed projects only), and return the remaining sets to the Project Engineer. For electronic submittals, the Project Structural Engineer or the CDOT Structural Reviewer shall return a copy of the reviewed and marked shop drawings to the Project Engineer and place a copy on CDOT ProjectWise©. This process supersedes the transmittal process outlined in the CDOT *Construction Manual* Section 105.2.3.

36.4.2 Working Drawings

Typically, working drawings are not formally reviewed by the Project Structural Engineer or returned to the Contractor. At the Project Engineer's request, the Project Structural Engineer may be asked to review certain working drawings such as shoring or falsework or to assist with interpreting Contractor working drawing submittals. If time and budget allow, a courtesy review for feasibility and conformity to contract requirements may be conducted. Review of bridge rail and expansion device drawings is suggested as time allows to avoid field issues. A conversation between the Project Structural Engineer and the Project Engineer is recommended before project advertisement to discuss expectations of working drawing reviews, budget, and scope.

36.4.3 Demolition and Girder Erection

Structure demolition plans and girder erection plans shall be reviewed for safety concerns and general feasibility. All safety issues must be addressed before commencing work.

36.5 REQUESTS FOR INFORMATION (RFI)/REQUESTS FOR REVISION (RFR)

On projects using a delivery method other than design-build, Contractor RFIs and RFRs shall first be administered through the Project Engineer. Requests are often for plan or specification clarification or a change in details, design or specification due to field conditions or variances. If a change is requested, the Contractor shall provide the solution; it is not the Project Structural Engineer's responsibility. The Project Structural Engineer shall make recommendations to the Project Engineer to allow, accept, delete, add, etc., the RFI/RFR. Direct correspondence between the Project Structural Engineer and the Contractor shall not occur, unless the Project Engineer directs otherwise. The Project Engineer will consider effects to the schedule, impacts to other work activities, costs, and contract requirements before final response is given to the Contractor. The Project Structural Engineer or Project Engineer shall place a copy of the RFI/RFR and response on Project Wise; coordination is required.

36.5.1 As-Constructed Plans

The Project Engineer is responsible for creating as-constructed plans based on information provided by the Contractor. The Project Structural Engineer or Project Engineer shall place a copy of the as-constructed plan set on Project Wise®; coordination is required.

36.6 BRIDGE CONSTRUCTION REVIEWS, FINAL INSPECTION

Upon all structure construction completion, the Project Engineer should ask the Project Structural Engineer to conduct a final walk-through inspection. This supersedes Section 101.103.8.3 of the CDOT *Construction Manual*, which calls for inspection of only major structures. If construction is complete with no final inspection, the Project Engineer should be contacted to arrange one. Typically, the Project Structural Engineer with the addition of a Staff Bridge representative (for consultant designs) shall perform the inspection. It is preferred for the Design Engineer to be involved during the inspection due to their familiarity with the project.

Local Agency projects that involve FHWA/FEMA funds will also require a final inspection of all structures before project acceptance. The CDOT Structural Reviewer shall work with their Local Agency Coordinator during the design phase of the project to inform local agencies of the requirement. The Local Agency Project Manager shall coordinate the final inspection with the Local Agency Project Engineer and invite the CDOT Local Agency Coordinator and the Project Structural Engineer at a minimum. The CDOT Structural Reviewer or Unit Leader should be invited as a courtesy but the primary responsibility is on the Project Structural Engineer and the Local Agency for identifying nonconformant work. Some regions may require a stamped memo from the Local Agency's licensed Engineer that the project is in conformance. Refer to the current edition of the Local Agency Manual for additional requirements and details.

The final inspection is considered a final walk through for observation and structure acceptance, and shall be held to determine whether the work was completed in reasonable conformance with the plans and specifications including any authorized changes. The intent is to show good stewardship of

the funds provided on the project. Any nonconformance shall be documented using the Owner Acceptance - Final Walkthrough Items Spreadsheet located in the Forms & Form Letters section on the website and emailed to the Project Engineer, the Region Unit Leader, the CDOT Structural Reviewer (if consultant design) and the Staff Bridge Senior Design & Construction Engineer. Supporting information such as photos, date of inspection, and attendees should be included. This "punchlist" of nonconforming items shall be addressed and resolved prior to acceptance of the project and signing off on Form 1212.

The final walk through of structures also allows an opportunity to receive feedback and input from the field on the effectiveness and constructability of plan details and specification requirements. All lessons learned shall be included in the Owner Acceptance - Final Walkthrough Items Spreadsheet.

36.7 ARCHIVING

All construction submittals that concern or relate to structures shall be archived in ProjectWise including Working Drawings, Shop Drawings, RFIs, NCRs, erection and demolition plans.

SECTION 37 QUALITY ASSURANCE AND QUALITY CONTROL

37.1 GENERAL REQUIREMENTS

All design construction documents, reports, studies, and any other documents delivered to CDOT must comply with the minimum requirements of this BDM and the documents referenced in the Policies and Procedures. Deliverables are subject to both Quality Assurance and Quality Control (QA/QC) as described herein.

37.2 PURPOSE

All entities (CDOT and Consultants) producing deliverables for CDOT must use a rigorous QA/QC program to accomplish the following objectives, which include but are not limited to:

- Ensure safe structures for the traveling public
- Provide structures that are low maintenance for the life of the structure
- Prevent problems from occurring during construction
- Provide cost-effective solutions
- Prevent errors
- Provide consistency
- Promote ingenuity

The purpose of this section is not to supplant QA/QC programs and policy already established internally within CDOT or with individual consulting firms but rather it describes the minimum requirements that must be included in a QA/QC program applied to a CDOT project. Unless otherwise described in this BDM, specific methodologies for conducting and documenting QA/QC procedures are the prerogative of the entity executing a project. For example, an independent technical review, described in Section 37.3, is required, but the entity performing the work is responsible for determining the exact procedure and forms necessary to perform the review and to document that it has occurred.

This section defines the types of QA/QC reviews, discusses project planning, and identifies the required QA/QC reviews for each design phase in the order in which each design phase occurs.

37.3 DEFINITIONS

For definitions not included in this section, refer to the Policies and Procedures Section of this BDM.

Quality Assurance (QA): The procedure that verifies and documents that established QC procedures have been implemented during the execution of a project. QA is performed through audits as defined below.

Quality Control (QC): A systematic procedure that checks the accuracy of design calculations, construction plans, specifications, and other pertinent documents to achieve the objectives noted in Section 37.2. When properly used, QC procedures detect and correct errors and omissions before a project is constructed. QC procedures include the independent design check, independent

technical review, constructability review, and CDOT structural review on consultant projects as defined below.

Design: Design includes generation of the following:

- Structure Selection Report
- Structural design calculations that support structural elements
- Bridge geometry
- Detailing construction drawings
- Quantity calculations
- Estimates of probable costs for the Structure Selection Report (Note that final design includes only quantity calculations, not cost estimates)
- Project Special Provisions
- Structure Load Rating (Refer to the CDOT Bridge Load Rating Manual for QA/QC requirements)
- Design calculations and detailing drawings resulting from changes during construction

The Project Structural Engineer is responsible for assigning these tasks to the Structural Design Engineer(s).

Independent Design Check: The process that uses a person or party separate from those who prepared the documents to verify the contract documents. This key QC requirement involves the Independent Design Engineer verifying all design work, drawings, specifications, quantities, reports and any construction changes generated by the Structural Design Engineer to ensure structural integrity, constructability, and satisfaction of all applicable standards listed in this BDM. As such, the independent design check, combined with the initial design, results in (1) two sets of complete design and quantity calculations, both stamped by a Colorado Licensed Professional Engineer and (2) a review set of the final plans where all discrepancies between design and the independent check have been resolved. It is recommended that the Independent Design Engineer have more experience than the Structural Design Engineer. For some simple designs such as design by observation based on M&S standards, a red/yellow check may suffice in lieu of an independent design check. A red/yellow check is defined as a review of the design calculations where all assumptions and calculations are yellowed out if correct and redlined if modifications are necessary. The red/yellow check shall also be stamped by a Colorado Licensed Professional Engineer.

Independent Technical Review: The independent technical review, also known as an independent design review or a technical peer review, involves reviewing all project deliverables, including the construction plans, specifications, and estimate of probable cost. This QC review includes:

- Conformance with generally accepted best practices
- Conformance with CDOT bridge design practices
- Interdisciplinary design coordination: for example, roadway geometry correctly reflected in the structure plans

- Constructability, biddability, and inspectability issues without solely relying on the constructability review as defined below

The engineer assigned to the independent technical review (referred to as the Independent Technical Reviewer) shall be experienced, knowledgeable, and independent of the development of the project documents.

Constructability Review: The constructability review involves reviewing the construction plans and specifications to minimize scope changes, reduce design-related change orders, and ensure the structure and details are buildable. This QC review includes the following:

- Constructability, which shall consider at a minimum, phasing, sequencing, detailing, material availability, construction equipment access, and appropriate inclusion and use of specifications.
- Biddability; for example, the construction plans and specifications are consistent and contain sufficient information for a Contractor to bid on a project.
- Inspectability and safety; for example, adequate access for an inspector to determine the condition of structural elements that require inspection. Inspectability shall include details such as ladder stops on slope paving, ladder supports at inspection hatches, appropriately sized hatches, diaphragm ports, and lock protectors.

Application of constructability reviews is based on the project complexity (Category I, II, or III) as described below:

- **Category I projects** include bridges using standard construction methods that are generally one or two spans, structures that use the CDOT M&S Standards, and simple repairs such as expansion joint replacement. The Project Structural Engineer or in rare cases an outside consultant can conduct the review.
- **Category II projects** include bridges with specialty features, longer bridge lengths than Category I projects, or a project team with insufficient experience with the type of construction involved in the project. Category II bridges may include cast-in-place post-tensioned concrete, curved steel plate girders, etc. An experienced Project Structural Engineer, a construction engineer, and possibly an outside consulting firm may conduct the constructability review.
- **Category III projects** include critical or complex structures as defined by superstructure and substructure type, geometry, construction methods, height, length, or feature intersected. Category III bridges may include concrete segmental construction, curved steel box girders, viaducts, major river crossings, etc. A highly experienced Project Structural Engineer, a highly experienced construction engineer, an outside consulting firm, or possibly a contractor may conduct the constructability review.

Quality Assurance (QA) Audit: A review of the contract documents to verify that the project QC procedures have been implemented.

CDOT Structural Review: On Consultant projects, a CDOT Structural Reviewer will be assigned to review the deliverables. This review generally includes similar aspects as the independent technical review but from an oversight perspective. Thorough reviews of the preliminary design submittals (as a minimum, Structure Selection Reports and FIR plans) and final design submittals are required.

37.4 QUALITY MANAGEMENT PLAN

All CDOT projects should have a project-specific Quality Management Plan (QMP) that identifies the scope of work, project objectives, schedule, deliverables, and QA/QC procedures that will be used to achieve a successful project. A QMP may already be a requirement of a Consultant QA/QC program, which can also be used for a CDOT project.

As part of the QMP, the following meetings should be used to initiate the project and to ensure that the project is on the right path throughout the design process:

- **Project Scoping Meeting:** A project scoping meeting should be used to discuss the project objectives, design criteria, critical issues, and procedures used to mitigate risk. From a structural design point of view, the Project Structural Engineer, CDOT Structural Reviewer, and key team members should attend this meeting.
- **Structure Status Meetings:** On Consultant projects, the Project Structural Engineer shall meet periodically with the CDOT Structural Reviewer to discuss the design work. The frequency of meetings should be established at the project scoping meeting. The frequency is based on project complexity. Attendance by the Resident Engineer and, as appropriate, other design team members (e.g., geotechnical, hydraulics, roadway, and traffic) is encouraged. Holding structure status meetings for CDOT designed projects is also encouraged.

37.5 QUALITY CONTROL/QUALITY ASSURANCE PROCEDURES

Each submittal, including all portions of the submittal, is subject to the independent design check, independent technical review, constructability review, QA audit, and CDOT structural review. The following briefly describe the reviews required in each design phase:

- **Preliminary Design (FIR):** A critical period in the life of a project where the direction of a project is determined. Independent design checks of elements that make up the FIR submittal are required, such as:
 - Geometric layout
 - Confirmation of structural elements that affect the recommended structure type (e.g., span lengths, girder type, and foundation type)
 - Quantities and the cost estimate
 - Data and conclusions in the Structure Selection Report. Refer to the Structure Selection Report Checklist in Section 2, Appendix 2A. The Staff Bridge Unit Leader shall approve the Structure Selection Report.

In addition, the Structure Selection Report should undergo a technical edit for grammar, spelling, and readability to both structural and non-structural engineers.

The independent technical review, constructability review, and CDOT structural review are also required to ensure that a project is on a successful path. When critical issues are not addressed during the FIR phase, they can have a significant impact on final design.

The CDOT Structural Reviewer, in consultation with the Staff Bridge Engineer and the Resident Engineer, shall select the project category (I, II, or III) during preliminary design. This will determine the appropriate level of constructability review for the project.

- **Final Design (FOR):** The Project Structural Engineer is responsible for originating or updating tasks defined in Section 37.3. During final design, the Independent Design Engineer shall be provided a complete set of FOR construction plans without any supporting calculations from the Structural Design Engineer. Through the independent design check, a second set of calculations is produced to support all appropriate design information in the plans, including, but not limited to, the following:
 - Design criteria
 - Geometry
 - All structural elements that support load
 - Devices that accommodate structure movements
 - Quantities

The exception to the independent design check process is situations when Strut and Tie models are used. They must be shared with the design checker to obtain concurrence on the models. To avoid late changes, concurrence should be obtained for the models as they are developed, rather than at the end of the design phase.

The independent design check also involves checking the FOR specifications, which includes the following:

- Determine if the CDOT Standard Specifications for Road and Bridge Construction adequately cover all aspects of construction in the plans.
- If the Standard Specifications are not adequate, determine if the CDOT Standard Special Provisions selected for the project are appropriate.
- If neither the Standard Specifications nor the Standard Special Provisions are adequate, Project Special Provisions are required and must be checked. CDOT provides Project Special Provisions and Bridge Design Worksheets (BDW) that can be modified for a project.
- QC documents, independent technical review, constructability review, and CDOT structural review are also required for the FOR documents. When the plans are complete, the initial block on the left side of the standard CDOT border shall be completed to identify the Designer

and checker for the structural design, detailing, and quantity calculations.

- **AD Plans & Specifications (P&S):** After the FOR meeting, all comments must be addressed and incorporated as appropriate. If items change the structural design or involve new structural items, they are subject to the previously described QA/QC procedures prior to final submittal. The AD P&S submittal shall be accompanied by a Final Detail Letter (FDL) certifying that the structural plans and specifications have been prepared in accordance with standards set by CDOT.
- **Documentation of Review Comments:** All comments received from the FIR and FOR meetings shall be tracked. The documentation should include at a minimum: (1) reviewer comments, (2) reference to the location in the reviewed document (e.g., sheet number, chapter, and section) for each comment, (3) comment responses, and (4) confirmation that each comment has been incorporated into the document as appropriate.
- **Review Comment Resolution:** Comment resolution from independent design checks, independent technical reviews, and constructability reviews shall be documented. The Project Structural Engineer and/or CDOT Structural Reviewer, as appropriate, are responsible for ensuring all comments and discrepancies are resolved. He or she shall make the final determination if comments and discrepancies are unable to be resolved between the Structural Design Engineer and the Independent Design Engineer or Independent Technical Reviewer.
- **QA Audit:** A person independent from the project team (not involved in producing project-related documents) and intimately familiar with the project QA/QC requirements is assigned to perform a QA audit. This person can be someone from the organization producing the documents or another organization contracted to provide QA audits. The QA audit verifies that the QC procedures have been implemented. A QA audit should occur before each deliverable is submitted.

Below are two examples of what an auditor may do to assure that a required quality procedure has been completed correctly for a set of construction plans:

1. The independent design check process includes creating a red-line drawing that shows suggested corrections, agreement for what corrections should be made, and demonstrates incorporation of the corrections to a clean drawing. The auditor reviews the red-line drawing to see evidence of a checking procedure and verifies that the initials block identifies the Designer and checker for design, detailing, and quantities. The auditor may also compare the red-line drawing to the clean drawing, not for the purpose of determining if the change is correct, but to verify changes have been incorporated.
2. An independent technical review of drawings/documents may be conducted by filling out a comment resolution form that documents the following:
 - a. Independent Technical Reviewer (reviewer) comments

- b. Structural Design Engineer's (originator) responses
- c. Initial and final disposition of the comment, e.g., accept, delete, clarify/discuss, incorporate in the next submittal
- d. Reviewer initials placed on the form after verifying that agreed upon dispositions are addressed as discussed

In this review, an auditor will verify that there is agreement between the originator and reviewer, and that the reviewer signed the form after verifying the disposition. The auditor may also verify that the change was made to the drawing.

SECTION 38 ALTERNATIVE DELIVERY

38.1 GENERAL REQUIREMENTS

38.1.1 Delivery Method Evaluation

Currently, several types of project delivery systems are available for publicly funded transportation projects. The most common systems are Design-Bid-Build, Construction Manager/General Contractor (CM/GC), and Design-Build. No single project delivery method is appropriate for every project. Each project must be examined individually to determine how it aligns with the attributes of each available delivery method. CDOT has developed a Project Delivery Selection Matrix to evaluate all methods for a project and ultimately to select the delivery method.

The latest version of the Project Delivery Selection Matrix can be found at <https://www.codot.gov/business/designsupport/adp-db-cmgc/pdsm>.

For CM/GC and Design-Build methods, the Designer is encouraged to review the latest CDOT manuals for each method. Both manuals can be found here at <https://www.codot.gov/business/designsupport/adp-db-cmgc>.

Public-Private Partnership is another alternative delivery method where funding is provided through a partnership with a private entity. For the purposes of bridge design, the Designer shall follow the same guidelines as set forth for Design-Build.

38.2 DESIGN-BID-BUILD

CDOT most commonly uses the Design-Bid-Build delivery format to develop plans and specifications. With this format, the design is completed with a complete set of plans and specifications before project advertisement. The plans and specifications are competitively bid on, and a Contractor is selected. The Designer shall follow the Structures Process Diagram presented in Policies and Procedures, Section E of this BDM when preparing designs, plans, and specifications. The design plans and specifications for advertisement/bid may name proprietary products but shall generally include two to three product options to promote competition unless it is an innovative application.

38.3 CONSTRUCTION MANAGER/GENERAL CONTRACTOR

Construction Manager/General Contractor or CM/GC is a method of delivery that allows CDOT to select a Contractor to provide feedback during the design phase. When consultants are used for design, the design consultant and the Contractor have independent contracts with CDOT. This method allows the Contractor to work with the Designer and CDOT to identify, minimize, and appropriately share risks; provide cost projections; and refine the project schedule. Once design is complete, CDOT and the Contractor negotiate the bid price and schedule of the construction contract. If CDOT agrees with the Contractor's bid and schedule, the contract is awarded and construction

begins. If CDOT and the Contractor cannot agree on the bid and schedule, the project is put out to bid in a manner like that of a Design-Bid-Build project.

On CM/GC projects, the Designer's role is similar to that on a Design-Bid-Build project. As part of the project development process, the Designer shall vet and incorporate Contractor comments and follow the Structures Process Diagram presented in Policies and Procedures, Section E of this BDM when preparing designs, plans, and specifications, except as noted herein. As part of the structure selection process, the Contractor's feedback may result in CDOT electing to eliminate certain structure types or span configurations due to cost, complexity, or risk. The Designer may incorporate Contractor preferred construction methods in the plans if other local contractors have the means to perform the work. The design plans and specifications for the Release for Construction submittal may name proprietary products but shall generally include 2 to 3 product options to promote competition unless it is an innovative application.

38.4 DESIGN-BUILD AND STREAMLINED DESIGN-BUILD

CDOT uses two types of Design-Build delivery systems:

- Streamlined Design-Build – A streamlined design-build delivery is a single step procurement. CDOT does not shortlist teams through a statement of qualifications (SOQ) submittal but relies on pre-qualified Contractors and Designers to bid and perform the work. The streamlined delivery system is primarily used for smaller structural projects that may have various site challenges, rapid schedules, or other complexities that allow innovative design and construction.
- Standard Design-Build – A standard design-build project is typically a large scope project with complexities like those of the streamlined design-build but on a much larger scale. On these projects, CDOT will first shortlist teams through a SOQ process. Then CDOT will select the winning team using a best value approach. A best value approach ties project goals to best value parameters, such as cost, time, scope, technical design considerations, and construction operational considerations. These parameters are evaluated using specific scoring criteria and entered into an evaluation formula to identify the apparent successful proposer.
- In either Design-Build delivery format, the Contractor is under contract with CDOT, whereas the Engineer-of-Record Designer is under contract with the Contractor.
- Designers may be called on for a variety of roles on a Design-Build project. A Designer may work as part of the owner representation team, which includes CDOT and consultant staff, or the Contractor's team, which includes the Contractor and consultant designers.

38.4.1 Owner Representation – Preliminary Design

Before advertisement of a Design-Build project, CDOT may require a Designer to develop conceptual level plans and to draft the technical requirements. This provides both a starting point for Design-Build teams in their pursuit of the project and serves as the technical requirements that the Contractor's team must follow through the final design for CDOT to accept the design.

Concept plans shall include information on minimum structure requirements. This is typically provided as a typical section and/or a general layout for each major structure. A joint agreement between CDOT Region and Staff Bridge shall determine the level of detail. Generally, design is progressed to a 10% to 20% level, depending on the complexity of the site and the project constraints.

The Designer preparing the technical requirements is encouraged to obtain CDOT's most recent Design-Build project's final structural technical requirements and to modify them to fit the current project. The Designer is encouraged to work directly with Staff Bridge to vet draft versions of the document before advertisement of the Draft Request for Proposals (RFP). After the Draft RFP is advertised, the Designer will modify the technical requirements as necessary until the Final RFP is published. When necessary, Addendums to the Final RFP may be required to provide final technical requirements to the Contractor team before receipt of proposals, proposal evaluations, and announcement of the apparent successful proposer.

38.4.2 Owner Representation – Delivery

After the contract is awarded on a Design-Build project, the Designer may be called on to review Contractor design submittals. This effort requires that the Designer review the plans, specifications, and calculations for conformance to the final technical requirements and associated design criteria. These design criteria include, but are not limited to, the CDOT BDM, the latest edition of the *AASHTO LRFD Bridge Design Specifications*, and project aesthetic requirements. The reviewer shall document all instances where the design does not meet the technical requirements or design criteria and provide comments to the CDOT Project Manager. Like other owner representation tasks, the Designer is encouraged to maintain communication with Staff Bridge and keep the unit informed of areas of concern.

38.4.3 Contractor's Designer

The Contractor's Designer shall provide designs, plans, and specifications as directed by the contract outlined for the Design-Build project through the Instructions to Proposers, Book 1 (where used), and Book 2 (Technical Requirements). The Designer may specify specific and proprietary items on the plans. These items should be selected from CDOT's Approved Product List, when applicable. The Contractor's specific means and methods may be incorporated into the design as long as the methodology of construction does not conflict with the technical requirements or limit future maintenance. Technical requirements will outline project deliverables.

38.5 ALTERNATIVE BRIDGE DESIGN SPECIFICATION

CDOT has the option to include an Alternative Bridge Design Specification project special provision for any bridge designed using the Design-Bid-Build delivery method. This special provision allows the Contractors who bid on the project to develop and price an alternative structure in lieu of bidding the default structure in the advertised plans. Acceptable alternative structure types and any other applicable design constraints for alternative structures will be delineated in the special provision.

Should a Contractor become the successful bidder with an alternative structure in the bid, the alternative structure will be designed and constructed using the Design-Build format of delivery for just that portion of the project. The alternative structure shall meet the design criteria and design deliverables as outlined in the special provision. Designer specification of proprietary items shall be as delineated for Design-Build projects. The Contractor's specific means and methods may be incorporated into the design as delineated for Design-Build projects.

SECTION 39 ACCELERATED BRIDGE CONSTRUCTION

39.1 GENERAL INFORMATION

This section provides general guidance for the use of accelerated bridge construction (ABC) techniques.

A standard practice for project delivery, ABC evaluates innovative materials, construction techniques, project planning, and design methods to safely and efficiently reduce construction time and traffic impacts for new and rehabilitated structures. CDOT is committed to using ABC as a tool to achieve the following goals:

- Embrace FHWA's Every Day Counts (EDC) initiatives
- Decrease and minimize maintenance of traffic (MOT) operations to reduce user costs associated with delays
- Encourage innovation
- Improve motorist and worker safety

To further strengthen CDOT's role as stewards of the taxpayers' dollars and to achieve the above goals, CDOT has developed tools and resource materials for evaluating ABC techniques to determine their applicability toward a given project. For Analytic Hierarchy Process (AHP) software downloads and specific resource materials mentioned in this section, refer to Accelerated Bridge Construction documents on CDOT's website at <https://www.codot.gov/business/designsupport/abc-documents>.

39.2 ABC EVALUATION OVERVIEW

39.2.1 Background

CDOT uses an ABC decision chart during project scoping to determine if ABC is appropriate for the project and site constraints. This chart was based on the FHWA manual entitled *Decision-Making Framework for Prefabricated Bridge Elements and Systems (PBES)*, May 2006. This process is based on a set of questions about specific constraints of each project. If certain thresholds are met, ABC was recommended.

Subsection 39.2.2 outlines the approach for the ABC decision-making process and how it is used during project development. The ABC Evaluation and Decision Matrix Workflow, shown in Figure 39-1, has been developed to graphically assist project engineers and planners in implementing the ABC process.

39.2.2 ABC Evaluation Process

The intent of the evaluation process is to apply some form of ABC on most projects. To encourage the use of ABC, a two-step process is presented as follows:

1. Complete the CDOT Prescoping ABC Rating spreadsheet (refer to Subsection 39.3.2 for additional information). The Design Team completes this spreadsheet at the pre-scoping level based on a general understanding of the project and its site constraints. If, according to the supplemental flowchart, the resulting ABC rating indicates little to no benefit in implementing ABC, the evaluation process is complete and is documented as part of this first step. This spreadsheet should be included in both simple and complex structure pre-scoping reports.
2. If the ABC rating indicates a benefit to implementing ABC, the Design Team shall execute the FHWA AHP software. This process uses a structured technique to organize and analyze only complex bridge construction decisions. It also provides a more in-depth evaluation to select the most appropriate ABC methods to meet the project goals and constraints. The ABC Construction Matrix (Figure 39-2) provides examples of construction methods with respect to project complexity. The second step will take place after pre-scoping but before completion of FIR level design efforts. This interactive process is completed with the CDOT specialty groups and led by the Project Engineer and a CDOT subject matter expert (SME). The Design Team shall capture and document for the project files summaries of each step of the decision process.

This ABC methodology shall be evaluated for all projects that include bridges. The final project submittal will include a justification letter written to the project file explaining why an ABC technique is or is not used. The Design Team shall also document the ABC decision process, including any supporting materials, in the Structure Selection Report (refer to Section 2.10 of this BDM for additional information) as part of FIR level design tasks.

Approval of ABC is at the discretion of each Resident Engineer/Region in coordination with the Unit Leader and should be communicated and approved at a level commensurate with the complexity of the ABC method and project cost. For example, a self-propelled modular transporter (SPMT) bridge move should receive the Regional Transportation Director's , Chief Engineer's concurrence for a bridge over light rail tracks, whereas use of prefabricated bridge elements may require approval from only the Unit Leader and Resident Engineer.

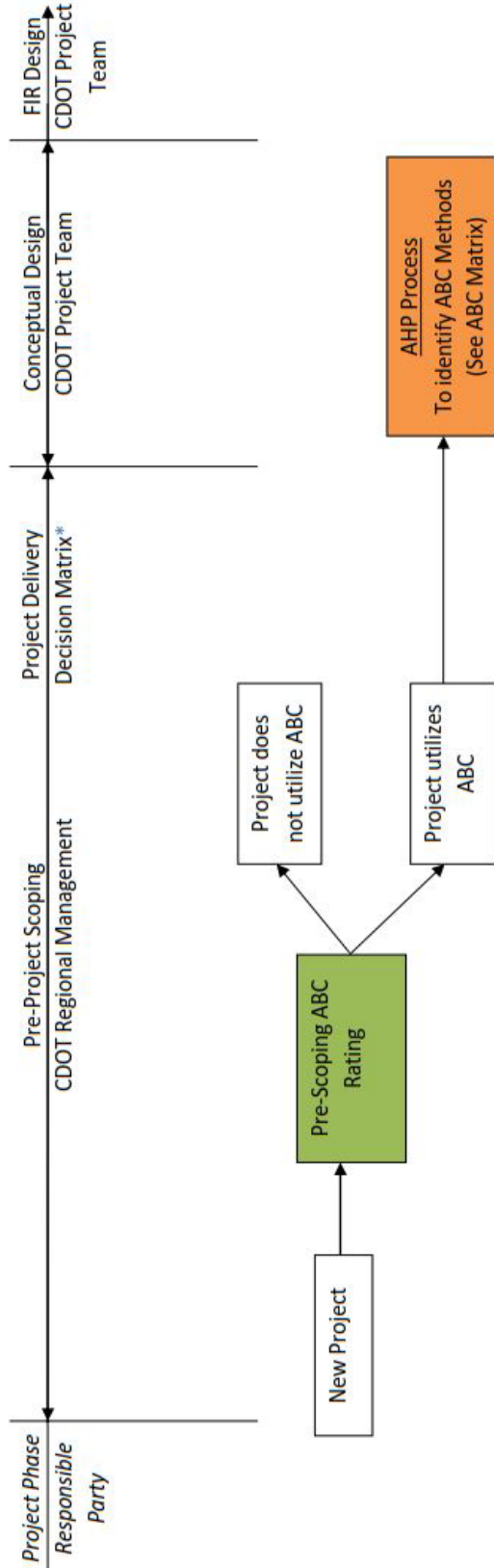
The ABC Workshop PowerPoint presentation (dated March 6, 2013) offers project-specific examples illustrating the use of the pre-scoping ABC rating and AHP software, as well as ABC project case histories. It is also recommended that the Design Team work with the designated SME for guidance and information about the use of the ABC materials. Subsection 39.3 discusses these resources in further detail.

39.3 ABC MATERIALS AND RESOURCE GUIDANCE

39.3.1 ABC Evaluation and Decision Matrix Workflow

Figure 39-1 graphically illustrates the two-phase approach for the ABC decision-making process from project inception to FIR level design efforts.

ABC Evaluation and Decision Matrix Workflow



**Contracting methods can work hand in hand with ABC delivery. Depending on the particular goals of the project and complexity of the ABC method, some contractual tools may be more beneficial than others. For each project, the Delivery Decision Matrix should be completed after the Pre-scoping ABC Rating form has been completed. Some contractual tools are listed below:*

Innovative Contracting Tools	A+B Contract Provision	Contract in Calendar Days	Fixed Completion Date Contract	Incentive-Disincentive Provision	Lane Rental Provision	Value Engineering Specification	Design Build Contract Delivery Method	CM/GC Contract Delivery Method
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Figure 39-1: ABC Decision Matrix Workflow

39.3.2 Pre-scoping ABC Rating

39.3.2.1 ABC Rating Procedure

The pre-scoping ABC rating spreadsheet is required during the bridge pre-scoping level and calculates an ABC rating score that accounts for all the project measures defined in Subsection 39.3.2.3, except environmental issues. This spreadsheet is located under ABC Documents on CDOT's website. Weighting factors have been assigned to each measure to coincide with FHWA's EDC initiatives and CDOT's goals. The Designer shall not modify the weighting factors for individual projects.

The values assigned to each project decision measure are multiplied by the corresponding weight factor. The ABC rating score is the ratio of the weighted score to the maximum score shown as a percentage and is categorized into three ranges: 0 to 20, 20 to 50, and over 50. The minimum score of 20 is intended to capture any project receiving a score of 5 in any one of the four most heavily weighted categories, while the higher threshold score of 50 is intended to capture any project receiving an average score of 3.5 in the four most heavily weighted categories. The range of scores is set to ensure that accelerated construction is commonplace when the measured benefit is more significant than the measured cost with respect to accomplishing FHWA EDC initiatives and CDOT's goals. Apply the ABC rating score to the flowchart to work toward a conclusion.

39.3.2.2 ABC Decision Flowchart

The ABC Rating Procedure described in Subsection 39.3.2.1 is the first step in determining if ABC is appropriate for a given project. The ABC Decision Flowchart applies the ABC rating score and then addresses Yes/No factors that are considered before making a final decision on the construction approach. Factors include project schedule, environmental concerns, total project cost, site conditions, and high-level indirect costs such as political capital, safety, or impacts to stakeholders.

Together, the ABC Rating Procedure and ABC Decision Flowchart are used to make a final determination of the appropriate construction methods for each project. If ABC is deemed beneficial to the project at the pre-scoping level, the Design Team should proceed to the second step in the evaluation process, applying the AHP software and discussing with CDOT specialty groups such as Staff Bridge, Utilities, Environmental, Traffic, and Hydraulics, to better identify site constraints, project goals, and preferred ABC technologies and delivery methods.

39.3.2.3 ABC Rating Procedure Measures

Using the Structure Inspection Assessment Report and Staff Bridge user costs spreadsheet in conjunction with preliminary knowledge of the project conditions, the Design Team determines the appropriate score for each ABC measure. The nine ABC measures described herein are incorporated into the Pre-Scoping ABC Rating procedure to help determine where the use of ABC is appropriate and to output the ABC rating score.

- **Average Daily Traffic (ADT)** – This is a measure of the volume of traffic traversing the bridge site. Use a value equal to the total number of vehicles on and under the bridge, where applicable. This measure accounts for the value of maintaining the interstate highway network by assigning the maximum score for this situation. This measure also addresses minimizing impacts to the traveling public during construction.
- **Delay/Detour Time** – This is a measure of the time required for vehicles to pass through or circumvent a construction site because of a project. It accounts for the time delays due to detours and construction induced congestion. If delays are anticipated for the roadways both on and under the bridge, enter the worst-case scenario. This measure addresses minimizing impacts to the traveling public during construction.
- **Bridge Importance** – This measure assigns a value for bridges on or over designated emergency evacuation routes or bridges that are economically crucial to servicing local communities and businesses. This measure addresses minimizing impacts to the traveling public by accelerating construction on these important roadways.
- **User Costs** – This is a measure of the financial impact a construction project has on the traveling public. While the contributing factors in calculating user costs are traffic delays and ADT, the duration of the impact to users is essential in measuring the burden to the traveling public. CDOT has instituted standard methods for calculating user costs using FHWA guidelines. The Design Team shall calculate user costs in coordination with the Regional Traffic Engineer to determine the total project cost for each construction option being evaluated, including, but not limited to, SPMT methods, slide-in bridge construction, prefabricated elements, or conventional construction. This measure addresses minimizing impacts to the traveling public during construction, reducing total project costs, and encouraging innovation.
- **Economy of Scale** – This measure accounts for repetition in structural elements and construction processes; how they relate to the overall project cost; and the potential savings to future projects. To account for repetition of substructure and superstructure elements, the number of spans for a proposed bridge is applied when quantifying economies of scale. This measure addresses reducing total project costs.
- **Safety** – This is a measure of the safety provided to the traveling public and contractor employees. A goal of implementing ABC methods is to reduce the amount of time motorists and workers are exposed to potentially hazardous construction environments. Project sites that require complex MOT schemes for extended periods of time are undesirable and impact the safety rating. This measure addresses improving worker and motorist safety during construction.
- **Railroad Impacts** – This is a measure of the impact to railroad operations. The quantity of railroad tracks and their importance to daily train operations are considered when determining this impact. This measure addresses total project cost, improving worker safety, and minimizing impacts to commerce.

- **Site Conditions** – This is a measure of the physical site constraints preventing the use of ABC methods. For example, vertical profile shifts over 1 ft. greatly impact, if not inhibit, the ability to accelerate construction. Additionally, time sensitive utilities may limit the time available for construction, local soil stability may preclude the use of heavy construction equipment, or adjacent ROW designations may limit staging opportunities. This measure addresses physical fatal flaws to the ABC delivery process.
- **Environmental Concerns** – The presence of endangered species or annual spawning seasons may shorten the opportunity for construction. In other cases, projects may have limitations due to wetlands, air quality, extreme weather, historical designations, or noise ordinances. ABC may be necessary to accomplish an acceptable level of impact on the surrounding environment. This measure does not specifically address a goal and is not a weighted factor in determining the ABC rating score; rather, it is included in the ABC Decision Flowchart to evaluate if ABC can provide appropriate mitigation to an environmental commitment or requirement.

39.3.3 ABC Matrix

The ABC Matrix (Figure 39-2) provides suggestions for accelerated construction techniques that may be applied depending on the complexity of the project. This matrix offers preliminary guidance only; the Design Team is encouraged to develop innovative solutions, especially if the chosen project delivery method is Construction Manager General Contractor or Design-Build. Conversely, the decision to execute ABC technologies may dictate the project delivery method because fast-track contracting methods are often tailored to Owner involvement and project goals. When using this matrix, it is important that the Design Team acknowledges total construction cost is not the primary consideration when determining suitable ABC methods where project constraints, for instance, favor public safety and/or user cost benefits.

39.3.4 ABC AHP Decision Tool Software

Refer to CDOT's website to download the AHP software and to access the complete ABC Decision Making Software materials, including definitions, user manual, and supplemental software development information.

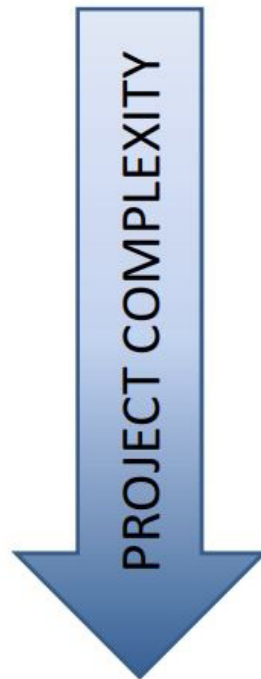
39.4 OTHER RESOURCES

FHWA's *Accelerated Bridge Construction Manual* provides detailed guidance to educate engineers further in ABC technologies, prefabricated bridge elements, construction techniques, and project planning and decision-making tools. Refer to FHWA's Accelerated Bridge Construction website (www.fhwa.dot.gov/bridge/abc/) for the most recent manual publication, webinars, case studies, and technical contacts.



Accelerated Bridge Construction Matrix

This matrix provides suggestions and previously utilized methods for accelerated bridge construction. It is not all inclusive nor intended to dictate any particular method.



Substructure	Approach, Embankment & Backfill	Superstructure	Superstructure Placement
	Pre-fabricated approach slabs	Adjacent Girders ²	
	Flowfill	Precast Deck Panels (partial depth) ²	
Pre-fabricated Pier Caps	Expanded Polystyrene (EPS) Geofoam	Pre-fabricated pedestrian bridge ²	
Pre-fabricated columns		Pre-fabricated box culvert ²	
Pre-fabricated foundations		Precast Deck Panels (full depth) ²	
Geosynthetic Reinforced Soil (GRS) Abutment ¹		Modular Girder and Deck elements ²	
Pre-fabricated wingwalls/backwalls ²		Post-tensioned concrete through beams ²	Heavy Lift Cranes
Continuous Flight Auger Piles (CFA)		Pre-fabricated truss or arch span ²	Skid or Slide In
			Longitudinal Bridge Launch
			Self Propelled Modular Transport (SPMT)

¹ FHWA Every Day Counts Initiatives

² Prefabricated Bridge Elements and Systems (PBES)

ABC Costs ABC method construction costs generally increase with project complexity. However, many methods of ABC may reduce the overall project cost, specifically where ABC methods can eliminate or reduce detours or traffic control.

Figure 39-2: ABC Matrix

Appendix A - Design Examples

Appendix A contains the following examples:

- Example 1 - Elastomeric Leveling Pad Method A
- Example 2 - Type I Bearing (Steel Reinforced) Method A
- Example 3 - Type I Bearing (Steel Reinforced) Method B
- Example 4 - Type II Bearing (Reinforced Bearing with PTFE) Method B
- Example 5 - Expansion Device (Strip Seal) 0-4 Inch
- Example 6 - Deck Design, Including Type 10 MASH Rail Collision
 - Example 6.1 - Deck Design
 - Example 6.2 – Type 10 MASH Strength Design
 - Example 6.3 - Barrier Type 9 Strength Design
 - Example 6.4 - Overhang Design
- Example 7 - Girder Haunch and Camber
 - Case 1: Bulb Tee Bridge
 - Case 2: Side-by-Side Box Girder Bridge
- Example 8 - Cantilever Wingwall Design Loads
- Example 9 - Seismic Zone 1 Design Example
- Example 10 - Sign Structure Foundation Design
- Example 11 - Cast-in-Place Concrete Cantilever Retaining Wall
- Example 12 - Rail Anchor Slab Design
- Example 13 - Vehicle Collision on a Pier

Disclaimer: These examples, associated software to develop them, and other files are intended for use by Consultants working for CDOT Staff Bridge in their development of projects for the Colorado Department of Transportation. Any other use is at the sole discretion of the user. The Colorado Department of Transportation makes the examples, associated software, and other files available "AS IS" and assumes no liability nor makes any warranty of any kind, including warranties of non-infringement, fitness or merchant ability whether expressed or implied, to the accuracy or functionality of these files. By using, referencing, and/or downloading any files, you are agreeing to this disclaimer. The examples provided are intended to provide common examples associated with bridge and structure design. They do not provide the full design requirements for structure design or analysis and the designer should finish all additional calculations and code checks to accomplish the full evaluation and analysis for the structure.

