Bridge and Roadway Structures Design Manual

Release 1

Preface

The Division of Bridges publishes the Bridge and Roadway Structures Design Manual (**BRSDM**) to provide engineering and detailing standards, criteria, and guidelines to designers and detailers who design and analyze bridges and highway-related structures for the Connecticut Department of Transportation (**CTDOT**).

The **BRSDM**, supplemented by other **CTDOT** manuals, Engineering and Construction Directives, Bulletins, and Policy Statements, is the vehicle by which the design and rehabilitation of bridges and roadway-related structures is implemented. Presented is a compilation of design, detail and plan presentation practices, specification interpretations and guidelines which constitute the Bridge Design Standard Practices of **CTDOT**.

The **BRSDM** is a replacement of the previously published **CTDOT** Bridge Design Manual (**BDM**). To better correlate information found in the **BDM** with the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications (**BDS**), the guidance previously found in the **BDM** has been reformatted and updated.

The **BRSDM** is a multi-volume series to provide guidance for project management, development, and structural design. The following volumes are presented within **BRSDM**:

- Volume 1: Project Management and Provisions
    - Part A: Project Management
    - Part B: Miscellaneous Design Provisions
- Volume 2: Structure Design Requirements
- Volume 3: Bridge Rehabilitation and Preservation

The use of the **BRSDM** is required of anyone performing design, analysis, or project administration of bridges and highway-related structures for the **CTDOT.**

While the **BRSDM** attempts to unify and clarify standard practices for design, it does not preclude justifiable deviations, subject to the concurrence of the **CTDOT**, provided the deviations are based on sound engineering principles. Good design practice will always require a combination of basic engineering principles, experience, and judgement in order to furnish the optimal design.

To reflect changes to standard practices, revisions to the **BRSDM** will be issued periodically as Engineering and Construction Directives or Bulletins. Engineering and Construction Directives and Bulletins are mandatory and supersede the current **BRSDM**. Recommendations for changes to the **BRSDM** are welcome and should be submitted in writing to the **CTDOT** via the Division Chief of Bridges or the feedback email provided on the **CTDOT** Internet Website. Note that the only recognized official version shall be the document that is provided on the **CTDOT** Internet Website.

Introduction

The design and details of all structures and structure components shall conform to the requirements set forth in the latest editions, including the interim or updated specifications, of the following publications, as modified and amended by the **BRSDM** and other **CTDOT** manuals and publications:

State of Connecticut, Department of Transportation: (CTDOT)

- Standard Specifications for Roads, Bridges, Facilities and Incidental Construction, Form (latest)
- Highway Design Manual
- Drainage Manual
- Geotechnical Engineering Manual
- Bridge Plan Notes Guide

American Association of State Highway and Transportation Officials: (AASHTO)

- AASHTO LRFD Bridge Design Specifications
- AASHTO Guide Specifications for LRFD Seismic Bridge Design
- AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete
- AASHTO LRFD Guide Specifications for Design of Concrete-Filled FRP Tubes
- AASHTO LRFD Guide Specifications for Accelerated Bridge Construction
- AASHTO LRFD Movable Highway Bridge Design Specifications
- AASHTO LRFD Road Tunnel Design and Construction Guide Specifications
- AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals
- Bridge Security Guidelines
- Guide Specifications for Bridges Vulnerable to Coastal Storms
- Guide Specifications for Design and Construction of Segmental Bridges,
- Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements
- Guide Specifications for Design of FRP Pedestrian Bridges
- Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members
- Guide Specifications for Seismic Isolation Design
- Guide Specifications for the Design of Concrete Bridge Beams Prestressed with Carbon Fiber-Reinforced Polymer (CFRP) Systems
- LRFD Guide Specifications for Accelerated Bridge Construction
- LRFD Guide Specifications for the Design of Pedestrian Bridges,
- LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals
- Technical Manual for Design and Construction of Road Tunnels - Civil Elements
- AASHTO LRFD Bridge Construction Specifications
- Guide Design Specifications for Bridge Temporary Works
- Construction Handbook for Bridge Temporary Works

- Guide Specifications for Wind Loads on Bridges During Construction
- A Policy on Design Standards - Interstate System
- A Guide to Standardized Highway Barrier Hardware
- Manual for Assessing Safety Hardware, Second Edition (2016)
- AASHTO Roadside Design Guide
- Standard Specifications for Transportation Materials and Methods of Sampling and Testing

American Railroad Engineering and Maintenance-of-Way Association (AREMA):

- Manual for Railway Engineering

American Welding Society (AWS):

- Bridge Welding Code ANSI/AASHTO/AWS D1.5
- Structural Steel Welding Code ANSI/AWS D1.1
- Structural Welding Code - Aluminum ANSI/AWS D1.2
- Structural Welding Code - Reinforcing Steel ANSI/AWS D1.4
- Guide for the Protection of Steel with Thermal Sprayed Coatings of Aluminum and Zinc and Their Alloys and Composites ANSI/AWS C2.18

American Society for Testing and Materials (ASTM):

- Annual Book of ASTM Standards

Federal Highway Administration (FHWA):

- FHWA Technical Advisory T5140.32, Uncoated Weathering Steel in Structures, dated October 3, 1989
- FHWA-IP-89-016, Design of Riprap Revetments, Hydraulic Engineering Circular No. 11 (HEC-11), March 1989
- FHWA-NHI-12-004, Stream Stability at Highway Structures, Hydraulic Engineering Circular No. 20 (HEC-20), 2012
- FHWA-HIF-12-004, Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18 (HEC-18), 2012
- FHWA-SA-92-010, Bridge Deck Drainage System, Hydraulic Engineering Circular No. 21 (HEC-21), May 1993
- FHWA-HRT-17-080, Design and Construction Guidelines for Geosynthetic Reinforced Soil Abutments and Integrated Bridge Systems, June 2018

Prestressed Concrete Institute (PCI):

- Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products MNL-116

Society for Protective Coatings (SSPC):

- Steel Structures Painting Manual, Vol. 1, Good Painting Practice
- Steel Structures Painting Manual, Vol. 2, Systems and Specifications

Designers shall become familiar with all FHWA memoranda and the Code of Federal Regulations as they pertain to the design of highway structures. Links to both can be found on the **CTDOT** State Bridge Design website.

The following is a list of abbreviated references used in the **BRSDM** for common references:

Reference	**Abbreviated Reference**
CTDOT Bridge and Roadway Structures Design Manual	BRSDM
CTDOT Standard Specifications for Roads, Bridges, Facilities and Incidental Construction	SSC
CTDOT Bridge Load Rating Manual	BLRM
CTDOT Highway Design Manual	HDM
CTDOT Drainage Manual	DRM
CTDOT Geotechnical Engineering Manual	GEM
CTDOT Digital Project Development Manual	DPDM
AASHTO LRFD Bridge Design Specifications	BDS
AASHTO LRFD Bridge Construction Specifications	CON
AASHTO LRFD Guide Specifications for Accelerated Bridge Construction	ABC
AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals	LTS
AASHTO Manual for Bridge Evaluation	MBE
AASHTO Manual for Assessing Safety Hardware	MASH
Manual for Railway Engineering	AREMA
Bridge Welding Code	AWS D1.5
Structural Steel Welding Code	AWS D1.1
Structural Welding Code – Aluminum	AWS D1.2
Structural Welding Code – Reinforcing Steel	AWS D1.4
Connecticut General Statutes	CGS

References throughout the **BRSDM** use the following syntax:

- To reference a publication, only the abbreviated reference in a bold font is used. For example, **BLRM.**

- To reference a division, section or table in a publication, the abbreviated reference in a bold font followed by a description with a numerical reference in brackets is used. For example, **BDS** [Table 3.5.1.1].

- To reference an article in a publication, the abbreviated reference in a bold font followed by a numerical reference in brackets is used. For example, **BDS** [3.6.1.2].

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Volume 1

Project Management and Provisions

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Part A

Project Management

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DESIGN CRITERIA

Bridges and other highway related structures shall be designed to meet the design criteria, requirements, and standards contained in or refenced by several **CTDOT** discipline-specific manuals. The basis of the design criteria, requirements, and standards are federal regulations, state statutes, **FHWA** policy and **CTDOT** policy, directives, preferences, and practices.