APPENDIX A
EXAMPLE 1 - ELASTOMERIC LEVELING PAD
METHOD A

GENERAL INFORMATION

Per CDOT Bridge Design Manual (BDM) Section 14.5.7, leveling pads are plain elastomeric pads (PEP) and are designed using Method A procedures in accordance with AASHTO LRFD 7th Edition Section 14.7.6

Leveling pads are primarily used with integral substructures and will not experience shear displacements in that condition. In addition, design for bearing rotation is implicit within Method A procedures (AASHTO C14.7.6.1). The Designer, however, shall confirm that the thickness of the leveling pad is sufficient to prevent girder-to-support contact as a result of anticipated girder rotations, girder skew, and roadway vertical geometry. Leveling pads used with integral substructures are designed for dead loads only, up to and including the deck pour, per BDM Section 14.5.7.

MATERIAL AND SECTION PROPERTIES

Leveling Pad Dimensions

Leveling Pad Width	W = 37.00 in	AASHTO 14.7.5.1
Leveling Pad Length	L = 10.00 in	AASHTO 14.7.5.1
Leveling Pad Thickness	$h_{ri} = h_{rt} = $ 0.75 in	Typically between 1/2" and 1"

Leveling Pad Material Properties

Shore A Durometer Hardness	Duro = 60 (min)	BDM 14.5.7
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Shear Modulus

The least favorable value is assumed since the material is specified by its hardness value (AASHTO 14.7.6.2)

G = 0.13 ksi	AASHTO T14.7.6.2-1
Check = 0.08 ksi < G < 0.25 ksi	OK AASHTO 14.7.6.2

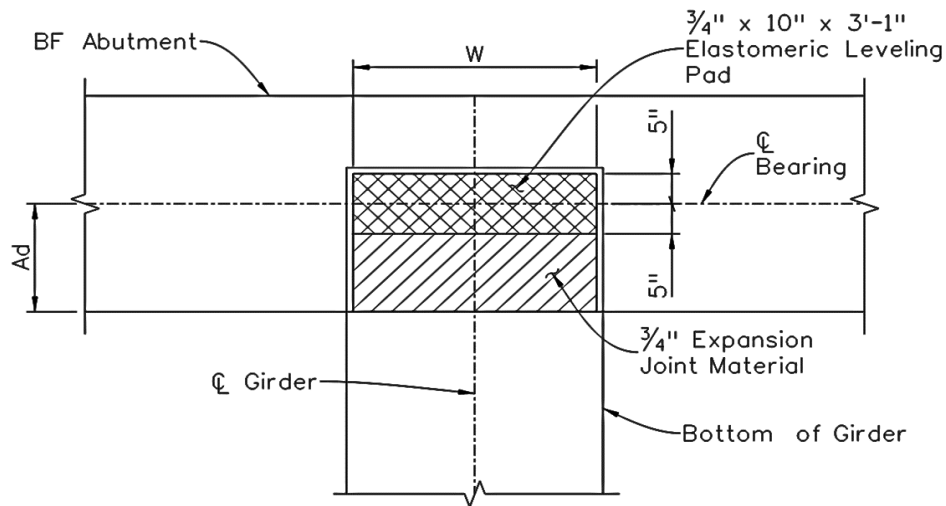


FIGURE 1 - LEVELING PAD DETAIL

BRIDGE GEOMETRY

Profile grade between supports % = **-1.50** %
 C bearing to FF Abutment A_d = **1.25** ft

BEARING ROTATIONS

Rotations include effects of girder camber. For all rotation values, positive indicates an upward rotation while negative indicates a downward rotation.

Service I Limit State Loads

Net girder rotations (camber plus dead loads) θ_d = **0.004** rad

Include a rotational allowance of 0.005 radians due to uncertainties in bearing fabrication and bearing seats. Per BDM 14.5.4, the flatness tolerance for bearing seat uncertainties is accounted for in the rotational allowance.

Construction Tolerance θ_r = **0.005** rad AASHTO 14.4.2.1

BEARING LOADS

Loads acting on the leveling pad are dead load girder reactions, up to and including the deck pour, at the service limit state. Loads are per bearing.

Service I Limit State

DL = **136.00** kip

SOLUTION**Shape Factor**

Total thickness of interior layer, h_{ri} , is equal to total elastomer thickness, h_{rt} ($h_{ri} = h_{rt}$)

Rectangular, plain bearing shape factor without holes:

$$S_i = \frac{LW}{2h_{ri}(L+W)} = \frac{(10.00 \times 37.00)}{[2 \times 0.75 \times (10.00 + 37.00)]} = 5.25 \quad \text{AASHTO 14.7.5.1-1}$$

Compressive Stress

AASHTO 14.7.6.3.2

The compressive stress of the leveling pad shall satisfy the criteria below for a PEP.

σ_s = average compressive stress due to total load from applicable service load combinations

$$\sigma_s \leq 1.00GS_i = 1.00 \times 0.13 \times 5.25 = 0.68 \text{ ksi} \quad \text{AASHTO 14.7.6.3.2-1}$$

and

$$\sigma_s \leq 0.80 \text{ ksi} \quad \text{AASHTO 14.7.6.3.2-2}$$

$$\sigma_s = \frac{DL}{LW} = \frac{136.00}{(10.00 \times 37.00)} = 0.37 \text{ ksi}$$

$$\text{Check } \sigma_s \leq 1.00GS_i \quad 0.37 \text{ ksi} < 0.68 \text{ ksi} \quad \text{OK}$$

$$\text{Check } \sigma_s \leq 0.80 \text{ ksi} \quad 0.37 \text{ ksi} < 0.80 \text{ ksi} \quad \text{OK}$$

Compressive Deflection

Compressive deflection under initial dead load of a PEP shall meet the following criteria in AASHTO 14.7.6.3.3 and 14.7.5.3.6. Total thickness of interior layer, h_{ri} , is equal to total elastomer thickness, h_{rt} ($h_{ri} = h_{rt}$). Note the graphs presented in Figure C14.7.6.3.3-1 apply to laminated bearings; equation C14.7.5.3.6-1 will be used in lieu of these graphs to determine the strain in the bearing pad under applicable stresses.

$$\delta_d \leq .09h_{ri} = 0.09 \cdot 0.75 = 0.068 \text{ in.} \quad \text{AASHTO 14.7.6.3.3}$$

$$\delta_d = \sum \varepsilon_d h_{ri} \quad \text{AASHTO 14.7.5.3.6-2}$$

$\varepsilon_d =$ dead load compressive strain in elastomeric pad

$$\varepsilon_d = \frac{\sigma_d}{4.8GS_i^2} \quad \text{AASHTO C14.7.5.3.6-1}$$

$$\sigma_d = \frac{DL}{LW} = 0.37 \text{ ksi}$$

$$\varepsilon_d = \frac{\sigma_d}{4.8GS_i^2} = 0.37 / (4.8 \cdot 0.13 \cdot 5.25^2) = 0.021$$

Check

$$\delta_d = \varepsilon_d h_{ri} = 0.021 \cdot 0.75 = 0.016 \text{ in} < 0.068 \text{ in} \quad \text{OK}$$

Stability

AASHTO 14.7.6.3.6

Total thickness of interior layer, h_{ri} , is equal to total elastomer thickness, h_{rt} ($h_{ri} = h_{rt}$).

Total bearing thickness shall not exceed the lesser of the following dimensions:

$$\frac{L}{3} = 10.00 / 3 = 3.33 \text{ in}$$

and

$$\frac{W}{3} = 37.00 / 3 = 12.33 \text{ in}$$

$$\text{Check} \quad h_{rt} = 0.75 \text{ in} < 3.33 \text{ in} \quad \text{OK}$$

Geometry

Confirm that the thickness of the leveling pad is adequate to prevent girder-to-support contact under anticipated girder rotations and roadway geometry. Assume rotations are about the centerline of bearing.

Maximum rotation, including compressive deflection effects, before bottom of girder comes in contact with the top of support:

$$\theta_{max} = \tan^{-1} \frac{(h_{rt} - \delta_d)}{A_d} = -\tan^{-1} [(0.75 - 0.016) / (1.25 \cdot 12)] = -0.0489 \text{ rad}$$

Rotation of girder due to profile grade of bridge between supports in question:

$$\theta_{grade} = \tan^{-1} \frac{\%}{100} = \tan^{-1} (-1.50/100) = -0.0150 \text{ rad}$$

Total girder rotation, including camber, dead loads, allowances for construction and bearing fabrication uncertainties, and roadway geometry.

$$\theta_{total} = \theta_d - \theta_r + \theta_{grade} = 0.0040 - 0.0050 + 0.0150 = -0.0160 \text{ rad}$$

Total rotations through the deck pour need to be less than the maximum rotation:

$$\theta_{max} \geq \theta_{total} \quad -0.0489 \text{ rad} > -0.0160 \text{ rad} \quad \text{OK}$$

APPENDIX A
EXAMPLE 2 - TYPE I BEARING (STEEL REINFORCED)
METHOD A

GENERAL INFORMATION

Per CDOT Bridge Design Manual (BDM) Section 14.5.8, reinforced bearing pads may be designed using Method A upon approval by Unit Leader in coordination with the Bearing SMEs. This example is in accordance with AASHTO LRFD 7th Edition Section 14.7.6.

This example assumes a concrete superstructure that can displace under the effects of temperature, creep, and shrinkage, and assumes a rectangular bearing similar to that shown in Figure 1. The structure is assumed to move freely in the longitudinal direction for the range of temperatures conforming to AASHTO 3.12.2.2 Procedure B. Design for rotation is implicit within Method A procedures per AASHTO C14.7.6.1 and is not investigated. The Designer, however, shall confirm that the thickness of the bearing pad is sufficient to prevent girder-to-support contact as a result of anticipated girder rotations, girder skew, and roadway vertical geometry.

MATERIAL AND SECTION PROPERTIES

Bearing Dimensions

Bearing Width	W =	25.00	in	AASHTO 14.7.5.1
Bearing Length	L =	12.00	in	AASHTO 14.7.5.1

Bearing Pad Layers

Exterior Elastomeric Thickness	$h_{re} =$	0.125	in	OK	< 70% h_{ri} AASHTO 14.7.6.1
Interior Elastomeric Thickness	$h_{ri} =$	0.500	in		
Steel Plate Thickness	$h_s =$	0.125	in		
No. of Steel Shim Plates	$n_{shims} =$	6			
No. of Interior Elastomer Layers	$n =$	5			AASHTO 14.7.6.1
Total Elastomer Thickness	$h_{rt} =$	2.750	in		
Total Bearing Height	$t =$	3.50	in	OK	2" minimum height per BDM 14.5.8

Bearing Material Properties

Shore A Durometer Hardness	Duro =	50	(min)	AASHTO 14.7.6.2
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Shear Modulus

The least favorable value is assumed at each check since the material is specified by its hardness value (AASHTO 14.7.6.2). The shear modulus of the elastomer is based on a temperature of 73°.

$G_{max} =$	0.130	ksi	AASHTO T14.7.6.2-1
$G_{min} =$	0.095	ksi	
Check =	$0.08 \text{ ksi} < G < 0.175 \text{ ksi}$	OK	AASHTO 14.7.6.2

Steel Shim Properties

Yield Strength of Steel	$F_y =$	36.00	ksi	AASHTO T6.4.1-1
Allowable Fatigue Threshold	$\Delta F_{TH} =$	24.00	ksi	AASHTO T6.6.1.2.3-1

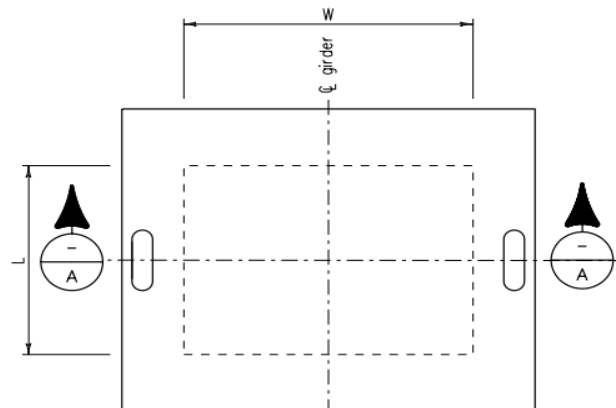


FIGURE 1 - BEARING DETAIL

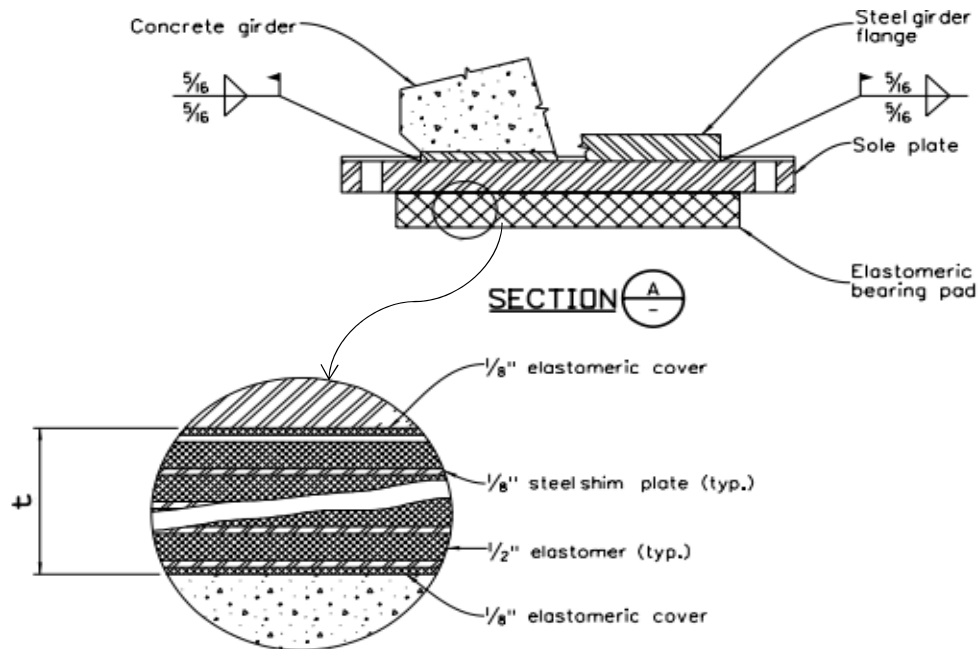


FIGURE 2 - TYPE I - STEEL REINFORCED BEARING DETAIL - SECTION

BEARING LOADS

Loads acting on the bearing are dead and live load girder reactions at the service limit state. Per AASHTO 14.4.1, dynamic load allowance is excluded from the live load influence. Loads are per bearing.

Service I Limit State Loads

DL = 102.00 kip
 LL = 43.00 kip

HORIZONTAL MOVEMENT

Shear deformations include movements from temperature, creep and shrinkage, prestressing effects, live loads, wind, braking, earthquake, and construction tolerances from service load combinations per AASHTO C14.4.1.

Uniform Temperature Movement Range:

Maximum temperature	$T_{max} =$	100	°F	AASHTO F3.12.2.2-1
Minimum temperature	$T_{min} =$	-20	°F	AASHTO F3.12.2.2-2
Coeff. of thermal expansion	$\alpha =$	6.0E-06	in/in/ °F	AASHTO 5.4.2.2
Expansion length	$L =$	75.00	ft = 900.00 in	
Service I Load Factor, TU	$\gamma_{TU} =$	1.20		AASHTO T3.4.1-1
AASHTO Reduction Factor	$\alpha_{AASHTO} =$	1.00		BDM 14.5.3

$\Delta_T = \alpha L(T_{max} - T_{min}) = 6.0E-6 * 900.00 * [100 - (-20)] = 0.65 \text{ in}$ AASHTO 3.12.2.3-1

Creep, Shrinkage, Elastic Shortening, Live Load, and Miscellaneous Movements:

Creep movement	$\Delta_{CR} =$	0.21	in
Shrinkage movement	$\Delta_{SH} =$	0.07	in
Elastic shortening	$\Delta_{EL} =$	0.00	in
Live load movement	$\Delta_{LL} =$	0.01	in
Miscellaneous movement	$\Delta_{MISC} =$	0.00	in

$\Delta_o =$ Maximum horizontal displacement of the superstructure
 $\Delta_s =$ Maximum shear deformation of the bearing modified to account for substructure stiffness

Assuming the substructure is stiff enough to prevent movement:

$$\Delta_o = \Delta_s = \sum \alpha_{AASHTO} \gamma_{TU} \Delta_T + \Delta_{CR} + \Delta_{SH} + \Delta_{EL} + \Delta_{LL} + \Delta_{MISC} = \text{AASHTO 14.7.5.3.2 \& BDM 14.5.3}$$

$$1.00 * 1.20 * 0.65 + 0.21 + 0.07 + 0.00 + 0.01 + 0.00 = 1.07 \text{ in}$$

SOLUTION

Shape Factor

Rectangular, steel reinforced bearing shape factor without holes:

$$S_i = \frac{LW}{2h_{ri}(L+W)} = \frac{(12.00 * 25.00)}{[2 * 0.50 * (12.00 + 25.00)]} = 8.11 \quad \text{AASHTO 14.7.5.1-1}$$

Confirm Method A is applicable for the design of the reinforced bearing pad per AASHTO 14.7.6.1

$$\text{Check } \frac{S_i^2}{n} < 22 = \frac{(8.11^2)}{5} = 13.15 < 22 \quad \text{OK} \quad \text{AASHTO 14.7.6.1}$$

Compressive Stress

AASHTO 14.7.6.3.2

The compressive stress of the bearing shall satisfy the criteria below for a steel reinforced elastomeric bearing pad. Since shear deformation is not prevented, a 10% increase in stress limits is not permitted (AASHTO 14.7.6.3.2).

σ_s = average compressive stress due to total load from applicable service load combinations

$$\sigma_s \leq 1.25 G_{min} S_i = 1.25 * 0.095 * 8.11 = 0.96 \text{ ksi} \quad \text{AASHTO 14.7.6.3.2-7}$$

and

$$\sigma_s \leq 1.25 \text{ ksi} \quad \text{AASHTO 14.7.6.3.2-8}$$

$$\sigma_s = \frac{DL + LL}{LW} = \frac{(102.00 + 43.00)}{(12.00 * 25.00)} = 0.48 \text{ ksi}$$

$$\text{Check } \sigma_s \leq 1.25 G_{min} S_i \quad 0.48 \text{ ksi} < 0.96 \text{ ksi} \quad \text{OK}$$

$$\text{Check } \sigma_s \leq 1.25 \text{ ksi} \quad 0.48 \text{ ksi} < 1.25 \text{ ksi} \quad \text{OK}$$

Compressive Deflection

Compressive deflection under instantaneous live load and initial dead load shall meet the following criteria in AASHTO 14.7.6.3.3 (Method A) and 14.7.5.3.6 (Method B). Note the design aids presented in Figure C14.7.6.3.3-1 are used in determining the compressive strain. For cases where the steel reinforced bearing pad material is specified by its shear modulus, equation C14.7.5.3.6-1 is used in lieu of these graphs to determine the strain in the bearing pad under applicable stresses. Refer to Example 3 accordingly.

Live Load Compressive Deflection

Minimizing deflection from instantaneous live loads is recommended when bridge joints are present. For jointless bridges, these criteria may be omitted. Method A requirements per 14.7.6.3.3 refer to Method B deflection checks per 14.7.5.3.6.

$$\delta_L \leq 0.125" \quad \text{AASHTO C14.7.5.3.6}$$

$$\delta_L = \sum \varepsilon_{Li} h_{ri} = \varepsilon_L h_{rt} \quad \text{compressive deflection due to live load} \quad \text{AASHTO 14.7.5.3.6-1}$$

ε_{Li} = instantaneous live load compressive strain in elastomeric pad determined from Figure C14.7.6.3.3-1

$\sigma_L =$ average compressive stress due to live load at the service limit state

$$\sigma_L = \frac{LL}{LW} = 43.00 / (12.00 * 25.00) = 0.14 \text{ ksi}$$

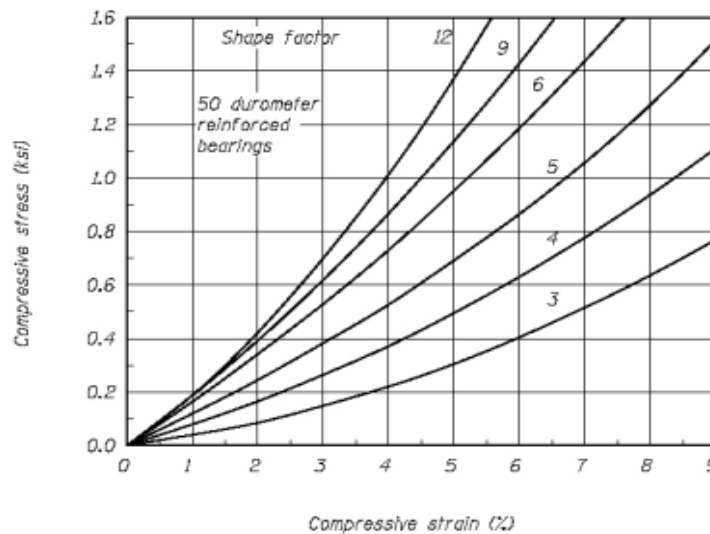


FIGURE 3 - AASHTO FIGURE C14.7.6.3.3-1

$\epsilon_{Li} =$ 0.005 AASHTO C14.7.6.3.3-1

$\delta_L = \epsilon_{Li} h_{rt} = 0.005 * 2.75 = 0.0138 \text{ in}$

Check $\delta_L \leq 0.125"$ 0.0138 in < 0.125 in **OK**

Dead Load Compressive Deflection

AASHTO does not have limitations on initial or long term dead load deflections. The following calculation is for demonstration only. Engineering judgment shall be used in evaluating appropriate allowable deflections in the bearing. Method A requirements per 14.7.6.3.3 refer to Method B deflection checks per 14.7.5.3.6.

Initial dead load deflection:

$\delta_d = \sum \epsilon_{di} h_{ri} =$ AASHTO 14.7.5.3.6-2

$\epsilon_{di} =$ initial dead load compressive strain in elastomeric pad determined from Figure C14.7.6.3.3-1

$\sigma_d =$ average compressive stress due to dead load at the service limit state

$$\sigma_d = \frac{DL}{LW} = 102.00 / (12.00 * 25.00) = 0.34 \text{ ksi}$$

$\epsilon_{di} =$ 0.0175 AASHTO C14.7.6.3.3-1

$\delta_d = \epsilon_{di} h_{rt} = 0.0175 * 2.75 = 0.0481 \text{ in}$

Long term dead load deflection:

$\delta_{lt} = \delta_d + \alpha_{cr} \delta_d$ AASHTO 14.7.5.3.6-3
 $\alpha_{cr} =$ 0.25 AASHTO T14.7.6.2-1

$\delta_{lt} = \delta_d + \alpha_{cr} \delta_d = 0.0481 + 0.25 * 0.0481 = 0.060 \text{ in.}$

Combined Live Load and Dead Load Compressive Deflection

$$\delta_s \leq .09h_{ri} = 0.09 \cdot 0.50 = 0.045 \text{ in.} \quad \text{AASHTO 14.7.6.3.3}$$

$$\delta_s = \sum \varepsilon_{si} h_{ri} \quad \text{compressive deflection due to live load and dead load} \quad \text{AASHTO 14.7.5.3.6-1}$$

ε_{si} = instantaneous live and dead load compressive strain in elastomeric pad determined from Figure C14.7.6.3.3-1

σ_s = average compressive stress due to total load from applicable service load combinations

$$\sigma_s = \frac{DL + LL}{LW} = 0.48 \text{ ksi}$$

$$\varepsilon_{si} = 0.026 \quad \text{AASHTO C14.7.6.3.3-1}$$

$$\delta_s = \varepsilon_{si} h_{ri} = 0.026 \cdot 0.50 = 0.0130 \text{ in}$$

$$\text{Check } \delta_s \leq .09h_{ri} \quad 0.0130 \text{ in} < 0.045 \text{ in} \quad \text{OK}$$

Shear Deformation

Total elastomer thickness = h_{rt}

$$h_{rt} \geq 2\Delta_s = 2 \cdot 1.07 = 2.14 \text{ in} \quad \text{AASHTO 14.7.6.3.4}$$

$$\text{Check } h_{rt} = 2.75 \text{ in} > 2.14 \text{ in} \quad \text{OK}$$

Stability

AASHTO 14.7.6.3.6

Total bearing thickness, t , shall not exceed the lesser of the following dimensions:

$$\frac{L}{3} = 12.00 / 3 = 4.00 \text{ in}$$

and

$$\frac{W}{3} = 25.00 / 3 = 8.33 \text{ in}$$

$$\text{Check } t = 3.50 \text{ in} < 4.00 \text{ in} \quad \text{OK}$$

Reinforcement

AASHTO 14.7.5.3.5

Note that holes are not present in the bearing. The allowable thickness does not need to be increased per AASHTO 14.7.5.3.5

The minimum thickness of steel reinforcement shall satisfy the following:

$$h_s \geq .0625 \text{ in}$$

and

(Service Limit State)

$$h_s \geq \frac{3h_{ri}\sigma_s}{F_y} = \frac{3 \cdot 0.500 \cdot 0.48}{36} = 0.020 \text{ in} \quad \text{AASHTO 14.7.5.3.5-1}$$

and

(Fatigue Limit State)

$$h_s \geq \frac{2h_{ri}\sigma_L}{\Delta F_{TH}} = \frac{2 \cdot 0.500 \cdot 0.14}{24.00} = 0.006 \text{ in} \quad \text{AASHTO 14.7.5.3.5-2}$$

Check

	$h_s =$	0.125 in	>	0.0625 in	OK
		0.125 in	>	0.020 in	OK
		0.125 in	>	0.006 in	OK

Anchorage (Bearing Pad Slip)

AASHTO 14.8.3

The bearing pad must be secured against horizontal movement if the shear force sustained by the deformed pad exceeds the minimum vertical force due to permanent loads modified for the concrete friction. G_{max} is used since the pad is stiffer at colder temperatures and will produce larger shear forces. Note the following example considers longitudinal deformations only; wind, breaking, and seismic loads shall also be considered as appropriate, in the direction of consideration.

$$H_b = \mu P_{min} \qquad \text{AASHTO 14.6.3.1-1}$$

and

$$H_b = G_{max} A \frac{\Delta_s}{h_{rt}} \qquad \text{AASHTO 14.6.3.1-2}$$

Combining equations:

$$\Delta_{s,allow} = \frac{\mu P_{min} h_{rt}}{G_{max} A} = \frac{(0.20 * 102.00 * 2.75)}{(0.13 * 300.00)} = 1.438 \text{ in.}$$

where

	$\mu =$	0.20		Coefficient of friction AASHTO C14.8.3.1
	$P_{min} = DL =$	102.00	kip	
	$A = LW =$	300.00	in ²	
	$h_{rt} =$	2.75	in	

Check

	$\Delta_{s,allow} =$	1.438 in	>	$\Delta_s =$	1.068 in OK
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In cases where Δ_s exceeds $\Delta_{s,allow}$, anchor bolts shall be sized and designed in accordance with those Articles specified in AASHTO 14.8.3

Geometry

The minimum unreinforced bearing pad thickness of 2 in. is assumed sufficient to prevent girder-to-support contact under the applied girder rotations and compressive deflections. Under extreme skews, large girder loads or rotations, and/or steep profile grades, the Designer shall confirm the bearing thickness. Refer to Example 1 - Elastomeric Leveling Pad.

APPENDIX A
EXAMPLE 3 - TYPE I BEARING (STEEL REINFORCED)
METHOD B

GENERAL INFORMATION

Per CDOT Bridge Design Manual (BDM) Section 14.5.8, steel reinforced bearing pads shall be designed using Method B. This example follows AASHTO LRFD 7th Edition Section 14.7.5.

This example assumes a steel superstructure that can displace under the effects of temperature and assumes a rectangular bearing shown below in Figures 1 and 2. Externally bonded plates are not used. The structure is assumed to move freely in the longitudinal direction only for the range of temperatures conforming to AASHTO 3.12.2.1 Procedure A.

MATERIAL AND SECTION PROPERTIES***Bearing Dimensions***

Bearing Width $W = 20.00$ in AASHTO 14.7.5.1
 Bearing Length $L = 15.00$ in AASHTO 14.7.5.1

Bearing Pad Layers

Exterior Elastomeric Thickness $h_{re} = 0.125$ in **OK** < 70% h_{ri} AASHTO 14.7.5.1
 Interior Elastomeric Thickness $h_{ri} = 0.500$ in
 Steel Plate Thickness $h_s = 0.125$ in
 No. of Steel Shim Plates $n_{shims} = 10$
 No. of Interior Elastomer Layers $n = 9$ AASHTO 14.7.5.3.3
 Total Elastomer Thickness $h_{rt} = 4.750$ in
 Total Bearing Height $t = 6.00$ in **OK** 2" minimum height per BDM 14.5.8

Bearing Material Properties

Elastomer Grade Grade = **3** (Zone C) BDM 14.5.8, & AASHTO Table & Figure 14.7.5.2-1

Shear Modulus

Design drawings shall specify the shear modulus of the elastomer at 73°. With an acceptance variation of $\pm 15\%$ of the specified value, the shear modulus used in design will vary. The shear modulus shall be taken as the least favorable value within the range to cause the more conservative outcome in the specific analysis being considered (AASHTO 14.7.5.2). The plan shear modulus below assumes a Durometer Hardness of 60.

$G_{plan} = 0.150$ ksi AASHTO T14.7.6.2-1
 $G_{max} = 0.173$ ksi
 $G_{min} = 0.128$ ksi
 Check = $0.08 \text{ ksi} < G < 0.175 \text{ ksi}$ **OK** AASHTO 14.7.5.2

Creep Deflection Factor $\alpha_{cr} = 0.35$ AASHTO T14.7.6.2-1

Steel Shim Properties

Yield Strength of Steel $F_y = 36.00$ ksi AASHTO T6.4.1-1
 Allowable Fatigue Threshold $\Delta F_{TH} = 24.00$ ksi AASHTO T6.6.1.2.3-1

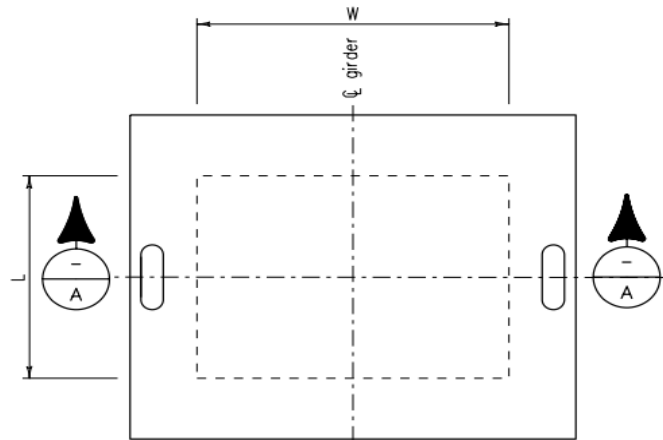


FIGURE 1 - TYPE I - STEEL REINFORCED BEARING DETAIL - PLAN

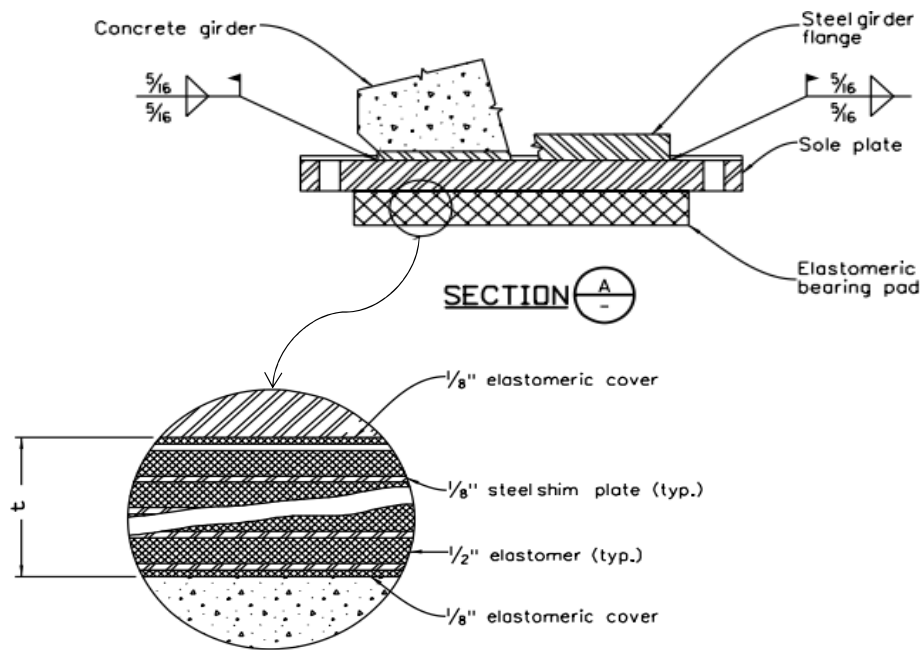


FIGURE 2 - TYPE I - STEEL REINFORCED BEARING DETAIL - SECTION

BEARING LOADS

Loads acting on the bearing are dead and live load girder reactions at the service limit state. Per AASHTO 14.4.1, dynamic load allowance is excluded from the live load influence. Loads are per bearing.

Service I Limit State Loads

DL = 115.00 kip
 LL = 85.00 kip

BEARING ROTATIONS

Rotations include effects of girder camber. For all rotation values, positive indicates a downward rotation while negative indicates an upward rotation. Note this example does not account for profile grade differences between supports.

Service I Limit State Rotations

Dead Load Rotations $\theta_d = -0.002$ rad
 Live Load Rotations $\theta_L = 0.001$ rad

Include a construction tolerance of 0.005 radians to account for uncertainties in bearing fabrication and bearing seat construction. Per BDM 14.5.4, the flatness tolerance for bearing seat uncertainties is accounted for in the construction tolerance.

Construction Tolerance $\theta_r = 0.005$ rad AASHTO 14.4.2.1

HORIZONTAL MOVEMENT

Shear deformations include movements from temperature, creep and shrinkage, prestressing effects, and miscellaneous movement from loads such as live and wind loads from service load combinations per AASHTO C14.4.1. Assume the bearings are not adjusted after construction; therefore, the 65 percent reduction in thermal movement range per AASHTO 14.7.5.3.2 is not included per BDM 14.5.3.

Uniform Temperature Movement Range:

Maximum temperature	$T_{max} = 120$	°F	AASHTO T3.12.2.1-1
Minimum temperature	$T_{min} = -30$	°F	AASHTO T3.12.2.1-1
Coeff. of thermal expansion	$\alpha = 6.5E-06$	in/in/°F	AASHTO 6.4.1
Expansion length	$L = 80.00$	ft = 960.00 in	
Service I Load Factor, TU	$\gamma_{TU} = 1.20$		AASHTO T3.4.1-1
AASHTO Reduction Factor	$\alpha_{AASHTO} = 1.00$		BDM 14.5.3

$\Delta_T = \alpha L (T_{max} - T_{min}) = 6.5E-6 * 960.00 * [120 - (-30)] = 0.94$ in AASHTO 3.12.2.3-1

Creep, Shrinkage, Elastic Shortening, Live Load, and Miscellaneous Movements:

Creep movement	$\Delta_{CR} = 0.00$	in
Shrinkage movement	$\Delta_{SH} = 0.00$	in
Elastic shortening	$\Delta_{EL} = 0.00$	in
Live load movement	$\Delta_{LL} = 0.04$	in
Miscellaneous movement	$\Delta_{MISC} = 0.50$	in

$\Delta_o =$ Maximum horizontal displacement of the superstructure
 $\Delta_s =$ Maximum shear deformation of the bearing modified to account for substructure stiffness

Assuming the substructure is stiff enough to prevent movement:

$\Delta_o = \Delta_s = \sum \alpha_{AASHTO} \gamma_{TU} \Delta_T + \Delta_{CR} + \Delta_{SH} + \Delta_{EL} + \Delta_{LL} + \Delta_{MISC} =$ AASHTO 14.7.5.3.2 & BDM 14.5.3

$1.00 * 1.20 * 0.94 + 0.00 + 0.00 + 0.00 + 0.04 + 0.50 = 1.66$ in

SOLUTION

Shape Factor

Rectangular, steel reinforced bearing shape factor without holes:

$S_i = \frac{LW}{2hri(L+W)} = (15.00 * 20.00) / [2 * 0.500 * (15.00 + 20.00)] = 8.57$ AASHTO 14.5.7.1-1

Computed Compressive Stresses

$$\sigma_s = \frac{DL + LL}{LW} = (115.00 + 85.00) / (15.00 * 20.00) = 0.67 \text{ ksi}$$

σ_s = average compressive stress due to total load from applicable service load combinations

$$\sigma_L = \frac{LL}{LW} = 85.00 / (15.00 * 20.00) = 0.28 \text{ ksi}$$

σ_L = average compressive stress due to live load at the service limit state (cyclic load)

$$\sigma_d = \frac{DL}{LW} = 115.00 / (15.00 * 20.00) = 0.38 \text{ ksi}$$

σ_d = average compressive stress due to dead load at the service limit state (static load)

Compressive Deflections

AASHTO 14.7.5.3.6

Live Load Compressive Deflection

Minimizing deflection from instantaneous live loads is recommended when bridge joints are present. For jointless bridges, these criteria may be omitted.

$$\delta_L \leq 0.125" \quad \text{AASHTO C14.7.5.3.6}$$

$$\delta_L = \sum \varepsilon_{Li} h_{rt} = \varepsilon_L h_{rt} \quad \text{AASHTO 14.7.5.3.6-1}$$

ε_{Li} = instantaneous live load compressive strain in elastomeric pad

$$\varepsilon_{Li} = \frac{\sigma_L}{4.8G_{min}S_i^2} = 0.28 / (4.8 * 0.13 * 8.57^2) = 0.006 \quad \text{AASHTO C14.7.5.3.6-1}$$

$$\delta_L = \varepsilon_{Li} h_{rt} = 0.006 * 4.750 = 0.030 \text{ in}$$

$$\text{Check } \delta_L \leq 0.125" \quad 0.030 \text{ in} < 0.125 \text{ in} \quad \text{OK}$$

Dead Load Compressive Deflection

AASHTO Method B does not have limitations on initial or long term dead load deflections. The following calculation is for demonstration only. Engineering judgment shall be used in evaluating appropriate allowable deflections in the bearing.

Initial dead load deflection:

$$\delta_d = \sum \varepsilon_{di} h_{rt} = \quad \text{AASHTO 14.7.5.3.6-2}$$

ε_{di} = initial dead load compressive strain in i th layer of elastomeric pad

$$\varepsilon_{di} = \frac{\sigma_d}{4.8G_{min}S_i^2} = 0.38 / (4.8 * 0.13 * 8.57^2) = 0.009 \quad \text{AASHTO C14.7.5.3.6-1}$$

$$\delta_d = \varepsilon_{di} h_{rt} = 0.009 * 4.750 = 0.040 \text{ in.}$$

Long term dead load deflection:

AASHTO 14.7.5.3.6-3

$$\delta_{lt} = \delta_d + \alpha_{cr} \delta_d = 0.040 + 0.35 * 0.040 = 0.055 \text{ in.} \quad \text{AASHTO T14.7.6.2-1}$$

Shear Deformations

Total elastomer thickness = h_{rt}

$$h_{rt} \geq 2\Delta_s = 2 * 1.66 = 3.33 \text{ in} \quad \text{AASHTO 14.7.5.3.2-1}$$

Check $h_{rt} = 4.75 \text{ in} > 3.33 \text{ in} \quad \text{OK}$

Combined Compression, Rotation, and Shear

AASHTO 14.7.5.3.3

For demonstration purposes, only rotation about the transverse direction is verified. The Designer shall evaluate the bearing about both the longitudinal and transverse axis as appropriate, especially in cases where the structure contains a significant skew (AASHTO C14.7.5.3.3). Cyclic loading shall consist of loads induced by traffic with all other loads considered static (AASHTO 14.7.5.3.3).

$$(\gamma_{a,st} + \gamma_{r,st} + \gamma_{s,st}) + 1.75(\gamma_{a,cy} + \gamma_{r,cy} + \gamma_{s,cy}) \leq 5.0 \quad \text{AASHTO 14.7.5.3.3-1}$$

and

$$\gamma_{a,st} \leq 3.0 \quad \text{AASHTO 14.7.5.3.3-2}$$

Axial Load Shear Strain

Axial strain from static loads: $\gamma_{a,st} = D_a \frac{\sigma_{s,st}}{G S_i}$ AASHTO 14.7.5.3.3-3

Axial strain from cyclic loads: $\gamma_{a,cy} = D_a \frac{\sigma_{s,cy}}{G S_i}$ AASHTO 14.7.5.3.3-3

where:

$D_a = 1.40$ AASHTO 14.7.5.3.3-4

$\sigma_{s,st} = \sigma_d =$ Compressive stress due to total static load at service limit state

$\sigma_{s,cy} = \sigma_L =$ Compressive stress due to cyclic load at service limit state

$$\gamma_{a,st} = D_a \frac{\sigma_{s,st}}{G_{min} S_i} = \frac{1.40 * 0.38}{0.13 * 8.57} = 0.491$$

$$\gamma_{a,cy} = D_a \frac{\sigma_{s,cy}}{G_{min} S_i} = \frac{1.40 * 0.28}{0.13 * 8.57} = 0.363$$

Rotational Shear Strain

Rotational strain from static loads: $\gamma_{r,st} = D_r \left(\frac{L}{h_{ri}} \right)^2 \frac{\theta_{s,st}}{n}$ AASHTO 14.7.5.3.3-6

Rotational strain from cyclic loads: $\gamma_{r,cy} = D_r \left(\frac{L}{h_{ri}} \right)^2 \frac{\theta_{s,cy}}{n}$ AASHTO 14.7.5.3.3-6

where

$D_r = 0.50$ AASHTO 14.7.5.3.3-7

$\theta_{s,st} = \theta_d + \theta_r =$ Maximum static service limit state design rotation

$\theta_{s,cy} = \theta_L =$ Maximum cyclic service limit state design rotation

$$\gamma_{r,st} = D_r \left(\frac{L}{h_{ri}} \right)^2 \frac{\theta_{s,st}}{n} = 0.50 (15.00 / 0.500) ^2 * (-0.002 + 0.005) / 9 = 0.150$$

$$\gamma_{r,cy} = D_r \left(\frac{L}{h_{ri}} \right)^2 \frac{\theta_{s,cy}}{n} = 0.50 (15.00 / 0.500) ^2 * (0.001) / 9 = 0.050$$

Shear Deformation Shear Strain

Shear strain from static loads: $\gamma_{s,st} = \frac{\Delta_{s,st}}{h_{rt}}$ AASHTO 14.7.5.3.3-10

Shear strain from cyclic loads: $\gamma_{s,cy} = \frac{\Delta_{s,cy}}{h_{rt}}$ AASHTO 14.7.5.3.3-10

where

$$\Delta_{s,st} = \Delta_s - \Delta_{LL} = 1.663 \text{ in}$$

$$\Delta_{s,cy} = \Delta_{LL} = 0.040 \text{ in}$$

$$\gamma_{s,st} = \frac{\Delta_{s,st}}{h_{rt}} = \frac{1.663}{4.750} = 0.350$$

$$\gamma_{s,cy} = \frac{\Delta_{s,cy}}{h_{rt}} = \frac{0.040}{4.750} = 0.008$$

Combined Shear Strains Checks

$$(\gamma_{a,st} + \gamma_{r,st} + \gamma_{s,st}) + 1.75(\gamma_{a,cy} + \gamma_{r,cy} + \gamma_{s,cy}) \leq 5.0$$

$$= 0.491 + 0.150 + 0 + 1.75(0.363 + 0.050 + 0.008) = 1.73 < 5.00 \quad \text{OK}$$

$$\gamma_{a,st} \leq 3.0 \quad \gamma_{a,st} = 0.491 < 3.00 \quad \text{OK}$$

Stability

If the following is satisfied, no further investigation of stability is required:

$$2A \leq B$$

where

$$A = \frac{1.92 \frac{h_{rt}}{L}}{\sqrt{1 + \frac{2.0L}{W}}} = \frac{1.92 * (4.750 / 15.00)}{\text{SQRT} [1 + (2 * 15.00) / 20.00]} = 0.38$$

$$B = \frac{2.67}{(S_i + 2.0)(1 + \frac{L}{4.0W})} = \frac{2.67}{(8.57 + 2.0) * [1 + 15.00 / (4.0 * 20.00)]} = 0.21$$

Note that if L is greater than W, stability shall be investigated by interchanging L and W. L= 15.00 in
W= 20.00 in

Check $2A = 2 * 0.38 = 0.77 > 0.21 = B$ **FAILS**

If the above criteria for stability are not satisfied, the following equations shall be investigated:

For a bridge deck that is free to translate horizontally:

For demonstration only. Designer shall determine movement capability of bridge on a case by case basis.

$$\sigma_s \leq \frac{G_{min}S_i}{2A - B} = \frac{0.13*8.57}{2*0.38 - 0.21} = 1.96 \text{ ksi} \quad \text{AASHTO 14.7.5.3.4-4}$$

Check $\sigma_s = 0.67 \text{ ksi} < 1.96 \text{ ksi}$ **OK Bearing is Stable**

For a bridge deck that is fixed against horizontal translation:

For demonstration only. Designer shall determine movement capability of bridge on a case by case basis.

$$\sigma_s \leq \frac{G_{min}S_i}{A - B} = \frac{0.13*8.57}{0.38 - 0.21} = 6.36 \text{ ksi} \quad \text{AASHTO 14.7.5.3.4-5}$$

Check $\sigma_s = 0.67 \text{ ksi} < 6.36 \text{ ksi}$ **OK Bearing is Stable**

Reinforcement

AASHTO 14.7.5.3.5

Note that holes are not present in the bearing. The allowable thickness does not need to be increased per AASHTO 14.7.5.3.5

The minimum thickness of steel reinforcement shall satisfy the following:

$$h_s \geq .0625 \text{ in}$$

and

(Service Limit State)

$$h_s \geq \frac{3h_{ri}\sigma_s}{F_y} = \frac{3*0.500*0.67}{36} = 0.028 \text{ in} \quad \text{AASHTO 14.7.5.3.5-1}$$

and

(Fatigue Limit State)

$$h_s \geq \frac{2h_{ri}\sigma_L}{\Delta F_{TH}} = \frac{2*0.500*0.28}{24.00} = 0.012 \text{ in} \quad \text{AASHTO 14.7.5.3.5-2}$$

Check

$h_s =$	0.125 in	>	0.0625 in	OK
	0.125 in	>	0.028 in	OK
	0.125 in	>	0.012 in	OK

Bearing Anchorage

AASHTO 14.7.5.4

For bearings without externally bonded plates, a restraint system is required to secure the bearing against horizontal movement if:

$$\frac{\theta_s}{n} \geq \frac{3\varepsilon_a}{S_i} \quad \text{AASHTO 14.7.5.4-1}$$

where

θ_s = total of static and cyclic service limit state design rotation. Cyclic component is multiplied by 1.75

ε_a = total of static and cyclic average axial strain. Cyclic component is multiplied by 1.75

$$\begin{aligned} \theta_s &= \theta_{s,st} + 1.75\theta_{s,cy} = \theta_d + \theta_r + 1.75\theta_L = \\ &= -0.002 + 0.005 + 1.75 * 0.001 = \quad \quad \quad 0.005 \text{ rad} \end{aligned}$$

$$\varepsilon_a = \varepsilon_{st} + 1.75\varepsilon_{cy} = \varepsilon_d + 1.75\varepsilon_L = \quad 0.009 + 1.75 * 0.006 = \quad 0.020$$

Check

$$\frac{\theta_s}{n} \geq \frac{3\varepsilon_a}{S_i} = \quad \frac{0.005}{9} = 0.001 \quad < \quad \frac{3 * 0.020}{8.57} = 0.007 \quad \text{FAILS} \quad \text{Restraint Required}$$

If the Engineer elects to use externally bonded plates, limitations on hydrostatic pressure per AASHTO 14.7.5.3.3-11 shall be satisfied.

Anchorage (Bearing Pad Slip)

AASHTO 14.8.3

The bearing pad must be secured against horizontal movement if the shear force sustained by the deformed pad exceeds the minimum vertical force due to permanent loads modified for the concrete friction. G_{max} is used since the pad is stiffer at colder temperatures and will produce larger shear forces. Note this example considers longitudinal deformations only; wind, braking, and seismic loads shall also be considered as appropriate, in the direction of consideration.

$$H_b = \mu P_{min} \quad \text{AASHTO 14.6.3.1-1}$$

and

$$H_b = G_{max} A \frac{\Delta_s}{h_{rt}} \quad \text{AASHTO 14.6.3.1-2}$$

Combining equations:

$$\Delta_{s,allow} = \frac{\mu P_{min} h_{rt}}{G_{max} A} = \quad 0.20 * 115.00 * 4.75 / (0.17 * 300.00) = \quad 2.11 \text{ in}$$

where

$$\begin{aligned} \mu &= 0.20 && \text{Coefficient of friction AASHTO C14.8.3.1} \\ P_{min} &= DL = 115.00 && \text{kip} \\ A &= LW = 300.00 && \text{in}^2 \\ h_{rt} &= 4.75 && \text{in} \end{aligned}$$

Check

$$\Delta_{s,allow} = \quad 2.11 \text{ in} \quad > \quad \Delta_s = \quad 1.66 \text{ in} \quad \text{OK}$$

In cases where Δ_s exceeds $\Delta_{s,allow}$, anchor bolts shall be sized and designed in accordance with those Articles specified in AASHTO 14.8.3

APPENDIX A

**EXAMPLE 4 - TYPE II BEARING (REINFORCED BEARING WITH PTFE)
METHOD B**

GENERAL INFORMATION

Per CDOT Bridge Design Manual (BDM) Section 14.5.9, Type II bearings are Type I bearings with a PTFE sliding surface. Type II bearings shall meet the same requirements as steel reinforced bearings, in addition to providing adequate slip on the sliding plane to accommodate horizontal movements without causing excessive bearing pad deformation. The following example is in accordance with Method B procedures per AASHTO LRFD 7th Edition Section 14.7.5.

This example assumes a concrete superstructure that can displace under the effects of temperature, creep, and shrinkage and assumes a rectangular bearing shown below in Figures 1 and 2. The bearing is fixed in the transverse direction and free to move longitudinally. Assume temperature movements conform to AASHTO 3.12.2.2 Procedure B. The PTFE surface is assumed unfilled and lubricated and no externally bonded plates are present.

MATERIAL AND SECTION PROPERTIES

Bearing Dimensions

Bearing Width	W =	24.00	in		AASHTO 14.7.5.1
Bearing Length	L =	10.00	in		AASHTO 14.7.5.1

Bearing Pad Layers

Exterior Elastomeric Thickness	$h_{re} =$	0.125	in	OK	< 70% h_{ri} AASHTO 14.7.5.1
Interior Elastomeric Thickness	$h_{ri} =$	0.500	in		
Steel Plate Thickness	$h_s =$	0.125	in		
No. of Steel Shim Plates	$n_{shims} =$	5			
No. of Interior Elastomer Layers	n =	5			AASHTO 14.7.5.3.3
Total Elastomer Thickness	$h_{rt} =$	2.625	in		
Total Bearing Height	t =	3.250	in	OK	2" minimum height per BDM 14.5.8

PTFE

PTFE thickness	$h_{PTFE} =$	0.094	in	OK	AASHTO 14.7.2.3
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Bearing Material Properties

Elastomer Grade	Grade =	3	(Zone C)		BDM 14.5.8, & AASHTO Table & Figure 14.7.5.2-1
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Shear Modulus

Design drawings shall specify the shear modulus of the elastomer at 73°. With an acceptance variation of ± 15% of the specified value, the shear modulus used in design will vary. The shear modulus shall be taken as the least favorable value within the range to cause the more conservative outcome in the specific analysis being considered (AASHTO 14.7.5.2). The plan shear modulus below assumes a Durometer Hardness of 60.

	$G_{plan} =$	0.150	ksi		AASHTO T14.7.6.2-1
	$G_{max} =$	0.173	ksi		
	$G_{min} =$	0.128	ksi		
	Check =	0.08 ksi < G < 0.175 ksi		OK	AASHTO 14.7.5.2

Creep Deflection Factor	$\alpha_{cr} =$	0.35			AASHTO T14.7.6.2-1
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Steel Shim Properties

Yield Strength of Steel	$F_y =$	36.00	ksi		AASHTO T6.4.1-1
Allowable Fatigue Threshold	$\Delta F_{TH} =$	24.00	ksi		AASHTO T6.6.1.2.3-1

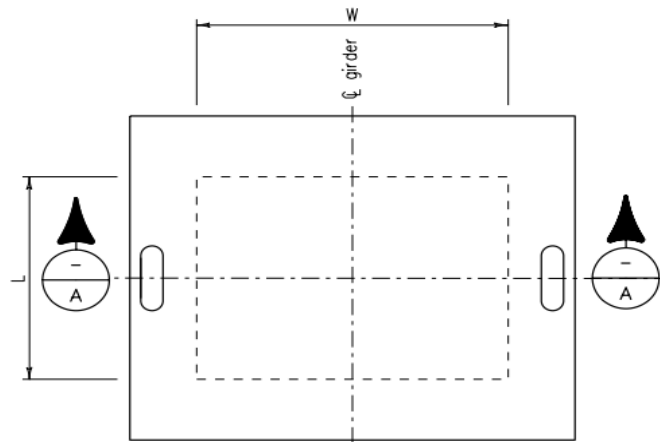


FIGURE 1 - TYPE II - STEEL REINFORCED BEARING PAD WITH PTFE - PLAN

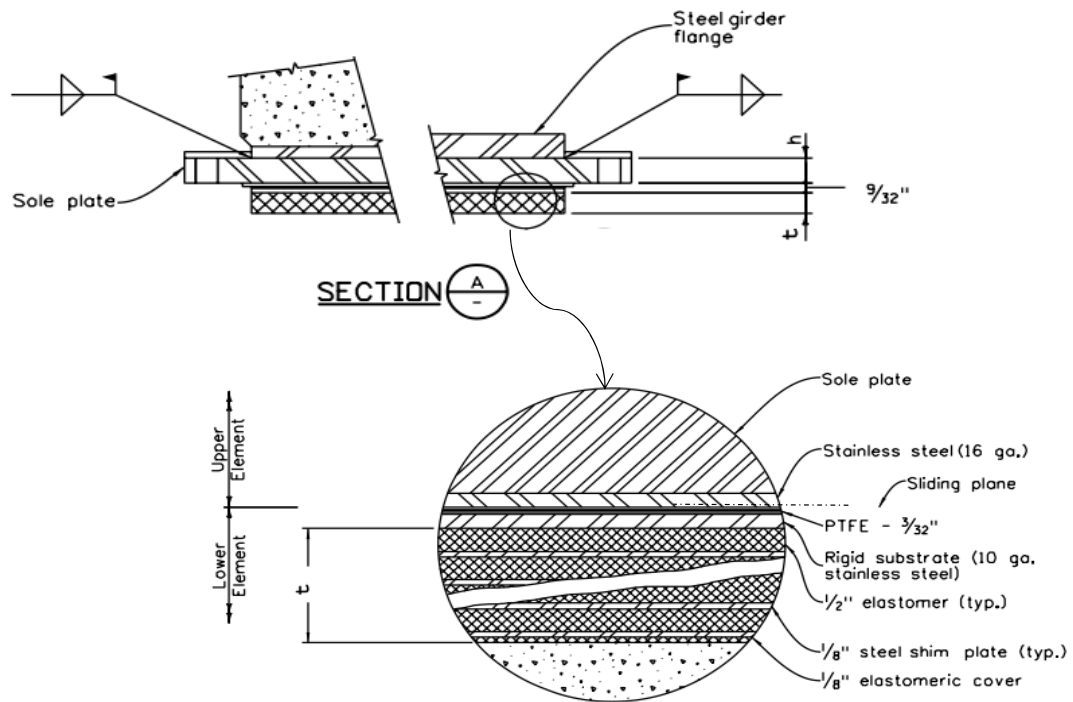


FIGURE 2 - TYPE II - STEEL REINFORCED BEARING PAD WITH PTFE - SECTION

BEARING LOADS

Loads acting on the bearing are dead and live load girder reactions at the service limit state. Per AASHTO 14.4.1, dynamic load allowance is excluded from the live load influence.

Service I Limit State Loads

DL = 200.00 kip
 LL = 60.00 kip

BEARING ROTATIONS

Rotations include effects of girder camber. For all rotation values, positive indicates a downward rotation while negative indicates an upward rotation. Note this example does not account for profile grade differences between supports.

Service I Limit State Rotations

Dead Load Rotations	$\theta_d =$	-0.001	rad
Live Load Rotations	$\theta_L =$	0.006	rad

Include a construction tolerance of 0.005 radians to account for uncertainties in bearing fabrication and bearing seat construction. Per BDM 14.5.4, the flatness tolerance for bearing seat uncertainties is accounted for in the construction

Construction Tolerance	$\theta_r =$	0.005	rad	AASHTO 14.4.2.1
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HORIZONTAL MOVEMENT

Shear deformations include movements from temperature, creep and shrinkage, prestressing effects, and miscellaneous movement from loads such as live and wind loads from service load combinations per AASHTO C14.4.1. Assume the bearings are not adjusted after construction; therefore, the 65 percent reduction in thermal movement range per AASHTO 14.7.5.3.2 is not included per BDM 14.5.3.

Uniform Temperature Movement Range:

Maximum temperature	$T_{max} =$	110	°F	AASHTO F3.12.2.2-1
Minimum temperature	$T_{min} =$	-10	°F	AASHTO F3.12.2.2-2
Coeff. of thermal expansion	$\alpha =$	6.0E-06	in/in/ °F	AASHTO 6.4.1
Expansion length	$L =$	150.00	ft = 1800.00 in	
Service I Load Factor, TU	$\gamma_{TU} =$	1.20		AASHTO T3.4.1-1
AASHTO Reduction Factor	$\alpha_{AASHTO} =$	1.00		BDM 14.5.3

$$\Delta_T = \alpha L (T_{max} - T_{min}) = 6.0E-6 * 1800.00 * [110 - (-10)] = 1.30 \text{ in} \quad \text{AASHTO 3.12.2.3-1}$$

Creep, Shrinkage, Elastic Shortening, and Miscellaneous Movements:

Creep movement	$\Delta_{CR} =$	0.58	in
Shrinkage movement	$\Delta_{SH} =$	0.60	in
Elastic shortening	$\Delta_{EL} =$	0.05	in
Live load movement	$\Delta_{LL} =$	0.10	in
Miscellaneous movement	$\Delta_{MISC} =$	0.00	in

$\Delta_o =$ Maximum horizontal displacement of the superstructure

$\Delta_s =$ Maximum shear deformation of the bearing modified to account for substructure stiffness

Assuming the substructure is stiff enough to prevent movement:

AASHTO
14.7.5.3.2 &
BDM Section
14.5.3

$$\Delta_o = \Delta_s = \sum \alpha_{AASHTO} \gamma_{TU} \Delta_T + \Delta_{CR} + \Delta_{SH} + \Delta_{EL} + \Delta_{LL} + \Delta_{MISC} = 1.00 * 1.20 * 1.30 + 0.58 + 0.60 + 0.05 + 0.10 + 0.00 = 2.89 \text{ in}$$

SOLUTION

Shape Factor

Rectangular, steel reinforced bearing shape factor without holes:

AASHTO 14.5.7.1-1

$$S_i = \frac{LW}{2h_{ri}(L+W)} = \frac{(10.00 * 24.00)}{[2 * 0.500 * (10.00 + 24.00)]} = 7.06$$

Computed Stresses

AASHTO 14.7.2.4

Average Contact Stresses

$$\sigma_s = \frac{DL + LL}{LW} = (200.00 + 60.00) / (10.00 * 24.00) = 1.08 \text{ ksi}$$

σ_s = average compressive stress due to total load from applicable service load combinations

$$\sigma_L = \frac{LL}{LW} = 60.00 / (10.00 * 24.00) = 0.25 \text{ ksi}$$

σ_L = average compressive stress due to live load at the service limit state (cyclic load)

$$\sigma_d = \frac{DL}{LW} = 200.00 / (10.00 * 24.00) = 0.83 \text{ ksi}$$

σ_d = average compressive stress due to dead load at the service limit state (static load)

Check

AASHTO T14.7.2.4-1

$$\sigma_s = 1.08 \text{ ksi} < \sigma_{Sallow} = 2.50 \text{ ksi} \quad \text{OK}$$

$$\sigma_d = 0.83 \text{ ksi} < \sigma_{DLallow} = 1.50 \text{ ksi} \quad \text{OK}$$

Edge Contact Stresses

AASHTO 14.7.2.4

The contact stress at the edge shall be determined by considering the maximum service moment transferred by the bearing, assuming a linear distribution of stress across the PTFE. The moment is assumed to occur at the centerline of the bearing, perpendicular to the direction of load.

$$M_s = (0.5E_c I) \frac{\theta_s}{h_{rt}} \quad \text{Service moment due to total load}$$

AASHTO

14.6.3.2-3

AASHTO

C14.6.3.2

where

$$E_c = 4.8G_{max} S_i^2 = 4.8 * 0.17 * 7.06^2 = 41.26 \text{ ksi}$$

AASHTO C14.6.3.2-1

$$I = \frac{WL^3}{12} = 24.00 * 10.00^3 / 12 = 2000.00 \text{ in}^4$$

$$\theta_s = \theta_d + \theta_r + \theta_L = -0.001 + 0.005 + 0.006 = 0.010 \text{ rad}$$

$$M_s = (0.5E_c I) \frac{\theta_s}{h_{rt}} = (0.5 * 41.26 * 2000.00) * (0.010 / 2.63) = 157.17 \text{ k-in}$$

$$\sigma_{s,edge} = \sigma_s + \frac{M_s}{1/6WL^2} = 1.08 + [157.17 / ((1/6) * 24.00 * 10.00^2)] = 1.48 \text{ ksi}$$

$$M_d = (0.5E_c I) \frac{\theta_D}{h_{rt}} \quad \text{Service moment due to dead load}$$

AASHTO

14.6.3.2-3

AASHTO

C14.6.3.2

where

$$\theta_D = \theta_d + \theta_r = -0.001 + 0.005 = 0.004 \text{ rad}$$

$$M_d = (0.5E_c I) \frac{\theta_D}{h_{rt}} = (0.5 * 41.26 * 2000.00) * (0.004 / 2.63) = 62.87 \text{ k-in}$$

$$\sigma_{d,edge} = \sigma_d + \frac{M_d}{1/6WL^2} = 0.83 + [62.87 / ((1/6) * 24.00 * 10.00^2)] = 0.99 \text{ ksi}$$

where $\frac{M_d}{1/6WL^2}$ is derived from My/I , where y is $L/2$, and I is $WL^3/12$

Check AASHTO T14.7.2.4-1

$$\sigma_{s,edge} = 1.48 \text{ ksi} < \sigma_{sallow} = 3.00 \text{ ksi} \quad \text{OK}$$

$$\sigma_{d,edge} = 0.99 \text{ ksi} < \sigma_{dallow} = 2.00 \text{ ksi} \quad \text{OK}$$

Coefficient of Friction of PTFE

AASHTO 14.7.2.5

For the given minimum temperature and the average pressure at the service limit state, interpolate between values in AASHTO T14.7.2.5-1. Assume a lubricated PTFE surface.

$\sigma_s = 1.083 \text{ ksi}$
 \downarrow

Temp. °F	Pressure (ksi)		
	1.0	1.083	2.0
68	0.030	0.0296	0.025
-10	0.044	0.0440	0.039
-13	0.045	0.0446	0.040

$T_{min} = -10 \rightarrow$

$\mu_{PTFE} = 0.044$

Shear Deformations

Since a low friction sliding surface is implemented, Δ_s need not be taken larger than the deformation corresponding to first slip (AASHTO 14.7.5.3.2). The minimum pressure will create the largest coefficient of friction and the largest movement.

The minimum service shear force transferred by the sliding surface at the specified minimum temperature:

$$H_b = \mu_{PTFE}(DL) = 0.044*(200.00) = 8.81 \text{ kip} \quad \text{AASHTO 14.6.3.1-1}$$

The deflection of the elastomeric bearing, before first slip of the sliding surface, is estimated as:

$$\Delta_{s,slip} = \frac{H_b h_{rt}}{G_{min} WL} = \frac{8.81*2.63}{0.13*24.00*10.00} = 0.76 \text{ in} \quad \text{AASHTO 14.6.3.1-2}$$

$$h_{rt} \geq 2\Delta_{s,slip} = 2*0.76 = 1.51 \text{ in} \quad \text{AASHTO 14.7.5.3.2-1}$$

Check $h_{rt} = 2.63 \text{ in} > 1.51 \text{ in} \quad \text{OK}$

The remaining movement is accommodated by the movement of the PTFE surface:

$$\Delta_{sPTFE} = \Delta_s - \Delta_{s,slip} = 2.89 - 0.76 = 2.13 \text{ in}$$

The Designer shall size the steel sliding surface, sole plate, anchor bolt holes, and edge distances accordingly to accommodate the above movement.

Compressive Deflections

AASHTO 14.7.5.3.6

Live Load Compressive Deflection

Minimizing deflection from instantaneous live loads is recommended when bridge joints are present. For jointless bridges, these criteria may be omitted.

$$\delta_L \leq 0.125" \quad \text{AASHTO C14.7.5.3.6}$$

$$\delta_L = \sum \varepsilon_{Li} h_{ri} = \varepsilon_{Li} h_{rt} \quad \text{AASHTO 14.7.5.3.6-1}$$

ε_{Li} = instantaneous live load compressive strain in elastomeric pad

$$\varepsilon_{Li} = \frac{\sigma_L}{4.8G_{min}S_i^2} = 0.25 / (4.8 * 0.128 * 7.06^2) = 0.008 \quad \text{AASHTO C14.7.5.3.6-1}$$

$$\delta_L = \varepsilon_{Li} h_{rt} = 0.008 * 2.625 = 0.022 \text{ in.}$$

Check $\delta_L \leq 0.125" \quad 0.022 \text{ in.} < 0.125 \text{ in.} \quad \text{OK}$

Dead Load Compressive Deflection

AASHTO Method B does not have limitations on initial or long term dead load deflections. The following calculation is for demonstration only. Engineering judgment shall be used in evaluating appropriate allowable deflections in the bearing.

Initial dead load deflection:

$$\delta_d = \sum \varepsilon_{di} h_{ri} = \varepsilon_{di} h_{rt} \quad \text{AASHTO 14.7.5.3.6-2}$$

ε_{di} = initial dead load compressive strain in i th layer of elastomeric pad

$$\varepsilon_{di} = \frac{\sigma_d}{4.8G_{min}S^2} = 0.83 / (4.8 * 0.13 * 7.06^2) = 0.027 \quad \text{AASHTO C14.7.5.3.6-1}$$

$$\delta_d = \varepsilon_{di} h_{rt} = 0.027 * 2.625 = 0.072 \text{ in.}$$

Long term dead load deflection:

AASHTO 14.7.5.3.6-3

$$\delta_{lt} = \delta_d + \alpha_{cr} \delta_d$$

$$= 0.072 + 0.35 * 0.072 = 0.097 \text{ in} \quad \text{AASHTO T14.7.6.2-1}$$

Combined Compression, Rotation, and Shear

AASHTO 14.7.5.3.3

For demonstration purposes, only rotation about the transverse direction is verified. The Designer shall evaluate the bearing about both the longitudinal and transverse axis as appropriate, especially in cases where the structure contains a significant skew (AASHTO C14.7.5.3.3). Cyclic loading shall consist of loads induced by traffic with all other loads considered static (AASHTO 14.7.5.3.3).

$$(\gamma_{a,st} + \gamma_{r,st} + \gamma_{s,st}) + 1.75(\gamma_{a,cy} + \gamma_{r,cy} + \gamma_{s,cy}) \leq 5.0 \quad \text{AASHTO 14.7.5.3.3-1}$$

and

$$\gamma_{a,st} \leq 3.0 \quad \text{AASHTO 14.7.5.3.3-2}$$

Axial Load Shear Strain

$$\text{Axial strain from static loads:} \quad \gamma_{a,st} = D_a \frac{\sigma_{s,st}}{G S_i} \quad \text{AASHTO 14.7.5.3.3-3}$$

$$\text{Axial strain from cyclic loads:} \quad \gamma_{a,cy} = D_a \frac{\sigma_{s,cy}}{G S_i} \quad \text{AASHTO 14.7.5.3.3-3}$$

where:

$$D_a = 1.40 \quad \text{AASHTO 14.7.5.3.3-4}$$

 $\sigma_{s,st} = \sigma_d =$ Compressive stress due to total static load at service limit state

 $\sigma_{s,cy} = \sigma_L =$ Compressive stress due to cyclic load at service limit state

$$\gamma_{a,st} = D_a \frac{\sigma_{s,st}}{G_{min} S_i} = \frac{1.40 * 0.83}{0.13 * 7.06} = 1.296$$

$$\gamma_{a,cy} = D_a \frac{\sigma_{s,cy}}{G_{min} S_i} = \frac{1.40 * 0.25}{0.13 * 7.06} = 0.389$$

Rotational Shear Strain

$$\text{Rotational strain from static loads:} \quad \gamma_{r,st} = D_r \left(\frac{L}{h_{ri}} \right)^2 \frac{\theta_{s,st}}{n} \quad \text{AASHTO 14.7.5.3.3-6}$$

$$\text{Rotational strain from cyclic loads:} \quad \gamma_{r,cy} = D_r \left(\frac{L}{h_{ri}} \right)^2 \frac{\theta_{s,cy}}{n} \quad \text{AASHTO 14.7.5.3.3-6}$$

where

$$D_r = 0.50 \quad \text{AASHTO 14.7.5.3.3-7}$$

 $\theta_{s,st} = \theta_d + \theta_r =$ Maximum static service limit state design rotation

 $\theta_{s,cy} = \theta_L =$ Maximum cyclic service limit state design rotation

$$\gamma_{r,st} = D_r \left(\frac{L}{h_{ri}} \right)^2 \frac{\theta_{s,st}}{n} = 0.50 (10.00 / 0.500) ^2 * (-0.001 + 0.005) / 5 = 0.160$$

$$\gamma_{r,cy} = D_r \left(\frac{L}{h_{ri}} \right)^2 \frac{\theta_{s,cy}}{n} = 0.50 (10.00 / 0.500) ^2 * (0.006) / 5 = 0.240$$

Shear Deformation Shear Strain

Shear strain from static loads: $\gamma_{s,st} = \frac{\Delta_{s,st}}{h_{rt}}$ AASHTO 14.7.5.3.3-10

Shear strain from cyclic loads: $\gamma_{s,cy} = \frac{\Delta_{s,cy}}{h_{rt}}$ AASHTO 14.7.5.3.3-10

where

$\Delta_{s,st} = \Delta_{s,slip} = 0.755$ in.

$\Delta_{s,cy} = \Delta_{LL} = 0.100$ in.

$\gamma_{s,st} = \frac{\Delta_{s,st}}{h_{rt}} = \frac{0.755}{2.625} = 0.288$

$\gamma_{s,cy} = \frac{\Delta_{s,cy}}{h_{rt}} = \frac{0.100}{2.625} = 0.038$

Combined Shear Strains Checks

$(\gamma_{a,st} + \gamma_{r,st} + \gamma_{s,st}) + 1.75(\gamma_{a,cy} + \gamma_{r,cy} + \gamma_{s,cy}) \leq 5.0$
 $= 1.296 + 0.160 + 0 + 1.75(0.389 + 0.240 + 0.038) = 2.91 < 5.00$ **OK**

$\gamma_{a,st} \leq 3.0$ $\gamma_{a,st} = 1.296 < 3.00$ **OK**

Stability

AASHTO 14.7.5.3.4

If the following is satisfied, no further investigation of stability is required:

$2A \leq B$ AASHTO 14.5.3.4-1

where

$A = \frac{1.92 \frac{h_{rt}}{L}}{\sqrt{1 + \frac{2.0L}{W}}} = \frac{1.92(2.625 / 10.00)}{\text{SQRT} [1 + (2*10.00) / 24.00]} = 0.37$ AASHTO 14.7.5.3.4-2

$B = \frac{2.67}{(S_i + 2.0)(1 + \frac{L}{4.0W})} = \frac{2.67}{(7.06 + 2.0) * [1 + 10.00 / (4.0*24.00)]} = 0.27$ AASHTO 14.7.5.3.4-3

Note that if L is greater than W, stability shall be investigated by interchanging L and W. L= 10.00 in W= 24.00 in

Check $2A = 2*0.37 = 0.74 > 0.27 = B$ **FAILS**

If the above criteria for stability are not satisfied, the following equations shall be investigated:

For a bridge deck that is free to translate horizontally:

For demonstration only. Designer shall determine movement capability of bridge on a case by case basis.

$$\sigma_s \leq \frac{G_{min} S_i}{2A - B} = \frac{0.13 * 7.06}{2 * 0.37 - 0.27} = 1.88 \text{ ksi} \quad \text{AASHTO 14.7.5.3.4-4}$$

Check $\sigma_s = 1.08 \text{ ksi} < 1.88 \text{ ksi}$ **OK Bearing is Stable**

If the above criteria for stability are not satisfied, the following equations shall be investigated:

For a bridge deck that is fixed against horizontal translation:

For demonstration only. Designer shall determine movement capability of bridge on a case by case basis.

$$\sigma_s \leq \frac{G_{min} S_i}{A - B} = \frac{0.13 * 7.06}{0.37 - 0.27} = 8.55 \text{ ksi} \quad \text{AASHTO 14.7.5.3.4-5}$$

Check $\sigma_s = 1.08 \text{ ksi} < 8.55 \text{ ksi}$ **OK Bearing is Stable**

Reinforcement

AASHTO 14.7.5.3.5

Note that holes are not present in the bearing. The allowable thickness does not need to be increased per AASHTO 14.7.5.3.5.

The minimum thickness of steel reinforcement shall satisfy the following:

$$h_s \geq 0.0625 \text{ in}$$

and

(Service Limit State)

$$h_s \geq \frac{3h_{ri}\sigma_s}{F_y} = \frac{3 * 0.500 * 1.08}{36} = 0.045 \text{ in} \quad \text{AASHTO 14.7.5.3.5-1}$$

and

(Fatigue Limit State)

$$h_s \geq \frac{2h_{ri}\sigma_L}{\Delta F_{TH}} = \frac{2 * 0.500 * 0.25}{24.00} = 0.010 \text{ in} \quad \text{AASHTO 14.7.5.3.5-2}$$

Check

$h_s = 0.125 \text{ in} > 0.0625 \text{ in}$ **OK**

$0.125 \text{ in} > 0.045 \text{ in}$ **OK**

$0.125 \text{ in} > 0.010 \text{ in}$ **OK**

Bearing Anchorage

AASHTO 14.7.5.4

For bearings without externally bonded plates, a restraint system is required to secure the bearing against horizontal movement if:

$$\frac{\theta_s}{n} \geq \frac{3\varepsilon_a}{S_i} \quad \text{AASHTO 14.7.5.4-1}$$

where

θ_s = total of static and cyclic service limit state design rotation. Cyclic component is multiplied by 1.75

ε_a = total of static and cyclic average axial strain. Cyclic component is multiplied by 1.75

$$\begin{aligned} \theta_s &= \theta_{s,st} + 1.75\theta_{s,cy} = \theta_d + \theta_r + 1.75\theta_L = \\ &= -0.001 + 0.005 + 1.75 \cdot 0.006 = \quad \quad \quad 0.015 \text{ rad} \end{aligned}$$

$$\varepsilon_a = \varepsilon_{st} + 1.75\varepsilon_{cy} = \varepsilon_d + 1.75\varepsilon_L = \quad 0.027 + 1.75 \cdot 0.008 = \quad 0.042$$

Check

$$\frac{\theta_s}{n} \geq \frac{3\varepsilon_a}{S_i} = \quad \frac{0.015}{5} = 0.003 \quad < \quad \frac{3 \cdot 0.042}{7.06} = 0.018 \quad \text{FAILS} \quad \text{Restraint Required}$$

If the Engineer elects to use externally bonded plates, limitations on hydrostatic pressure per AASHTO 14.7.5.3.3-11 shall be satisfied.

Anchorage (Bearing Pad Slip)

AASHTO 14.8.3

The bearing pad must be secured against horizontal movement if the shear force sustained by the deformed pad exceeds the minimum vertical force due to permanent loads modified for the concrete friction. The allowable slip of the pad is calculated and compared to the pad deformation using both G_{max} and G_{min} to determine the controlling scenario. Note this example considers longitudinal deformations only; wind, braking, and seismic loads shall also be considered as appropriate, in the direction of consideration.

$$H_b = \mu P_{min} \quad \text{AASHTO 14.6.3.1-1}$$

and

$$H_b = G_{min} A \frac{\Delta_s}{h_{rt}} \quad \text{AASHTO 14.6.3.1-2}$$

Combining equations:

$$\Delta_{s,allow} = \frac{\mu P_{min} h_{rt}}{G_{min} A} = \quad 0.20 \cdot 200.00 \cdot 2.63 / (0.13 \cdot 240.00) = \quad 3.43 \text{ in}$$

where

$$\begin{aligned} \mu &= 0.20 && \text{Coefficient of friction AASHTO C14.8.3.1} \\ P_{min} &= DL = 200.00 && \text{kip} \\ A &= LW = 240.00 && \text{in}^2 \\ h_{rt} &= 2.63 && \text{in} \end{aligned}$$

Check

$$\text{Using } G_{min}: \quad \Delta_{s,allow} = 3.43 \text{ in.} \quad > \quad \Delta_{s,slip} = 0.76 \text{ in.} \quad \text{OK}$$

$$\text{Using } G_{max}: \quad \Delta_{s,allow} = 2.54 \text{ in.} \quad > \quad \Delta_{s,slip} = 0.56 \text{ in.} \quad \text{OK}$$

In cases where $\Delta_{s,slip}$ exceeds $\Delta_{s,allow}$, anchor bolts shall be sized and designed in accordance with those Articles specified in AASHTO 14.8.3

APPENDIX A

EXAMPLE 5 - EXPANSION DEVICE (STRIP SEAL)

0 - 4 INCH

GENERAL INFORMATION

Assuming a 340-ft multi-span, precast, prestressed BT63 girder superstructure with a 20 deg. skew, determine the range of movement for a 0-4 inch expansion device due to temperature, creep, and shrinkage. Specify the installation gap sizes for temperatures ranging from -30 °F to 120 °F, at 10 degree increments, for placement in the expansion device construction drawing. The following example is in accordance with AASHTO LRFD 7th Edition Section 14.5. Refer to CDOT Bridge Design Manual (BDM) Section 14 for additional information and movement considerations. Assume temperature movements conform to AASHTO 3.12.2.2 Procedure B. Stiffnesses in the supporting elements may affect thermal length contribution and may not be symmetrical, this example assumes the stiffness in supporting elements are symmetrical. The 340-ft length includes the approach slabs.

PROJECT VARIABLES

Bridge Properties

Superstructure Type	Bridge	Concrete	
Expansion Length	L =	170.00	ft
Skew	Skew =	20	°
Thermal Coefficient	$\alpha =$	6.0E-06	in./in./°F

Measured from a line normal to bridge \perp
AASHTO 5.4.2.2

Creep and Shrinkage

Total Creep and Shrinkage Strain	$\epsilon_{CR\&SH} =$	0.0002	in./in.	AASHTO 5.4.2.3.2 & AASHTO 5.4.2.3.3
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Temperature Range

Maximum Temperature	$T_{max} =$	110	°F	AASHTO F3.12.2.2-1
Minimum Temperature	$T_{min} =$	-10	°F	AASHTO F3.12.2.2-2
Strength Load Factor, TU	$\gamma_{TU} =$	1.20		AASHTO 14.5.3.2 & T3.4.1-1

Expansion Device Dimensions

Cold Temperature Opening	$A_C =$	4.00	inch	Maximum recommended gland opening
Hot Temperature Opening	$A_H =$	0.50	inch	Minimum recommended gland opening
Minimum Installation Opening	$A_I =$	1.50	inch	Required for placement of gland
Rail Width (min.)	E =	1.25	inch	CDOT B-518-1

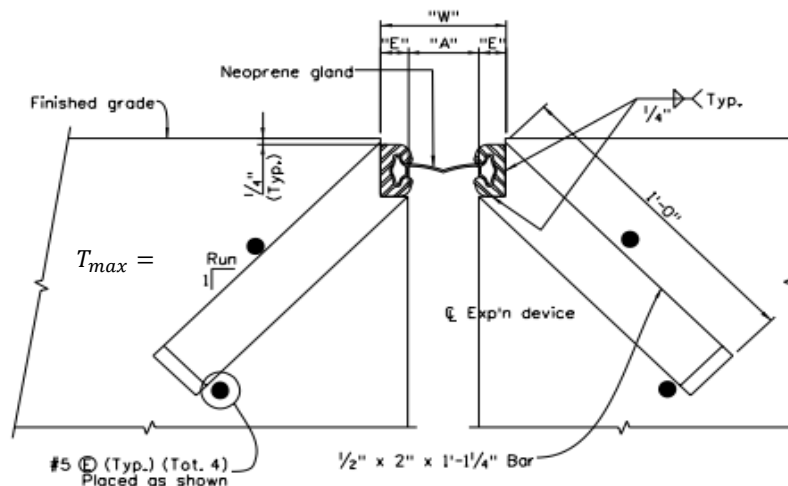


FIGURE 1 - BRIDGE EXPANSION DEVICE (0 - 4 INCH)

SOLUTION

For demonstration, the following solution assumes a structure temperature of 60 °F at the time of expansion device installation. The Designer shall determine "A" and "W" for the additional installation temperatures accordingly as shown in the completed table below.

$$T_i = 60 \text{ } ^\circ\text{F}$$

The total horizontal joint movement shall not exceed the maximum manufacturer recommended joint opening:

$$HM = L(\gamma_{TU}\Delta T\alpha + \varepsilon) \cos(\text{Skew}) = (170.00)*(12)[1.20*(110.00-(-10.00))*6.0E-06+0.0002]\cos(20.00)$$

$$HM = 2.04 \text{ in.} < 4.00 \text{ in.} \quad \mathbf{OK}$$

Maximum cold temperature fall if installed at T_i :

$$\Delta T_C = T_i - T_{min} = 60 - (-10) = 70 \text{ } ^\circ\text{F}$$

Maximum hot temperature rise if installed at T_i :

$$\Delta T_H = T_{max} - T_i = 110 - (60) = 50 \text{ } ^\circ\text{F}$$

Maximum superstructure contraction (joint expansion) caused by a fall in temperature from T_i :

$$A_{expn} = L(\gamma_{TU}\Delta T_C\alpha + \varepsilon) \cos(\text{Skew}) = (170.00)*(12)[1.20*70.00*6.0E-06+0.0002]\cos(20.00)$$

$$A_{expn} = 1.35 \text{ in.}$$

Maximum superstructure expansion (joint contraction) caused by a rise in temperature from T_i :

$$A_{cont} = L(\gamma_{TU}\Delta T_H\alpha - \varepsilon) \cos(\text{Skew}) = (170.00)*(12)[1.20*50.00*6.0E-06-0.0002]\cos(20.00)$$

$$A_{cont} = 0.31 \text{ in.}$$

Check that the factored cyclic joint movement does not exceed 3.50 in. per BDM 14.4.4

$$\Delta_{cyclic} = A_{expn} + A_{cont} = 1.35 + 0.31 = 1.66 \text{ in.} < 3.50 \text{ in.} \quad \mathbf{OK}$$

Dimension "A" at the given installation temperature needs to accommodate the hot and cold temperature movement ranges within the capabilities of the 0-4 in. joint.