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When a deviation from controlling or non-controlling criteria appears to be warranted designers shall submit a request that explains the issue or problem, provides a proposed solution, and presents justification for the requested deviation. There are 2 different types of requests a designer can submit: a design exception or design variance. Design exceptions are required when existing or proposed design elements do not meet the controlling criteria. Design variances are required when existing or proposed design elements do not meet non-controlling criteria.

Requests for either a design exception or a design variance shall be made early in the preliminary design phase of projects.

CONTROLLING AND NON-CONTROLLING CRITERIA

Controlling and non-controlling criteria for **CTDOT** projects are described in the **HDM**. Controlling criteria includes the **FHWA** controlling criteria as well as additional **CTDOT** controlling criteria related to highway design. The **HDM** describes the requirements for applying the criteria.

Per the CFR (23 CFR 625) (eCFR :: 23 CFR Part 625 -- Design Standards for Highways) and **FHWA** policy (Revisions to the Controlling Criteria for Design and Documentation for Design Exceptions - Geometric Design - Design - Federal Highway Administration (dot.gov)), for projects on the NHS, including those involving new construction, reconstruction, resurfacing, restoration or rehabilitation, must meet the following 10 controlling design criteria: Design Speed, Lane Width, Shoulder Width, Horizontal Curve Radius, Superelevation Rate, Stopping Sight Distance, Maximum Grade, Cross Slope, Vertical Clearance, and Design Loading Structural Capacity. Stopping sight distance (SSD) applies to horizontal alignments and vertical alignments except for sag vertical curves. Of the 10 controlling criteria, only design loading structural capacity and design speed apply to all NHS facility types. The remaining eight criteria are applicable only to "high-speed" NHS roadways, defined as Interstate highways, other freeways, and roadways with a design speed greater than or equal to 50 mph.

Controlling criteria for design is generally associated with design speed and geometric requirements, both vertical and horizontal, for the roadway design based on a chosen design standard. Since vehicular bridges and other highway related structures are a part of a larger highway transportation system, their designs are also governed by the controlling criteria.

Criteria related to bridges include vertical clearance, bridge width and design loading structural capacity.

Although, no longer a **FHWA** controlling criteria, bridge width is still included as a **CTDOT** controlling criteria. For additional information on bridge width, refer to **BRSDM** [V2-2.3.3.3] and the **HDM**.

For additional information on clearances, both horizontal and vertical, refer to **BRSDM** [V2-2.3.3] and the **HDM**.

For any design exception for vertical clearance on Interstate Routes I-84, I-91, I-95 and I-395, the **CTDOT** must receive concurrence from the Military Surface Deployment and Distribution Command Transportation Engineering Agency (SDDCTEA). Refer to the **HDM** for additional information.

DESIGN LOADING STRUCTURAL CAPACITY

All vehicular bridges in projects involving new construction, reconstruction, resurfacing, restoration, rehabilitation, spot improvements, and preservation/preventative maintenance shall meet the design loading structural capacity criterion independent of the highway system, location, design classification, functional classification, or design speed of the roadway.

The requirements for the design loading structural capacity have been provided to supplement the **HDM** and clarify the application of the criterion.

Generally, projects initiated to address existing bridge deficiencies include a scope of structural work that will result in meeting the Design Loading Structural Capacity criteria. If a project does not include a scope of structural work sufficient to meet the Design Loading Structural Capacity criteria, actions must be taken to address the controlling design criteria, such as modifying the project scope of structural work or initiating a new project.

Construction work on bridges that do not meet the Design Loading Structural Capacity criteria shall be performed in a manner, in stages and sequences for example, that does not compromise the structural integrity of the bridge to perform its function while under construction. In no case shall the structural capacity of a bridge that does not meet the Design Loading Structural Capacity criteria be permanently reduced by the construction work.

Per the CFR and **FHWA**, the design loading structural capacity is a fundamental controlling criterion that applies to all vehicular bridges in projects on the NHS involving new construction, reconstruction, resurfacing, restoration, or rehabilitation. The **CTDOT** applies this criterion to all vehicular bridges independent of the highway system, location, design classification, functional classification, or design speed of the roadway.

New construction, reconstruction, resurfacing, restoration, or rehabilitation are terms used in 23 CFR 625 to describe project scopes of work. These scopes of work are described in the **HDM** along with the scopes of work for 2 other terms: spot improvements and preservation/preventative maintenance.

Bridges must satisfy the requirements of the **BRSDM** to meet the Design Loading Structural Capacity criteria. Since not all components and elements of a bridge are load rated, the **MBE** design load rating at the inventory level cannot be used to justify meeting the Design Loading Structural Capacity criteria.

Based on the findings of research and **FHWA**'s assessment and experience, the following is a brief discussion on the controlling criteria for the Design Loading Structural Capacity (Federal Register :: Revision of Thirteen Controlling Criteria for Design; Notice and Request for Comment):

> "Design Loading Structural Capacity is related to the strength and service limit state designs, not to traffic operations or the likelihood of traffic crashes. Previously called `structural capacity,' FHWA proposes to clarify that the applicable criterion covered herein relates to the design of the structure, not the load rating. Design loading structural capacity is important in maintaining a consistent minimum standard for safe load-carrying capacity and deviations from this criterion should be extremely rare. Design loading structural capacity is proposed to be retained as a controlling criterion regardless of the design speed for the project. Exceptions to design loading structural capacity on the NHS could impact the mobility of freight, emergency and military vehicles, and the traveling public and requires additional coordination with the FHWA Office of Infrastructure."

Although the CFR (23 CFR 625) allows modifications to existing bridges to be designed by the latest AASHTO Standard Specifications, the **CTDOT** has chosen to design modifications to existing bridges to meet the **BDS**.

Related to the design loading structural capacity is the non-controlling criterion, bridge load rating. Vehicular bridges shall satisfy the bridge load rating requirements of **BRSDM** [V1B-Load Ratings].

DESIGN EXCEPTIONS AND DESIGN VARIANCES

DESIGN EXCEPTIONS

Design exceptions are required when existing or proposed controlling criteria do not meet the requirements presented in **CTDOT** manuals.

For the process on obtaining a design exception, refer to the **HDM**. A meeting is required to discuss proposed design exceptions. The **HDM** provides guidance to designers on information that should be prepared and submitted for review in advance of the meeting. The **HDM** includes information on how to arrange the meeting and requirements for who should attend.

Design speed values and crash analysis for design exceptions for design loading structural capacity are not required.

DESIGN VARIANCES

Design variances are required when existing or proposed design criteria do not meet non-controlling criteria.

Consider the following examples:

Case 1:

A project includes the superstructure replacement of a grade separation bridge that carries a local road over an interstate route. The bridge will carry a lane of traffic in each direction and have no sidewalks. Per **BRSDM** [V2-2.5.2.6.3], for all highway and pedestrian bridges, the criteria for span to depth ratios is mandatory. During the preliminary design phase, the designer has discovered that the span to depth ratio cannot be met without violating the controlling criteria for vertical clearance to the roadway below or the vertical curve geometry of the roadway carried by the structure. The designer can meet the controlling criteria if a bridge design variance to the span to depth ratio design criteria is determined to be acceptable.

Case 2:

A major structure rehabilitation project includes the deck replacement of an existing bridge. During the preliminary design phase, the designer confirms that the bridge satisfies the requirements in the **BDS**. As a result, the controlling criteria for the design loading structural capacity is met. However, the designer discovers that the design load rating at the inventory level for the bridge with the new deck will be 1.14. Per **BRSDM** [V1B-Load Ratings], for a major structure rehabilitation, the minimum acceptable rating factor for the design load rating at the inventory level is 1.20. Since the rating factor is less than the minimum acceptable value, either the structure should be strengthened to increase the rating factor, or a design variance for the bridge load rating should be requested.

Variances to non-controlling structural design criteria related to bridge and other highway related structures are referred to as bridge design variances.

Prior to submitting a request for a bridge design variance, contact the **CTDOT** bridge project manager responsible for overseeing the design phase of the project for guidance and assistance in determining if a bridge design variance is necessary.

Requests for bridge design variances shall use the latest template format and include the following:

- Project number, town, location (route number, roadway name, crossing), bridge number, and a brief project description
- Project information (functional classification, design classification, AADT, % trucks, NHS or non-NHS)
- Description of substandard structural design criteria, note required criteria value, including constraints preventing the criteria from being met
- alternatives considered, include alternatives that meet the "required value" and alternatives that may not meet the "required value", supporting documents, calculations, and details
- Identify selected alternative and justification for the selection
- Submit/approval signature page

Example submit/approval signature page:

Submitted by:

_________________Date: _______
Name - Project Engineer

Recommended for Approval By:

_________________Date: _______
Name - Project Manager

_________________Date: _______
Name - Principal Engineer

Approved By:

_________________Date: _______
Name – Bridge Division Chief

Design speed values and crash analysis for bridge design variances to non-controlling structural design criteria is not required.