The maximum opening the joint is allowed at the installation temperature is the recommended maximum opening minus the maximum joint expansion under cold temperatures.

$$A_{max} = A_C - A_{expn} = 4.00 - 1.35 = 2.65 \text{ in.}$$

The minimum opening the joint is allowed at the installation temperature is the recommended minimum opening plus the maximum joint contraction under hot temperatures.

$$A_{min} = A_H + A_{cont} = 0.50 + 0.31 = 0.81 \text{ in.}$$

The "A" dimension is determined as the value midway between A_{max} and A_{min} . The "A" value specified in the plans should be at least the minimum gland opening required for installation. If the temperature is too warm, causing a narrow joint opening, waiting for a drop in the air temperature is an option prior to gland installation.

$$A = \frac{A_{max} + A_{min}}{2} = \frac{(2.65 + 0.81)}{2} = 1.73 \text{ in.}$$

Check $A \geq A_i = 1.73 \text{ in.} > 1.50 \text{ in.} \quad \text{OK}$

The "W" dimension specified in the plans shall be the total width of the expansion device, measured as the gland opening "A" plus the two rails on either side, E

$$W = 2E + A = 2 \times 1.25 + 1.73 = 4.23 \text{ in.}$$

Comprehensive Expansion Device Table

Air Temp. T_i (°)	ΔT_C (°)	ΔT_H (°)	A_{max} (in)	A_{min} (in)	"A"	"W"
-30	-20	140	3.89	2.05	2.97	5.47
-20	-10	130	3.75	1.91	2.83	5.33
-10	0	120	3.62	1.77	2.69	5.19
0	10	110	3.48	1.63	2.56	5.06
10	20	100	3.34	1.50	2.42	4.92
20	30	90	3.20	1.36	2.28	4.78
30	40	80	3.06	1.22	2.14	4.64
40	50	70	2.93	1.08	2.00	4.50
50	60	60	2.79	0.94	1.87	4.37
60	70	50	2.65	0.81	1.73	4.23
70	80	40	2.51	0.67	1.59	4.09
80	90	30	2.37	0.53	1.45	3.95
90	100	20	2.24	0.39	1.31	3.81
100	110	10	2.10	0.25	1.18	3.68
110	120	0	1.96	0.12	1.04	3.54
120	130	-10	1.82	Too Small	1.82	4.32

Note "A" dimension is less than required for installation. Wait for drop in structure temperature before installing joint.

The "A" dimension values provided are based on a joint with a minimum opening of 0.5 in. and a maximum opening of 4 in. The Contractor shall adjust the "A" dimension values for joints fabricated with different minimum and maximum opening dimensions accordingly.

APPENDIX A

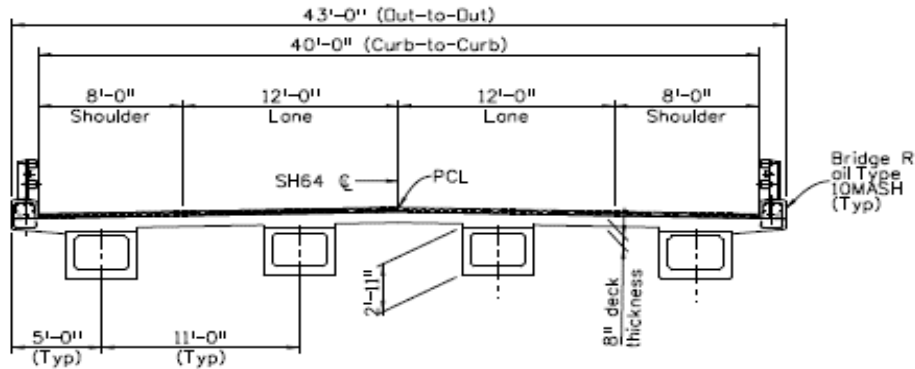
**EXAMPLE 6 - DECK DESIGN INCLUDING TYPE 10 MASH RAIL COLLISION
EXAMPLE 6.1 - DECK DESIGN**

GENERAL INFORMATION

Based on AASHTO LRFD Bridge Design Specifications 9.6.1, there are 3 methods of deck analysis:

1. Approximate Elastic Method, or "Equivalent Strip" Method (AASHTO 4.6.2.1)
2. Refined Methods (AASHTO 4.6.3.2)
3. Empirical Design Method (AASHTO 9.7.2)

This design example uses the Approximate Elastic Method (Equivalent Strip Method), in which the deck is divided into transverse strips, assumed to be supported on rigid supports at the center of the girders.



TYPICAL SECTION

MATERIAL AND SECTION PROPERTIES

Structure Type	CIP Concrete Deck		
Girder Spacing, maximum	$S_{Gdr} =$	11.0	ft
Number of girders	$N_{Gdr} =$	4	ea
Overall Deck width	$W_{deck} =$	43.0	ft
Deck slab thickness	$t_{deck} =$	8	in
Overhang thickness (average)	$t_{OH} =$	9.67	in
Concrete top cover	$c_{Top} =$	2.0	in
Concrete bottom cover	$c_{Bot} =$	1.0	in
Wearing surface	$t_{WS} =$	3.0	in
			AASHTO T 5.10.1-1 & BDM 5.4.3
			AASHTO T 5.10.1-1
Concrete strength	$f'_c =$	4.5	ksi
Reinforcement strength	$f_y =$	60.0	ksi (Minimum yield strength of grade 60 steel)
Concrete density	$W_c =$	0.150	kcf
Deck overlay density	$W_{WS} =$	0.147	kcf
Allowance for future utilities	$W_{util} =$	0.005	ksf
			BDM 3.4.2
Resistance factors	$\phi_{STR} =$	0.9	(strength limit state)
	$\phi_{EE} =$	1.0	(extreme event limit state)
Correction factor for source aggregate	$K_1 =$	1	
Modulus of elasticity of reinforcement	$E_s =$	29000.0	ksi
Modulus of elasticity of concrete	$E_c =$	4435.3	ksi
			AASHTO 5.4.3.2
			AASHTO 5.4.2.4
$E_c = 120,000K_1W_c^2f'_c{}^{0.33}$			
Modular ratio	$n = E_s/E_c =$	6.54	
Girder Type	Box Girder		
Girder web thickness	web =	4.0	in
Girder top flange width	flange =	48.0	in

Barrier Type	Type 10MASH	
CY of concrete for barrier section	$A_B =$	0.059 CY/ft
Barrier Weight	$W_{barrier} =$	0.289 kip/ft

(Refer to CDOT bridge Worksheet B-606-10MASH for more details)

UNFACTORED DEAD LOADS

Based on Table 3-22c, Continuous Beams Moment and Shear Coefficients - Equal Spans, Equally Loaded, in terms of wl^2 , $+M = 0.080$ and $-M = 0.100$ and will be used for this design

+Moment in terms of wl^2	0.08
-Moment in terms of wl^2	0.10

$W_{deck} =$	$8.00 \text{ in} / 12 * 0.15 \text{ kcf} =$	0.1 klf
$W_{WS} =$	$3.00 \text{ in} / 12 * 0.147 \text{ kcf} =$	0.037 klf

Positive Moment

$+M_{deck} =$	$0.100 \text{ klf} * (11.00 \text{ ft})^2 * 0.08 =$	0.968 k-ft/ft
$+M_{WS} =$	$0.037 \text{ klf} * (11.00 \text{ ft})^2 * 0.08 =$	0.355 k-ft/ft

Negative Moment

$-M_{deck} =$	$0.100 \text{ klf} * (11.00 \text{ ft})^2 * 0.10 =$	1.21 k-ft/ft
$-M_{WS} =$	$0.037 \text{ klf} * (11.00 \text{ ft})^2 * 0.10 =$	0.444 k-ft/ft

UNFACTORED LIVE LOADS

Live load moment can be determined by using AASHTO LRFD Bridge Design Specifications Appendix A4 T.A4-1. This table lists positive and negative moments per unit width of the deck with various girder spacings and various distances from the design section to the centerline of girders. This table is based on the equivalent strip method and interpolation is allowed when needed.

Deck superstructure type	b	AASHTO T4.6.2.2.1-1
Design section =	At the face of the supporting component	24.00 in AASHTO 4.6.2.1.6

Girder spacing, S=	11.0 ft	
Maximum Live Loads per unit width:		
Positive Moment from LL	$+M_{LL} =$	7.46 kip-ft/ft AASHTO T. A4-1
Negative Moment from LL	$-M_{LL} =$	4.52 kip-ft/ft AASHTO T. A4-1

FACTORED DESIGN LOADS

Concrete decks must be investigated for strength, service and extreme limit states. Fatigue and fracture limit states do not need to be investigated (AASHTO 9.5).

$M_u = \eta [Y_{DC}M_{DC} + Y_{DW}M_{DW} + m Y_{LL}(M_{LL} + IM)]$
 $\eta = 1.0$ load modifier
 γ - load factors specified in AASHTO T.3.4.1-1, T.3.4.1-2
 m - multiple presence factor, included in values from AASHTO T. A4-1
 IM - dynamic load allowance, included in values from AASHTO T. A4-1

Load Combination	Load Factors			Design Moments	
	Y_{DC_max}	Y_{DW_max}	Y_{LL}	$+M_{LL}$	$-M_{LL}$
Strength I	1.25	1.5	1.75	14.80	-10.09
Service I	1	1	1	8.78	-6.17

Note - it is conservative to use minimum load factors for positive values of M_{100} and M_{200} and negative values of M_{150} .

Controlling positive factored moment	$+Mu =$	14.80 kip-ft/ft
Controlling negative factored moment	$-Mu =$	-10.09 kip-ft/ft

DECK SLAB STRENGTH DESIGN

Design of deck reinforcement, including flexural resistance, limits of reinforcement, and control of cracking is based on AASHTO LRFD Bridge Design Specifications 5.7.3 (typical rectangular beam design). The following design method can be used for normal weight concrete with specified compressive strengths up to 15.0 ksi. Refer to Section 9, Deck and Deck Systems, of this BDM for information about acceptable deck reinforcement sizes and spacing.

Width of the design section $b = 12.0$ in.
Resistance factor for tension-controlled section $\phi_{STR} = 0.9$ AASHTO 5.5.4.2

Positive Moment Capacity (bottom reinforcement)

Try Bar size # **5**
Bar spacing $s = 6.0$ in.
Bar diameter $d_b = 0.625$ in.
Bar area $A_b = 0.31$ in.²

Area of steel per design strip

$$A_s = b (A_b / s) = 12.0 \text{ in.} * 0.310 \text{ in.}^2 / 6.0 \text{ in.} = 0.62 \text{ in.}^2$$

Effective depth of section

$$d_s = t_{Deck} - c_{Bot} - 1/2 d_b = 8.0 \text{ in.} - 1.0 \text{ in.} - 0.625 \text{ in.} / 2 = 6.69 \text{ in.}$$

Depth of equivalent stress block

$$a = \frac{A_s f_y}{0.85 f'_c b} = 0.62 \text{ in.}^2 * 60.0 \text{ ksi} / (0.85 * 4.5 \text{ ksi} * 12 \text{ in.}) = 0.81 \text{ in.}$$

Factored flexural resistance $+\phi M_n = \phi A_s f_y \left(d_s - \frac{a}{2} \right) =$

$$= 0.90 * 0.62 \text{ in.}^2 * 60.0 \text{ ksi} * (6.69 \text{ in.} - 0.81 \text{ in.} / 2) / 12 \text{ in./ft.} = 17.53 \text{ kip-ft.}$$

$$Check +\phi M_n > +M_u: \quad 17.53 > 14.80 \quad \mathbf{OK}$$

Negative Moment Capacity (top reinforcement)

Try Bar size # **5**
Bar spacing $s = 5.0$ in.
Bar Diameter $d_b = 0.625$ in.
Bar Area $A_b = 0.31$ in.²

Area of steel per 1.00 ft. design strip

$$A_s = B (A_b / s) = 12 \text{ in.} * 0.310 \text{ in.}^2 / 5.00 \text{ in.} = 0.74 \text{ in.}^2$$

Effective depth of section

$$d_s = t_{Deck} - c_{Top} - 1/2 d_b = 8.0 \text{ in.} - 2.0 \text{ in.} - 0.625 \text{ in.} / 2 = 5.69 \text{ in.}$$

Depth of equivalent stress block

$$a = \frac{A_s f_y}{0.85 f'_c b} = 0.74 \text{ in.}^2 * 60.0 \text{ ksi} / (0.85 * 4.5 \text{ ksi} * 12 \text{ in.}) = 0.97 \text{ in.}$$

Factored flexural resistance $-\phi M_n = \phi A_s f_y \left(d_s - \frac{a}{2} \right) =$

$$= 0.90 * 0.74 \text{ in.}^2 * 60.0 \text{ ksi} * (5.69 \text{ in.} - 0.97 \text{ in.} / 2) / 12 \text{ in./ft.} = 17.41 \text{ kip-ft.}$$

$$Check -\phi M_n > -M_u: \quad 17.41 > 10.09 \quad \mathbf{OK}$$

Minimum Reinforcement

AASHTO 5.6.3.3

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, $M_r = \phi M_n$, at least equal to the lesser of:

- 1.33 times the positive factored ultimate moment
- Cracking moment

$$Cracking \text{ moment} \quad M_{cr} = \gamma_3 \left[(\gamma_1 f_r + \gamma_2 f_{cpe}) S_c - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \right] \quad \text{AASHTO 5.6.3.3-1}$$

When simplified by removing all values applicable to prestressed and noncomposite sections, this equation becomes the following:

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c$$

Where:

Flexural cracking variability factor	$\gamma_1 =$	1.6	(non-segmental brg.)	AASHTO 5.6.3.3
Ratio of specified min. yield strength to ultimate tensile strength	$\gamma_3 =$	0.67	(A615 steel)	
Concrete density modification factor	$\lambda =$	1.0		AASHTO 5.4.2.8
Modulus of rupture	$f_r = 0.24 \lambda \sqrt{f'_c} =$	0.509 ksi		AASHTO 5.4.2.6

Section modulus of design section $S_c = \frac{bh^2}{6} = \frac{bt_{deck}^2}{6} = 12.0 \text{ in.} * (8.0 \text{ in.})^2 / 6 = 128 \text{ in.}^3$

Check Positive Moment reinforcement

$$Check + \phi M_n \geq min \left\{ \begin{array}{l} 1.33 (+M_u) = 1.33 * 14.80 \text{ kip-ft.} = 19.68 \text{ kip-ft.} \\ M_{cr} = 0.67 * 1.60 * 0.51 \text{ ksi} * 128.0 \text{ in.}^3 / 12 \text{ in./ft.} = 5.82 \text{ kip-ft.} \end{array} \right.$$

17.53 > 5.82 **OK**

Check Negative Moment reinforcement

$$Check - \phi M_n \geq min \left\{ \begin{array}{l} 1.33 (-M_u) = 1.33 * 10.09 \text{ kip-ft.} = 13.42 \text{ kip-ft.} \\ M_{cr} = 0.67 * 1.60 * 0.51 \text{ ksi} * 128.0 \text{ in.}^3 / 12 \text{ in./ft.} = 5.82 \text{ kip-ft.} \end{array} \right.$$

17.41 > 5.82 **OK**

CONTROL OF CRACKING AT SERVICE LIMIT STATE

Cracking is controlled by the spacing of mild steel reinforcement in the layer closest to the tension face, which shall satisfy the following (need not be less than 5.00 in.):

$$s \leq \frac{700 \gamma_e}{\beta_s f_{ss}} - 2d_c \quad \text{AASHTO 5.6.7-1}$$

- In which:
- $\gamma_e =$ **1.00** - exposure factor (1.0 for Class 1 and 0.75 for Class 2) (assume waterproofing membrane is used)
 - β_s - ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face
 - f_{ss} - calculated tensile stress in mild steel reinforcement at the service limit state ($\leq 0.60 f_y$ ksi)
 - d_c - thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto. For calculation purposes, d_c need not be taken greater than 2 in. plus the bar radius

Check Cracking at the Bottom of Deck (spacing of Positive Moment reinforcement):

$$d_c = c_{Bot} + 1/2 d_b = 1.00 \text{ in.} + 0.625 \text{ in.} / 2 = 1.31 \text{ in.}$$

$$\beta_s = 1 + \frac{d_c}{0.7(t_{deck} - d_c)} = 1 + 1.31 \text{ in.} / [0.7(8.0 \text{ in.} - 1.31 \text{ in.})] = 1.28$$

$$\text{Tension reinforcement ratio } \rho = \frac{A_s}{bd_s} = 0.62 \text{ in.} / (12 \text{ in.} * 6.69 \text{ in.}) = 0.008$$

$$k = \sqrt{2n\rho + (n\rho)^2} - n\rho = 0.271$$

$$j = 1 - k/3 = 0.910$$

$$f_{ss} = \frac{+M_{u,service}}{A_s j d_s} = 8.78 \text{ kip-ft.} * 12 \text{ in./ft.} / (0.62 \text{ in.} * 0.91 * 6.69 \text{ in.}) = 27.95 \text{ ksi}$$

$$s_{max} = \frac{700 \gamma_e}{\beta_s f_{ss}} - 2d_c = 700 * 1.00 / (1.28 * 27.95 \text{ ksi}) - 2 * 1.31 \text{ in.} = 16.94 \text{ in.}$$

Spacing of positive moment reinforcement used in the design = 6.00 in.

Check spacing used $\leq s_{max}$: 6.00 < 16.94 **OK**

Check Cracking at Top of Deck (spacing of Negative Moment reinforcement):

$$d_c = c_{Top} + 1/2 d_b = 2.0 \text{ in.} + 0.625 \text{ in.} / 2 = 2.31 \text{ in.}$$

$$\beta_s = 1 + \frac{d_c}{0.7(t_{Deck} - d_c)} = 1 + 2.31 \text{ in.} / [0.7 * (8.0 \text{ in.} - 2.31 \text{ in.})] = 1.58$$

Tension reinforcement ratio $\rho = \frac{A_s}{bd_s} = 0.74 \text{ in.} / (12 \text{ in.}^2 * 5.69 \text{ in.}) = 0.011$

Modular ratio $n = E_s / E_c = 29000 \text{ ksi} / 4435 \text{ ksi} = 6.54$

$$k = \sqrt{2n\rho + (n\rho)^2} - n\rho = 0.313$$

$$j = 1 - k/3 = 0.896$$

$$f_{ss} = \frac{-M_{u_service}}{A_s j d_s} = 6.17 \text{ kip-ft.} * 12 \text{ in./ft.} / (0.74 \text{ in.}^2 * 0.90 * 5.69 \text{ in.}) = 19.55 \text{ ksi}$$

$$s_{max} = \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = 700 * 1.00 / (1.58 * 19.55 \text{ ksi}) - 2 * 2.31 \text{ in.} = 18.03 \text{ in.}$$

Spacing of negative moment reinforcement used in the design = 5.00 in.

Check spacing used $\leq s_{max}$: 5.00 < 18.03 **OK**

Check tensile stress at service limit state doesn't exceed 0.60fy

$$f_{ss} = 19.55 \text{ ksi}$$

$$0.60 f_y = 36 \text{ ksi} = 0.60 * 60 \text{ ksi}$$

Check $f_{ss} \leq 0.60 f_y$ **OK**

LONGITUDINAL REINFORCEMENT

Minimum reinforcement is required in all directions to accommodate shrinkage and temperature changes near the surface of the slab. Longitudinal reinforcement on each face shall meet the following:

$$A_s \geq \frac{1.3b t_{Deck}}{2(b + t_{Deck})f_y} \quad \text{AASHTO 5.10.6-1}$$

$$0.11 \leq A_s \leq 0.60 \quad \text{AASHTO 5.10.6-2}$$

$$A_{s,min} = 1.3 * 12.0 \text{ in.} * 8.0 \text{ in.} / [2 (12.0 \text{ in.} + 8.0 \text{ in.}) 60.0 \text{ ksi}] = 0.052 \text{ in.}^2/\text{ft.}$$

$$A_{s,min} = 0.11 \text{ in.}^2/\text{ft.} \quad \text{- controls}$$

Per Section 9.6 of the CDOT BDM, the minimum longitudinal reinforcing steel in the top of the concrete bridge deck shall be #4 @ 6.00 in. Longitudinal reinforcement in the bottom of the deck slab can be distributed as a percentage of the primary reinforcement for positive moment.

Top reinforcement try **# 4 @ 6.00 in** on center: $A_s = 0.40 \text{ in.}^2/\text{ft.}$
 Check $A_s \geq A_{s,min}$ **OK**

Effective span length $S = S_{Gdr} - \text{girder width} = 11.0 \text{ ft.} - 48.0 \text{ in.} / 12 \text{ in./ft.} = 7 \text{ ft.}$
 AASHTO 9.7.2.3

Amount of reinforcement required in secondary direction in the bottom of the slab

$$\frac{220}{\sqrt{S}} \leq 67\% \quad \frac{220}{\sqrt{S}} = 83\% \quad \text{Use - 67\%} \quad \text{AASHTO 9.7.3.2}$$

Area of primary reinforcement for positive moment = 0.62 in.²/ft.
 Required area of bottom longitudinal steel: $A_{s_Req} = 67\% * 0.62 \text{ in.}^2/\text{ft.} = 0.42 \text{ in.}^2/\text{ft.}$

Bottom reinforcement try **# 5 @ 8.00 in.** on center: $A_s = 0.465 \text{ in.}^2/\text{ft.}$
 Check $A_s \geq A_{s,min}$ **OK**
 Check $A_s \geq A_{s_Req}$ **OK**

DECK SECTION SUMMARY		
Deck thickness		8.00 in.
Top Transverse Reinforcement	# 5 @	5.00 in.
Bottom Transverse Reinforcement	# 5 @	6.00 in.
Top Longitudinal Reinforcement	# 4 @	6.00 in.
Bottom Longitudinal Reinforcement	# 5 @	8.00 in.

EXAMPLE 6.2 - TYPE 10 MASH STRENGTH DESIGN

GENERAL INFORMATION

CDOT Bridge Rail Type 10MASH consists of a concrete parapet and a metal rail. The resistance to transverse vehicular impact loads shall be determined as specified in AASHTO LRFD Bridge Design Specifications A13.3.3. End impact is not considered. See CDOT Worksheet B-606-10MASH for barrier details.

The TL-4 maximum capacity of Type 10 MASH is shown for overhang example.

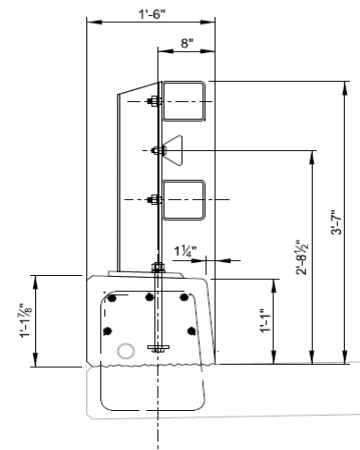
Overall barrier height	$H_B =$	43.0	in.	
Concrete cover (For SS rebars)	$c =$	1.5	in.	
Resistance factors	$\phi_{EE} =$	1	(Extreme Event)	AASHTO 1.3.2.1
	$\phi_S =$	0.8	(A325 bolts in shear)	AASHTO 6.5.4.2
	$\phi_T =$	0.8	(A325 bolts in tension)	AASHTO 6.5.4.2
Test level	MASH	TL-4		AASHTO T.A13.2-1
Transverse design force	$F_t =$	80.0	kips	See table below
Impact force distribution	$L_t =$	5.0	ft.	See table below

CONCRETE PARAPET

Height	$H_W =$	13.4375	in.
Width at base	$d =$	18.0	in.
Concrete Compressive Strength	$f'_c =$	4.5	ksi
Reinforcing Steel	$f_y =$	75.0	ksi

RAIL POST

Type	W6x20
Steel grade	ASTM A-572, Grade 50
Post spacing	$L =$ 10 ft. (max)
Effective height	$H_R =$ 32.5 in.
Area of post	$A_{Post} =$ 5.87 in. ²
Web depth	$D =$ 5.47 in.
Web thickness	$t_W =$ 0.26 in.
Flange thickness	$t_F =$ 0.37 in.
Flange width	$b_F =$ 6.02 in.
Depth of W beam	$d_b =$ 6.2 in.
	F_y (post) = 50 ksi
	Z_{x-x} (post) = 14.9 in. ³
$M_n = M_p = F_y Z$ (F7-1 AISC Manual)	$M_{post} =$ 62.08 kip-ft



AISC Table 1-1

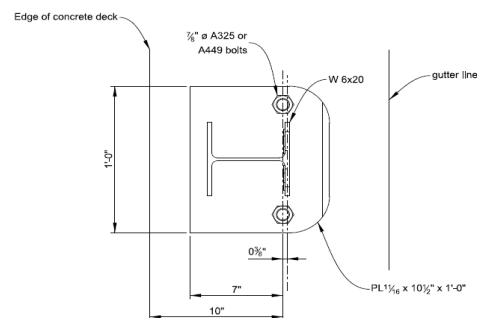
RAIL TUBES

Type	HSS 6x6x1/4
Steel grade	ASTM A-1085
Area of one tube	$A_{Tube} =$ 5.59 in. ²
Number of tubes	$n_{Tubes} =$ 2 ea.
	F_y (tube) = 50.0 ksi
	Z (tube) = 11.2 in. ³
$M_n = M_p = F_y Z$ (F7-1 AISC Manual)	$M_p =$ 93.33 kip-ft

AISC Table 1-12

BASE PLATE

Width of base plate	$W_b =$ 12.0 in.
Thickness of base plate	$t_b =$ 0.6875 in.
Distance to bolts	$d_{bo} =$ 10.0 in.
Bolt diameter	$\emptyset =$ 0.875 in.
Min tensile strength	$F_{ub} =$ 120.0 ksi
Number of bolts	$n_b =$ 2



The bar is hooked with a vertical embedment: 7 in Then the development fraction is:
 Development length factor = 0.82

Area of steel per design strip $A_s = \text{Bar Area} * b / \text{Stirrup spacing} = 0.20 \text{ in.}^2/\text{ft.}$
 Effective depth of section $d_s = d - c - 1/2 \text{ Stirrup Dia.} = 16.25 \text{ in.}$

Depth of equivalent stress block $a = \frac{A_s f_y}{0.85 f'_c b} = 0.32 \text{ in.}$

Flexural moment resistance $M_c = \phi_{EE} A_s f_y \left(d_s - \frac{a}{2} \right) = 19.73 \text{ kip-ft./ft.}$

Critical length of yield line failure pattern $L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2} \right)^2 + \frac{8H_W(M_b + M_W)}{M_c}} = 7.38 \text{ ft.}$

There is no additional resistance at the top of the parapet in addition to M_W , $M_b = 0 \text{ kip-ft.}$

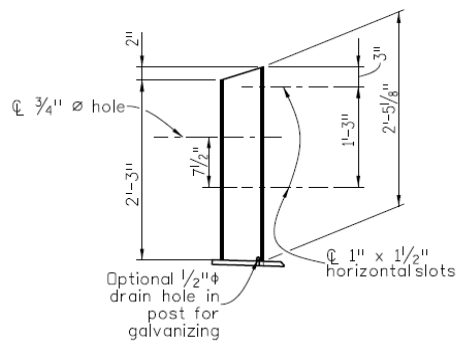
3. Determine R_W (nominal railing resistance to transverse load) within a wall segment.

$$R_W = \left(\frac{2}{2L_c - L_t} \right) \left(8M_b + 8M_W + \frac{M_c L_c^2}{H_W} \right) = 259.97 \text{ kip} \quad \text{AASHTO A13.3.1-1}$$

4. Calculate maximum post capacity P_p .

a. Plastic moment capacity of the post

Yielding of post $M_{\text{post}} = 62.08 \text{ kip-ft}$
 CG of impact force above curb $H_R - H_W = 19.06 \text{ in}$
 Maximum shear force at base of the post, P_p to cause post failure
 $M_{\text{post}} / (H_R - H_W)$ $P_{p1} = 39.08 \text{ kip}$



POST ELEVATION

b. Weld connection strength

Thickness of the weld $t_{\text{weld}} = 0.313 \text{ in}$
 Effective thickness $0.77 * t_{\text{weld}}$ $t_{\text{weff}} = 0.22 \text{ in}$

Calculate fillet weld strength as a line (Design of Welded Structures by Blodgett)

$S_W = (2 * b * d + \frac{d^2}{3}) * t_{\text{weff}}$ $S_W = 19.32 \text{ in}^3$
 Strength of the weld $F_{EXX} = 70.00 \text{ ksi}$
 Maximum weld moment $M_{\text{weld}} = 67.63 \text{ kip-ft} \quad (0.6 * F_{EXX} * S_W)$
 Maximum shear force at base $P_{p2} = 42.58 \text{ kip}$

c. Bolt shear strength

Shear resistance $R_n = 0.45 A_b F_{ub} N_s$ AASHTO 6.13.2.7-2
 $R_n = 0.45 * (\pi * 7/8 \text{ in}^2) / 4 * 120.0 \text{ ksi} * 2$ $P_{p3} = 64.94 \text{ kip}$

d. Concrete breakout shear strength

Spacing of bolts $b_{\text{spa}} = 9.00 \text{ in}$ ACI 318 17.7.2

Since the spacing of the anchors is less than 3 times the bolt distance d_b , the bolts must be treated as a group

Area resisting breakout $A_{VC} = 585 \text{ in}^2 \quad (9.0 \text{ in} + 3 * 10.0 \text{ in}) * 1.5 * 10.0 \text{ in}$
 Maximum area = $n_b * 4.5 d_{bo}^2 = 900 \text{ in}^2$ $A_{VCO} = 450 \text{ in}^2$

$$V_{cb} = \frac{A_{VC}}{A_{VCO}} \psi_{ec,v} \psi_{ed,v} \psi_{c,v} \psi_{h,v} V_b$$

There is no eccentricity in shear loading and so modification factor for eccentricity $\psi_{ec,V} = 1.0$ ACI 318 17.7.2.3
 Edge distances (along the curb) > 1.5 x bolt distance and so modification factor for edge distance $\psi_{ed,V} = 1.0$ ACI 318 17.7.2.4
 Analysis indicates no cracking at service loads and so modification factor for concrete $\psi_{c,V} = 1.4$ ACI 318 17.7.2.5

Anchor embedment $h_{ef} = 10.75$ in
 $1.5 * d_{bo} = 15.00$ in

$$\psi_{h,V} = \sqrt{\frac{1.5d_b}{h_{ef}}} = 1.181 \quad \text{ACI 318 17.7.2.6}$$

Basic shear strength is minimum of $V_{b1} = \left(7 \left(\frac{l_e}{\phi}\right)^{0.2} \sqrt{\phi}\right) \lambda_a \sqrt{f'_c} (d_{bo})^{1.5}$ $V_{b2} = 9 \lambda_a \sqrt{f'_c} (d_{bo})^{1.5}$

Load bearing length in shear $l_e = 7$ in $V_{b1} = 21.05$ kip $V_{b2} = 19.09$ kip
 (Min of h_{ef} and 8ϕ)

$\lambda_a = 1.0$ for normal weight concrete

Basic shear strength $V_b = 19.09$ kip

Shear strength $P_{p4} = 41.05$ kip

e. Bolt tensile strength (Ignore self weight)

$$\phi N_{sa} = \phi A_{se} N f_{uta} \quad A_{se} = \frac{\pi}{4} \left(d_a - \frac{0.9743}{n_t} \right)^2$$

Bolt tensile strength $f_{uta} = 120.0$ ksi Bolt outside diameter $d_a = 0.895$ in
 $\phi = 0.75$ Number of threads/in. $n_t = 9$ in
 $A_{se} = 0.486$ in²
 $N_{sa} = 43.75$ kip

Tensile strength of 2 bolts = $N_s = 87.50$ kip

Equating tension and compression, depth of compression $c = N_s / (0.85 * f_c * W_b) = 1.91$ in
 Moment lever arm = $7" - c/2 = 6.05$ in

Moment capacity based on bolt tensile capacity $M_{bolt} = 44.09$ kip-ft
 $M_{bolt} / (H_R - H_W) = P_{p5} = 27.76$ kip

Minimum strength of post in shear $P_p = 27.76$ kip

4. Calculate collision tensile force in deck T and collision moment M_{CT} .

The resistance of each component of a combination bridge rail shall be determined as specified in Article A13.3.1 and A13.3.2 of the AASHTO code. The flexural strength of the rail shall be determined over one and two spans. The resistance of the combination parapet and rail shall be taken as the lesser of the resistances determined for the two failure modes.

Impact at Midspan (3 spans) (Other odd spans didn't control and so not included)

Number of spans $N = 3$

Yielding of all rails $M_p = 93.33$ kip-ft

Impact force distribution $L_t = 5.00$ ft

post spacing $L = 10.00$ ft

$$R_R = \frac{16M_p + (N - 1)(N + 1)P_p L}{2NL - L_t} = \frac{(16 * 93.33 \text{ kip-ft} + 0) / (2 * 3 \text{ ft} * 10.00 \text{ ft} - 5.00 \text{ ft})}{R_R = 67.53 \text{ kip}} \quad \text{AASHTO A13.3.2-1}$$

$$\bar{R} = R_R + R_W = (67.53 \text{ kip} + 259.97 \text{ kip}) = 327.50036 \text{ kip} \quad \text{AASHTO A13.3.3-1}$$

Designing deck overhang for strength > strength of rails and curb is conservative. Therefore, design only for maximum MASH F_t loads. Assume the rails fail during impact and curb resists the remaining load.

Therefore Use $R_w =$ 12.47 kip (80.00 kip - 67.53 kip) Single span $\bar{R} =$ 80.00 kip
 AASHTO A13.3.3-2

$$\bar{Y} = \frac{R_R H_R + R_w H_w}{\bar{R}} = \frac{(67.53 \text{ kip} * 32.50 \text{ in.} + 12.47 \text{ kip} * 13.44 \text{ in.})}{80.00 \text{ kip}}$$

Y = 29.53 in

$$T = \frac{R_w}{L_C + 2H_w} \quad T_{\text{mid}} = 1.30 \text{ kip/ft}$$

$$M_{CT} = T * H_w \quad M_{CT\text{mid}} = 1.45 \text{ kip-ft/ft}$$

Impact at Post (2 spans)

(Other even spans didn't control and so not included)

Number of spans

$$N = 2$$

Impact force distribution
 post spacing

$$L_t = 5.00 \text{ ft}$$

$$L = 10.00 \text{ ft}$$

$$R'_R = \frac{16M_p + N^2 P_p L}{2NL - L_t} = \frac{(16 * 93.33 \text{ kip-ft} + 2^2 * 27.76 * 10.00 / (2 * 2 * 10.00 \text{ ft} - 5.00 \text{ ft}))}{2 * 2 * 10.00 \text{ ft} - 5.00 \text{ ft}}$$

AASHTO A13.3.2-2

$$R'_R = 74.39 \text{ kip}$$

$$R'_w = \frac{R_w H_w - P_p H_R}{H_w} = \frac{(259.97 \text{ kip} * 13.44 \text{ in.} - 27.76 \text{ kip} * 32.50 \text{ in.})}{13.44 \text{ in.}}$$

AASHTO A13.3.3-5

$$R'_w = 192.84 \text{ kip}$$

$$\bar{R} = P_p + R'_R + R'_w = 27.76 \text{ kip} + 74.39 \text{ kip} + 192.84 \text{ kip} = 294.99 \text{ kip}$$

AASHTO A13.3

Use $\bar{R} =$ 80 kip Ignore R'_w and use reduced $R'_R =$ 52.24 kip (80.00 kip - 27.76 kip)

AASHTO A13.3.3-4

$$\bar{Y} = \frac{P_p H_R + R'_R H_R + R'_w H_w}{\bar{R}} = \frac{(27.76 \text{ kip} * 32.50 \text{ in.} + 52.24 \text{ kip} * 32.50 \text{ in.} + 0.00 \text{ kip} * 13.44 \text{ in.})}{80.00 \text{ kip}}$$

Y = 32.5 in

$$T = \frac{P_p}{W_b + d_b + 2H_w} \quad T_{\text{post}} = 7.26 \text{ kip/ft}$$

$$M_{CT} = T * \bar{Y} \quad M_{CT\text{post}} = 19.66 \text{ kip-ft/ft}$$

Use greater of the two failure modes \ $M_{ct} =$ 19.66 kip-ft/ft $T =$ 7.26 kip/ft

SUMMARY

Impact at post controls the design as the transfer width is narrower than the impact between posts

Use the following data for Deck overhang design at the front face of the curb (Test Level 4):

Controlling Axial Load Per Unit Length of the Deck	$T_{\text{Axial}} =$	7.26 kip/ft.
Deck Overhang Moment	$M_{ct} =$	19.66 kip-ft./ft.

EXAMPLE 6.3 - BARRIER TYPE 9 STRENGTH DESIGN

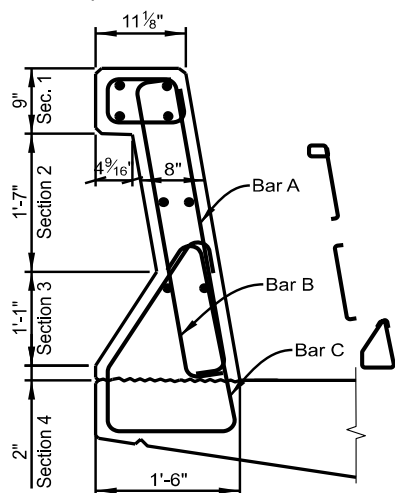
GENERAL INFORMATION

The CDOT Bridge Rail Type 9 design follows the AASHTO LRFD Bridge Design Specifications A13.3.1 design procedure for concrete railings, using strength design for reinforced concrete. The following calculations show case of impact within barrier segment, assuming that barrier will be extended past the limits of the bridge. For cases concerning impact at end of the barrier, refer to AASHTO Appendix A13. The applied design force (F_t) and the longitudinal length of distribution of the impact force (L_t) in this example is from the research conducted under NCHRP Project 22-20(2). **The TL-4 maximum capacity of Type 9 is shown for overhang example.**

Overall barrier height	$H_B =$	43.00	in.	
Concrete strength	$f'_c =$	4.50	ksi	
Reinforcement strength	$f_y =$	75.00	ksi	
Concrete cover	$c =$	1.50	in.	
Resistance factor	$\phi =$	1.00	(Extreme Event)	AASHTO 1.3.2.1
Test level		TL-4		AASHTO T A13.2-1
Transverse design force	$F_t =$	80.00	kips	
Impact force distribution	$L_t =$	5.00	ft.	

BARRIER FLEXURAL CAPACITY

1. Determine M_c : flexural resistance of cantilevered parapet about an axis parallel to the longitudinal axis of the bridge at midspan.



<u>Barrier Dimensions</u>	Sec. 1	Sec. 2	Sec. 3	Sec. 4	
Section top width	11.125	8.00	8.00	17.63	in.
Section bottom width	12.56	8.00	17.63	18.00	in.
Section height	9.00	19.00	13.00	2.00	in.

- A_s - area of steel per design strip
- h - section height
- d_{avg} - average section width
- d_s - effective depth of design section
- b_c - width of design strip (taken as 1 ft per AASHTO Section 13)
- a - depth of equivalent stress block

$$\phi M_n = \phi A_s f_y \left(d_s - \frac{a}{2} \right) \quad M_c = \sum_1^n \phi M_n / b_c$$

1st vertical rebar (Bar A) **# 4 @ 9 in.** Bar Diameter = 0.500 in.
 Bar Area = 0.20 in.²

	A _s (in. ²)	h (in.)	d _{avg} (in.)	d _s (in.)	b _c (in.)	k = .85f' _c b	a = A _s f _y /k (in.)	φM _n (kip-ft.)	M _C (kip-ft./ft.)
Section 1	0.27	9.00	11.84	10.09	12.00	45.90	0.44	16.46	3.45
Section 2	0.27	19.00	8.00	6.25	12.00	45.90	0.44	10.05	4.44
Section 3	0.27	13.00	12.81	11.06	12.00	45.90	0.44	18.07	5.46
Section 4	0.27	2.00	17.81	16.06	12.00	45.90	0.44	26.41	1.23
Barrier M _C (Bar A) =									14.58

2nd vertical rebar (Bar B) **# 4 @ 18 in.**

	A _s (in. ²)	h (in.)	d _{avg} (in.)	d _s (in.)	b _c (in.)	k = .85f' _c b	a = A _s f _y /k (in.)	φM _n (kip-ft.)	M _C (kip-ft./ft.)
Section 1	0.13	9.00	11.84	5.59	12.00	45.90	0.22	4.57	0.96
Section 2	0.00	19.00	8.00	1.75	12.00	45.90	0.00	0.00	0.00
Section 3	0.13	13.00	12.81	6.56	12.00	45.90	0.22	5.38	1.63
Section 4	0.13	2.00	17.81	11.56	12.00	45.90	0.22	9.54	0.44
Barrier M _C (Bar B) =									3.03

3rd vertical rebar (Bar C) **# 4 @ 12 in.**

	A _s (in. ²)	h (in.)	d _{avg} (in.)	d _s (in.)	b _c (in.)	k = .85f' _c b	a = A _s f _y /k (in.)	φM _n (kip-ft.)	M _C (kip-ft./ft.)
Section 1	0.00	9.00	11.84	10.09	12.00	45.90	0.00	0.00	0.00
Section 2	0.00	19.00	8.00	6.25	12.00	45.90	0.00	0.00	0.00
Section 3	0.20	13.00	12.81	11.06	12.00	45.90	0.33	13.62	4.12
Section 4	0.20	2.00	17.81	16.06	12.00	45.90	0.33	19.87	0.92
Barrier M _C (Bar C) =									5.04

Grand Total Barrier M_C = 22.65

2. Determine M_w: flexural resistance of the parapet about its vertical axis.

Back face horizontal reinforcement **# 4** Bar Diameter = 0.50 in.
 Bar Area = 0.20 in.²

	No. of Bars	A _s (in. ²)	h (in.)	d _{avg} (in.)	d _s (in.)	b (in.)	k = .85f' _c h	a = A _s f _y /k (in.)	φM _w (kip-ft.)
Section 1	2	0.40	9.00	11.84	9.59	9.00	34.43	0.87	22.90
Section 2	1	0.20	19.00	8.00	5.75	19.00	72.68	0.21	7.06
Section 3	1	0.20	13.00	9.63	7.38	13.00	49.73	0.30	9.03
Section 4	0	0.00	2.00	17.81	15.56	2.00	7.65	0.00	0.00
Barrier M _w =									38.98

3. Rail resistance within a wall segment.

$$R_W = \left(\frac{2}{2L_C - L_t} \right) \left(8M_b + 8M_W + \frac{M_C L_C^2}{H} \right) \quad \text{AASHTO A13.3.1-1}$$

$$L_C = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2} \right)^2 + \frac{8H(M_b + M_W)}{M_C}} \quad \text{AASHTO A13.3.1-2}$$

Additional flexural resistance at top of wall $M_b = 0.00$ kip-ft.
 Critical length of yield line $L_C = 9.96$ ft.
 Nominal transverse load resistance $R_W = 125.86$ kips

Capacity Check Check $R_W > F_t$: 125.86 > 80.00 **OK**

BARRIER INTERFACE SHEAR CAPACITY

AASHTO 5.7.4

Evaluate the shear capacity of the cold joint to transfer nominal resistance R_W between the deck and railing.
 Neglect effects of barrier Dead Load and assume that the surface of the deck is not roughened.

Interface width considered in shear transfer $b_v = 18.00$ in.
 Interface length considered in shear transfer $L_v = 12.00$ in.

Shear contact area $A_{CV} = b_v L_v = 216.00$ in.²

Shear reinforcement at front face **# 4 @ 12 in.** Bar Area : 0.2 in.²

Area of shear reinforcement $A_{VF} = 12 \text{ in.} * 0.20 \text{ in.} / 12 \text{ in.} = 0.2 \text{ in.}^2/\text{ft.}$

Check $A_{vf} \geq \frac{0.05A_{cv}}{f_y} = 0.144$ **OK** AASHTO 5.7.4.2-1

Permanent compression force from barrier weight (neglected) $P_c = 0.00$ kip

For concrete placed against clean concrete surface, free of laitance, but not intentionally roughened

Cohesion factor $c = 0.075$ ksi AASHTO 5.7.4.4
 Friction factor $\mu = 0.6$
 Shear factor 1 $K_1 = 0.2$ (Fraction of concrete strength available to resist interface shear)
 Shear factor 2 $K_2 = 0.8$ ksi (Limiting interface shear resistance)

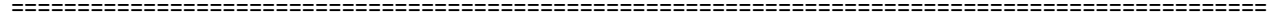
$$V_n = \min \left\{ \begin{array}{l} K_1 f'_c A_{CV} = 0.20 * 4.50 \text{ ksi} * 216.0 \text{ in.} = 194.4 \text{ kip} \\ K_2 A_{CV} = 0.80 * 216.0 \text{ in.} = 172.8 \text{ kip} \\ c A_{CV} + \mu (A_{VF} f_y + P_C) = 0.075 \text{ ksi} * 216 \text{ in.} + 0.60 (0.20 \text{ in.} * 75 \text{ ksi} + 0 \text{ kip}) = 25.20 \text{ kip} \end{array} \right. \quad \text{AASHTO 5.7.4.3}$$

Resistance factor $\phi = 1.00$ (Extreme Event) AASHTO 1.3.2.1

Factored Shear Resistance $\phi V_n = 25.20$ kip

Shear force acting on the barrier per 1.00 ft. strip $V_u = \frac{R_W}{L_C} = 12.64$ kip/ft.

Capacity Check Check $\phi V_n > V_u$: 25.20 > 12.64 **OK**



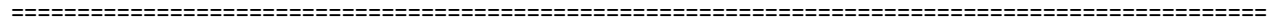
OVERHANG DESIGN DATA

Barrier Type 9 satisfies all checks outlined in AASHTO LRFD Bridge Design Specifications Appendix 13. Use the following data for Deck overhang design at the front face of the curb when Barrier Type 9 is used (Test Level 4):

$$T_{Axial} = R_W / (L_C + 2H_B)$$

AASHTO A13.4.2-1

Axial Load Per Unit Length of the Deck	$T_{Axial} = 7.35$ kip/ft.
Moment Capacity of the Barrier	$M_c = 22.65$ kip-ft./ft.



EXAMPLE 6.4 - OVERHANG DESIGN

GENERAL INFORMATION

Bridge deck overhang shall be designed for three separate design cases: AASHTO A13.4.1

- Case 1 - Horizontal and longitudinal forces from vehicle collision load (Extreme Event II limit state)
- Case 2 - Vertical force from vehicle collision load (Extreme Event II limit state)
- Case 3 - Vertical Dead and Live Load at the overhang section (Strength I limit state)

The deck overhang region shall be designed to have resistance larger than the MASH impact forces. Therefore, analysis of MASH barriers must be done. Refer to Example 6.2 for detailed strength calculations for Barrier Type 10 MASH.

Barrier type	Type 10MASH		
Width of barrier base	$W_B =$	18.0	in.
Barrier weight	$W_{Barrier} =$	0.289	kip/ft. (see Deck Design)
Deck overlay density	$W_{WS} =$	0.147	kcf Section 3.4.2
Concrete density	$W_C =$	0.15	kcf
Barrier center of gravity	$X_{C.G.} =$	12.63	in.
Axial load per unit length	$T_{Axial} =$	7.26	kip/ft. (refer to Type 10MASH Strength Design)
Deck Overhang Moment	$M_C =$	19.66	kip-ft./ft. (refer to Type 10MASH Strength Design)
Critical length of yield line	$L_C =$	7.38	ft. (refer to Type 10MASH Strength Design)
Overhang width	$S_{OH} =$	5.00	ft.
Edge of deck to edge of flange	$S_{Gdr_Edge} =$	3.00	ft.
Overhang minimum depth	$t_{OH(min)} =$	8.00	in.
Overhang maximum depth	$t_{OH(max)} =$	10.00	in. (at exterior edge of flange)
Concrete top cover	$c_{Top} =$	2.00	in. AASHTO T.5.10.1-1
Concrete strength	$f'_c =$	4.5	ksi
Reinforcement strength	$f_y =$	60	ksi
Test Level		TL-4	
Transverse design force	$F_t =$	80	kips
Impact force distribution	$L_t =$	5	ft
Vertical Design Force	$F_V =$	22	kips
Longitudinal distribution of Vertical force	$L_V =$	18	ft

Controlling Load Combinations	Load Factors			
	Y_{DC}	Y_{DW_max}	Y_{CT}	Y_{LL}
Extreme Event II	1.00	1.00	1.00	0.50
Strength I	1.25	1.50	0.00	1.75

AASHTO T3.4.1-1

The deck overhang is designed to resist an axial tension force and moment from vehicular collision (CT) acting simultaneously with the Dead Load (DC/DW) and Live Load (LL) moment. The critical section shall be taken at the face of the box girder (AASHTO 4.6.2.1.6). In addition, Extreme Event II combination is also checked at the face of the curb. Loads are assumed to be distributed at a 45 degree angle starting from the base plate.

DESIGN CASE 1: Extreme Event II (Transverse Collision) at the face of the curb

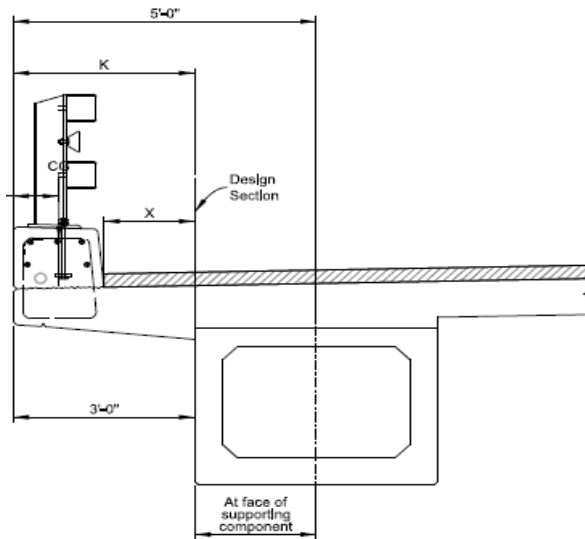
Distance from edge of deck to design section	$K =$	1.50	ft.	AASHTO 4.6.2.1.6
Distance from barrier face to design section	$X =$	0.00	ft.	
Depth of the section under consideration	$h_{Design} =$	9.00	in. (may add min haunch depth if needed, conservative to use constant deck depth)	

Bending moments from dead load of structural components and nonstructural attachments:

Barrier	$M_{DC-Barrier} = W_{Barrier} * (K - X_{C.G.}) =$	$0.289 \text{ kip/ft.} * (1.50 \text{ ft.} - 12.63 \text{ in.} / 12 \text{ in./ft.}) =$	0.129 kip-ft./ft.
Deck	$M_{DC-Deck} = W_C * t_{OH(min)} * K^2 / 2 =$	$0.150 \text{ kcf} * 8 \text{ in.} / 12 \text{ in./ft.} * (1.50 \text{ ft.})^2 / 2 =$	0.113 kip-ft./ft.

Additional overhang concrete $M_{DC-Add} = 0.5 W_C * S_{Gdr_Edge} (T_{OH(max)} - T_{OH(min)}) * (K - 2/3 S_{Gdr_Edge}) =$
 $= 0.5 * 0.150 \text{ kcf} * 1.50 \text{ ft.} * (10.0 \text{ in.} - 8.0 \text{ in.}) / 12 \text{ in./ft.} * (1.50 \text{ ft.} - 2/3 * 1.50 \text{ ft.}) =$ 0.009 kip-ft./ft.

Total DC = $M_{DC-Barrier} + M_{DC-Deck} + M_{DC-Add} =$ 0.13 kip-ft.+0.11 kip-ft.+0.009 kip-ft. = 0.251 kip-ft./ft.



Development length of transverse reinforcement should be considered. It can be dealt with in a variety of methods

Bending moments from wearing surfaces and utilities:

$$\text{Deck overlay } M_{DW-WS} = W_{WS} * 3 \text{ in.} * X^2 / 2 = 0.147 \text{ kcf} * 3 \text{ in.} / 12 \text{ in./ft.} * (0.00 \text{ ft.}) / 2 = 0.000 \text{ kip-ft./ft.}$$

Both design bending moment and design axial tension are calculated based on the properties of the barrier on the deck. See Type 10MASH tab for information on strength design.

$$\text{Bending moment from vehicular collision } M_{CT} = M_C = 19.66 \text{ kip-ft./ft.}$$

$$\text{Design factored moment (Extreme Event II, Case I) } \mu_1 = 1.0M_{DC} + 1.0M_{DW} + 1.0M_{CT} = 0.251 \text{ kip-ft.} + 0.000 \text{ kip-ft.} + 19.66 \text{ kip-ft.} = 19.92 \text{ kip-ft./ft.} \quad \text{AASHTO 3.4.1, A13.4.1}$$

DESIGN CASE 2: Extreme Event II (Vertical Collision) at the face of the curb

Vertical and Longitudinal collision cases will not control generally and so other critical sections are not included.

Lever arm for vertical collision	$I_a =$	0.448	ft
Vertical Design Force	$F_V =$	22.00	kips
Longitudinal distribution of Vertical force	$L_V =$	18.00	ft

$$\text{Bending moment on overhang due to vertical forces } M_{V-CT} = F_V * I_a / L_V = 22 \text{ kip} * 0.45 \text{ ft.} / 18.00 \text{ ft.} = 0.547 \text{ kip-ft./ft.}$$

$$\text{Dead Load moment } M_{DC} = 0.25 \text{ kip-ft./ft.}$$

$$\text{Design factored moment (Extreme Event II, Case I) } \mu_2 = 1.0M_{DC} + 1.0M_{CT} = 0.547 \text{ kip-ft./ft.} + 0.251 \text{ kip-ft./ft.} = 0.798 \text{ kip-ft./ft.} \quad \text{AASHTO 3.4.1, A13.4.1}$$

DESIGN CASE 3: STRENGTH I (At the face of the girder)

The overhang is designed to resist gravity forces from the Dead Load of structural components and attachments to the cantilever, as well as a concentrated Live Load positioned 12.00 in. from the face of the barrier.

For decks with overhangs not exceeding 6.00 ft. measured from the centerline of the exterior girder to the face of a structurally continuous concrete railing, the outside row of wheel loads may be replaced with a uniformly distributed line load of 1.0 klf intensity per AASHTO LRFD Bridge Design Specifications 3.6.1.3.4.

=====

Distance from edge of deck to design section	K =	3	ft.	
Distance from barrier face to design section	X =	1.5	ft.	
Depth of the section under consideration	$h_{Design} =$	10.00	in.	
Distance from LL application to design section	z =	0.5	ft.	
Live Load multiple presence factor	m =	1.00		AASHTO T.3.6.1.1.2-1
Dynamic load allowance	IM =	0.33		AASHTO 3.6.2

Bending moment from Dead Loads (equal to the loads calculated for Design Case 1)

Barrier	$M_{DC-Barrier} =$	0.562	kip-ft./ft.	
Deck	$M_{DC-Deck} =$	0.45	kip-ft./ft.	
Add. overhang concrete	$M_{DC-Add} =$	0.038	kip-ft./ft.	
Deck overlay	$M_{DW-WS} =$	0.041	kip-ft./ft.	AASHTO 3.6.1.3.4

Bending moment from live load	$M_{LL} =$	1.0 klf * 0.50 ft. =	0.5	kip-ft./ft.
Design factored moment (Strength I)	$Mu_3 = 1.25M_{DC} + 1.50M_{DW} + 1.75m(M_{LL} + IM) =$			
	$= 1.25 * 1.05 \text{ kip-ft./ft} + 1.50 * 0.041 \text{ kip-ft./ft} + 1.75 * 1.00 * 1.33 * 0.50 \text{ kip-ft./ft} =$		2.54	kip-ft./ft.

Design Summary (By observation, other load cases will not control and are not included in this example)

Design Case 1	$M_{u1} =$	19.916	kip-ft./ft.	
Design Case 2	$M_{u2} =$	0.798	kip-ft./ft.	
Design Case 3	$M_{u3} =$	2.538	kip-ft./ft.	
Controlling Case =	$Mu1 =$	19.916	kip-ft./ft.	DESIGN CASE 1 CONTROLS

Design axial tensile load $T_{Axial} =$ 7.26 kip/ft.

Top transverse reinforcement: Bar size **# 5** (see Deck Design)
 Bar spacing s = **5** in.

Bottom transverse reinforcement: Bar size **# 5** (see Deck Design)
 Bar spacing s = **6** in.

Area of top steel per design strip $A_{St} = b (A_b / s) =$ 12 in. * 0.31 in. / 5.0 in. = 0.744 in.²/ft.

Area of bottom steel per design strip $A_{Sb} = b (A_b / s) =$ 12 in. * 0.31 in. / 5.0 in. = 0.62 in.²/ft.

Steel in each layer resisting tension $A_{ten} = T_{axial} * 0.5 / F_y =$ 7.26 kip * 0.5 / 60.0 ksi = 0.061 in.²/ft.

Area of top steel per design strip resisting moment $A_{st} - A_{ten} =$ 0.74 sq. in. - 0.06 sq. in. = 0.683 in.²/ft.

Effective depth of section $d_s = h_{Design} - c_{Top} - 1/2 d_b =$ 9 in. - 2 in. - 0.625 in. / 2 = 6.688 in.

Depth of equivalent stress block

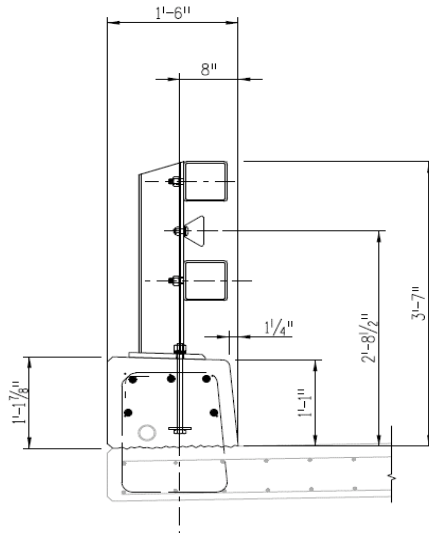
$$a = \frac{A_s * f_y}{0.85 f'_c b} = 0.68 \text{ sq. in.} * 60.00 \text{ ksi} / (0.85 * 4.50 \text{ ksi} * 12 \text{ in.}) = 0.893 \text{ in.}$$

Factored flexural resistance $\phi_{EE} M_n = \phi_{EE} \left[A_s * f_y \left(d - \frac{a}{2} \right) \right] =$
 $1.0 * 0.68 \text{ sq. in.} * 60.00 \text{ ksi} * (6.69 \text{ in.} - 0.89 \text{ in.} / 2) =$ 21.328 kip-ft./ft.

21.328 > 19.916 **OK**

=====

BARRIER TYPE 10 MASH CENTER OF GRAVITY (Steel Only)



Description	Unit wt lb/ft	Distance from deck out (in.)	Length (ft)	Number	Weight lb	Wx lb-in.
Tubes 6 x 6 x 1/4	19.02	13.50	10.00	2	380.40	5135.40
Post W6 x 20	20.00	7.4	2.339	1	46.78	346.17
Base Pl 10.5 x 12 x 3/4	26.80	8.25	0.75	1	20.10	165.81
				Total	447.28	5647.38

CG from deck out = **12.63** in.

Case 1: Bulb Tee Bridge

PROBLEM STATEMENT

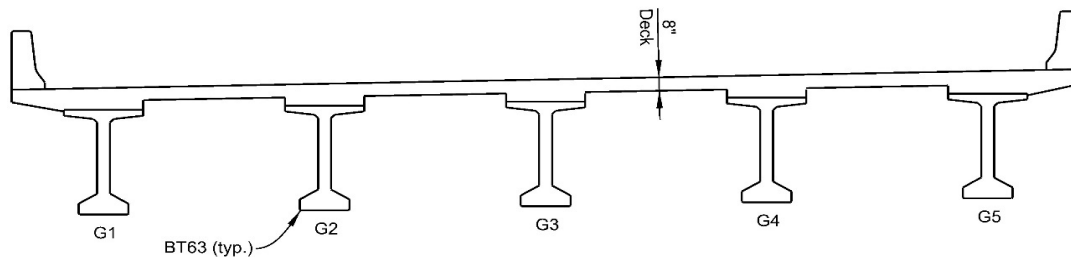
Case 1 illustrates how to set the haunch at supports for a BT girder bridge. Partial depth precast deck panels will be allowed, thus a minimum haunch thickness of 1 in. will be maintained at all locations. At supports, an additional 0.5 in. is provided for construction tolerance, giving a total min. haunch of 1.5 in. required at supports. See Section 5.5.2.1 of this BDM for more information.

The profile grade of the bridge is a crest vertical curve, with the bridge alignment on a horizontal curve with a constant cross-slope. The bridge is supported by chorded girders. The example shows how both the vertical and horizontal deck geometrics affect the deck profile above the girders, and thereby affect the haunch depths.

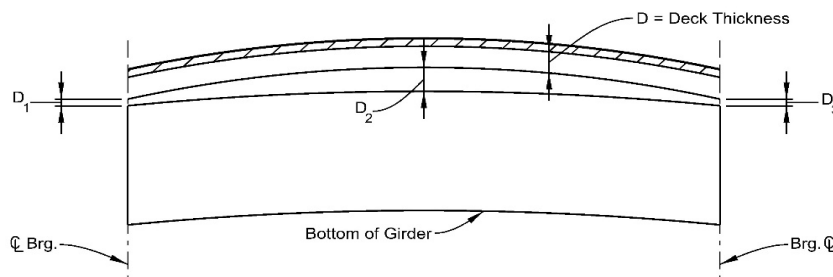
For this example, the design $f'c$ per BDM Section 5.3.1.2 was used for the given predicted girder cambers and DL deflections, not the optional actual values permitted in BDM Section 5.5.2.1.D.

The dead load deflections given in this example do not contain an increase for long-term effects, permissible per BDM Section 5.5.2.1.E of this BDM.

Positive values indicate upward camber or deflection.



Bridge Section View



Girder Elevation View

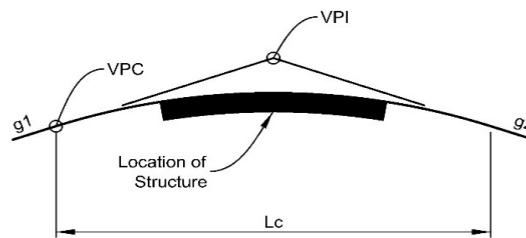
GIVENS

Girder span length, L =	100	ft.
Deck cross-slope, CS =	0.06	ft./ft.
Proposed haunch at CL brg. at CL girder, $D_1 = D_3$ =	3.00	in.
Assumed weighted average haunch for DL, $D_{avg,DL}$ =	5.81	in. (may require iteration)
Girder top flange width, B_{tf} =	43	in.
Dead load deflection, Δ_{DL} =	-1.51	in. (includes superimposed DL)
Predicted girder camber at deck placement, C_{dp} =	3.43	in. ($C_{dp} = P/S \text{ Camber} - \Delta_{Girder \text{ Self Weight}}$)

GIVENS (Continued):

Vertical Curve Data:

Station at VPI =	5+00.00
Elevation at VPI =	5280
STA @ CL abut. 1, G1 =	4+50.00
STA @ CL abut. 2, G1 =	5+50.00
Curve length, L_c =	400 ft.
Grade in, g_1 =	8.0 %
Grade out, g_2 =	-8.0 %



Horizontal Curve Data:

Radius at G1 CL brg, $R = 1275$ ft. (may not be equal to radius of HCL)

CALCULATIONS

Step 1: Profile effect due to vertical curve

$$ELEV_x = ELEV_{VPC} + g_1 * x + \left(\frac{r}{2}\right) * x^2$$

$$r = \frac{(g_2 - g_1)}{L_c} \text{ (g in \% and } L_c \text{ in STA)}$$

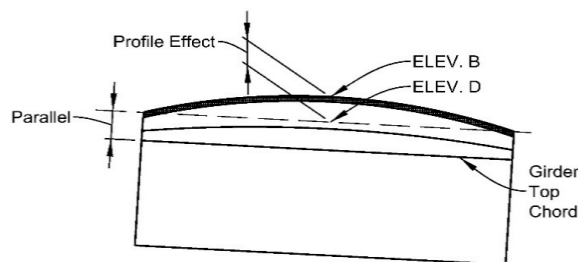
$$ELEV_{VPC} = ELEV_{VPI} - \frac{g_1}{100} * (STA_{VPI} - STA_{VPC})$$

$$STA_{VPC} = STA_{VPI} - \frac{L_c}{2}$$

$$r = -4.000 \text{ \% / STA}$$

$$STA_{VPC} = 3+00.00$$

$$ELEV_{VPC} = 5264.00$$



	X (STA)	$g_1 * x$	$r/2 * x^2$	ELEV	
CL Abut. 1	1.50	12.00	-4.50	5271.50	ELEV _A
Midspan	2.00	16.00	-8.00	5272.00	ELEV _B
CL Abut. 2	2.50	20.00	-12.50	5271.50	ELEV _C

$$\text{Profile effect 1, } \delta_{PE1} = (ELEV_B - ELEV_D) * 12 \frac{\text{in.}}{\text{ft.}}$$

$$ELEV_D = 0.5 * (ELEV_A + ELEV_C)$$

$$ELEV_D = 5271.50$$

$$\delta_{PE1} = 6.00 \text{ in.}$$

CALCULATIONS (Continued):

Step 2: Profile effect due to chorded girders

$$\text{Profile effect 2, } \delta_{PE2} = -M * CS * 12 \frac{\text{in.}}{\text{ft.}}$$

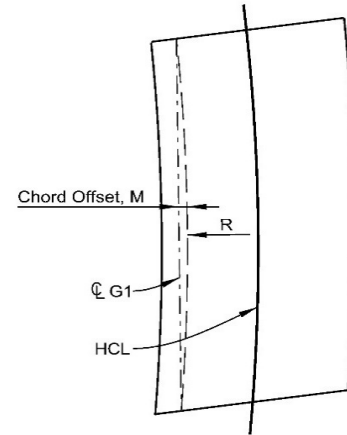
$$\text{Chord offset, } M = \frac{L}{2} * \tan \frac{\alpha}{4}$$

$$\text{Intersection angle of curve along chord, } \alpha = \frac{360 * L}{2\pi R}$$

$$\alpha = 4.49 \text{ }^\circ$$

$$M = 0.98 \text{ ft.}$$

$$\delta_{PE2} = -0.71 \text{ in.}$$



Step 3: Combined profile effect

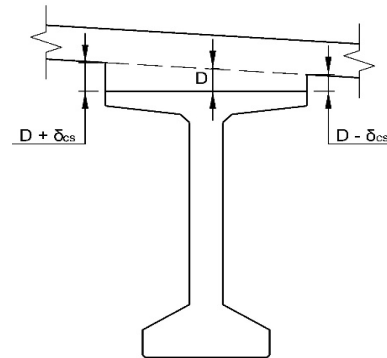
$$\text{Profile effect, } \delta_{PE} = \delta_{PE1} + \delta_{PE2}$$

$$\delta_{PE} = 5.29 \text{ in.}$$

Step 4: Cross-slope effect

$$\text{Cross-slope effect, } \delta_{CS} = \frac{B_{tf} * CS}{2}$$

$$\delta_{CS} = 1.29 \text{ in. (+/-)}$$



Step 5: Check minimum estimated haunch at supports

$$\text{Estimated haunch, } D_{1,\min} = D_1 - \delta_{CS}$$

$$D_{1,\min} = 1.71 \text{ in.}$$

OK, $D_{1,\min} >$ minimum haunch thickness at supports of 1.50 in.

Step 6: Check estimated haunch at midspan

$$\text{Estimated haunch at midspan, } D_2 = \frac{D_1 + D_3}{2} - \Delta_{DL} - C_{dp} + \delta_{PE}$$

$$D_2 = 6.37 \text{ in. @ CL Girder}$$

Step 7: Verify assumed weighted average haunch for DL

$$\text{Actual average haunch for DL, } D_{\text{avg,DL}} = \frac{(D_1 + 10 * D_2 + D_3)}{12}$$

BDM Eq. 5-1

$$D_{\text{avg,DL}} = 5.81 \text{ in.}$$

OK, $D_{\text{avg,DL}}$ matches assumed average haunch used for dead loads

Note: D_2 may be used as the haunch thickness at midspan for the following items:

- Calculating Δ_{DL} reported on the girder sheet and used in setting deck elevations
- Calculating haunch concrete quantities

CALCULATIONS (Continued):Step 8: Calculate camber tolerances per BDM 5.5.2.1.D

$$\text{Over-camber tolerance, } \delta_{\text{over}} = 0.20 * C_{\text{dp}} \geq +1.0 \text{ in.}$$

$$\delta_{\text{over}} = 1.00 \text{ in.}$$

$$\text{Under-camber tolerance, } \delta_{\text{under}} = -0.50 * C_{\text{dp}} \leq -1.0 \text{ in.}$$

$$\delta_{\text{under}} = -1.72 \text{ in.}$$

Step 9: Account for over-camber

$$\text{Minimum haunch at midspan, } D_{2,\text{over}} = D_2 - \delta_{\text{over}} - \delta_{\text{CS}}$$

$$D_{2,\text{over}} = 4.08 \text{ in. (at edge of flange)}$$

OK, $D_{2,\text{over}} >$ minimum haunch thickness of 1.00 in. if girders over-camber by 20%

Step 10: Account for under-camber

$$\text{Maximum haunch at midspan, } D_{2,\text{under}} = D_2 - \delta_{\text{under}}$$

$$D_{2,\text{under}} = 8.08 \text{ in.}$$

$$\text{Weighted average haunch for DL, } D_{\text{avg,DL,under}} = \frac{(D_1 + 10 * D_{2,\text{under}} + D_3)}{12}$$

BDM Eq. 5-1

$$D_{\text{avg,DL,under}} = 7.24 \text{ in.}$$

$$\text{DL defl. (revised using } D_{\text{avg,DL,under}}), \Delta_{\text{DL,under}} = -1.58 \text{ in. (from software)}$$

$$\text{Residual camber} = C_{\text{dp}} + \delta_{\text{under}} + \Delta_{\text{DL,under}}$$

$$\text{Residual camber} = 0.13 \text{ in.}$$

OK, girder maintains positive camber if under-cambered by 50%

Note: Girder has been designed for all strength and service criteria using the following:

- $D_{2,\text{under}}$ as the haunch at midspan for composite section properties
- $D_{\text{avg,DL,under}}$ as the weighted average haunch thickness for dead load
- Girder design compressive strength, f_c per BDM Section 5.3.1.2

CONCLUSION

A proposed haunch of 3 in. at CL of girder at supports passed all required checks. The haunch at supports was intentionally minimized to avoid an excessively thick haunch at midspan.

The example shows how a crest vertical curve adds to the haunch thickness at midspan and, in this case, results in a thicker estimated haunch at midspan than at supports. The haunch thickness at midspan is partially offset by the apparent sag effect of chording girders on a horizontally curved bridge deck.

Other geometric situations that will impact the haunch depth include flared girders and deck cross-slope transitions.

EXAMPLE 8: CANTILEVER WINGWALL DESIGN LOADS

Problem Statement

Example 8 covers the design of a wingwall cantilevered off a standard CDOT integral abutment. The example illustrates the following items:

- The 20 ft. length (measured as shown in Figures 1 & 2) used in Example 8 is the maximum length permitted for cantilevered wingwalls per BDM Section 11.3.6.1.
- The example wingwall is skewed 30°, which is the maximum allowed for an integral abutment per BDM Section 11.3.1.
- At-rest earth pressure is required for skewed wingwalls per BDM Section 11.3.6.2.
- Per BDM Section 11.3.6.2, a portion of the earth pressure acting on the buried part of the wingwall may be neglected, as shown in Figure 1 below. Equations are provided to assist in calculating the resultant wingwall force effects from the trapezoidal shape of earth pressure.
- Force effects are summarized at the two design sections shown in Figure 2. Design Section A is the critical design section for the wingwall. Design Section B summarizes the force-effects transferred to the abutment .

Assumptions

- The backfill is assumed to be sufficiently drained so that hydrostatic pressure does not develop.
- Example 8 assumes that no settlement of the backfill is anticipated. See BDM Section 11.3.6.1 for guidance when significant settlement is expected.

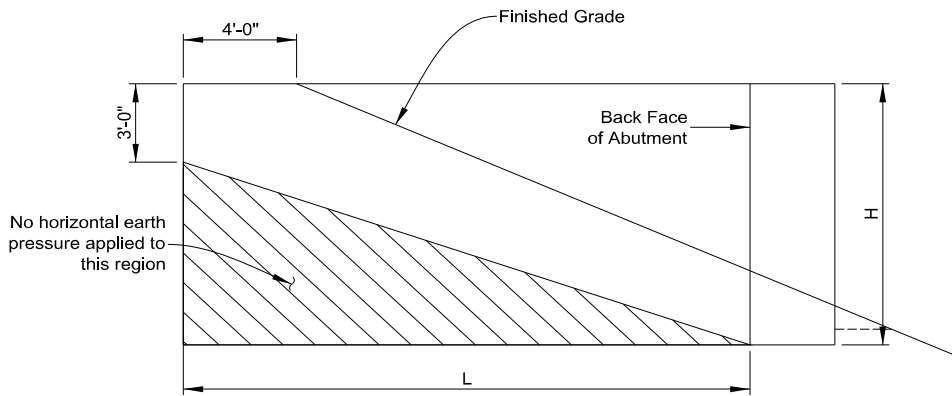


Figure 1 - Wingwall Elevation

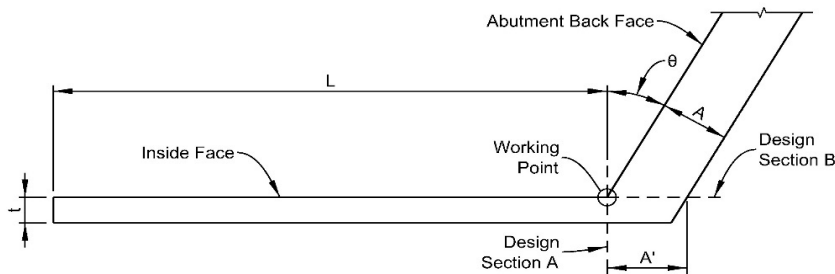
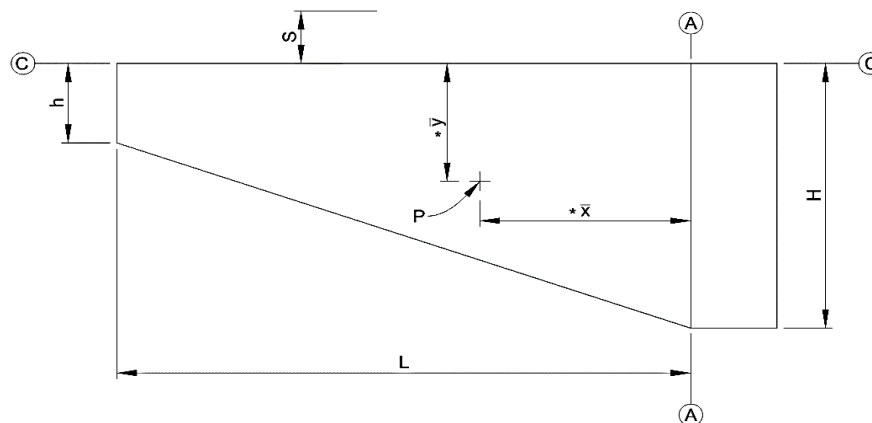


Figure 2 - Partial Plan

Givens

Wingwall Height, H =	10.00	ft.	
Wall Thickness, t =	1.00	ft.	
Live Load Surcharge Height, S =	2.00	ft.	BDM 11.3.6.2
End Height, h =	3.00	ft.	
Wingwall Length, L =	20.00	ft.	
Abutment Width, A =	3.00	ft.	
Skew Angle, θ =	30.00	degrees	
Backfill Unit Weight, γ^1 =	0.130	kcf (CDOT Class 1)	
Angle of Internal Friction of Backfill, Φ^1 =	34.0	degrees	
Dead Load Factor, γ_{DC} =	1.25		AASHTO 3.4.1
Horizontal Earth Pressure Factor, γ_{EH} =	1.35	for at-rest pressure	AASHTO 3.4.1
Live Load Surcharge Factor, γ_{LS} =	1.75		AASHTO 3.4.1
Unit Weight of Concrete, γ_c =	0.15	kcf	

¹Provided by Geotechnical Engineer.



* \bar{x} = distance from back face of abutment to location of Service thrust (\bar{x}_S) or Ultimate thrust (\bar{x}_U)
 \bar{y} = distance from top of wall to location of Service thrust (\bar{y}_S) or Ultimate thrust (\bar{y}_U)

Figure 3 - Horizontal Load Geometry

Calculations

Earth Pressure

Earth pressure moments are calculated about the A and C axes shown in Figure 3. The total thrust, P, due to horizontal earth pressure and live load surcharge, is also calculated and located. The following equations are adopted from a Caltrans design aid; the derivations are not provided.

At-rest Lateral Earth Pressure Coefficient,	$k_0 = 1 - \sin \Phi$	AASHTO	Eq.
	= 0.441		3.11.5.2-1
Effective Fluid Weight, W	$= \max[k_0 * \gamma, 0.057 \text{ kcf}]$	BDM	11.3.6.2
	= 0.057 kcf		

Service Limit State:

$$\begin{aligned} \text{Service Moment, } M_{S_{AA}} &= \frac{WL^2}{24} (3h^2 + (H + 4S)(H + 2h)) \\ &= 301 \quad \text{kft} \end{aligned}$$

$$\begin{aligned} \text{Service Moment, } M_{S_{CC}} &= \frac{WL}{12} (2ShH + (H + h + 2S)(H^2 + h^2)) \\ &= 188 \quad \text{kft} \end{aligned}$$

$$\begin{aligned} \text{Service Thrust, } P_S &= \frac{WL}{6} (H^2 + (h + H)(h + 3S)) \\ &= 41.5 \quad \text{kip} \end{aligned}$$

$$\begin{aligned} \bar{x}_S &= \frac{M_{S_{AA}}}{P_S} \\ &= 7.26 \quad \text{ft., from back face of abutment} \end{aligned}$$

$$\begin{aligned} \bar{y}_S &= \frac{M_{S_{CC}}}{P_S} \\ &= 4.55 \quad \text{ft., from top of wall} \end{aligned}$$

Strength Limit State:

$$\begin{aligned} \text{Effective Surcharge height, } S' &= S \frac{\gamma_{LS}}{\gamma_{EH}} \quad \text{Nominal depth of live load surcharge is increased} \\ &\quad \text{to account for the difference in load factors} \\ &= 2.59 \quad \text{ft.} \end{aligned}$$

$$\begin{aligned} \text{Ultimate Moment, } M_{U_{AA}} &= \gamma_{EH} \frac{WL^2}{24} (3h^2 + (H + 4S')(H + 2h)) \\ &= 455 \quad \text{kft} \end{aligned}$$

$$\begin{aligned} \text{Ultimate Moment, } M_{U_{cc}} &= \gamma_{EH} \frac{WL}{12} (2S'hH + (H + h + 2S')(H^2 + h^2)) \\ &= 276 \quad \text{kft} \end{aligned}$$

$$\begin{aligned} \text{Ultimate Thrust, } P_U &= \gamma_{EH} \frac{WL}{6} (H^2 + (h + H)(h + 3S')) \\ &= 61.9 \quad \text{kip} \end{aligned}$$

$$\begin{aligned} \bar{x}_U &= \frac{M_{U_{AA}}}{P_U} \\ &= 7.35 \quad \text{ft., from back face of abutment} \end{aligned}$$

$$\begin{aligned} \bar{y}_U &= \frac{M_{U_{CC}}}{P_U} \\ &= 4.45 \quad \text{ft., from top of wall} \end{aligned}$$

Self Weight:

$$\begin{aligned} \text{Service Wall Weight, } V_s &= HLt * \gamma_c \\ &= 30.0 \quad \text{kip} \end{aligned}$$

$$\begin{aligned} \text{Ultimate Wall Weight, } V_u &= \gamma_{DC} V_s \\ &= 37.5 \quad \text{kip} \end{aligned}$$

$$\begin{aligned} \text{Service Moment at Design Section A, } M_{S_{wall}} &= V_s * \frac{L}{2} \\ &= 300 \quad \text{kft} \end{aligned}$$

$$\begin{aligned} \text{Ultimate Moment at Design Section A, } M_{U_{wall}} &= V_u * \frac{L}{2} \\ &= 375 \quad \text{kft} \end{aligned}$$

Design Section A Summary

Primary Horizontal Reinforcement

$$M_{S_{AA}} = 301 \quad \text{kft}$$

$$M_{S_{AA}, \text{ per ft.}} = M_{S_{AA}}/H$$

$$= 30.1 \quad \text{kft/ft}$$

$$M_{U_{AA}} = 455 \quad \text{kft}$$

$$M_{U_{AA}, \text{ per ft.}} = M_{U_{AA}}/H$$

$$= 45.5 \quad \text{kft/ft.}$$

These moments are used to design the primary horizontal reinforcement along the inside face of the wingwall for a 1 ft. wide section with a depth of t. For example calculations of reinforced concrete design, see BDM Design Examples 6 and 11. Per calculations not shown, #8 bars at 6 in. spacing are selected as primary reinforcing. All wingwall reinforcement is required to be corrosion resistant, in accordance with BDM Section 5.4.5.