Requests shall be submitted by the bridge project engineer. The request will be reviewed and evaluated by the **CTDOT** bridge project manager and **CTDOT** Bridge Principal Engineer. All reviewers must recommend the request for approval for it to proceed. The **CTDOT** Bridge Division Chief shall make the final decision to approve or reject the request for a bridge design variance.

DESIGN REVIEW SUBMISSIONS

The following list outlines the design process and describes the submissions required for the design of bridges, buried structures and retaining walls. It should not be regarded as fully complete. The following items, where applicable, should be submitted in the order listed to the **CTDOT** for review and approval:

1. Environmental Review of the site,
2. Hydrologic Study,*
3. Preliminary Hydraulic Study (including any temporary facility as required),*
4. Preliminary Design Plans (30% Design Submission)
5. Scour Analysis (draft / final),*
6. Structure Type Studies or Rehabilitation Study Report,
7. Railroad Clearance Diagram,
8. Semi-Final Design plans (60% Design Submission) and Structure Geotechnical Report,
9. Final Hydraulic Study,*
10. Final Plans for Review (90% Design Submission),
11. Incorporation of Review Comments, and
12. Final Submission.

* for structures crossing a waterway

All plans for milestone submissions should be prepared in accordance with the **DPDM** and **CTDOT** Digital Design Environment Guide. All details shall be drawn to scale. Plans for individual bridges shall be self-contained sets. On large projects with multiple retaining walls or resurfacing projects with numerous bridges, these structures may be combined into one set of plans for efficiency of detailing.

All milestone submissions shall be submitted to the **CTDOT** in accordance with the **DPDM**. Specific requirements and materials, such as level symbology and seed files, are available from the **CTDOT** Digital Design Environment Guide.

HYDROLOGIC STUDY

Prior to the start of the structure design and prior to the start of a Hydraulic Study to determine the waterway opening, the design discharge shall be calculated and submitted to **CTDOT** for approval. All pertinent “backup” data shall be submitted to facilitate the review process. This work shall conform to the latest edition of the **DRM**.

PRELIMINARY HYDRAULIC STUDY

A Hydraulic Study is required if the structure requires work within the floodplain of a watercourse or stream with a watershed area exceeding one square mile. All work within the floodplain must meet the requirements of Sections 13a-94, 22a-344 and 25-68b through 25-68h of the **CGS** along with the **DRM**. If a floodway is established, every effort should be made to avoid encroachment into it. Note, certain activities, such as the construction of bridge piers within the floodway may be acceptable provided there is no increase in the “with floodway” water surface profile for the base flood or the ten-year flood. Prior to the preparation of a Structure Type Study, a preliminary Hydraulic Study must be prepared and submitted for review and approval.

SCOUR ANALYSIS

Foundations, for both new and existing structures, adjacent to or within waterways subjected to scour shall be evaluated in according with the **DRM**. The results of the evaluation, presented in a scour evaluation report, shall be submitted to the **CTDOT** for review and action. For additional information and requirements, refer to **BRSDM** [V2-2.4.4].

STRUCTURE SCOPING: STRUCTURE TYPE STUDIES

Structure Type Studies shall be prepared for each new highway, pedestrian and railway crossing. The studies should consider the safety, serviceability, maintainability, constructability, permit requirements, economics and aesthetics of the proposed structures. The studies shall be developed after careful appraisal of the site conditions, foundation conditions, hydraulic and drainage conditions, design discharge and scour potential, coordination with DEEP fisheries, rights of way, utilities, and highway limitations (including maintenance and protection of traffic and environmental impacts) both present and future. Additional studies may be requested if the **CTDOT** finds the original proposals unsuitable or inadequate.

Multiple studies done just for quantity are not desired but only those studies that show promise or feasibility within the parameters herein should be pursued. For a group of bridges in a Contract, structure type should be similar so that similarity of construction details may result in economy of costs. Repetition of a structure type merely for ease of design is to be avoided. Attention to detail in the aesthetic of the structure is to be kept foremost in mind. New materials and developments may be incorporated into the design of the proposed structure with the prior approval of the **CTDOT**.

Where the structure is required to have more than one span, the resulting multi-span structure shall be designed as jointless bridge decks to eliminate the need for deck joints.

The structure type studies shall incorporate or otherwise resolve all requirements and constraints from applicable studies, reports and analysis, including Complete Streets, developed by groups both within and outside the **CTDOT** for the crossing location.

The structure studies are to be prepared in pdf format. US Customary units of measurement shall be used in all studies. The sheets are to be numbered and each structure study is to be indexed. Construction costs and Life Cycle Cost Analysis shall be prepared for each structure type. One complete quantity and cost estimate sheet per study or structure shall be prepared. Contingencies, Minor Item Allowances and Inflation shall be included in the cost estimate for each alternative in accordance with the Programming requirements of the **CTDOT** Cost Estimating Guidelines.

The structure studies shall be submitted for review. A meeting will be held to review the structure studies and select the type of structure to be designed. Upon approval of the structure type, the Designer shall be authorized to proceed with the preparation of the Semi-Final Design Plans.

STRUCTURE SCOPING: BRIDGE REHABILITATION STUDY

A Bridge Rehabilitation Study is a documented process conducted during the Preliminary Design Phase for the purpose of determining the Final Design scope of work for bridge preservation or rehabilitation. A Bridge Rehabilitation Study shall be conducted on all projects. Elements of a Rehabilitation Study include:

- Information Collection
- LRFD Analyses
- Rehabilitation Study Report (RSR)
- Presentation
- Determination and Report of Meeting

LRFD ANALYSES

Rehabilitation sub-scope is a means of classifying work on bridge components in a simple, meaningful way. One or more sub-scopes can be included in an overall scope of rehabilitation work.

ANALYSIS NEEDS BY REHABILITATION SUB-SCOPE

Bridge components shall be evaluated by performing an analysis in accordance with **BDS** for the common bridge rehabilitation sub-scopes listed in table below. If multiple sub-scopes are selected from the table below, designers should consider analyzing the components in all the selected sub-scopes. If a sub-scope is added to the overall scope, designers shall check the table to determine if additional component(s) are recommended to be analyzed.

Table - Bridge Components to be considered in LRFD Analysis

Rehabilitation Sub-scope		Existing Bridge Components included in LRFD Analysis											
		Deck	Bridge Parapets	Girder System	Truss System	Bearings (*)	Abutments	Piers	Wingwalls/ Endwall	Foundations	Box Culverts	Pipes (Span > 6.0’)	Rigid Frame/Arches
1	Beam End Repair	-	-	✓	-	-	-	-	-	-	-	-	-
2	Deck Replacement	-	-	$	$	$	$	$	-	$	-	-	-
3	Parapet/Railing Modification	-	✓	$	$	-	-	-	-	-	-	-	-
4	Bridge Widening	✓	✓	✓	✓	✓	$	$	$	$	-	-	-
5	Deck Patching	-	-	-	-	-	-	-	-	-	-	-	-
6	Superstructure Replacement	-	-	-	-	-	✓	✓	-	✓	-	-	-
7	Superstructure Strengthening	-	-	✓	✓	✓	✓	✓	-	✓	-	-	-
8	Superstructure Preservation/Repair	-	-	$	$	-	-	-	-	-	-	-	-
9	Substructure Repair	-	-	-	-	-	✓	✓	-	-	-	-	-
10	Substructure Strengthening	-	-	-	-	-	✓	✓	✓	✓	-	-	-
11	Substructure Replacement	-	-	-	-	-	-	-	-	-	-	-	-
12	Bearing Replacement	-	-	-	-	-	-	-	-	-	-	-	-
13	Rehabilitation of Buried Structures	-	-	-	-	-	-	-	$	-	✓	✓	✓

Key:
✓ This component should be analyzed in association with the rehabilitation scope.
(*) Depending on existing bearing conditions and types.
$ Only analyze these components when there is a change in loading associated with the scope.