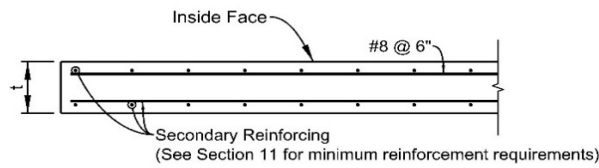


Figure 4 - Primary Horizontal Reinforcement

Top Horizontal Reinforcement

$$M_{S_{\text{wall}}} = 300 \quad \text{kft}$$

$$M_{U_{\text{wall}}} = 375 \quad \text{kft}$$

These moments are used to design the required top reinforcing bars in the wingwall for a section of width t and depth of H. Per calculations not shown, the primary horizontal reinforcing provided above is sufficient to resist the imposed moment; no additional bars are needed.

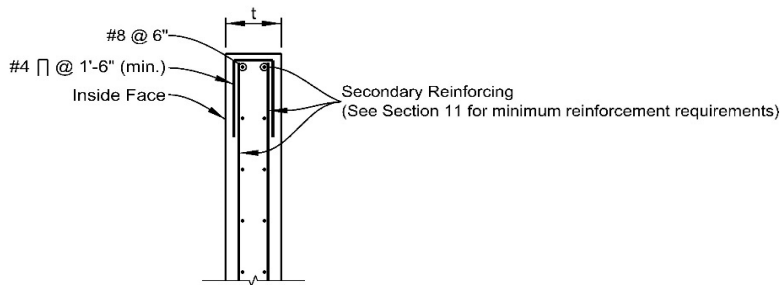


Figure 5 - Top Horizontal Reinforcement

Wingwall Reinforcement Details

See Figures 11.6-1, 11.6-2, and 11.7-1 of the Bridge Detail Manual for additional wingwall reinforcement details, including development of top and primary horizontal bars into the abutment.

Design Section B Summary

Earth pressure and dead loads are ultimately transferred to, and must be resisted by, the abutment and its supporting foundation elements. This section resolves earth pressure and self-weight forces into design forces and moments about centroidal axes of the abutment, and at Design Section B (see Figure 2).

The abutment width along the skew, $A' = A/\cos(\theta)$
 $= 3.46 \text{ ft.}$

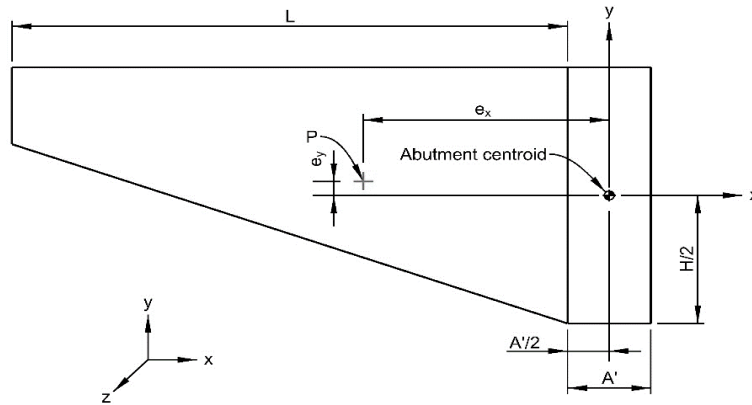


Figure 6 - Abutment Eccentricities

Service Limit State:

Tension, $P_s = 41.5 \text{ kip}$

Shear, $V_s = 30.0 \text{ kip}$

$$e_{x_s} = \bar{x}_s + \frac{A'}{2}$$

$$= 8.99 \text{ ft.}$$

$$e_{y_s} = \frac{H}{2} - \bar{y}_s$$

$$= 0.454 \text{ ft.}$$

$$M_y, \text{ Service} = P_s * e_{x_s}$$

$$= 373 \text{ kft}$$

$$M_x, \text{ Service} = P_s * e_{y_s}$$

$$= 18.8 \text{ kft}$$

$$T_z, \text{ Service} = V_s \left(\frac{L + A'}{2} \right)$$

$$= 352 \text{ kft}$$

Strength Limit State:

$$\text{Tension, } P_u = 61.9 \quad \text{kip}$$

$$\text{Shear, } V_u = 37.5 \quad \text{kip}$$

$$e_{x_U} = \bar{x}_U + \frac{A'}{2}$$

$$= 9.08 \quad \text{ft.}$$

$$e_{y_U} = \frac{H}{2} - \bar{y}_U$$

$$= 0.548 \quad \text{ft.}$$

$$M_y, \text{ Ultimate} = P_U * e_{x_U}$$

$$= 562 \quad \text{kft}$$

$$M_x, \text{ Ultimate} = P_U * e_{y_U}$$

$$= 34.0 \quad \text{kft}$$

$$T_z, \text{ Ultimate} = V_U \left(\frac{L + A'}{2} \right)$$

$$= 440 \quad \text{kft}$$

The shear, tension, torsion, and bi-axial moments summarized above are concurrent and must be resisted by the abutment. Careful detailing is required to provide adequate capacity and sufficient reinforcement development at Design Section B. See Figure 11-13 of the BDM for reinforcement details at the wingwall/abutment interface.

Conclusion

This design example shows the primary calculations needed to develop design forces for a cantilever wingwall supported by an integral abutment. While all force effects were calculated for completeness, it is noted that for this example the following force effects are negligible: self-weight shear at sections A & B, self-weight moment M_{wall} at Section A, and earth pressure moment M_x at Section B.

Other configurations, such as a cantilever wingwall attached to a semi-integral abutment cap, need to resist the same loading as illustrated in this design example. However, in this case, the structural section available to resist the wingwall forces is reduced because the wingwall is supported only by the abutment cap. It is noted that the aforementioned force effects that are typically inconsequential for an integral abutment are more critical for this configuration.

Example 9: Seismic Zone 1 Design Example

Problem Statement

Most bridges in Colorado fall into the Seismic Zone 1 category. Per AASHTO, no seismic analysis is required for structures in Zone 1. However, seismic criteria must be addressed in this case. This example illustrates the seismic-specific code requirements associated with bridges in Zone 1, including:

AASHTO 4.7.4

- Determination of seismic zone
- Horizontal connection forces
- Minimum support length requirements
- Substructure transverse reinforcement requirements

This example bridge is a skewed, 2-span, steel I-girder bridge supported by semi-integral abutments and a multi-column pier, with a drop style pier cap and each column supported by a single caisson (see Figures 1 and 2). The caisson reinforcing clear cover allows the same reinforcing cage diameter to be used for both column and caisson.

Fixed Type 1 bearings are used at the pier while expansion Type 1 bearings are used at the abutments. Anchor bolts projecting through a sole plate are assumed as the restraint mechanism at the bearings, with the holes in the sole plate slotted in the longitudinal direction at the abutments. Note that integral abutments would typically be specified for a bridge with this span arrangement, but expansion abutments are included for illustrative purposes.

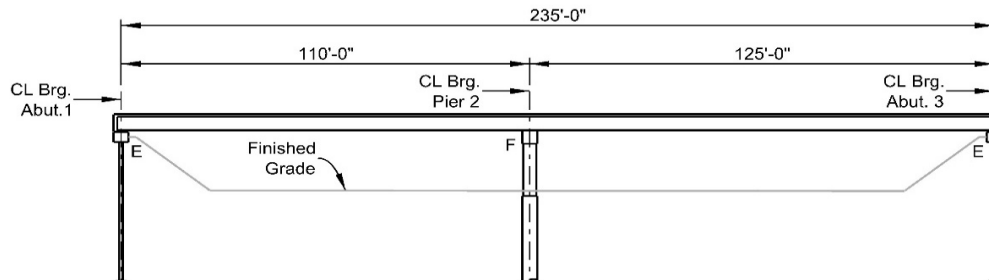


Figure 1 - Bridge Layout and Longitudinal Fixity

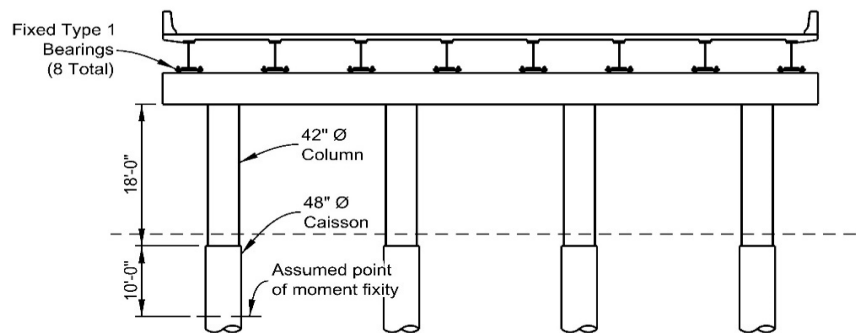


Figure 2 - Pier 2 Elevation

Givens

Total Bridge Length, L =	235.00	ft.	
Pier 2 Column Height, H =	18.00	ft.	See Figure 2
Bridge Skew, S =	5.00	degrees	
Abutment Support Length =	36.00	in.	
Extreme Event I LL Factor, γ_{EQ} =	0.00		AASHTO 3.4.1
Earthquake Load Factor, γ =	1.00		AASHTO 3.4.1
Permanent Vertical Reaction at Abut. 1, R_1^1 =	494	kip per Abutment	
Permanent Vertical Reaction at Pier 2, R_2^1 =	1759	kip per Pier	
Permanent Vertical Reaction at Abut. 3, R_3^1 =	561	kip per Abutment	
Column Diameter, D =	42.0	in.	
Column Clear Cover =	2.00	in.	
Caisson Diameter, D_c =	48.0	in.	
Caisson Clear Cover =	5.00	in.	
Assumed Depth to Moment Fixity ² =	10.00	ft.	See Figure 2
f'_c , Column =	4.50	ksi	
f'_c , Caisson =	4.00	ksi	
f_y =	60.00	ksi	

Seismic Design Parameters:³

Site Class =	D		
PGA =	0.103 g	A_S =	0.165 g
S_S =	0.212 g	S_{DS} =	0.338 g
S_1 =	0.053 g	S_{D1} =	0.127 g

¹ These values are the unfactored total for the support.

² Assumed for this example, Designers should determine analytically for each project.

³ Provided by Geotechnical Engineer for an event with a 7% probability of exceedance in 75 years.

Determination of Seismic Zone

Bridges are assigned to seismic zones based on the S_{D1} parameter and Table 3.10.6-1 in AASHTO, re-created here:

Acceleration Coefficient, S_{D1}	Seismic Zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

Since $S_{D1} = 0.127 < 0.15$, the bridge is located in Seismic Zone 1.

Horizontal Connection Force:

AASHTO 3.10.9.2

For bridges in Zone 1, the horizontal design connection force is a function of the acceleration coefficient, A_S .

Since $A_S = 0.165 \geq 0.05$, the minimum horizontal design connection force is 0.25 times the vertical reaction due to tributary permanent load and the tributary live loads assumed to exist during an earthquake. For this example, the tributary live load is assumed to be zero. See BDM Section 3.12 for guidance on the value of γ_{EQ} to use when performing a seismic analysis for bridges in other seismic zones.

This calculation is performed for both longitudinal and transverse directions.

Longitudinal Direction

Since the abutment bearings allow expansion in the longitudinal direction, the superstructure is restrained only by the 8 fixed bearings at Pier 2. Any passive soil resistance that may develop behind the abutments is ignored. The design connection force in the longitudinal direction at Pier 2 is 0.25 times the sum of the permanent vertical reactions at all supports.

$$\begin{aligned} \text{Tributary reaction at Pier 2} &= R_1 + R_2 + R_3 \\ &= 2814 \quad \text{kip} \\ 0.25 \text{ times reaction} &= 704 \quad \text{kip} \end{aligned}$$

The factored horizontal design connection force for each bearing:

$$= 1.0 \cdot 704 / 8 = 88.0 \quad \text{kip}$$

Transverse Direction

The superstructure is restrained in the transverse direction at all three supports. Therefore, the design connection forces in the transverse directions are a function of the permanent vertical reactions at each support. Each support has 8 bearings.

$$\begin{aligned} \text{Tributary reaction for Abutment 1, } R_1 &= 494 \quad \text{kip} \\ 0.25 \text{ times reaction} &= 124 \quad \text{kip} \end{aligned}$$

The factored horizontal design connection force for each bearing at Abutment 1:

$$= 1.0 \cdot 124 / 8 = 15.5 \quad \text{kip}$$

$$\begin{aligned} \text{Tributary reaction for Pier 2, } R_2 &= 1759 \quad \text{kip} \\ 0.25 \text{ times reaction} &= 440 \quad \text{kip} \end{aligned}$$

The factored horizontal design connection force for each bearing at Pier 2:

$$= 1.0 \cdot 440 / 8 = 55.0 \quad \text{kip}$$

$$\begin{aligned} \text{Tributary reaction for Abutment 3, } R_3 &= 561 \quad \text{kip} \\ 0.25 \text{ times reaction} &= 140 \quad \text{kip} \end{aligned}$$

The factored horizontal design connection force for each bearing at Abutment 3:

$$= 1.0 \cdot 140 / 8 = 17.5 \quad \text{kip}$$

Resolution of Horizontal Connection Forces

Because the bearing devices provide horizontal restraint for the bridge, Designers should verify the capacity of the following items with respect to the connection force: the girder to sole plate connection, the sole plate to anchor bolt connection, the anchor bolt, and anchor bolt anchorage into concrete.

The transverse and longitudinal connection forces determined above are simplified approximations AASHTO allows for Zone 1, in lieu of performing a refined seismic analysis using stiffness based force distribution. As such, the horizontal and longitudinal connection forces need not be combined as described in AASHTO 3.10.8, the provisions of which are predicated on a perpendicular seismic analysis.

Adequate resistance of the connection force shall be verified at any connection (not necessarily just bearing devices) whose failure could cause loss of support or structure instability, as described in AASHTO C3.10.9.2. Previous versions of AASHTO required that the connection force be addressed from the point of application through the substructure and into the foundation elements. However, the 2015 Interim Revisions to AASHTO removed this requirement.

Minimum Support Length Requirements

AASHTO 4.7.4.4

Because no longitudinal restraint is provided at Abutment 1 or 3, the support lengths must meet the requirements of AASHTO 4.7.4.4. Note that bearings with anchors in slotted holes are not considered restrained in the direction of the slots.

The minimum support length, N, measured normal to the centerline of bearing is:

$$N = (8 + 0.02L + 0.08H)(1 + 0.000125 * S^2)$$

AASHTO 4.7.4.4-1

where:

L =

Length of bridge deck to the adjacent expansion joint or to the end of the bridge deck

H =

Average height of columns supporting the bridge deck from the abutment to the next expansion joint (definition for abutments only)

S =

Skew of support measured from line normal to span (degrees)

$$N = (8 + 0.02 * 235' + 0.08 * 18')(1 + 0.000125 * 5^2)$$

$$N = 14.2 \text{ in.}$$

The percentage of N required for a given seismic zone and A_s is shown in AASHTO Table 4.7.4.4-1. For Seismic Zone 1 and with $A_s = 0.165$, 100% of N (14.2 inches) is required. The support length provided is 36 in., thus the minimum support requirements are satisfied.

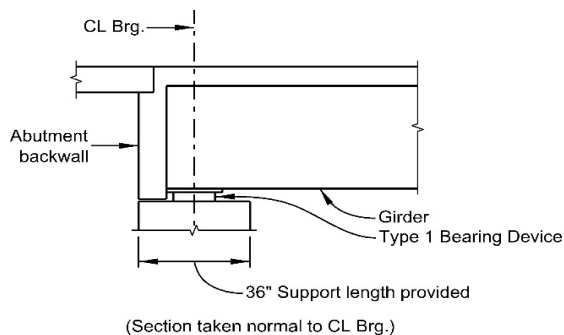


Figure 3 - Abutment Support Length

Substructure Transverse Reinforcement Requirements

AASHTO 5.11

In addition to connection force requirements, for bridges in the high end of Seismic Zone 1 where the response acceleration coefficient S_{D1} is greater than 0.10, transverse confinement reinforcement is required in the expected plastic hinge regions. AASHTO 5.11.2 assumes the plastic hinges zones to be located at the top and bottom of columns. However, the actual locations of plastic hinges depend on support geometry and boundary conditions and must be determined on a project-specific basis. Transverse confinement reinforcement need only be provided in the expected plastic hinge regions.

AASHTO 5.11.2

Since $S_{D1} = 0.127$, confinement reinforcement as specified in AASHTO 5.11.4.1.4 and 5.11.4.1.5 must be provided.

Transverse Reinforcement for Confinement at Plastic Hinges

AASHTO
5.11.4.1.4

Seismic hoop or spiral transverse reinforcement is required in the expected plastic hinge regions. Per BDM Section 5.4.9, CDOT prefers spirals for confinement reinforcement of round elements.

For a circular member, the volumetric ratio, ρ_s , of spiral reinforcement shall satisfy either of the following:

$$\rho_s = \frac{4A_{sp}}{D_{core}s} \geq 0.45 * \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_y} \quad \text{AASHTO 5.6.4.6-1}$$

$$\rho_s = \frac{4A_{sp}}{D_{core}s} \geq 0.12 \frac{f'_c}{f_y} \quad \text{AASHTO 5.11.4.1.4-1}$$

where:

- f'_c = specified 28-day compressive strength of concrete (ksi)
- f_y = minimum yield strength of reinforcing (ksi) ≤ 75.0 ksi
- A_g = gross area of concrete section (in.²)
- A_c = area of the core measured to the outside diameter of the spiral (in.²)
- A_{sp} = cross-sectional area of spiral or hoop (in.²)
- D_{core} = core diameter of column measured to the outside of spiral or hoop (in.)
- s = pitch of spiral or vertical spacing of hoops (in.)

Recall that:

Column Diameter, D =	42.0	in.
Column Clear Height, H =	18.0	ft.
Column Clear Cover =	2.00	in.
Caisson Diameter, D _c =	48.0	in.
Caisson Clear Cover =	5.00	in.

Column Spiral:

$$\text{Core diameter, } D_{\text{core}} = D - 2 * (\text{clear cover})$$

$$D_{\text{core}} = 38.0 \text{ in.}$$

$$A_g = \pi \left(\frac{D}{2} \right)^2$$

$$A_g = 1385 \text{ in.}^2$$

$$A_c = \pi \left(\frac{D_{\text{core}}}{2} \right)^2$$

$$A_c = 1134 \text{ in.}^2$$

The volumetric ratio of spiral reinforcement, ρ_s , must satisfy either of the following:

$$\rho_s \geq 0.45 * \left(\frac{1385 \text{ in.}^2}{1134 \text{ in.}^2} - 1 \right) \frac{4.5 \text{ ksi}}{60 \text{ ksi}} \quad \text{AASHTO 5.6.4.6-1}$$

$$\geq 0.0075$$

$$\rho_s \geq 0.12 \frac{4.5 \text{ ksi}}{60 \text{ ksi}} \quad \text{AASHTO 5.11.4.1.4-1}$$

$$\geq 0.0090$$

$$\rho_s, \text{ min} = 0.0075$$

AASHTO 5.11.4.1.5 limits the spacing of confinement reinforcement to 1/4th the member diameter, D, or 4.0 in. The 4.0 in. maximum spacing controls.

$$\text{Try \#5 spirals at pitch, } s = 4.00 \text{ in.}$$

$$\text{\#5 diameter} = 0.625 \text{ in.}$$

$$\text{Spiral diameter, } d_s = D_{\text{core}} - 0.625"$$

$$d_s = 37.38 \text{ in}$$

The required area of one leg of the spiral, A_{sp} :

$$A_{\text{sp}} = \frac{\rho_s, \text{ min} * s * D_{\text{core}}^2}{4 * d_s} \cong \frac{\rho_s * s * D_{\text{core}}}{4}$$

$$A_{\text{sp}} = 0.29 \text{ in.}^2$$

As a #5 bar has a cross-sectional area of 0.31 in.², using #5 spirals at a 4.0 in. pitch satisfies the confinement requirements.

Lap splices of the confinement reinforcement in the hinge zone are not permitted; rather, splices shall be made by full-welded splices or by full-mechanical connections. AASHTO C5.11.4.1.4 also recommends spacing longitudinal bars a maximum of 8 in. to help confinement (see Figure 4).

AASHTO
5.11.4.1.4

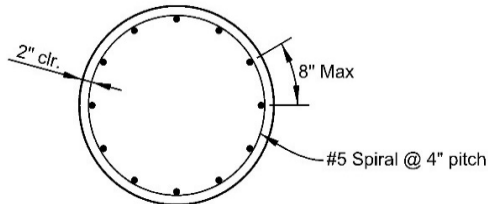


Figure 4 - Column Confinement Reinforcement

Caisson Spiral:

$$\text{Core diameter, } D_{\text{core}} = D_c - 2(\text{clear cover})$$

$$D_{\text{core}} = 38.0 \text{ in.}$$

$$A_g = \pi \left(\frac{D_c}{2} \right)^2$$

$$A_g = 1810 \text{ in.}^2$$

$$A_c = \pi \left(\frac{D_{\text{core}}}{2} \right)^2$$

$$A_c = 1134 \text{ in.}^2$$

The volumetric ratio of spiral reinforcement, ρ_s , must satisfy either of the following:

$$\rho_s \geq 0.45 * \left(\frac{1810 \text{ in.}^2}{1134 \text{ in.}^2} - 1 \right) \frac{4.0 \text{ ksi}}{60 \text{ ksi}}$$

$$\geq 0.0179$$

AASHTO 5.6.4.6-1

$$\rho_s \geq 0.12 \frac{4.0 \text{ ksi}}{60 \text{ ksi}}$$

$$\geq 0.0080$$

AASHTO 5.11.4.1.4-1

$$\rho_s, \text{ min} = 0.008$$

$$\text{Try \#5 spirals at pitch, } s = 4.00 \text{ in.}$$

$$\text{\#5 diameter} = 0.625 \text{ in.}$$

$$\text{Spiral diameter, } d_s = D_{\text{core}} - 0.625"$$

$$d_s = 37.38 \text{ in.}$$

The required area of one leg of the spiral, A_{sp} :

$$A_{\text{sp}} = \frac{\rho_s, \text{ min} * s * D_{\text{core}}^2}{4 * d_s} \cong \frac{\rho_s * s * D_{\text{core}}}{4}$$

$$A_{\text{sp}} = 0.31 \text{ in.}^2$$

A #5 spiral at a 4.0 in. pitch satisfies the confinement requirements (see Figure 5).

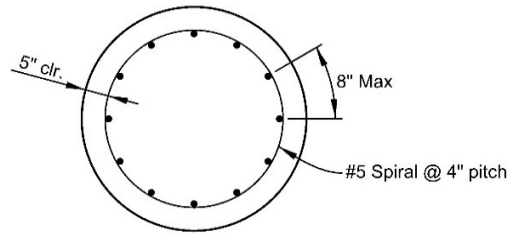


Figure 5 - Caisson Confinement Reinforcement

AASHTO
5.11.4.1.5

Spacing of Transverse Reinforcement for Confinement

AASHTO 5.11.4.1.5 gives guidance on the required lengths where confinement reinforcement is required. As the example column and caisson have similar flexural stiffnesses and capacities, their seismic behavior, including location of plastic hinges, is expected to be similar to that of a pile bent. Therefore, the provisions of AASHTO 5.11.4.1.5 that pertain to pile bents are followed. Further, the column clear height parameter is increased by the assumed depth to fixity to more accurately reflect the bending height of the column/caisson element.

At the top of the column, confinement reinforcement must be provided over a length not less than:

- the maximum cross-sectional column dimension,

$$\text{Column Diameter, } D = 3.50 \text{ ft.}$$

- 1/6th of the bending height of the column/caisson,

$$1/6 * (H + 10') = 4.67 \text{ ft.} < \text{Controls}$$

- or 18 in.

$$18.0 \text{ in.} = 1.50 \text{ ft.}$$

And extend into the adjoining pier cap for a distance not less than:

- one-half the maximum column dimension

$$D/2 = 1.75 \text{ ft.} < \text{Controls}$$

- or 15 in.

$$15.0 \text{ in.} = 1.25 \text{ ft.}$$

AASHTO
5.11.4.3

In accordance with the provisions for pile bents, confinement reinforcement must be provided in the caisson over a length extending from 3.0 times the diameter below the point of moment fixity in the caisson to a height of one diameter, but not less than 18 in., above the mud line.

Figure 6 shows the resulting hinge zones and reinforcement.

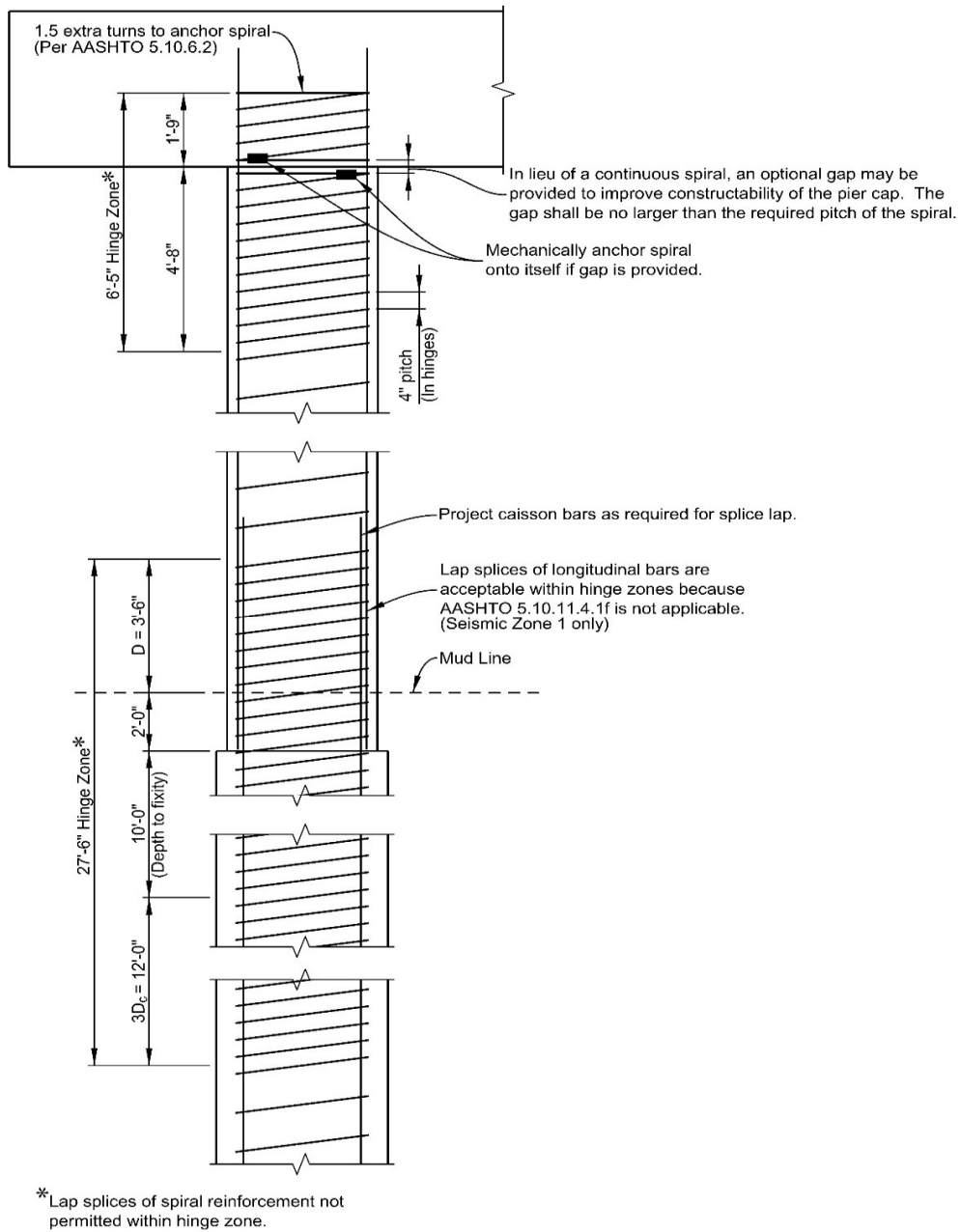


Figure 6 - Hinge Zone Reinforcement

Conclusion

Horizontal design connection forces and minimum seat lengths are typically critical for bridges that use bearing devices, which the example bridge highlighted. Guidelines for other common CDOT situations with respect to horizontal connection forces are as follows:

- Standard CDOT integral abutments that are designed and detailed per BDM Section 11.3.1 are considered restrained in all directions and may be assumed to meet horizontal design connection force requirements by inspection.
- The typical CDOT “pinned” piers where the girders are embedded in concrete pier diaphragms that are connected to the pier cap with a single line of dowels, require Designers to check the doweled connection to the diaphragm for the horizontal connection force. Shear friction at the pier diaphragm to pier cap interface should be used as the resistance.

The example also showed the transverse confinement reinforcement requirements (applicable when $0.10 \leq S_{D1} \leq 0.15$) for the common CDOT configuration of a single caisson supporting each column of a multi-column pier, and where the caisson and column are of similar size. The following guidelines are applicable to other common CDOT substructure configurations, when $0.10 \leq S_{D1} \leq 0.15$:

- Transverse confinement reinforcement for hinging need not be specified at the tops of columns that exhibit cantilever behavior in both horizontal directions, regardless of the S_{D1} magnitude. This is because a plastic hinge cannot form where there is no significant moment development possible.
- For the situation where a significantly larger caisson is used under each column and the column bars are embedded into the caisson, the lower hinge during an earthquake is likely to occur at the bottom of column, not within the caisson. In this case, the hinge zone for the column may use the actual clear column height to establish the upper and lower column hinge zone limits. The caisson is then considered an adjoining member, and the column’s confinement reinforcement should be extended into the caisson as required in AASHTO 5.11.4.3. The caisson’s transverse reinforcement need not meet the special requirements for confinement at plastic hinges.
- For the situation where a caisson is significantly smaller than the column that is used, and the caisson bars project into the column, the lower hinge during an earthquake is likely to occur in the caisson. The “pile bent” criteria shown in the example should be used to establish the top and bottom hinge zone limits, except that confinement reinforcement need not be provided for the bottom of column as no plastic hinge is expected there. The caisson transverse confinement reinforcement should be extended into the column as required in AASHTO 5.11.4.3 for adjoining members.

Example 9 followed AASHTO LRFD provisions for Seismic Zone 1. As an alternative, Designers may follow the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Note that the equivalent to AASHTO LRFD Seismic Zone 1 is Seismic Zone A in the guide specifications.

EXAMPLE 10 - SIGN STRUCTURE FOUNDATION DESIGN

GENERAL INFORMATION

Example Statement: Example 10 demonstrates a design procedure for a drilled shaft foundation for a cantilever sign structure. The cantilever supports a sign panel attached to the horizontal support. The example is only for the design of the shaft foundation. It does not discuss cover design of the members and attachment.

The design follows the LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, First Edition 2015, with 2017 updates (AASHTO LTS), with references to AASHTO LRFD Bridge Design Specifications, 8th Edition (AASHTO). Example 10 was designed with a geotechnical investigation performed on the soil. If one does not have geotechnical data, it is CDOT's preference to use the Brom's method in Section 13 of the AASHTO LTS to determine shaft embedment.

MATERIAL PROPERTIES

Concrete: CDOT Concrete Class BZ

Concrete Compressive Strength	$f_c =$	4	ksi
Concrete Unit Weight	$\gamma_c =$	150	pcf

Steel: Reinforcing Steel

Grade 60 Reinforcing Steel	$f_y =$	60	ksi
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Steel: Steel Members

Steel Density	$\gamma_{steel} =$	490	pcf
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Aluminum: Sign Panels

Aluminum Density	$\gamma_{aluminum} =$	175	pcf
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SIGN STRUCTURE GEOMETRY INFORMATION (Refer to Figure 1)

Pole Length	$L_{pole} =$	22.00	ft.
Pole Base Diameter (outside diameter, o.d.)	$\phi_{pole-B} =$	15.50	in.
Pole Top Diameter (o.d.)	$\phi_{pole-T} =$	12.50	in.
Pole Wall Thickness	$t_{pole} =$	0.1875	in.
Depth to Arm	$D_{arm} =$	1.50	ft.
Arm Length	$L_{arm} =$	16.00	ft.
Arm Base Diameter (o.d.)	$\phi_{arm-B} =$	10.00	in.
Arm End Diameter (o.d.)	$\phi_{arm-E} =$	6.25	in.
Arm Wall Thickness	$t_{arm} =$	0.1875	in.
Shaft Depth	$D_{shaft} =$	13.00	ft.
Shaft Diameter	$\phi_{shaft} =$	36	in.
Number of Sign Panels		1	

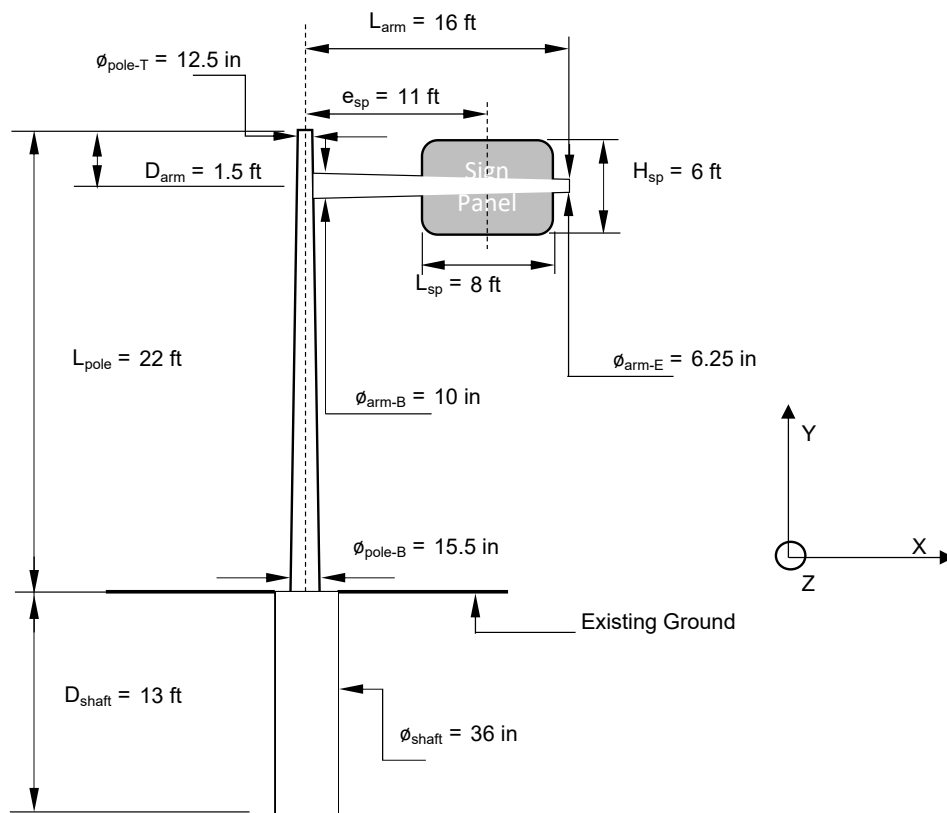


Figure 1 - Sign Structure Geometry Information

SIGN PANEL GEOMETRY INFORMATION

	Length	Height	e_{sp}	Area
Sign Panel 1	8.00 ft.	6.00 ft.	11.00 ft.	48.00 ft. ²

1. LOAD CALCULATION

Use the load combinations and factors from AASHTO LTS T3.4-1 for all loads acting on the sign structure. Determine the loads at the top of the shaft foundation:

APPLIED LOADS

AASHTO LTS 3

(Other loads not listed here may be applicable for different design cases.)

DC - dead load of structural components and nonstructural attachments

LL - live load is considered for designing members for walkways and service platforms

ICE - ice and wind on ice do not practically control and have been removed from the specifications

W - wind load is based on the pressure of the wind acting horizontally on all components

Dead Loads (DC)

AASHTO LTS 3.5

*Weight is based on the typical weight of steel and aluminum

Pole Weight	DC ₁ =	0.61	kip	
Arm Weight	DC ₂ =	0.25	kip	
Sign Weight	DC ₃ =	0.15	kip	*Assumed 7/32" Sign Thickness
Misc. Weight (Anchors and Sign Support)	DC ₄ =	0.08	kip	*Assumed to be 50% of Sign Weight

Live Loads (LL)

AASHTO LTS 3.6

Is LL applicable? no

Ice Loads (ICE)

AASHTO LTS 3.7

Is ICE applicable? no

Wind Loads (W)

AASHTO LTS 3.8

Mean Recurrence Interval	MRI =	1700		BDM 32.3.1.3
Basic Wind Speed	V =	120.00	mph	BDM 32.3.1.3
Height and Exposure Factor for Signs and Arm	K _z =	0.90		AASHTO LTS Eq. 3.8.4-1
Height and Exposure Factor for Pole	K _z =	0.86		AASHTO LTS Eq. 3.8.4-1
Directionality Factor	K _d =	0.85		AASHTO LTS 3.8.5
Gust Effect Factor	G =	1.14		AASHTO LTS 3.8.6
Velocity Conversion Factor - Ext Event	C _{v-Ext} =	0.80		AASHTO LTS 3.8.7
	C _v V d = C _v V φ _{pole-avg} =	112.00		
Velocity Conversion Factor	C _v =	1.00		AASHTO LTS 3.8.7
	C _v V d = C _v V φ _{pole-avg} =	140.00		
Drag Coefficient for Members	C _{d-members} =	0.45		AASHTO LTS 3.8.7
Drag Coefficient for Sign Panels	C _{d-sp} =	1.19	*rounded up	AASHTO LTS 3.8.7
Wind Pressure on Members	$P_z = 0.00256 K_z K_d G V^2 C_d =$	14.50	psf	AASHTO LTS Eq. 3.8.1-1
Wind Pressure on Sign Panels	$P_z = 0.00256 K_z K_d G V^2 C_d =$	38.35	psf	AASHTO LTS Eq. 3.8.1-1
Pole Surface Area (along x axis)	A _{1x} =	25.67	ft. ²	
Pole Surface Area (along z axis)	A _{1z} =	25.67	ft. ²	
Arm Surface Area (along x axis)	A _{2x} =	10.83	ft. ²	
Sign Panels Surface Area (along x axis)	A _{3x} =	48.00	ft. ²	
Wind Load (x-direction)	$W = \Sigma A * P_z = W_x =$	0.37	kip = A _{1z} * P _{z-members}	
Wind Load on Signs (z-direction)	W _{z-sign} =	1.84	kip = A _{3x} * P _{z-sign panels}	
Wind Load on Arm (z-direction)	W _{z-arm} =	0.16	kip = A _{2x} * P _{z-members}	
Wind Load on Pole (z-direction)	W _{z-pole} =	0.37	kip = A _{1x} * P _{z-members}	

UNFACTORED LOADS AND MOMENTS AT TOP OF SHAFT

Moments taken about the centerline of the shaft

Load	Description	Load Direction (x,y,z)	Load (kip)	Moment Arm (ft.)	Moment Direction (x,y,z)	Moment at the Top of the Caisson (kip-ft.)
DC ₁	Pole Weight	Y	0.61	0.00	Z	0.00
DC ₂	Arm Weight	Y	0.25	4.31	Z	1.10
DC ₃	Sign Weight	Y	0.15	11.00	Z	1.68
DC ₄	Misc. Weight	Y	0.08	11.00	Z	0.84
LL	Live Load	Y	0.00	0.00	Z	0.00
W _{x-pole}	Wind on Pole	X	0.37	6.73	Z	2.51
W _{z-sign/arm}	Wind on Signs & Arm	Z	2.00	20.50	X	40.95
W _{z-sign}	Wind on Signs	Z	1.84	11.00	Y	20.25
W _{z-arm}	Wind on Arm	Z	0.16	4.31	Y	0.68
W _{z-pole}	Wind on Pole	Z	0.37	11.00	X	4.09

LOAD COMBINATIONS

AASHTO LTS T3.4-1

Load Combination	γ_{DC}	γ_{LL}	γ_w	Application
Strength I	1.25	1.60	-	Gravity
Extreme Ia	1.10	-	1.00	Wind max
Extreme Ib	0.90	-	1.00	Wind min
Service I	1.00	-	1.00	Translation

SUMMARY OF FACTORED LOADS AND MOMENTS AT TOP OF SHAFT

Moments taken about the centerline of the shaft

$$U = \gamma_{DC}DC + \gamma_{LL}LL + \gamma_w W$$

Load Combination	Axial (kip)	Moment about x-axis (kip-ft.)	Moment about y-axis* (kip-ft.)	Moment about z-axis (kip-ft.)	Shear in the x-axis (kip)	Shear in the z-axis (kip)
Strength I	1.37	-	-	4.53	-	-
Extreme Ia	1.20	45.05	20.92	6.49	0.37	2.37
Extreme Ib	0.98	45.05	20.92	5.77	0.37	2.37
Service I	1.09	45.05	20.92	6.13	0.37	2.37

*M_y to be used for torsion calculation

2. SHAFT CAPACITY

Run static L-PILE analysis with parameters from geotechnical report and calculated factored loads.

L-PILE INPUT

Soil Properties

*From Geotechnical Report

Top of Boring Elevation	$E_{\text{boring top}} =$	5297.00
Bottom of Boring Elevation	$E_{\text{boring bot}} =$	5270.00
Top of Shaft Elevation	$E_{\text{caisson top}} =$	5297.50
Bottom of Shaft Elevation	$E_{\text{caisson bot}} =$	5284.50

Top of Soil Elev.	Soil Type	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)	ϵ_{50}	k (pci)
5297.00	Stiff Clay w/o free water using k	120.00	0.00	2000.00	0.006	500.00
5290.00	Stiff Clay w/o free water using k	130.00	0.00	2500.00	0.005	1000.00

Shaft Section Properties

Section	Round Concrete Shaft		
Length of Section	$D_{\text{shaft}} =$	13.00	ft.
Length of Section in Bedrock	$D_{\text{rock}} =$	5.50	ft.
Section Diameter	$\phi_{\text{shaft}} =$	36	in.
Longitudinal Rebar Size	#	8	
Longitudinal Rebar Count		13	
Concrete Cover to Inside Edge of Stirrup Bar		3.625	in.
Stirrup Size	#	5	
Stirrup Spacing		12	in.

BDM 5.4.3

INPUT LOADS

L-Pile models in only one plane, therefore:

Shear in the X Direction is paired with Moment in the Z Direction

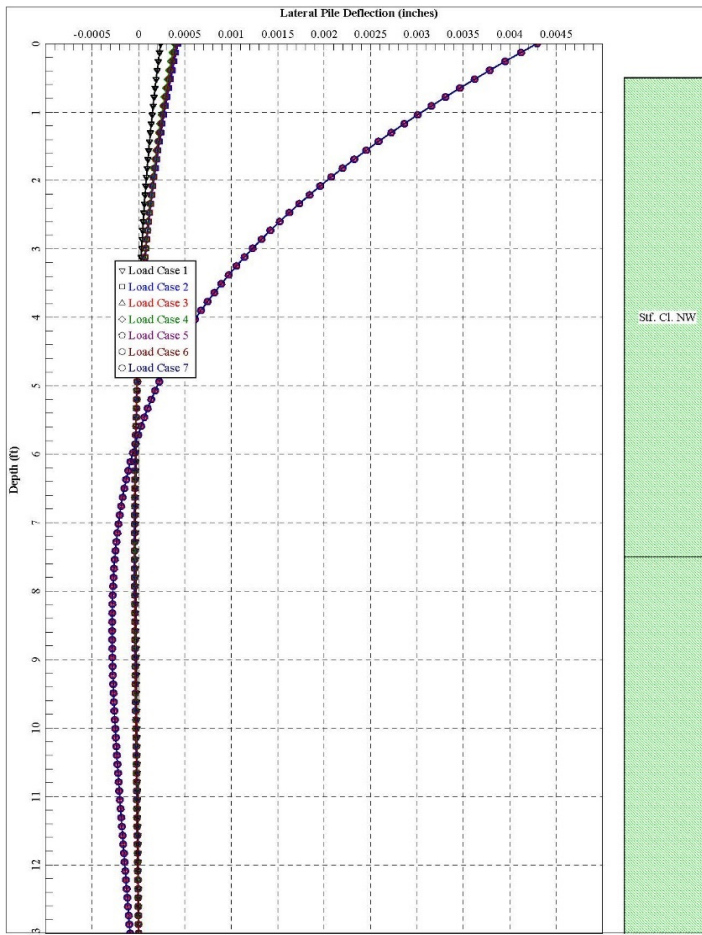
Shear in the Z Direction is paired with Moment in the X Direction

Load Case	Pile-Head Loading Condition	Shear (lb)	Moment (lb-in)	Axial (lb)
1	1	0	54,347	1,367
2	1	372	77,892	1,203
3	1	2,370	540,557	1,203
4	1	372	69,196	984
5	1	2,370	540,557	984
6	1	372	73,544	1,093
7	1	2,370	540,557	1,093

L-PILE OUTPUT

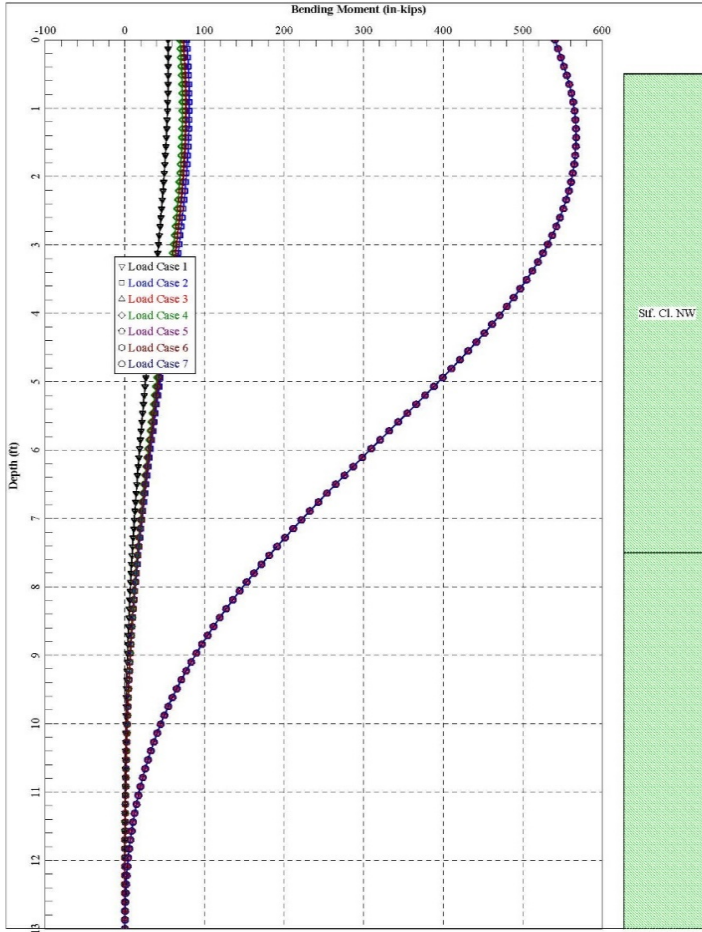
*Agg size assumed to be 0.75"

Reinforcement	13 #8		
Clear Distance Between Bars	5.64	in.	
Spacing Check for Min Spacing	>		
Min Clear Allowed, Max(1.5d _b , 1.5*Agg Size, 1.5") =	1.50	in.	AASHTO 5.10.3.1.1
Min Clear Allowed, Max(5*Agg Size, 5") =	5.00	in.	AASHTO 5.12.9.5.2
Area of Steel	10.27	in. ²	
Percentage of Steel	1.01%		
	>		
	0.80%		AASHTO 5.12.9.5.2
Maximum Pile-Head Deflection	0.0043	in.	
Maximum Shear Force	7,261	lbs	
Maximum Bending Moment	567,170	lb-in	
Axial Thrust at Max Moment Case	1,203	lbs	



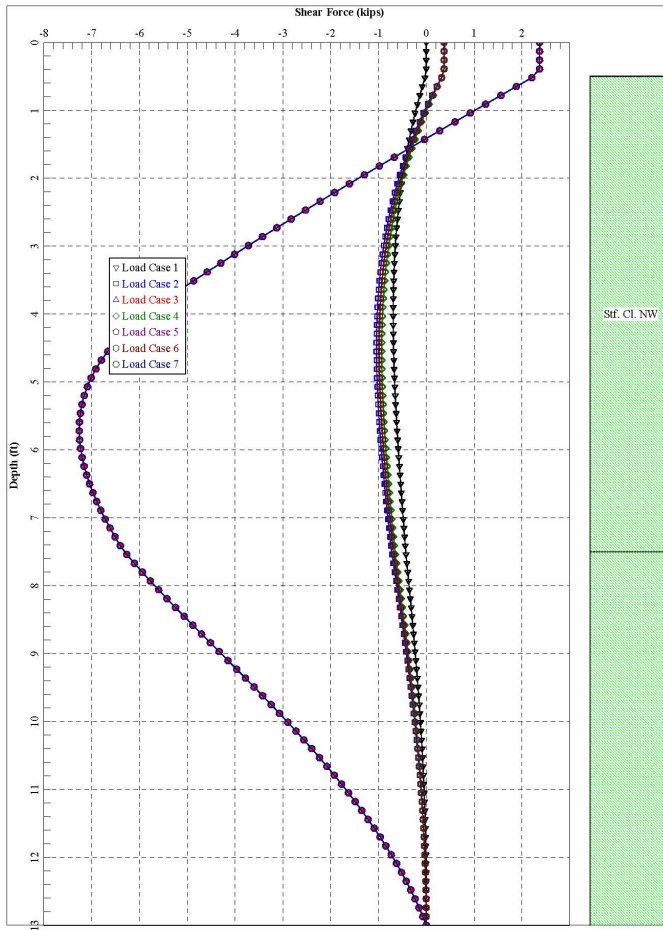
Lateral Pile Deflection (in.) vs Depth (ft.)

The maximum deflection, at the top of the caisson is 0.0043", which is considered zero; therefore, the shaft is deemed stable for the length used per the Engineer's judgment.



Bending Moment (in-kip) vs Depth (ft.)

The maximum factored moment is less than the maximum resistance moment. The shaft is considered stable per the reinforcement and size.



Shear Force (kips) vs Depth (ft.)

AXIAL RESISTANCE

Unit End Bearing Resistance	$q_p =$	18.00	ksf	Geotechnical Report	
Unit Side Resistance	$q_s =$	1.00	ksf	Geotechnical Report	
End Bearing Factor	$\phi_{qp} =$	0.40		Geotechnical Report	
Side Resistance Factor	$\phi_{qs} =$	0.45		Geotechnical Report	
Shaft End Bearing Area	$A_{shaft} = \pi d^2 / 4 =$	$A_{shaft} =$	7.07	ft. ²	
Shaft Perimeter	$P_{shaft} = \pi d =$	$P_{shaft} =$	9.42	ft.	
Depth in Bedrock		$D_{rock} =$	5.50	ft.	
End Bearing Resistance	$\phi_{qp} q_p A_{shaft} = \phi_{qp} R_p =$		50.89	kip	AASHTO Eq. 10.8.3.5-2
Side Shear Resistance	$\phi_{qs} q_s P_{shaft} D_{rock} = \phi_{qs} R_s =$		23.33	kip	AASHTO Eq. 10.8.3.5-3
Ultimate Shaft Resistance	$R_R = \phi R_n = \phi_{qp} R_p + \phi_{qs} R_s$		74.22	kip	AASHTO Eq. 10.8.3.5-1
Applied Vertical Load		15.15	kip	F_y max plus DL of shaft	
		<			
		74.22	kip		
		OK!			

BENDING RESISTANCE

L-Pile provides Nominal Moment Resistance for each axial value.

The maximum factored applied moment from each L-Pile case with varying axial is compared to the nominal moment resistance provided by L-Pile.

$$\phi M_n = M_u \geq M_{applied}$$

$\phi = 0.75$ AASHTO 5.5.4.2

Load Case	Axial (lb)	Nominal Moment Resistance, M_n (kip-in.)	ϕ	Ultimate Moment Resistance, M_u (kip-in.)	Factored Applied Moment, $M_{applied}$ (kip-in.)	Check
5	984	8,472.87	0.75	6,354.65	540.56	OK!
7	1,093	8,474.12	0.75	6,355.59	540.56	OK!
3	1,203	8,475.37	0.75	6,356.53	540.56	OK!
1	1,367	8,477.25	0.75	6,357.94	54.35	OK!

SHEAR AND TORSION RESISTANCE

*The side shear resistance of soil for torsion effects is checked at the end of this example.

Shear Force	$V_u = 7.26$	kip	
Torsion	$M_y = T_u = 20.92$	k-ft.	
Flexure	$M_u = 45.05$	k-ft.	
Tension	$N_u = 15.15$	kip	
Phi for Shear and Torsion	$\phi = 0.90$		AASHTO 5.5.4.2

Concrete Cover to Reinforcing & Bar Size:

Side Cover	$clr = 3.00$	in.
Stirrup Bar Diameter	$d_{stirrup} = 0.63$	in.

Nominal Resistance	$M_n = 706.07$	k-ft.	L-Pile Output
Area of Flexural Reinforcement	$A_f = 5.14$	in. ²	Half of the reinforcement in shaft
Dia of Circle Passing Through Long. Reinf	$D_r = 27.75$	in. ³	
Depth to Flexural Reinforcement	$d_s = 26.83$	in.	$= D_{shaft}/2 + D_r/\pi$

Torsional Cracking Moment AASHTO 5.7.2.1

Area of Concrete Perimeter	$A_{cp} = 1,018$	in. ²	
Concrete Perimeter	$p_c = 113.10$	in.	
Compressive Stress at Centroid of Section	$f_{pc} = 0.00$	ksi	
	$T_{cr} = 0.126K\lambda\sqrt{f'_c} \frac{A_{cp}^2}{p_c}$		AASHTO Eq.5.7.2.1-4

$$K = \sqrt{1 + \frac{f_{pc}}{0.126\lambda\sqrt{f'_c}}} \leq 2.0 \quad \text{AASHTO Eq.5.7.2.1-6}$$

$$K = 1.00$$

Torsional Cracking Moment	$T_{cr} = 2,308.54$	k-in.	
	$0.25\phi T_{cr} = 519.42$	k-in.	
	>		AASHTO Eq. 5.7.2.1-3
	$T_u = 251.08$	k-in.	

Torsional effects can be neglected

Design Factored Shear Force	$V_u = 7.26$	kip
-----------------------------	--------------	-----

Shear Stress on Concrete AASHTO 5.7.2.8

Effective Shear Depth	$d_v = \max \left\{ \begin{array}{l} \frac{M_n}{A_s f_y} \\ 0.9 * d_s \\ 0.72 * h \end{array} \right.$		
	$M_n / A_s f_y = 27.50$	in.	Maximum
	$0.9 * d_s = 24.15$	in.	
	$0.72 * h = 25.92$	in.	
	$d_v = 27.50$	in.	

Shear Stress	$v_u = \frac{ V_u }{\phi b_v d_v} = v_u = 0.0081$	ksi	AASHTO Eq. 5.7.2.8-1
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Transverse Reinforcement

Transverse Reinforcement is required where: $V_u > 0.5\phi V_c$ AASHTO Eq. 5.7.2.3-1

$$V_u = 7.26 \text{ kip}$$

<

$$0.5\phi V_c = 115.15 \text{ kip}$$

Transverse reinforcement not necessary

Minimum Transverse Reinforcement $A_{v, \min} \geq 0.0316\lambda\sqrt{f'_c} \frac{b_v S}{f_y}$ AASHTO Eq. 5.7.2.5-1

$$A_{v, \min} \geq 0.46 \text{ in.}^2$$

<

$$A_{v, \text{prov'd}} = 0.62 \text{ in.}^2$$

OK!

Maximum Spacing of Transverse Reinforcement AASHTO 5.7.2.6

$$v_u = 0.008 \text{ ksi}$$

<

$$0.125f'_c = 0.500 \text{ ksi}$$

If $v_u < 0.125f'_c$, then:

$$s_{max} = 0.8d_v \leq 24.0$$

AASHTO Eq. 5.7.2.6-1

If $v_u \geq 0.125f'_c$, then:

$$s_{max} = 0.4d_v \leq 12.0$$

AASHTO Eq. 5.7.2.6-2

$$s_{max} = 22.00 \text{ in.}$$

>

$$s_{v, \text{prov'd}} = 12.00 \text{ in.}^2$$

OK!

Maximum Nominal Shear Resistance AASHTO 5.7.3.3

Nominal Shear Resistance $0.25f'_c b_v d_v = V_n = 990.01 \text{ kip}$ AASHTO Eq. 5.7.3.3-2

$$\phi V_n = 891.01 \text{ kip}$$

>

$$V_u = 7.26 \text{ kip}$$

OK!

$$\epsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u|\right)}{E_s A_s}$$

AASHTO Eq. 5.7.3.4.2-4

Net Longitudinal Tensile Strain $\epsilon_s = 0.0002$

For sections containing at least the minimum amount of transverse reinforcement specified in Art. 5.7.2.5, the value of β may be determined by the following equation:

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \quad \text{AASHTO Eq. 5.7.3.4.2-1}$$

$$\beta = 4.09$$

$$\theta = 29.81 \quad \text{AASHTO Eq. 5.7.3.4.2-3}$$

Nominal Shear Resistance of Concrete $V_c = 0.0316\beta\lambda\sqrt{f'_c}b_vd_v$ AASHTO Eq. 5.7.3.3-3

$$V_c = 255.88 \text{ kip}$$

$$V_u = 7.26 \text{ kip}$$

Side Shear Resistance of Soil in Torsion

Per CDOT's experience, the soil torsion capacity may control the shaft length. If the drilled shaft sees torsion, the following applicable checks should be completed. Refer to Report No. CDOT-DTD-R-2004-8 for equations and procedure.

Cohesive Soil Resistance

It is CDOT's approach that the soil resistance to torsion in cohesive soils is based on the drilled shaft embedment area into the soil, neglecting the top 1.5' of section length. Perform the following check if the drilled shaft is in cohesive soil.

Torsion	$T_u = 20.92$ k-ft.
Soil profile used for example	cohesion, $s_u = 2000$ psf
Assumed Phi for Torsion, per SF = 1.25	$\phi = 0.80$
Section Diameter	$\phi_{shaft} = 36$ in
Length of Section	$D_{shaft} = 13.00$ ft.

Drilled shaft side resistance $T_s = \frac{\pi \cdot \phi_{shaft}^2}{2} (D_{shaft} - 1.5\phi_{shaft}) \cdot s_u$

Drilled shaft toe resistance $T_t = \frac{\pi \cdot \phi_{shaft}^3}{12} s_u$

$$T_s = 240.33 \text{ k-ft.}$$

$$T_t = 14.14 \text{ k-ft.}$$

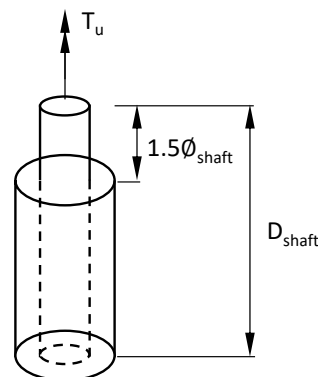
Nominal Total Torsion Resistance $T_n = (T_s + T_t) = 254.47 \text{ k-ft.}$

$$\phi T_n = 203.58 \text{ k-ft.}$$

>

$$T_u = 20.92 \text{ k-ft.}$$

OK!



Cohesionless Soil Resistance

It is CDOT's approach that the soil resistance to torsion in cohesionless soils is based on the drilled shaft embedment into the soil. Perform the following check if the drilled shaft is in cohesionless soil.

Torsion	$T_u =$	20.92	k-ft.
Soil profile used for example	unit weight, $\gamma =$	120	pcf
Soil profile used for example	friction angle, $\phi =$	30.00	degrees
Assumed Phi for Torsion, per SF = 1.25	$\phi =$	0.80	
Section Diameter	$\phi_{shaft} =$	36.00	in
Length of Section	$D_{shaft} =$	13.00	ft.
Weight of Section	$W =$	13.78	kip

Drilled shaft side resistance $T_s = \frac{\pi \cdot \phi_{shaft}^2}{2} D_{shaft} \cdot r_s$

Drilled shaft toe resistance $T_t = \frac{\phi_{shaft}}{3} W \cdot \tan\phi$

Coefficient of lateral earth pressure $K = \frac{2D_{shaft}}{3\phi_{shaft}} (1 - \sin\phi) = 1.44$

Unit shaft side resistance $r_s = K\gamma \frac{D_{shaft}}{2} \tan\phi = 0.65$ ksf

$T_s = 119.55$ k-ft.

$T_t = 7.96$ k-ft.

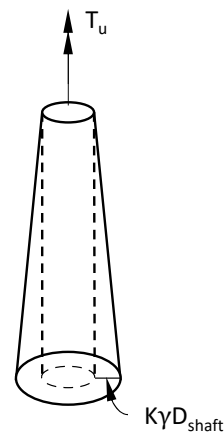
Nominal Total Torsion Resistance $T_n = (T_s + T_t) = 127.51$ k-ft.

$\phi T_n = 102.00$ k-ft.

>

$T_u = 20.92$ k-ft.

OK!



**EXAMPLE 11 - CAST-IN-PLACE CONCRETE CANTILEVER
RETAINING WALL**

GENERAL INFORMATION

Example 11 demonstrates design procedures for cast-in-place cantilever retaining walls supported on spread footing in conformance with AASHTO and Section 11.5 of this BDM. Horizontal earth pressure is applied based on the Coulomb earth pressure theory.

Example Statement: The retaining wall supports 15'-0" of level roadway embankment measured from top of wall to top of footing. The wall will be built adjacent to the roadway shoulder where traffic is 2 ft. from the barrier face. The wall stem is 1'-6" wide to accommodate mounting a Type 7 Bridge Rail to the top of wall. See Figure 3.

Starting Element Size Assumptions:

Total Footing Width = 70% to 75% of the design height

Footing Thickness = 10% of the design height

Toe Width = 10% of design height

MATERIAL PROPERTIES

Soil: CDOT Class 1 Backfill-Drained

Footing bears on soil

Soil unit weight	$\gamma_s =$	0.130	kcf	
Angle of internal friction (backfill)	$\phi =$	34	deg	
Wall-backfill friction angle	$\delta = 2/3\phi =$	22.67	deg	
Coefficient of active earth pressure	$K_a =$	0.261	(Coulomb)	AASHTO Eq. 3.11.5.3-1
Coefficient of passive earth pressure	$K_p =$	7.60		AASHTO Fig. 3.11.5.4-1
Active equivalent fluid weight	$EFW (a) = K_a \gamma_s =$	0.036	kcf (36 pcf min)	BDM 11.5
Passive equivalent fluid weight	$EFW (p) = K_p \gamma_s =$	0.988	kcf	

Subgrade: for bearing and sliding

Nominal design values are typically provided in the project-specific geotechnical report.

Nominal soil bearing resistance	$q_n =$	7.50	ksf	
Angle of internal friction (subgrade)	$\phi_{Sub} =$	20	deg (for sliding)	
Wall-subgrade friction angle	$\delta_{Sub} = 2/3\phi_{Sub} =$	13.33	deg (for shear key design)	
Nominal soil sliding coefficient	$\mu_n = \tan \phi_{Sub} =$	0.36		AASHTO C.10.6.3.4

Concrete: CDOT Concrete Class D

Concrete compressive strength	$f_c =$	4.50	ksi
Concrete unit weight	$\gamma_c =$	0.150	kcf

Bridge Rail Type 7

Type 7 bridge rail weight	$w_{rail} =$	0.486	klf
Center of gravity from wall back face	$X_{C.G.} =$	6.84	in. (see Bridge Worksheet B-606-7A)

RESISTANCE FACTORS

When not provided in the project-specific geotechnical report, refer to the indicated AASHTO sections.

Bearing	$\phi_b =$	0.55	AASHTO T.11.5.7-1
Sliding (concrete on soil)	$\phi_T =$	1.00	AASHTO T.11.5.7-1
Sliding (soil on soil)	$\phi_{T\ s-s} =$	1.00	AASHTO T.11.5.7-1
Passive pressure	$\phi_{ep} =$	0.50	AASHTO T.10.5.5.2.2-1
Extreme event	$\phi_{EE} =$	1.00	AASHTO 11.5.8

WALL GEOMETRY INFORMATION

See Figure 1.

Stem Height	H =	15.00	ft.	
Top of Wall Thickness	T _{Top} =	1.50	ft.	
Bottom of Wall Thickness	T _{Bot} =	1.75	ft.	
Width of footing	B =	10.00	ft.	
Thickness of Footing	T _F =	1.25	ft.	
Toe Distance	S =	2.75	ft.	
Height of fill over the toe	H _{TF} =	2.00	ft.	BDM 11.5.1
Minimum Footing embedment \geq 3 ft..	H _{TF} + T _F =	3.25	ft.	OK BDM 11.5.1
Bridge Rail Type 7 Height	H _B =	2.92	ft.	
Wall Backface to vertical surcharge	R =	2.00	ft.	
Live Load Surcharge height	h _{Sur} =	2.00	ft.	AASHTO Table 3.11.6.4-2
Vehicle Collision Load (TL-4)	P _{CT} =	54.00	kip	AASHTO Table A13.2-1
Collision Load Distribution	L _t =	3.50	ft.	AASHTO Table A13.2-1
Top of wall to point of collision impact on rail	h _{CT} =	2.67	ft.	

1. STABILITY CHECKS

Use the load combinations and factors from AASHTO 11.5.6 and BDM Section 11.5.1 for all loads acting on the retaining wall. Evaluate the retaining wall for the following:

1. Eccentricity
2. Sliding
3. Bearing

Note: The Geotechnical Engineer is responsible for evaluating global stability with consideration for both footing width and embedment.

APPLIED LOADS

Loads not listed here may be applicable for different design cases.

DC - dead load of structural components and nonstructural attachments

EH - horizontal earth pressure load

EV - vertical pressure from dead load of earth fill

CT - vehicular collision force

LS - live load surcharge

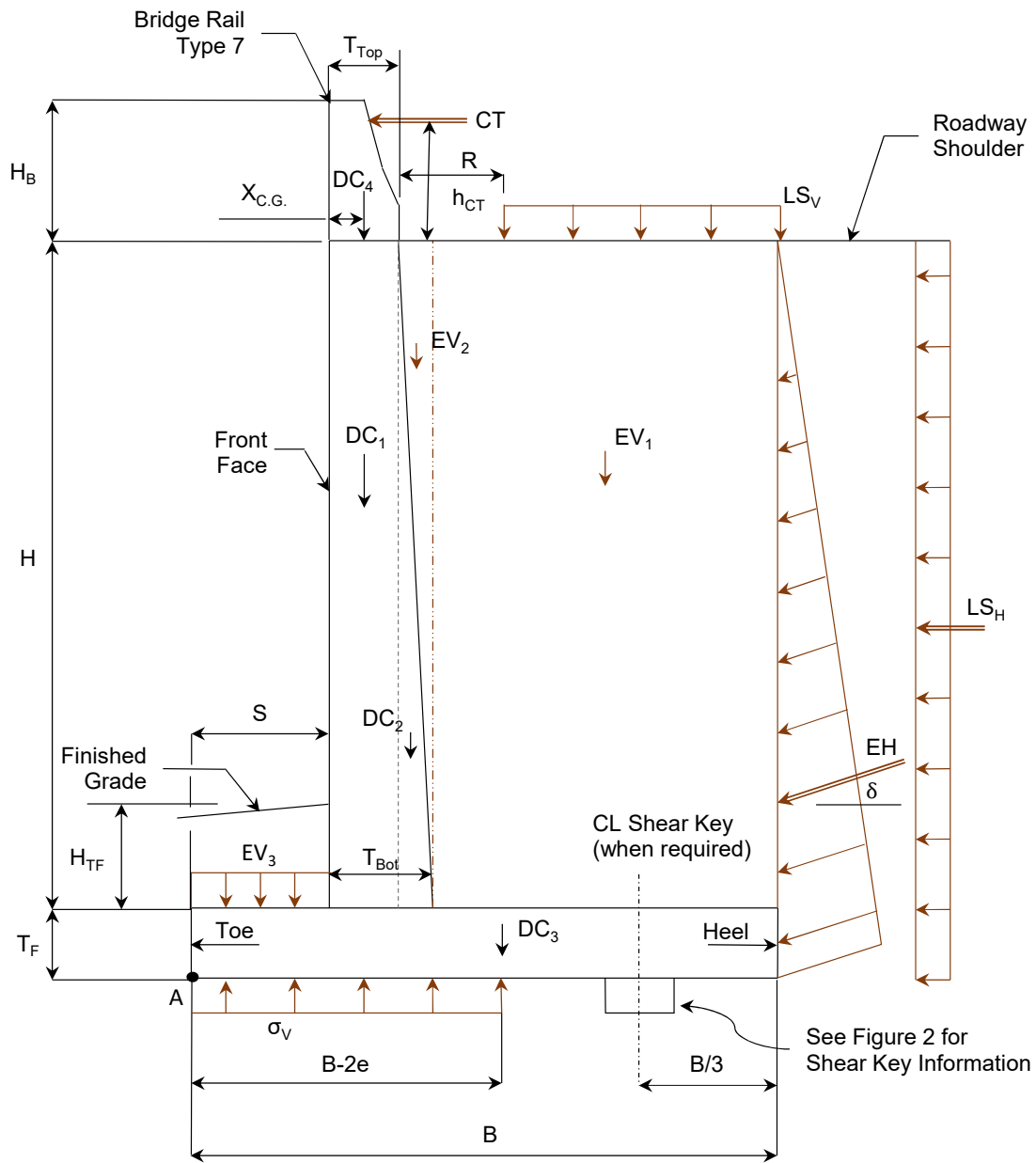


Figure 1 - Typical Section

Summary of Unfactored Loads and Moments

Resolve moments about Point A (see Figure 1 - Typical Section)

Vertical Loads & Moments				
Load Type	Description	V (kip/ft.)	Moment Arm (ft.)	MV (kip-ft./ft.)
DC ₁	Stem dead load	3.38	3.50	11.83
DC ₂	Stem dead load	0.28	4.33	1.21
DC ₃	Footing dead load	1.88	5.00	9.40
DC ₄	Barrier dead load	0.49	3.32	1.63
EV ₁	Vertical pressure from dead load of fill on heel	10.73	7.25	77.79
EV ₂	Vertical pressure from dead load of fill on heel	0.24	4.42	1.06
EV ₃	Vertical pressure from dead load of fill on toe	0.72	1.38	0.99
EH _V	Vertical component of horizontal earth pressure	1.83	10.00	18.30
LS _V	Vertical component of live load surcharge	0.98	8.13	7.97

Horizontal Loads and Moments				
Load Type	Description	H (kip/ft.)	Moment Arm (ft.)	MH (kip-ft./ft.)
EH _H	Horizontal component of horizontal earth pressure	4.39	5.42	23.79
LS _H	Horizontal component of live load surcharge	1.17	8.13	9.51
CT	Vehicular collision load	2.61	18.92	49.38

$$EH_V = \sin(\delta) EH = \sin(\delta) 0.5 EFW(a) (H + T_F)^2$$

$$EH_H = \cos(\delta) EH = \cos(\delta) 0.5 EFW(a) (H + T_F)^2$$

$$LS_V = \gamma_s h_{sur} (B - S - T_{Top} - R)$$

$$LS_H = EFW(a) h_{sur} (H + T_F)$$

Note: The collision force (CT) is assumed to be distributed over a length of "L_t" ft. at the point of impact and is also assumed to spread downward to the bottom of the footing at a 45° angle. Conservatively, CT is assumed at the end of the wall where the force distribution occurs in one direction. See Figure 11-20 in Section 11 of this BDM.

Reinforcement between the Bridge Rail Type 7 and the wall interface is assumed to be adequate to transfer the collision load from the rail through the wall to the footing.

$$CT = P_{CT} / (L_t / 2 + (h_{CT} + H + T_F))$$

Load Combinations

The table that follows summarizes the load combinations used for the stability and bearing checks of the wall. To check sliding and eccentricity, load combinations Strength Ia and Extreme Event IIa apply minimum load factors to the vertical loads and maximum load factors to the horizontal loads. To check bearing, load combinations Strength Ib, Strength IV, and Extreme Event IIb apply maximum load factors for both vertical and horizontal loads.

CT load is considered with Extreme Event II limit state when checking eccentricity, sliding, and bearing.

Note: LS_H, LS_V, and EH_H are not included in Extreme Event IIa or IIb. It is assumed that the horizontal earth pressure is not activated due to the force of the collision deflecting the wall away from the soil mass at the instant of collision.

LS_V is not applied when analyzing sliding and overturning; rather, it is applied only for load combinations that are used to analyze bearing (AASHTO 11.5.6, Figure C11.5.6-3a).

The service limit state is used for the crack control check and settlement.

Total factored force effect: $Q = \sum \eta_i \gamma_i Q_i$ AASHTO 3.4.1-1

where Q_i = force effects from loads calculated above

Load Modifiers:	Ductility	η _D =	1.00	AASHTO 1.3.3-1.3.5
	Redundancy	η _r =	1.00	
	Operational Importance	η _I =	1.00	

Load Factors:

Load Combination	γ _{DC}	γ _{EV}	γ _{LS_V}	γ _{LS_H}	γ _{EH}	γ _{CT}	Application
Strength Ia	0.90	1.00	-	1.75	1.50	-	Sliding, Eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	-	Bearing, Strength Design
Strength IV	1.50	1.35	-	-	1.50	-	Bearing
Extreme IIa	0.90	1.00	-	-	-	1.00	Sliding, Eccentricity
Extreme IIb	1.25	1.35	-	-	-	1.00	Bearing
Service I	1.00	1.00	1.00	1.00	1.00	-	Wall Crack Control

Summary of Load Groups:

Load Combination	Vertical Load & Moment		Horizontal Load & Moment	
	V (kip/ft.)	MV (kip-ft.)/ft.	H (kip/ft.)	MH (kip-ft.)/ft.
Strength Ia	19.86	128.95	8.63	52.33
Strength Ib	27.78	179.27	8.63	52.33
Strength IV	27.57	171.34	6.59	35.69
Extreme IIa	17.12	101.50	2.61	49.38
Extreme IIb	23.32	137.87	2.61	49.38
Service I	20.53	130.18	5.56	33.30

Eccentricity (Overturning) Check

When a shear key is required to prevent sliding, the passive resistance shall be ignored.

Maximum eccentricity limit: $e_{max} = B/3 = 3.33$ ft. AASHTO 10.6.3.3

$$e_{actual} = \frac{B}{2} - \frac{\Sigma M_V - \Sigma M_H}{\Sigma V}$$

Strength Ia: $X = (\Sigma M_V - \Sigma M_H) / \Sigma V = (128.95 - 52.33) / 19.86 = 3.86$ ft.
 $e = 10.0 / 2 - 3.86 = 1.14$ ft. $e_{actual} < e_{max}$ **OK**

Extreme IIa: $X = (\Sigma M_V - \Sigma M_H) / \Sigma V = (101.50 - 49.38) / 17.12 = 3.04$ ft.
 $e = 10.0 / 2 - 3.04 = 1.96$ ft. $e_{actual} < e_{max}$ **OK**

Bearing Resistance Check

When a shear key is required to prevent sliding, the passive resistance shall be ignored.

Vertical stress for wall supported on soil: $\sigma_v = \frac{\Sigma V}{B - 2e}$ AASHTO 11.6.3.2-1

Nominal soil bearing resistance $q_n = 7.50$ ksf
 Factored bearing resistance $q_R = \phi_b q_n = 4.13$ ksf
 $q_{R_EE} = \phi_{EE} q_n = 7.50$ ksf Extreme event

Strength Ib: $X = (\Sigma M_V - \Sigma M_H) / \Sigma V = (179.27 - 52.33) / 27.78 = 4.57$ ft.
 $e = B / 2 - X = 10.0 / 2 - 4.57 = 0.43$ ft.
 $\sigma_v = \Sigma V / (B-2e) = 27.78 / (10.0 - 2(0.43)) = 3.04$ ksf
 $\sigma_v < q_R$ **OK**

Strength IV: $X = (\Sigma M_V - \Sigma M_H) / \Sigma V = (171.34 - 35.69) / 27.57 = 4.92$ ft.
 $e = B / 2 - X = 10.0 / 2 - 4.92 = 0.08$ ft.
 $\sigma_v = \Sigma V / (B-2e) = 27.57 / (10.0 - 2(0.08)) = 2.80$ ksf
 $\sigma_v < q_R$ **OK**

Extreme IIb: $X = (\Sigma M_V - \Sigma M_H) / \Sigma V = (137.87 - 49.38) / 23.32 = 3.79$ ft.
 $e = B / 2 - X = 10.0 / 2 - 3.79 = 1.21$ ft.
 $\sigma_v = \Sigma V / (B-2e) = 23.32 / (10.0 - 2(1.21)) = 3.08$ ksf
 $\sigma_v < q_{R_EE}$ **OK**

Sliding Check

AASHTO 10.6.3.4

Per AASHTO 11.6.3.5, passive soil pressure shall be neglected.

Strength Ia and Extreme IIa:

Maximum total Horizontal force $\Sigma H = 8.63$ kip / ft.
 Maximum total Vertical force $\Sigma V = 19.86$ kip / ft.
 Nominal passive resistance $R_{ep} = 0.00$ kip / ft. AASHTO 11.6.3.5
 For concrete cast against soil $C = 1.00$ AASHTO EQ 10.6.3.4-2
 Nominal soil sliding coefficient $\mu_n = \tan \phi_{Sub} = 0.360$
 Nominal sliding resistance $R_\tau = C \Sigma V \mu_n = 1.0 (19.86) (0.360) = 7.15$ kip / ft.

Factored resistance against failure by sliding

$$R_R = \phi R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep} = 1.00 (7.15) + 0.50 (0.0) = 7.15 \text{ kip / ft.}$$

$$R_R < \Sigma H \quad \text{Shear Key is Required}$$

Shear Key Design

1. Assume shear key dimensions.
2. Center line of the shear key is approximately B/3 from the heel edge of the footing; see BDM Section 11.5.1.
3. Passive soil pressure at the toe shall be neglected; only include passive pressure due to the inert block (c) (see AASHTO 11.6.3.5).
4. Depth of inert block is taken to be the sum of the key depth and the effective wedge depth. This example follows this methodology. Conservatively, effective wedge depth can be ignored, allowing inert block to be equal to shear key depth.
5. Per BDM Section 11.5.1, the top 1 ft. of fill at the toe shall be ignored for all design cases.
6. The Designer may choose to add weight of the shear key for eccentricity and bearing analysis once shear key dimensions are confirmed. For this example, weight of the key is ignored.

Shear key depth	$d_{Key} =$	1.00	ft.
Shear key width	$T_{Key} =$	1.50	ft.
Heel of footing to centerline shear key	$K =$	3.50	ft.
Toe of footing to front face of shear key	$X_{Key} =$	5.75	ft.
Soil cover above the footing toe	$H_{TF} =$	2.00	ft.
Shear friction angle of subgrade	$\delta_{sub} = 2/3\phi_{sub} =$	13.33	deg.
Inert block depth	$c = d_{Key} + X_{Key} \tan(\delta_{sub}) =$	2.36	ft.
Top of fill to top of shear key	$y_1 =$	2.25	ft.
Top of fill to bottom of inert block	$y_2 =$	4.61	ft.