ANALYSIS REQUIREMENTS BY REHABILITATION SUB-SCOPE

Analysis requirements will vary depending on the sub-scope proposed for rehabilitation. Each rehabilitation sub-scope shall include one or more of the following requirements:

- Bridge load rating analysis in accordance with **BRSDM** [V1B] – Load Rating, including beam end analysis and construction loading. Construction loading includes:
    - CT-TLC
    - Construction Loads (**BRSDM** [V2-3.17CT])
    - **SSC** – 1.07.05 – Load Restrictions

A new load rating analysis is required if any of the following is true:

- Additional critical section loss exists that is not considered in the load rating that is on file (the Designer shall perform an As-Inspected load rating).
- The live load evaluated in the most current load rating on file does not meet the **BLRM** requirements.
- Additional dead load exists that was not considered in the most current load rating.

- Earthquake analysis for horizontal restraint in accordance with **BRSDM** [V2-3.10] and for beam seat length in accordance with **BDS** [4.7.4.4]. Rehabilitation sub-scopes may afford an opportunity to address earthquake force effects. Such opportunity may include installing earthquake horizontal restraint or increasing beam seat length.
- Analysis for Vehicular collision force (**BDS** [Section 13]).
- Hydraulic analysis, which may include hydrologic analysis as well.
- Scour Analysis (**BRSDM** [V2-2.6.4]):
    - Scour evaluation
    - Structural evaluation of pile-supported substructures with piles exposed due to scour.
- Analysis for all other force effects required by **BDS** for new designs as directed by the Bridge Principal Engineer. The opportunities to address these force effects are limited and will only be considered for critical bridges as identified by **CTDOT**.

1. Beam End Repair

Analysis considerations: bridge load rating.

When analyzing a beam end for the need for repair, a bridge load rating evaluation is required to determine beam end reactions. Evaluating an existing beam end is different than designing a new beam and requires analysis of different modes of failure to determine the capacity of the existing beam end. Beams that were under-designed and beams with section loss may exhibit modes of failures that are not accounted for in **BDS**. For new beam ends, **BDS** eliminates certain modes of failure from consideration by requiring the Designer to meet a minimum web Depth-to-Thickness ratio. Prevention of failure of a new bearing stiffener by local buckling is ensured by following the minimum ratio for bearing stiffener Width-to-Thickness recommended in the **BDS**. The Designer is reminded that deterioration to the web that does not extend in front of the bearing does not cause beam shear failure. A beam end analysis program, CT-BeamEnd, is available at:

https://portal.ct.gov/DOT/State-Bridge-Design/Load-Rating/Load-Rating

2. Deck Replacement
Analysis considerations: bridge load rating, earthquake, construction loading, scour analysis.

A bridge load rating evaluation is required to determine the force effects of dead and live loads from the proposed deck on the existing superstructure. Unless waived by **CTDOT**, an analysis of the substructure is also required to determine if the proposed force effects may be accommodated by the substructure with or without modification. Depending on the capacity of the superstructure and substructure to accommodate these forces, the rehabilitation scope may need to change. The proposed deck is often thicker than the existing deck and the overlay is often thicker and denser as well. The parapet also likely has a different cross-section and therefore different weight than the existing parapet. In some cases, new utilities are added to the bridge during the deck replacement because of the opportunity that it presents. The dead load force effect from the utilities shall also be considered. Corresponding force effects exerted on the substructure shall also be analyzed.

Analysis for earthquake loading shall be performed.

When a deck is to be replaced, there is an opportunity during this capital investment to evaluate the substructure for scour and propose potential scour countermeasures if required.

3. Parapet/Railing Modification
Analysis considerations: bridge load rating, vehicle collision force.

The term “railing” in **BDS** refers to traffic barrier when discussing both concrete parapet and open bridge rail. The term “modification” for the purpose of this sub-section refers to changes to bridge railing to bring it into compliance with current MASH requirements. There are two analyses of the deck-overhang and superstructure elements associated with this sub-scope:

- Vehicle impact force effect imposed.
- Increased dead load effect from modified railing loads.

The purpose of analyzing the deck-overhang and superstructure elements is to determine if these components must be strengthened or replaced in the RSR recommendations.

The analysis associated with vehicle impacts on the railing itself is discussed in conjunction with the design of the railing system and will not be discussed here.

4. Bridge Widening
Analysis considerations: bridge load rating, earthquake, scour analysis.

Bridge widening may include span bridges as well as buried structures.

a. *Span Bridges*:
Load Rating analysis of existing bridge components due to increased dead or live loads resulting from a bridge widening may be necessary as follows:

- When adding a traffic lane, the proposed widening may impose influence from a proposed traffic lane on the existing beams. Load rating of existing superstructure and substructure components shall be performed.
- The existing fascia beam may experience additional dead load from the widened superstructure and shall be analyzed.
- Bearings and substructure shall also be evaluated for additional dead and live loads.

Earthquake analysis is required for any bridge widening project due to the increased mass of the superstructure that must be restrained horizontally.

When a bridge is to be widened, there is an opportunity during this capital investment to evaluate the substructure for scour and propose potential scour countermeasures if required.

b. *Buried Structures:*
Widening of buried structures may involve additional length of structures to be constructed. No analysis of the proposed structure is required under this sub-scope.

Widening of buried structures may include the addition of fill above a portion of the existing structure. The structure shall be analyzed for the additional earth load and shall also consider any additional live load effects. Such widening may also require extension of wingwalls and headwalls to retain additional fill and possibly support live load surcharge. The analysis shall evaluate these components as well.

5. Deck Patching
Analysis considerations: none.

6. Superstructure Replacement
Analysis considerations: bridge load rating, earthquake, scour analysis, other force effects as directed.

When a superstructure can be replaced with an identical superstructure, no bridge load rating analysis is required. An earthquake analysis shall be performed to identify the need for horizontal restraint of the superstructure.

For most superstructure replacements, the dead load of the bridge is likely to increase, so a bridge load rating analysis is required to determine the dead load force effect on the substructure. This analysis shall also consider increased live load effect as well. For most superstructure replacement, a bridge load rating is the only analysis requirement, unless directed otherwise by the Department.

When a superstructure is to be replaced, there is an opportunity during this capital investment to evaluate the substructure for scour and propose potential scour countermeasures if required.

For select bridges, the substructure shall be analyzed for all the force effects required by **BDS**.

7. Superstructure Strengthening
Analysis considerations: bridge load rating.

For the purpose of this discussion, “superstructure” refers to the beam or girder system supporting the bridge deck. The term “strengthening” refers to an action that results in increased capacity of an existing member beyond its as-built capacity. This does not include repairs intended to restore as-built capacity or a portion thereof.

Superstructure strengthening sub-scope requires that the entire bridge be analyzed for additional load effects. Superstructures that are strengthened to accommodate additional load effects may impose those load effects on the bearings and substructures, which should also be analyzed.

8. Superstructure Preservation/Repair
Analysis considerations: bridge load rating, earthquake.

For each of the following superstructure preservation/repair treatments, consider the following analyses:

a. *Structural Repair*: When deterioration or other damage exists, a current structural analysis shall be used to determine if a structural repair is required. The analysis shall consider beam ends as well. See sub-scope 1 above for analysis requirements. The term “structural repair” should not be confused with “superstructure strengthening.” For this sub-scope, “structural repair” for superstructures refers to the addition of steel plates or other materials such as Ultra-High Performance Concrete (UHPC) to restore a specific capacity to a superstructure element, but not necessarily the as-built capacity.

The current analysis shall evaluate the entire load path from where the load is applied to the point of support. All possible failure modes along that load path shall be considered to determine if structural repair is necessary. It is possible that multiple failure modes exist and should be documented clearly in the project files and in the RSR.

Analysis for earthquake loading shall be performed.

b. *Preservation and Cosmetic Treatment:* The goal of preservation is to protect the superstructure from deterioration and to increase the life of the structure. The goal of cosmetic treatment is to improve appearance of a bridge component. When structural repair is not needed by analysis, the Designer shall determine if a preservation-type or cosmetic treatment is necessary or desired.

If a preservation or cosmetic treatment is specified, **no analysis is required unless the proposed changes affect the structure's load carrying capacity.**

9. Substructure Repair

Analysis considerations: bridge load rating, earthquake, other force effects as directed, scour analysis (major repair only).

a. *Concrete Substructures:*
There are different levels of repair associated with concrete substructures. There are reflected in the item names:

- Surface Repair Concrete – no analysis required.
- Structural Repair Concrete – for concrete bent structures that are heavily deteriorated, a stability evaluation may be required by the Department for horizontal forces generated by wind and earthquake. Should stability be a concern, the Designer may recommend installation of temporary bracing to stabilize the structure temporarily.

b. *Steel Substructures:*
An analysis of deteriorated steel components is required to determine if there is adequate capacity. Some possible failure modes that can occur in deteriorated steel substructure include:

- Local and global buckling in columns
- Global stability
- Yielding in compression
- Flexure: tension, compression and local and global buckling of compression elements
- Shear

If steel substructure elements are not deteriorated, no analysis is required unless requested by the Department.

For steel substructures, an earthquake analysis is required to determine if horizontal restraint and beam seat length are adequate. If significant deterioration is present, failure mechanisms may form that make the substructures unstable when lateral earthquake force is applied. In such cases the substructures shall be checked for earthquake lateral force effects.

When a major repair to concrete or steel substructures is to be performed, there is an opportunity during this capital investment to evaluate the substructure for scour and propose potential scour countermeasures if required.

10. Substructure Strengthening

Analysis considerations: bridge load rating, earthquake, other force effects as directed, scour analysis.

a. *Substructure types*

- Solid-Wall: no analysis required
    - Abutment
    - Pier
- Caps and Columns:
    - Concrete:
        - Multiple columns with at least two pier columns and one pier cap. This may include one or more cantilevered caps. Bridge load rating analysis is required when increased loads and/or moments are identified on the substructure. The Designer is reminded that this type of substructure contains both compression and beam-type elements, including cantilever beams. The Designer shall also take into consideration increased loadings on the columns due to continuity of the pier cap.
        - Single column with balanced or unbalanced hammer-head pier cap: bridge load rating analysis is required when increased loads and/or moments are identified on the substructure. The analysis shall focus on not only concentric loading of the column but shall also focus on moments in the pier column due to unbalanced loads and horizontal force effects.
    - Steel: an analysis of deteriorated steel components is required to determine if there is adequate capacity. Some possible failure modes that can occur in deteriorated steel substructure include local and global buckling in columns, global stability, yielding in compression, flexure: tension, compression and local and global buckling of compression elements, and shear.

      If steel substructure elements are not deteriorated, no analysis is required unless requested by the Department.

b. *Foundations:* geotechnical analysis shall be performed to determine if the existing foundations are adequate to support the desired loads. Foundation types include:

- Spread footing
- Pile cap

An earthquake analysis is also required to determine if horizontal restraint and beam seat length are adequate for steel substructures.

For select bridges, the substructure shall be analyzed for all the force effects required by **BDS** as directed.

When a bridge substructure is to be strengthened/replaced, there is an opportunity during this capital investment to evaluate the substructure for scour and propose potential scour countermeasures if required.

11. Substructure Replacement
Analysis considerations: the scope of analysis for substructure replacement shall be included in the scope for design of the proposed substructure.

12. Bearing Replacement
Analysis considerations: the scope of analysis for bearing replacement shall be included in the scope for design of the proposed bearings.

13. Rehabilitation of Buried Structures
Analysis considerations: bridge load rating, hydraulic analysis, scour analysis.

Buried structures include:

- Box Culverts
- Pipes with Span $\geq$ 6.0 ft.
- Rigid Frame/Arches

When a buried structure is scoped for rehabilitation, a current bridge load rating analysis performed in accordance with the **BLRM** is required to compare against the minimum acceptable rating factor in **BRSDM** [V1B] – Load Rating. All modifications of buried structures that include the addition of concrete inverts or liners are considered major structure rehabilitations. The analysis of a liner or a new invert is not included in this sub-scope of rehabilitation. Such design analysis should be included in the scope for design of the new elements.

Repairs to box culverts and pipes may include the addition of concrete inverts or liners. Both repair options may have negative consequences on hydraulic capacity. For rehabilitation of these buried structures, the rehabilitation is typically only performed after it is determined by hydraulic analysis that the repair will not adversely affect the hydraulic capacity. Repairs may cause an increased velocity of water in the structures. Therefore, a scour analysis shall also be performed to determine if roughness elements or scour countermeasures are required to be installed.

Concrete box culverts, rigid frames and arches may require surface or structural repairs to the concrete. These types of repairs typically do not require structural or hydraulic analysis. Analysis may be required if such concrete repairs must encroach into the hydraulic opening to:

- improve concrete cover
- increase thickness of concrete to improve capacity.

Scour analysis is not required for box culverts, but if the velocity of the flow is increased due to restriction of flow by repairs, erosion of the natural channel downstream of the outlet is possible and shall be investigated. Frames and arches founded on spread footings or deep foundations shall be evaluated for scour. If a valid and current scour evaluation is on file, this analysis can be waived by the Department.

RAILROAD CLEARANCE DIAGRAM

The Designer shall develop a “Railroad Clearance Diagram” and an “Approval of Railroad Clearance” form for approval by the Railroad and the **CTDOT**. The Designer shall begin coordination with the Railroad prior to RSR or Structure Type Study to determine allowable clearances for the determination of scope of work and complete the approval form by Design Approval.

See **CTDOT** State Bridge Design Publications Website for approval form template.

SEMI-FINAL DESIGN PLANS (60% DESIGN SUBMISSION)

Upon approval of the structure type studies or the Rehabilitation Study Report, and following notification authorizing the start of the final design phase, the Designer shall prepare Semi-Final Design Plans for all bridges, box culverts and retaining walls.

The Semi-Final Design plans should be prepared on full size sheets in pdf format. US Customary units of measurement shall be used in all plans. All details shall be drawn to scale. Extraneous information not relevant to the construction of the structure should not be shown on the plans. This includes miscellaneous topographic information such as trees, shrubs, signs, utility poles and other items that are detailed on the highway plans.

The Semi-Final Design plans shall contain the following:

a. Site Plan - A plan showing the location of the structure and approaches, topographical data including original and final contours, adjacent ramp and intersecting roadways and channels, if any, etc.

b. General Plan - A “Structure Plan” showing baseline stationing, controlling horizontal dimensions, span lengths, skew angle and clearances for the structure and approaches.

Projected below the “Structure Plan” should be an “Elevation” view showing the proposed structure with controlling dimensions and clearances, footing elevations, foundations, pertinent water and rock elevations, etc.

A typical cross section of the structure showing lane and shoulder arrangements, sidewalks if required, bridge railings, member spacing, slab thickness, and other pertinent details. For box culverts, this cross section shall show the number and size of the cells and type of construction, precast or cast-in-place.

The “General Plan” should also include a table of “Transportation Dimension and Weight” in accordance with **BRSDM** [V1A] – Transportation Dimensions and Weights and the “Notice to Bridge Inspectors” block in accordance with **BRSDM** [V1A] and **CTDOT Bridge Plan Notes Guide**.

c. Boring Plan(s) - Borings shall be plotted in accordance with **BRSDM** [V1A] – Boring Logs.

d. Stage Construction Plans, if applicable.

e. Pier Plan(s) - A pier “Plan” and “Elevation,” if applicable, showing the proposed pier with controlling dimensions, footing elevations, foundation, etc.

f. Additionally, architectural aspects of the structure shall be noted, on the SL/D plans, such as bridge railing, pier and abutment configuration, surface treatment, etc.

The inspection access features, if required, should be shown on the Semi-Final Design Plans. The Semi-Final Design plans shall be submitted to the Bridge Safety and Evaluation Unit for review. The Bridge Safety and Evaluation Unit review should indicate one or more of the following:

- No special inspection access features required.
- The inspection access features shown are adequate.
- Certain inspection access features shown are not required.
- The following additional inspection access features are required.

The Designer shall submit the Semi-Final Design plans, along with a copy of the Structure Geotechnical Report for review and approval. Upon approval of the Semi-Final Design plans, the Designer will be authorized to proceed with the development of the final Contract documents.

SOIL AND FOUNDATION INVESTIGATION

Subsurface exploration and testing programs shall be performed to provide pertinent and sufficient information for the design of substructures and retaining walls. The subsurface exploration and testing programs shall also provide pertinent and sufficient information for the design and construction of temporary support elements (sheet piling, cofferdams, soldier pile and lagging, etc.). The investigations shall conform to the **GEM**.

STRUCTURE GEOTECHNICAL REPORT

A Structure Geotechnical Report shall be prepared for each structure in accordance with the **GEM**. The Report shall include any information necessary for the proper design of all structural elements and components that may be influenced by subsurface conditions. The Report should include, but not be limited to, boring logs, excavation requirements, foundation recommendations, soil and rock properties and capacities, axial and lateral pile capacities, design criteria, backfill and drainage requirements, and related special provisions.