Passive equivalent pressure	EFW (p) =	0.988	kcf
Nominal soil sliding coefficient	$\mu_n =$	0.360	
Coefficients of friction (factored):	$\mu_u = \phi_T \mu_n =$	1.00 (0.360) =	0.360 (concrete-soil)
	$\mu_{u\ s-s} = \phi_{T\ s-s} \mu_n =$	1.00 (0.360) =	0.360 (soil-soil)
	$\mu_{u\ EE} = \phi_{EE} \mu_n =$	1.00 (0.360) =	0.360 (extreme event)

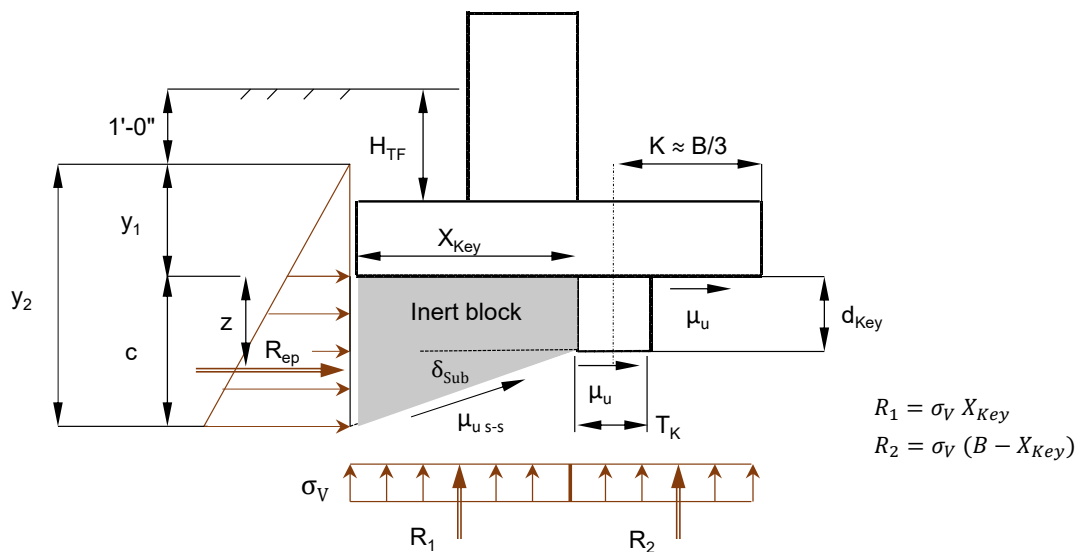


Figure 2 - Shear Key

Shear resistance between soil and foundation: $\phi_{\tau} R_{\tau} = C R_1 \mu_{u s-s} \cos \delta_{Sub} + C R_2 \mu_u$ (Strength Ia)
 $\phi_{EE} R_{\tau} = C R_1 \mu_{u EE} \cos \delta_{Sub} + C R_2 \mu_{u EE}$ (Extreme IIa)

$$X = (\Sigma M_V - \Sigma M_H) / \Sigma V \qquad e = \frac{B}{2} - X \qquad \sigma_v = \frac{\Sigma V}{B - 2e}$$

Load Combination	ΣV (kip/ft.)	ΣMV (kip-ft./ft.)	ΣMH (kip-ft./ft.)	X (ft.)	e (ft.)	σ_v (ksf)	R1 (kip/ft.)	R2 (kip/ft.)	ϕR_{τ} (kip/ft.)
Strength Ia	19.86	128.95	52.33	3.86	1.14	2.57	11.42	8.44	7.04
Extreme IIa	17.12	101.50	49.38	3.04	1.96	2.82	9.84	7.28	6.07

Passive resistance of soil available throughout the design life of structure:

$$R_{ep} = EFW(p)0.5 (y_1 + y_2) c = 0.988 * 0.5 (2.25 + 4.61) 2.36 = 8.00 \text{ kip}$$

Factored resistance against failure by sliding:

AASHTO 10.6.3.4

Strength Ia: Maximum total Horizontal force $\Sigma H = 8.63 \text{ kip}$
 $R_R = \phi R_n = \phi_{\tau} R_{\tau} + \phi_{ep} R_{ep} = 7.04 + 0.50 (8.00) = 11.04 \text{ kip}$
 $R_R > \Sigma H \quad \text{OK}$

Extreme IIa: Maximum total Horizontal force $\Sigma H = 2.61 \text{ kip}$
 $R_R = \phi R_n = \phi_{EE} R_{\tau} + \phi_{ep} R_{ep} = 6.07 + 0.50 (8.00) = 10.07 \text{ kip}$
 $R_R > \Sigma H \quad \text{OK}$

2. STRENGTH DESIGN

Concrete compressive strength	$f_c =$	4.50	ksi	
Yield strength of the reinforcement	$f_y =$	60.00	ksi	
Concrete unit weight	$\gamma_c =$	0.150	kcf	
Correction factor for source aggregate	$K_1 =$	1.00		AASHTO 5.4.2.4
Modulus of elasticity of reinforcement	$E_s =$	29000	ksi	AASHTO 5.4.3.2
Modulus of elasticity of concrete	$E_c = 120,000 K_1 \gamma_c^2 f_c'^{0.33} =$	4435.31	ksi	AASHTO 5.4.2.4
Modular ratio	$n = E_s / E_c =$	6.54		AASHTO 5.6.1
Compression zone factor	$\beta_1 = 0.85 - (f'c - 4.0)0.05 =$	0.825		AASHTO 5.6.2.2
Resistance factor for flexural-tension control	$\phi_f =$	0.90		AASHTO 5.5.4.2
Resistance factor for shear-tension control	$\phi_v =$	0.90		AASHTO 5.5.4.2
Design width	$b =$	12.00	in.	

2.1 STEM WALL DESIGN

Summary of Unfactored Horizontal Loads and Moments at the Bottom of the Stem:

Load Type	Description	H (kip/ft.)	Moment Arm (ft.)	MH (kip-ft./ft.)
EH _H	Soil	3.74	5.00	18.70
LS _H	Surcharge	1.08	7.50	8.10

Summary of Load Groups:

Load Combination	Horizontal Load & Moment	
	Vu (kip/ft.)	Mu (kip-ft./ft.)
Strength I _b	7.50	42.23
Service I	4.82	26.80

It has been assumed that the load combination Strength I_b generates the maximum moment at the interface of the stem wall and footing. However, the Designer should check all possible load combinations, including extreme event, and select the combination that produces the maximum load for the design of the stem.

Note: The Designer/Engineer is encouraged to use engineering judgment to determine the moment and required area of reinforcing steel at other points of the stem for tall walls (H ≥ 10.0') to reduce the amount of steel required at higher elevations.

2.1.1 Flexure Design

AASHTO 5.6.3.2

Design of vertical reinforcement bars at back face of stem

Assumed bar size	Bar =	# 5	
Factored applied moment	M _{u Str} =	42.23	kip-ft. / ft.
Concrete clear cover	r =	2.00	in.
Bar diameter	d _b =	0.625	in.
Bar area	A _b =	0.310	in ²
Effective Depth	d _e = T _{Bot} - r - d _b / 2 =	1.75' (12) - 2" - 0.625" / 2 =	18.69 in.

Try # 5 @ 6.0" on center:

Design steel area	A _s = A _b b / spa =	0.310 (12) / 6 =	0.620 in ² /ft.
Distance from compression fiber to neutral axis	C _b = $\frac{A_s f_y}{\beta_1 0.85 f_c' b}$ =	0.620 (60) / (0.825 * 0.85 * 4.5 * 12) =	0.982 in.
Equivalent Stress Block	a = β ₁ C _b =	0.825 (0.982) =	0.810 in.
Nominal Flexural Resistance	M _n = A _s f _y (d _e - $\frac{a}{2}$) =	0.620 (60) (18.69 - 0.810 / 2) =	56.68 kip-ft.
Factored Flexural Resistance	M _R = φ _f M _n =	0.90 (56.68) =	51.01 kip-ft.
		M _R >	M _{u Str} OK

Maximum Reinforcement: Provision deleted in 2005

Minimum Reinforcement:

AASHTO 5.6.3.3

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_R, at least equal to the lesser of 1.33M_{u Str} or M_{cr}

Member width	b =	12.00	in.
Member depth	d = T _{Bot} =	21.00	in.
Distance to Neutral Axis	y _t = T _{Bot} / 2 =	10.50	in.
Stem moment of inertia	I _g = b d ³ / 12 =	9261.0	in ⁴
Section modulus	S _{nc} = S _c = I _g / y _t =	882.0	in ³

Concrete Modulus of Rupture	$f_r = 0.24\sqrt{f'_c} = 0.509$	ksi	AASHTO 5.4.2.6
Cracking moment,	$M_{cr} = y_3[(y_1 f_r + y_2 f_{cpe})S_c - M_{dnc}(Y_c/Y_{nc} - 1)]$:	AASHTO 5.6.3.3-1
Flexural cracking variability factor	$y_1 = 1.600$		
Prestress variable factor	$y_2 = 0.000$		
Ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement	$y_3 = 0.670$	for A615, Grade 60 steel	
Compressive stress due to prestress force	$f_{cpe} = 0.000$	ksi	
Total unfactored dead load moment	$M_{dnc} = 0.000$	kip-in.	
Cracking moment,	$M_{cr} = 0.670 [(1.60 * 0.509 + 0) * 882.0 - 0] / 12 = 40.11$	kip-ft./ft.	- controls
Factored applied moment *1.33	$1.33 M_{u\ str} = 56.17$	kip-ft./ft.	
Factored flexural resistance	$M_R = 51.01$	kip-ft./ft.	
	$M_R > \min(M_{cr}, 1.33M_{u\ str})$		OK

Control of cracking by distribution of reinforcement: AASHTO 5.6.7

Exposure condition class	2	Use Class 2 for the stem, Class 1 for the footing and key	
Exposure factor	$\gamma_e = 0.75$		
Thickness of concrete cover	$d_c = 2" + d_b / 2 = 2" + 0.625 / 2 = 2.31$	in.	
Reinforcement Ratio	$\rho = A_s / b d_e = 0.620 / (12 * 18.69) = 0.003$		
Modular ratio	$n = 6.54$		
	$k = \sqrt{2n\rho + (n\rho)^2} - n\rho = 0.179$		
	$j = 1 - k/3 = 0.940$		
Service applied moment	$M_{u\ serv} = 26.80$	kip-ft.	
Tensile stress in steel	$f_{ss} = M_{u\ serv} * 12 / (A_s j d_e) = 26.80(12) / (0.620 * 0.940 * 18.69) = 29.52$	ksi	
	$\beta_s = 1 + \frac{d_c}{0.7(T_{Bot} - d_c)} = 1 + 2.31 / 0.7 (1.75 * 12 - 2.31) = 1.18$		
Maximum spacing	$s_{max} = \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = 700(0.75) / (1.18 * 29.52) - 2(2.31) = 10.45$	in.	
Spacing provided	$s_{prov} = 6.00$	in.	
	$s_{prov} < s_{max}$		OK

2.1.2 Shear Design AASHTO 5.7.3.3

Shear typically does not govern the design of retaining walls. If shear becomes an issue, the thickness of the stem should be increased. Ignore benefits of the shear key (if applicable) and axial compression.

Factored shear load	$V_{u\ str} = 7.50$	kip/ft.	
Effective Depth	$d_v = \max(d_e - C_b/2, 0.9 d_e, 0.72 T_{Bot}) = 18.20$	in (shear)	AASHTO 5.7.2.8

Per AASHTO 5.7.3.4.1, this section does not qualify for simplified procedure for determining shear resistance parameters. General procedure will be used (AASHTO 5.7.3.4.2).

Longitudinal tensile strain in the section
$$\epsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po} \right)}{E_s A_s + E_p A_{ps}}$$

Removing all prestress steel unknowns, the equation will be as follows:
$$\epsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u| \right)}{E A}$$

$$E_s A_s$$

Where,

Factored moment	$M_u = \max (M_{u \text{ str}}, V_{u \text{ str}} * d_v) =$	42.23	kip-ft./ft.
Factored axial force	$N_u = 1.25 (DC_1 + DC_2 + DC_4) =$	-5.19	kip
Area of steel on the flexural tension side	$A_s =$	0.620	in ² / ft.
Modulus of elasticity of reinforcement	$E_s =$	29,000	ksi
Longitudinal tensile strain in the section	$\epsilon_s =$	0.00182	in / in

Parameter β for sections with no transverse reinforcement
$$\beta = \frac{4.8}{(1 + 750\epsilon_s)} \frac{51}{(39 + s_{xe})}$$

Where,

Crack spacing parameter (1)
$$s_x = \min \left\{ \begin{array}{l} d_v = 18.20 \text{ in} \\ s = 12.00 \text{ in (see below - \#4 @ 12")} \\ \text{if } A_{s_layer} \geq 0.003b_e s_x = 0.67 \text{ in}^2 \end{array} \right.$$

$$s_x = 18.20 \text{ in}$$

Crack spacing parameter (2)
$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} = 18.20 \text{ in} \quad (12.0 \text{ in} \leq s_{ex} \leq 80.0 \text{ in})$$

Where, max aggregate size
$$a_g = 0.75 \text{ in}$$

Shear resistance parameter
$$\beta = \frac{4.8}{(1 + 750\epsilon_s)} \frac{51}{(39 + s_{xe})} = 1.81 \quad \text{AASHTO 5.7.3.4.2}$$

Concrete density modification factor
$$\lambda = 1.00 \quad \text{AASHTO 5.4.2.8}$$

Nominal Shear Resistance
$$V_c = 0.0316\beta\lambda\sqrt{f'_c}bd_v = 0.0316(2)(1)\sqrt{4.50}(12)(18.20) = 26.50 \text{ kip}$$

Factored Shear Resistance
$$V_R = \phi_v V_c = 0.90(26.50) = 23.85 \text{ kip}$$

Retaining wall footings and stems are typically unreinforced for shear. Confirm

transverse reinforcement is not required by design,
$$0.5 V_R > V_{u \text{ str}} \quad \text{AASHTO 5.7.2.3}$$

$$0.5 V_R = 11.93 \text{ kip}$$

$$0.5 V_R > V_{u \text{ str}} \quad \text{OK}$$

2.1.3 Shrinkage and Temperature Reinforcement Design

AASHTO 5.10.6

Horizontal reinforcement at each face of stem and vertical reinforcement at front face of stem

Try <u># 4 @ 12.0" on center:</u>	Design steel area	$A_s = 0.200 \text{ in}^2$	
	Check	$A_s \geq \frac{1.30 b T_{Bot}}{2(b + T_{Bot})f_y} =$	0.083 in ² OK
	Check	$0.11 \leq A_s \leq 0.60$	OK

2.2 FOOTING HEEL DESIGN

The critical section for shear and moment is at the back face of the stem wall (C5.13.3.6). The heel is designed to carry its self weight and the soil block above it. Conservatively, it is common to ignore upward soil reaction under the footing heel, thus Strength 1b is not checked. For shear in footings, the provisions of 5.8.2.4 are not applicable, thus $\phi V_c \geq V_u$.

Summary of Unfactored Vertical Loads and Moments at the Back Face of the Stem:

Load Type	Description	V (kip/ft.)	Moment Arm (ft.)	M (kip-ft.)/ft.
DC	Heel dead load	1.03	2.75	2.83
EV ₁	Vertical pressure from dead load of fill on heel	10.73	2.75	29.51

Summary of Load Groups:

Load Combination	Vertical Load & Moment	
	V _u (kip/ft.)	M _u (kip-ft.)/ft.
Strength IV	16.03	44.08
Service I	11.76	32.34

By inspection, load combination Strength IV generates a maximum moment at the interface of the footing heel and stem wall. However, the Designer should check all possible load combinations and select the combination that produces the maximum load for the design of the footing.

For reinforcement design, follow the procedure outlined in Section 2.1. Exposure Class I can be used for cracking check. Results of the design are as follows (also shown on Figure 3):

Transverse horizontal bar at top of footing - # 6 @ 6.0"
 Longitudinal reinforcement, top and bottom of footing - # 4 @ 12.0"

2.3 FOOTING TOE DESIGN

The critical section for shear is d_v from front face of wall stem and, for moment, is at the front face of wall stem (C5.13.3.6). Section is designed to resist bearing stress acting on toe. This example conservatively ignores the soil on top of the toe. For shear in footings, the provisions of 5.8.2.4 are not applicable, thus $\phi V_c \geq V_u$.

Controlling loads:

Maximum bearing stress (factored) $\sigma_v = 3.08$ ksf (from bearing resistance check)
 Factored shear $V_{u\ str} = \sigma_v S = 8.47$ kip/ft.
 Factored bending moment $M_{u\ str} = V_u S/2 = 11.65$ kip-ft./ft.

Service loads:

$X = (\Sigma M_v - \Sigma M_H) / \Sigma V = (130.18 - 33.30) / 20.53 = 4.72$ ft.
 $e = B / 2 - X = 10.0 / 2 - 4.72 = 0.28$ ft.
 $\sigma_v = \Sigma V / (B - 2e) = 20.53 / (10.0 - 2(0.28)) = 2.17$ ksf
 Factored shear $V_{u\ serv} = \sigma_v S = 5.97$ kip/ft.
 Factored bending moment $M_{u\ serv} = V_u S/2 = 8.21$ kip-ft./ft.

For reinforcement design, follow the procedure outlined in Section 2.1. Results of the design are as follows (also shown on Figure 3):

Transverse horizontal bar at bottom of toe - # 5 @ 6.0"

Note: Check that the toe length and footing depth can accommodate development length of the hooked bar past the design plane.

2.4 SHEAR KEY DESIGN

The critical section for shear and moment is at the interface with the bottom of the footing. Shear key reinforcing is designed to resist passive pressure determined in the sliding analysis. Passive pressure load resultant is located at

distance "z" from the bottom of footing, if using inclined wedge (see Figure 2).

Passive pressure against inert block $R_{ep} = 8.00$ kip

$$\text{Moment arm } z = (0.5K_p\gamma_s y_1 c^2 + 0.333K_p\gamma_s c^3) / R_{ep} =$$

$$= [0.5 (7.60)(0.130)(2.25)^2 + 0.333 (7.60)(0.130)(2.36)^3] / 8.00 = 1.31 \text{ ft.}$$

Factored bending moment for key design $M_{u \text{ str}} = 10.48$ kip-ft./ft.

For reinforcement design, follow the procedure outlined in Section 2.1. Results of the design are as follows (also shown on Figure 3):

Vertical 'U' bars at front and back face of shear key -

4 @ 6.0"

Longitudinal reinforcement in shear key -

4 @ 12.0"

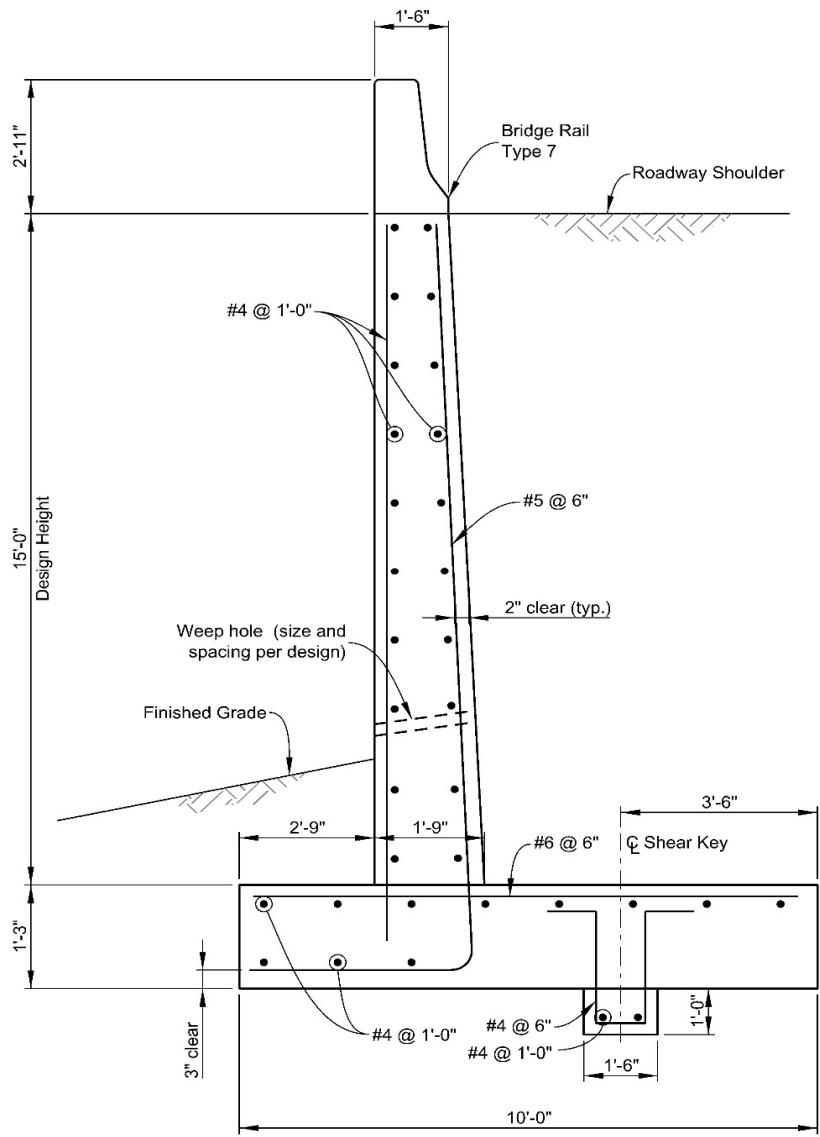


Figure 3 - Final Wall Section

EXAMPLE 12: RAIL ANCHOR SLAB DESIGN

General Information

Rail anchor slabs have been used with good performance under Allowable Strength Design (ASD) practices. AASHTO LRFD has since become the design standard and uses impact loads significantly larger than those in ASD. The LRFD impact loads result in a rail anchor slab that is much larger than what has proven successful in the past. NCHRP Report 663 concluded that AASHTO LRFD dynamic impact loads result in an overly conservative design for rail anchor slabs. New guidelines were established and validated through finite element modeling and full scale testing. NCHRP Report 663 recommends that a static load equivalent (L_s) of 10 kip be used to design rail anchor slabs for overturning and sliding design in lieu of AASHTO LRFD impact loads from Chapter 13. The static load equivalent of 10 kip is appropriate for designing rail anchor slabs for TL-4 test levels.

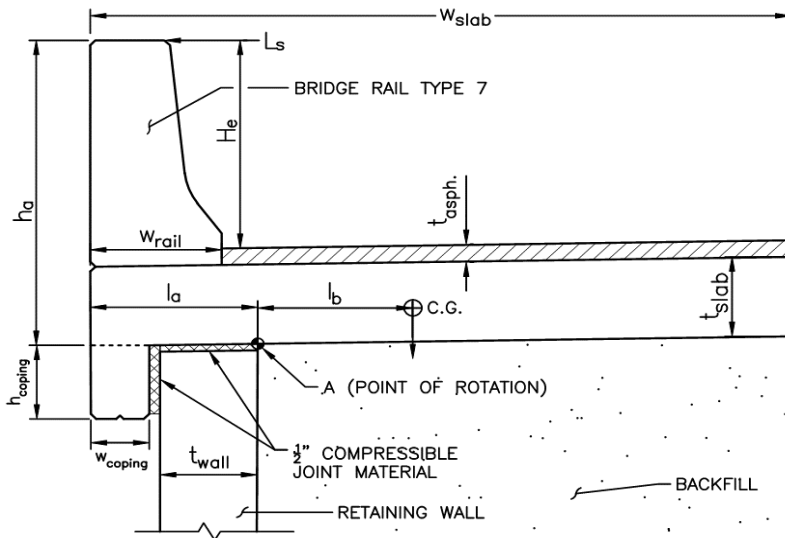


Figure 1 - Rail Anchor Slab

Rail Anchor Slab Inputs

This example illustrates the design of a rail anchor slab based on recommendations from NCHRP Report 663. Dimensions for this example are taken from CDOT Standard Sheet B-504-V1. Refer to this standard for additional details.

Concrete Unit Weight	$\gamma_{conc.} =$	0.150	kcf	CDOT BDM 3.4.4.1
Asphalt Unit Weight	$\gamma_{asph.} =$	0.147	kcf	CDOT BDM 3.4.2
Rail Anchor Slab Width	$W_{slab} =$	8	ft.	
Rail Anchor Slab Thickness	$t_{slab} =$	12	in.	
Rail Anchor Slab Length	$l_{rail} =$	30.0	ft.	(Length between expansion joints)
Asphalt Overlay Thickness	$t_{asph.} =$	3	in.	
Bridge Rail Type 7 Width	$W_{rail} =$	18	in.	
Coping Depth	$h_{coping} =$	12	in.	
Coping Width	$W_{coping} =$	8	in.	
Retaining Wall Thickness	$t_{wall} =$	12	in.	

Rail Anchor Slab Overturning

The overturning moment (M_o) caused by the impact of the vehicle shall be less than the stabilizing moment (M_n) created by the rail anchor slab dead weight. As show in Figure 1, the point of rotation, Point A, is assumed to be at the top, back face of the retaining wall and the structural backfill. In this design example, compressible joint material is placed on top of the wall to protect it, allowing the rail anchor slab to rotate before coming into contact with the wall. The maximum length of rail anchor slab assumed to resist the overturning moment is 60 ft. This limit is assumed to be the extents of rigid body behavior in rail anchor slabs, and is often governed by the spacing of expansion joints perpendicular to the CL of the roadway.

$$\phi M_n = \phi \sum(\text{DL Moments}) \geq M_u = \gamma_{CT} M_r$$

$$M_r = L_s H_a$$

NCHRP Report 663 (7-3,7-4)

Test Level		TL-4		CDOT BDM 13.3.3
Resistance Factor	$\phi =$	0.9		NCHRP Report 663 A1.4.3
Collision Load Factor	$\gamma_{CT} =$	1.0	(Extreme Event II)	AASHTO Table 3.4.1-1
Static Load Equivalent	$L_s =$	10.0	kip	NCHRP Report 663
Height of Impact Above Roadway	$H_e =$	32	in.	AASHTO A13.2-1
Dist. from B.F. Rail to 'Pt. A'	$l_a =$	1.71	ft.	
Dist. from C.G. to 'Pt. A'	$l_b =$	See table below		
Dist. from Impact Load to 'Pt. A'	$h_a =$	3.92	ft.	$(h_a = H_e + t_{asph.} + t_{slab})$
Factored Overturning Moment	$M_u =$	39.2	k-ft.	

To calculate M_n , the dead loads are tabulated and multiplied by the distance from their center of gravity to Point A (l_b). The distance between expansion joints in this example is 30 ft.

Tabulation of Dead Load Moments about Point A

$$\text{Weight} = \text{Area} * \gamma_{conc.}$$

$$\text{Moment} = \text{Weight} * l_b$$

$$\text{Total DL Moment} = \text{Moment} * l_{rail}$$

Ref. B-606-7A for rail weight and C.G. from BDM Ex. 6

	Height (ft.)	Width (ft.)	Weight (k/ft.)	l_b (ft.)	Moment (k-ft/ft.)	Total DL Moment (k-ft.)
Type 7 Bridge Rail			0.486	-1.14	-0.55	-16.6
Coping	1.00	0.67	0.10	-1.38	-0.14	-4.1
Slab	1.00	8.00	1.20	2.29	2.75	82.5
Asphalt	0.25	6.50	0.24	3.04	0.72	21.7
					$M_n =$	83.5 k-ft.
		$M_u =$	39.2 k-ft.	<	$\phi M_n =$	75.2 k-ft. OK

Rail Anchor Slab Sliding

Check the rail anchor slab for resistance to sliding (P_n) along its base. The weight of the slab and rail resists the impact load through friction between the concrete-soil interface. In the absence of project-specific soil information, the coefficient of friction is taken from AASHTO Table 3.11.5.3-1. The soil is assumed to be a silty or clayey fine to medium sand.

$$\phi_T P_n = \phi_T W_{DL} \tan \phi_f \geq P_u = \gamma_{CT} L_s$$

$$W_{DL} = \sum(\text{Weight}) l_{rail}$$

NCHRP Report 663 (7-1,7-2)

Static Load Equivalent	$L_s =$	10.0	kip	NCHRP Report 663	
Total Factored Horizontal Force	$P_u =$	10.0	kip		
Total Dead Load	$W_{DL} =$	60.7	kip		
Coefficient of Friction	$\tan \phi_f =$	0.35		AASHTO Table 3.11.5.3-1	
Sliding Resistance	$P_n =$	21.3	kip		
Sliding Resistance Factor	$\phi_T =$	0.8		AASHTO Table 10.5.5.2.2-1	
Factored Sliding Resistance	$\phi P_n =$	17.0	kip		
	$P_u =$	10.0 kip	<	$\phi P_n =$	17.0 kip OK

Rail Anchor Slab Reinforcing

The critical section of the rail anchor slab shall have sufficient flexural strength (M_n) to resist the impact load and the overhanging dead load of the structural components past Point A. NCHRP Report 663 recommends designing the slab to the appropriate impact loading from AASHTO Table 13.2-1. The structural integrity of the components was not evaluated for use with the static equivalent loads.

$$w_{s.o.} = t_{slab} l_a \gamma_{conc.} \quad e_{s.o.} = l_a / 2 \quad M_{slab} = w_{s.o.} e_{s.o.}$$

(weight due to asphalt is negligible)

Slab Overhang Weight	$w_{s.o.} =$	0.26 k / ft.
Slab Overhang Eccentricity	$e_{s.o.} =$	-0.85 ft.
Slab Overhang Moment	$M_{slab} =$	-0.22 k-ft/ ft.

The impact load is distributed over the length L_C of the barrier rail. L_C is the critical length of the yield line as calculated in Example 6.3 of this BDM. The rail anchor slab reinforcing is placed in the top of the slab and is designed in a 1 ft. strip.

Critical Yield Line Length	$L_C =$	10.74 ft.	AASHTO A13.3.1-2, Ex. 6
Impact Load	$F_t =$	54.0 kip	AASHTO A13.2-1
Dead Load Factor	$\gamma_{DC} =$	1.25	AASHTO Table 3.4.1-2
Collision Factor	$\gamma_{CT} =$	1.00	AASHTO Table 3.4.1-1

$$M_u = \gamma_{DC} (M_{slab} + M_{rail} + M_{coping}) + \gamma_{CT} F_t H_e / L_C$$

(Moment for rail and coping calculated in table)

Factored Moment	$M_u =$	14.54 k-ft./ ft.
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Design Section

$$\Phi M_n = \Phi A_s f_s (d_s - a/2) \quad \text{AASHTO 5.6.3.2.2}$$

$$a = A_s f_y / 0.85 f'_c b$$

$$c = \beta_1 / a$$

$$\beta_1 = 0.85 - 0.05 (f'_c - 4) \geq 0.65, \text{ for } f'_c > 4 \text{ ksi} \quad \text{AASHTO 5.6.3.2.2}$$

$$d = h - C_{TOP} - d_{bar} / 2 \quad \text{AASHTO 5.6.2.2}$$

$$\epsilon_s = 0.003 (d - c) / c \quad \text{AASHTO 5.6.2.1}$$

Reinforcement Strength	$f_y =$	60 ksi	
Concrete Strength	$f'_c =$	4.5 ksi	Concrete Class D
Stress Block Factor	$\beta_1 =$	0.825	
Strip Width	$b =$	12 in.	
Section Height	$h =$	12 in.	(thickness of the slab)
Top Reinforcing Cover	$C_{TOP} =$	2.5 in.	(Ref. B-504-V1)
Resistance Factor	$\Phi =$	0.9	(assume tension controlled) AASHTO 5.5.4.2.1
Depth to Reinforcing	$d =$	9.19 in.	(assume #5 bar)

Try a Reinforcing Pattern	Try	# 5 Bar @	8 " C.C.
Diameter of Reinforcing	$d_{bar} =$	0.625 in.	
Area of Steel per Strip	$A_{s-prov.} =$	0.47 in ²	

Net Tensile Strain	$\epsilon_s =$	0.034	>	0.005	OK
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$M_u =$	14.54 k-ft/ ft.	<	$\Phi M_n =$	18.59 k-ft/ ft.	OK
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USE #5 bars @ 8" C.C.

(Typically these bars are also placed in the bottom mat)

Check Minimum Reinforcement Requirement

AASHTO 5.6.3.3

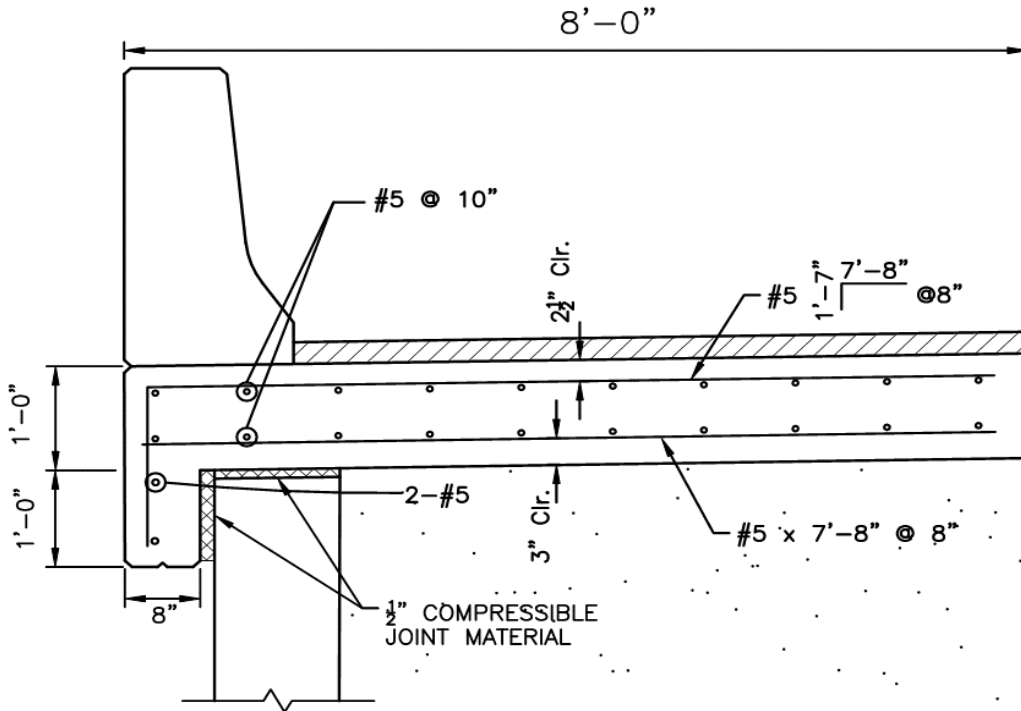
Flexural Cracking Variability Factor	$\gamma_1 =$	1.6	
Ratio f_y/f_u	$\gamma_3 =$	0.67	
Modulus of Rupture	$f_r = 0.24 (f'_c)^{0.5} =$	0.51 ksi	AASHTO 5.4.2.6
Section Modulus	$S_c = 1/6 b h^2 =$	288 in ³	

Cracking Moment	$M_{cr} = \gamma_3 \gamma_1 f_r S_c =$	13.1	k-ft./ ft.	CONTROLS min reinf. & is < ϕM_n
1.33 * Factored Moment	$1.33 M_u =$	19.3	k-ft./ ft.	OK

Note: Check Crack Control by Distribution of Reinforcement
 Note: Check Development Length of Flexural Bars
 Note: Check Temperature and Shrinkage Steel

AASHTO 5.6.7
 AASHTO 5.10.8.2.4
 AASHTO 5.10.6

Rail Anchor Slab Summary



EXAMPLE 13 - VEHICLE COLLISION ON A PIER

GENERAL INFORMATION

Example 13 illustrates pier design to provide structural resistance to withstand the vehicular collision using LEAP Bridge Substructure software. The only load case investigated in this example is Extreme Event II. Other load cases are not discussed for this example but should be investigated in the complete pier design.

References and Software Used:

- AASHTO LRFD 8th edition
- CDOT Bridge Structural Worksheets
- LEAP Bridge Concrete CONNECT Edition, Version 16.02.00.01, Substructure Module

LEAP Bridge Concrete Model Description:

The pier under design is a middle support of a two-span, 60-ft.-wide (out to out) bridge. Pier cap is 60 ft. long, 4.5 ft. wide, and 4 ft. deep, supported on three 20 ft. tall columns, spaced at 22 ft. The superstructure consists of six BT54 girders spaced at 10.5 ft. with an 8 in. deck. Columns are round, 4 ft. in diameter, supported on drilled shafts, 4.5 ft. in diameter. Refer to Figure 1 for details.

The following were also assumed in modeling the pier in the LEAP Bridge Substructure program:

- The end of the column is fixed at the top of the drilled shaft
- The drilled shaft point of fixity is located at 3x drilled shaft diameter = 13.5 ft.
- Total drilled shaft length is 5x drilled shaft diameter = 23 ft.

The Designer should use the project geotechnical information and a suitable design tool to determine the drilled shaft point of fixity and required drilled shaft total length and enter it in the LEAP Bridge Substructure program.

Material properties used (refer to BDM Section 5.3):

Pier cap concrete strength	$f'_c =$	4500.00	psi
Column concrete strength	$f'_c =$	4500.00	psi
Drilled shaft concrete strength	$f'_c =$	4000.00	psi
Concrete density	$\gamma_c =$	150.00	pcf
Steel yield strength	$f_y =$	60.00	ksi

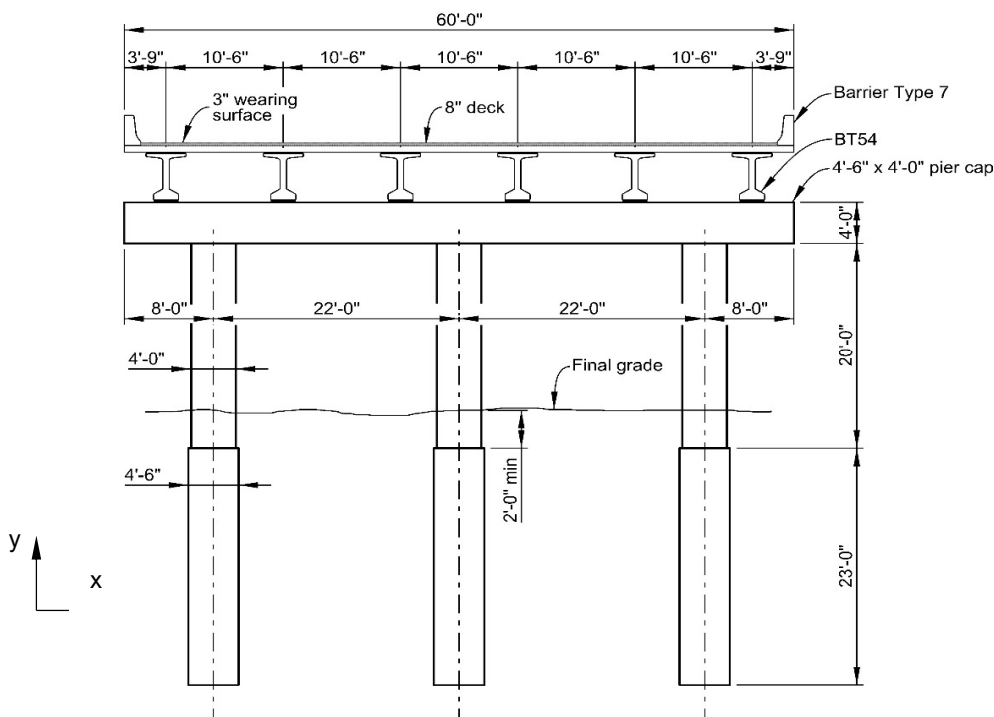


Figure 1 - Design Section

Applied Loads:

In this example, the only loads included in the analysis are collision loads and dead loads, due to the improbable coincidence of other loads (BDM Section 3.5.2). Designer may choose to include live loads, but in most cases shear from vehicle impact will control the design.

DC - dead load of structural components and nonstructural attachments

DW - dead load of wearing surfaces and utilities

CT - vehicular collision force

If the LEAP Bridge Superstructure model is available, DC and DW loads may be imported to pier model. Otherwise, they can be autogenerated as shown below.

- DC:
1. Slab and girder dead loads - autogenerated from superstructure input, $\gamma = 150$ pcf
 2. Barrier dead loads - total load per foot = 486 plf (see Structural Worksheets B-606-7B)

- DW:
1. Wearing surface total load per foot = $36.67 \text{ psf} \times (57 \text{ ft clear roadway width}) = 2090.19 \text{ plf}$ (refer to BDM Section 3.4.2)

CT: Equivalent static load = 600 kip, applied in a direction of 0 to 15 degrees, 5 ft. above the ground (AASHTO 3.6.5.1)

In the case of a multi-column pier, the Designer must investigate the collision force CT acting on each column separately and select the one with the maximum shear force. The Designer should then check the shear capacity of the column. The Designer is responsible for determining the most conservative load cases taking into account both directions of travel under the bridge and the geometry of the bridge.

Note: The critical design section for a column is at the point of impact. The Designer should add additional check points near the impact from the 'Structure Model' menu to get information needed for design.

Column length in LEAP Bridge Substructure model (includes drilled shaft length to fixity)

$$L = 1/2 \times 4 \text{ ft. cap depth} + 20 \text{ ft.} + 13.5 \text{ ft.} = 35.50 \text{ ft.}$$

Point of application of CT from drilled shaft point of fixity

$$y_1 = 5 \text{ ft.} + 2 \text{ ft. drilled shaft cover} + 13.5 \text{ ft.} = 20.50 \text{ ft.}$$

Case Number	Case Name	Col. No.	ϕ	Magnitude		Location y_1 / L
				X	Z	
Case No. 1	CT1	1	0	600.00	0.00	0.58
Case No. 2	CT2	1	15	579.56	155.29	0.58
Case No. 3	CT3	2	15	579.56	155.29	0.58
Case No. 4	CT4	3	15	579.56	155.29	0.58

Summary of CT Load Cases

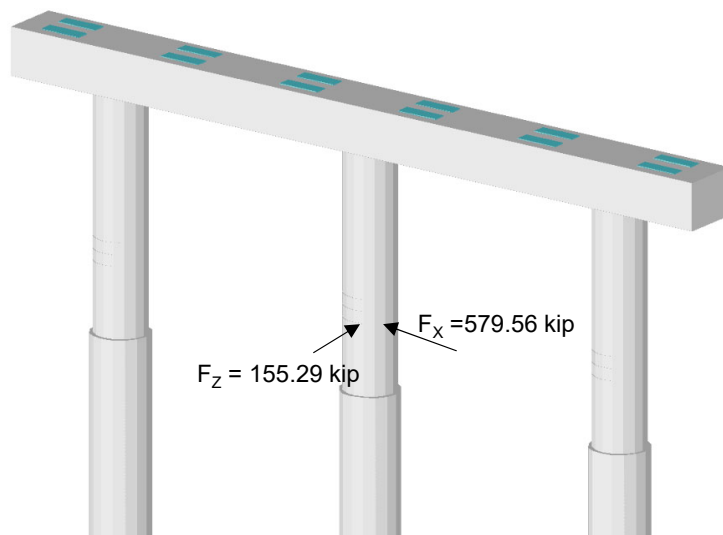


Figure 2 - CT Load Application

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Analysis of the columns is performed using the P-delta method. See below for the following outputs from LEAP Bridge Substructure:

- Summary of load combinations used in the design
- Controlling column design results
- Detailed shear design calculation for controlling column

Summary of Design:

Column reinforcement -	main rebar	22 #10 bars, equally spaced
	shear reinforcement	#4 ties @ 6"

Design of a drilled shaft in the Extreme Event collision case is similar to the design of a drilled shaft in the Strength cases and will not be shown for this example. The Designer must account for the collision load in the drilled shaft design by applying Extreme Event loads from the bottom of the column to the top of the caisson. It is recommended that the Designer use a suitable design tool to analyze shaft-soil interaction to determine stability and strength requirements.