The Report shall be made entirely with US Customary units of measurement. The Report shall be submitted for review and approval. A copy of the Report shall be submitted with the Semi-Final Design Plans, Specifications and Estimate.

FINAL HYDRAULIC STUDY AND SCOUR REPORTS

Final Hydraulic Study and Scour Reports based on the selected structural type must be prepared and submitted. The Final Hydraulic Study should address any concerns presented during the Preliminary Hydraulic Study and should contain a Hydrology Section as approved by the **CTDOT** in addition to the detailed hydraulic analysis. The hydraulic and scour data should be tabulated on the plans.

FINAL PLANS FOR REVIEW (90% DESIGN SUBMISSSION)

As part of the "Final Plans for Review Submission," the Designer shall submit the following structure related items. The actual number of copies required varies and must be coordinated with the individual Project Engineer for the particular job:

- Final Plans for Review,
- Specifications,
- proposal estimates,
- Soils Report – Structure,
- Final Hydraulic Report,
- design computations,
- load rating package,
- quantity computations,
- structure costs with estimated steel weights (if applicable), and
- Final Scour Report.

The "Final Plans for Review" shall be complete. All bridge plans not prepared by the **CTDOT** shall be signed by the responsible party from the Consultant Engineer or the Municipality.

Incomplete submissions of plans, specifications or other data required for the Final Plans for Review Submission will not be accepted. The structural material submitted and the design of the same should be well coordinated with the roadway and utility plans and shall satisfy the needs of maintenance and protection of traffic.

The “Final Plans for Review” for structures incorporating special features to facilitate inspection and items requiring special attention will be submitted to the Bridge Safety & Evaluation Unit for review. They will indicate whether these features are adequate for future inspection and return the plans with comments or signify that the plans are satisfactory.

INCORPORATION OF REVIEW COMMENTS

The various submissions will be reviewed, and the review comments will be forwarded to the Designer. All comments received shall be incorporated into the design prior to the next submission or mutually resolved. Written responses to all comments are required, even if only by noting “Incorporated”.

FINAL SUBMISSION

Upon completion of the review of the “Final Plans for Review,” all plans, specifications and cost estimates that require modifications will be returned to the Designer for incorporation of the review comments.

REQUIREMENTS FOR FINAL CONTRACT DOCUMENTS

The Contract documents include the Final Plans and Specifications necessary to complete the contemplated construction work for a project.

US Customary units of measurement shall be used in all plans and specifications. All layout dimensions and elevations shall be given as decimal dimensions in feet. The following note shall be placed in the General Notes:

When dimensions are given to less than three decimal places, the omitted digits shall be assumed to be zeros

Detail dimensions (those not normally measured by the surveyors) should be given in feet and inches.

FINAL PLANS

The final plans should be prepared in accordance with the **DPDM** and **CTDOT** Digital Design Environment Guide. All details shall be drawn to scale. Plans for individual bridges shall be self-contained sets. On large projects with multiple retaining walls or resurfacing projects with numerous bridges, these structures may be combined into one set of plans for efficiency of detailing.

The Designer shall prepare final Contract plans for all structures.

Existing structures (houses, garages, storage tanks, etc.), which will be demolished before the project is constructed, shall not be indicated on the structure plans. The location of the existing foundation should be noted on the Contract drawings. Any existing drainage facilities that are in conflict with footings, retaining walls, etc. should be shown on the plans.

The use of the phrase “by others” on Contract plans is considered acceptable as long as the reference to whom the “others” are is specified within the Contract plans.

All final plans shall be submitted to the **CTDOT** in accordance with the **DPDM**. Specific requirements and materials, such as level symbology and seed files, are available from the **CTDOT** Digital Design Environment Guide.

PRESENTATION OF DRAWINGS

The following is the recommended order for the presentation of structure plans and generally follows the order of construction:

- General Plan (one or two sheets),
- Layout Plan (if required),
- Boring Logs,
- Stage Construction Plans,
- Foundation Plans,
- Abutment and Wingwall Plans,
- Pier Plans (if required),
- Framing Plans,
- Beam and Girder Details,
- Bearing Details,
- Slab Plans,
- Slab and Approach Slab Details,
- Joint Details,
- Metal Bridge Rail Detail Sheet (if required),
- Pedestrian Railing or Bicycle Railing Detail Sheet (if required),
- Protective Fence Detail Sheet (if required),
- Deck Drainage Details (Scuppers and Piping if required),
- Electrical Detail Sheet,

- Utility Sheets (if required), and
- Existing Structure Plans (if required).

BORING LOGS

The boring logs shall be in US Customary units of measurement and shall be shown on the plans. The format of the boring logs plotted on the plans shall be identical to the format of the **CTDOT**'s standard boring log forms. A list of boring log abbreviations used for describing the soil and rock, such as colors, textures, properties, and types, shall also be shown on the plans.

QUANTITIES

Quantities shall be tabulated and shown on the "Detailed Estimate Sheet" only.

TRANSPORTATION DIMENSIONS AND WEIGHTS

The maximum transportation lengths, widths and height of bridge members along with the maximum transportation weight shall be shown on the "General Plan." The following is a sample of the information required:

Member	Shipping Length	Shipping Height	Shipping Width	Shipping Weight
G-1	115 ft	9 ft	10 ft	118,000 lbs.

HIGH, LOW & FLOOD WATER ELEVATIONS

For structures over tidal waterways, the "General Plan" shall indicate the mean high water and mean low water elevations. For structures over non-tidal waterways, the plans shall indicate the watershed area, the magnitude, frequency and the water surface elevation for the design flood, as well as the normal water surface elevation.

NOTICE TO BRIDGE INSPECTOR

The Designer shall note on the General Plan any item that would require special attention, such as fracture critical members, during inspection of the structure. This information shall be contained in the "Notice to Bridge Inspectors" block as shown in **CTDOT Bridge Plan Notes Guide**.

COORDINATE TABULATION

The Designer shall tabulate coordinates on each set of bridge plans for structures on a horizontal curve. These coordinates shall be tied into the Connecticut Coordinate Grid System. Coordinates shall be tabulated for the following:

- locations of working points,
- ends of wingwalls,
- ends of slabs,
- ends of approach slabs, and
- intersections of the centerlines of bearings at the abutments and piers with:
    - construction centerlines,
    - baselines,
    - points of application of grade,
    - gutterlines, and
    - centerlines of stringers.

BEAM OR GIRDER LENGTHS

The horizontal lengths of beams or girders measured center to center of bearings along the centerline of the member shall be shown on the plans.

UTILITY LOCATIONS

All existing underground utilities, including drainage facilities, in the vicinity of the construction must be shown on the General Plan and on all foundation drawings. It is imperative that utilities adjacent to but not actually within the excavation limits also be shown since heavy equipment, pile driving, or other deep foundation work may impact them. The size, type, owner and location of the utility must be given.

SPECIFICATIONS

STANDARD SPECIFICATIONS FOR ROADS, BRIDGES, FACILITIES AND INCIDENTAL CONSTRUCTION

This is the category of specifications that is commonly referred to as the “Standard Specs.” They are the basic construction specifications that describe and define the requirements of those items of construction most commonly used in **CTDOT** construction contracts. These specifications are in the charge of the “Standing Committee on Standard Specifications,” otherwise known as the “Specifications Committee.”

Amendments, additions to, or deletions from the **SSC** are accomplished through Committee action. The need for a particular action is usually brought to the attention of the Committee by those intimately concerned with the particular specification.

These specifications undergo constant change as new methods, materials and technology become available. The vehicle for accomplishing permanent change to a standard specification is the Supplemental Specification discussed in the following section.

SUPPLEMENTAL SPECIFICATIONS

As previously stated, the Supplemental Specifications permanently add to, delete, or otherwise revise the **SSC**. Prior to publishing and disseminating these specifications, they must have been approved by the Specifications Committee and the Federal Highway Administration.

The Supplemental Specifications are issued twice a year by the Specifications Committee, containing all the current supplements and errata that have been issued since the acceptance of the last set of **SSC**.

The Supplemental Specifications date that is to be referenced in the Contract will be associated with the Final Design Plans date.

The Supplemental Specifications may be considered part of the **SSC**. When a new set of standard specifications is accepted, these are automatically incorporated. The **SSC** set with the supplements merged into it are posted on the Department's webpage.

SPECIAL PROVISIONS

In those cases where neither the standard specification nor subsequent supplemental specifications are found to be adequate, or where no specification exists, a special provision must be prepared.

The **CTDOT** has developed and maintains lists of standardized special provisions known as "Owned Special Provisions." The purpose of these special provisions is to establish uniformity in the specification of materials and construction methods, and to have a person (Subject Matter Expert) or Department Unit responsible for updates.

These "Owned Special Provisions," available on the Department's web page, shall be inserted into the Contract documents unchanged. The Designer is responsible for the correct application of the recurring special provisions to each project. Written permission from the listed owner must be obtained should a change to an "Owned Special Provision" be required.

The Department's Contract Development Section web page has guidance for the format and content of special provisions. The Designer should make sure to include any necessary materials and what quality (testing) is necessary to accomplish the specified work.

CONTRACTOR DESIGNED ITEMS

For all items requiring the Contractor to provide designs for permanent structural features, special provisions shall be included in the Contract requiring the Contractor to provide PDF copies of all design plans. These PDFs shall conform to the **CTDOT**'s standard format for structural design plans and shall be signed and sealed by a Connecticut Professional Engineer. The Designer can include a reference to **SSC** [1.05.02] and any specific design requirements in the special provision.

INSPECTION MANUALS

For movable bridges, segmental bridges and other bridges as directed by **CTDOT**, the Designer shall provide an inspection manual. The manual should contain the instructions, procedures, check lists, diagrams and details necessary to perform a complete in-depth inspection of the various members and components of the bridge. Inspection manuals shall be submitted with the final submission for review.

DESIGN CALCULATIONS

Reviewed design calculations shall be submitted in accordance with the **DPDM** at FDP. Design calculations amendments shall be promptly submitted when changes to any design condition occurs and prior to any acceptance of change. The submissions shall include the design calculations for all phases of construction and the final condition for all structures affected by the Contract. Computer program input and output files shall also be included in the submittal if used in the Design. This requirement applies to Design Calculations only, Load Rating Package submission requirements are provided in **BRSDM** [V1B – Load Ratings].

Bridge and Roadway Structures Design Manual

Release 1

Volume 1

Part B

Miscellaneous Design Provisions

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ABUTMENTS, PIERS AND WALLS

GENERAL

A foundation serves to transmit the forces acting on the abutments, piers, or walls into the ground. Foundations are classified as either shallow or deep. Spread footings are shallow foundations. Driven piles, micropiles and drilled shafts are deep foundations.

Foundation type is generally based on the anticipated (structure) loads, underlying soil conditions, scour potential, and site constraints along with the ease and cost of construction.

SPECIAL PIER REQUIREMENTS

For new bridges and bridges undergoing major structure rehabilitation, the pier caps and columns of all single column and multi-bent piers shall be evaluated and shall meet a capacity to demand ratio of 1.2.

IDENTIFICATION NUMBERS

All abutments and piers shall be identified by numbers which start with the number “one” and progress consecutively but separately in the direction of stationing of the roadway, such as, Abutment 1, Pier 1, Abutment 2, etc.

All wingwalls shall be identified by a combination of a number and a letter (alphanumerical), such as 1A or 1B. The number used must correspond to the abutment to which the wingwall is attached. Looking up station, the letter “A” indicates the wingwall is on the left and the letter “B” indicates the wingwall is on the right.

Retaining walls shall be identified by three numbers that start at 101 and progress consecutively in the direction of stationing of the roadway, such as Retaining Wall 101, Retaining Wall 102. Parallel walls along both edges of roadway beginning at the same station are to follow wingwall rules. These numbers may designate a proprietary wall, a proprietary embankment wall, a cast-in-place wall or a soil nail wall. A table shall be provided in the Contract identifying the relationship between the wall number, type and site number of the wall, and location as in the following:

RETAINING WALL NUMBER	DESCRIPTION	LOCATION
101	Embankment Wall – Site 1	Station 10+00 to 12+50
102	Retaining Wall – Site 2	Station 25+50 to 32+50
103	Cast-in-place – Site 3	Station 70+00 to 72+50
104	Retaining Wall – Site 4	Station 80+00 to 82+50

EXCAVATION

Contract items for structure excavation, unless the work is included under other items, are required for the removal of all material of whatever nature necessary for the construction of foundations for bridges, box culverts, retaining walls and other structures. The items specified in the Contract depend on the type of material removed, earth or rock, and whether or not separate payment will be made for the work related to cofferdams and dewatering.

On any project where only some of the structures and/or their components require cofferdams and some do not, a combination of structure excavation items shall be shown in the Contract. For structures and components requiring "Cofferdam and Dewatering", pay limits and limits of the cofferdam shall be clearly delineate on the contract documents.

CONSTRUCTION REQUIRING COFFERDAM AND DEWATERING

A cofferdam is a structure that retains water and soil that allows the enclosed area to be pumped out and excavated in the dry to permit construction.

At water crossings, where structures or their components are located partially or wholly in the water and the bottom of the footing is below water level, or where a considerable flow or concentration of water is present that cannot be diverted, partly or wholly, from the site, the contract shall include the following item:

ITEM NAME	PAY UNIT
Cofferdam and Dewatering	L.F.

The hydraulic design of the cofferdam should be done in accordance with the **DRM**.

The contract shall also include either one or both of the following items, as required for the type of material removed:

ITEM NAME	PAY UNIT
Structure Excavation – Earth (Excluding Cofferdam and Dewatering)	C.Y.
Structure Excavation – Rock (Excluding Cofferdam and Dewatering)	C.Y.

Where underwater (tremie) concrete may be used to seal the bottom of a cofferdam to allow dewatering, the weight of the tremie concrete, adjusted for buoyancy, shall be added to pile and foundation loads for design purposes.

CONSTRUCTION IN THE DRY

Where structures or their components are to be constructed in the dry or where water may be temporarily directed away from an excavation, eliminating the need for a cofferdam, the contract shall also include either one or both of the following items, as required for the type of material removed:

ITEM NAME	PAY UNIT
Structure Excavation – Earth (Complete)	C.Y.
Structure Excavation – Rock (Complete)	C.Y.

The temporary redirection of water or water courses, either partially or wholly, from an excavation or site, must be coordinated with hydraulic studies and DEEP, Federal and State permit submittals. At the sites where water is directed away from an excavation, the following item shall be included in the contract.

ITEM NAME	PAY UNIT
Handling Water (Site No. XX)	L.S.

BACKFILL REQUIREMENTS

Unless otherwise directed, all abutments, wingwalls and retaining walls shall be backfilled with Pervious Structure Backfill to the limits described below. Pervious Structure Backfill is a clean, granular soil.

Indicate a wedge of Pervious Structure Backfill above a slope line starting at the top of the heel and extending upward at slope of 1:1½ (rise to run) to the bottom of the subbase. In cut situations, the following note, with a leader pointing to the slope line, shall be placed in the contract:

Slope line except where undisturbed material obtrudes within this area.

The analysis of unbalanced backfill shall be in accordance with **BRSDM** [V2-3.11.9CT].

SUBSURFACE DRAINAGE

Subsurface drainage shall be accomplished with the use of weepholes extending through the wall stems, or underdrains placed along the wall stems. Subsurface drainage for proprietary retaining walls shall conform to the owned special provisions governing their design and construction.

WEEPHOLES AND BAGGED STONE

Except for structures placed on embankments, 4 inch diameter weepholes, sloped 1:8 (rise to run), shall be placed approximately 1 foot above the finished grade at the front face of the wall stem. For structures placed on embankments, the weepholes shall be extended through the slope with an outlet. Weepholes shall not drain onto adjacent sidewalks. Weepholes should be spaced at approximately 8 to 10 foot intervals unless conditions warrant a closer spacing. The spacing and invert elevations of the weepholes shall be shown in an elevation view.

The cost of furnishing and installing weepholes is included in the cost of the concrete. Bagged Stone is paid under “Pervious Structure Backfill.”

UNDERDRAINS AND OUTLETS

Underdrains shall have a 6 inch nominal diameter, perforated and placed at the base of the stem and sloped a minimum of 1%. Underdrains shall be either connected to the roadway drainage or to a free outlet. The location and limits of the underdrain shall be shown in plan view. The invert elevations shall be shown in an elevation view. Outlets for underdrains shall consist of pipe laid in a trench and refilled with earth. The size and type of outlet pipe shall be the same as that of the underdrain to which it is connected, except that it shall not be pervious to water.

Underdrains shall be paid for under the item “6 inch Structure Underdrain.” Outlets shall be paid under “6 inch Outlets for Underdrain.” When an underdrain is connected to the roadway drainage, the pipe beyond the face of the wall stem or the end of the wall shall be shown in the contract to be included in the roadway items, and should be coordinated with the roadway designer.

SUBSURFACE DRAINAGE SELECTION CRITERIA

FULL HEIGHT ABUTMENTS

At abutments in cut situations, either an underdrain or weepholes may be used with the latter being preferred. Weepholes should be used at abutments located on fills. When there is a sidewalk in front of any abutment, an underdrain should be used. Where this type of abutment is used at water crossings, drainage shall be provided by weepholes.

PERCHED ABUTMENTS

At abutments in wet cuts, an underdrain should be used. At abutments in dry cuts and fills, extended weepholes should be used. If the total length of the extended weepholes exceeds what’s required for underdrains, the latter is preferred.

For walls with a fully exposed face adjacent to a sidewalk, an underdrain should be used. For walls with a fully exposed face not adjacent to a sidewalk, weepholes should be used. For walls with a partially exposed face, an underdrain should be used.

SLOPE PROTECTION

Provisions shall be made for protection of earth slopes in front of abutments on bridges over State highways, local roads, railroads and waterways. The slope of the embankment in front of the abutment shall be no steeper than 1:2 (rise to run).

SELECTION CRITERIA

The type of slope protection shall generally conform to the following criteria:

- Crushed Stone for Slope Protection shall be used under structures overpassing Interstate highways, railroads and waterways. Protection between the edge of the shoulder and the toe of the slope should be founded on a 6 inch granular fill base or geotextile. The limits of this base should be shown in the contract and shall be include in the estimated structure quantities.
- Concrete Block Slope Protection shall be used under structures overpassing State highways and local roads. The block shall be anchored or mortared into place to prevent vandalism. The use of granite block is not permitted due to its higher cost. Cast-in-place concrete is not permitted due to cracking and settlement of existing installations.
- Abutment slope protection for bridges over waterways should be designed in accordance with the procedures outlined in **HEC-18** or successor documents as well as documents referenced therein.

LIMITS OF SLOPE PROTECTION

The limits of slope protection shall cover the complete area, exclusive of sidewalks, from the edge of the shoulder to the face of the abutment stem and transversely within lines parallel to and 2 feet outside of the bridge rails.

INSPECTION SHELF

Provisions for inspection access (for bridge inspectors) shall be provided on all slopes. On stems with exposed heights less than or equal to 5 feet, access shall be provided by a shelf at the top of the slopes. On stems with exposed heights greater than 5 feet, access may be by a shelf at the top of the slopes or ladder stops on the slope itself. The contract shall include details of the intersection of the shelf and the slope along the wingwalls.

SURFACE TREATMENTS

In general, abutments, piers and walls shall be faced with standard formed concrete. Surface treatments other than standard formed concrete should only be considered in the following situations:

- When the structure has been determined by the **CTDOT** to be architecturally or historically significant.
- If there is a desire expressed for special surface treatments during the public involvement process of the project. The basis for surface treatments should involve the character of the area in which the wall is to be built. The use of special surface treatments should be coordinated with the town or city administration.
- Where the structure is to be built on a designated State scenic highway.
- The structure is part of the Merritt Parkway. For these structures, and in accordance with the Merritt Parkway Bridge Restoration Guide, every attempt should be made to replicate the appearance and structure type that was originally built.

- The treatment of the structure is part of a right-of-way settlement with a property owner. For instance, if a property has an existing stone wall that is to be removed and relocated, the owner may request that the replacement wall also have a stone surface.

If special surface treatments are desired for a particular structure, every attempt should be made to achieve architectural aesthetics by means of shape and form, not through surface treatments alone. Surface treatments should generally be used in conjunction with the shape of the structure.

FORM LINERS

When the use of surface treatments has been determined to be appropriate, the preferred method is the use of concrete form liners. Form liners offer a lower cost alternative to stone veneer. There is a wide variety of form liners available for different architectural treatments. Linear corrugated form liners should be avoided since it is difficult to hide joint lines and form tie holes. Form liners that replicate stone are preferred since the random nature of the surface makes it easy to hide form tie holes.

SIMULATED STONE MASONRY

In more sensitive areas, where the look of real stone is required, the use of simulated stone masonry may be considered. Simulated stone masonry utilizes a flexible form liner system and color stains or dry-colorant admixtures to provide the aesthetic appeal of natural stone with the durability of reinforced concrete.

STONE VENEER

Stone veneer is not recommended for use in lieu of form-liners due to maintenance concerns. The use of stone veneer on concrete should only be considered in very sensitive areas where the increased cost and maintenance can be justified. Stone veneer shall only be used with approval from the **CTDOT**.

ARCHITECTURAL TREATMENTS

If the appearance of stone is desired, architectural form liners should be used. These liners are significantly less costly than stone veneer. Several of the proprietary retaining walls can be built with form liners resembling stone. The Designer should contact the approved wall manufacturers for specifics about available form liners. Every effort should be made to keep the surface treatment similar for all the wall types specified.

REQUIREMENTS FOR WALLS

The following is a list of appropriate retaining wall types that may be considered:

1. <u>Non-Proprietary:</u> Precast and Cast-In-Place Reinforced Concrete
2. <u>Proprietary:</u> Prefabricated Modular Wall Systems
Mechanically Stabilized Earth Walls (precast concrete)

Mechanically Stabilized Earth Embankment Walls (dry-cast block)

A design is required for the non-proprietary walls only. The Contractor shall be responsible for the structural/internal design of the proprietary walls. For projects where proprietary retaining walls are included, the walls will be bid as a lump sum for each site. The Designer shall clearly define the horizontal, vertical, and transverse pay limits in the contract.

The **CTDOT** maintains a list of approved proprietary retaining walls for each category listed above. No other proprietary retaining walls will be allowed.

WALL SELECTION CRITERIA

The Designer shall select the appropriate retaining walls for each site. The Designer may need to contact wall manufacturers to ensure that each wall will be suitable at each site, and fit within the available right-of-way. The following general criteria should be followed for the selection of appropriate retaining walls:

WALLS < 8 FEET (MEASURED FROM FRONT GRADE TO BACK GRADE)

EMBANKMENT WALLS

It is not necessary to design a cast-in-place retaining wall as an alternate; however, the Designer shall lay out the embankment wall in the contract with at least the following information:

- Retaining wall plan view with all required dimensions, contours, property lines, utilities, etc.
- Retaining wall elevation view showing top and bottom elevations, approximate step locations, existing and finished grade, etc. Where required, the Designer shall also show the location of railings or fences required to be attached to the top of the wall.
- Typical sections (schematic) of the wall showing pay limits and minimum drainage requirements.
- Borings and soils information including the maximum factored bearing resistance.
- Temporary Sheeting required for excavation.

CAST-IN-PLACE WALLS

For locations where embankment walls are not appropriate (in accordance with the criteria listed above), a cast-in-place wall should be designed and detailed.
At the discretion of the **CTDOT**, proprietary walls may also be allowed if the wall is very long resulting in a large overall area. The Designer shall provide the same information for proprietary walls as required in the Section “Walls < Than 5,000 ft^2 of Vertical Face Area (Measured to Bottom of Footing).”

If the appearance of stone is desired, architectural form liners should be used. These liners are significantly less costly than stone veneer. If there are multiple walls on a project, the surface treatment shall be similar for each wall.

WALLS > 8 FEET (MEASURED FROM FRONT SLOPE TO BACK SLOPE)

WALLS < THAN 5,000 FT^2 OF VERTICAL FACE AREA (MEASURED TO BOTTOM OF FOOTING)

For this situation, a cast-in-place wall should be designed to be bid against the proprietary walls. The Contractor may be able to build the cast-in-place wall with their own forces at a lower cost. For these situations, the Designer shall completely design and detail the cast-in- place wall. For the proprietary retaining wall, schematic typical cross sections combined with the cast-in-place details should be enough for the proprietary wall manufacturers to design their walls.

The Designer shall provide a list of the specific wall types allowed for each site. For instance, the Designer may limit the selection based on the available right of way at a site.

For mechanically stabilized earth walls with metallic soil reinforcements that are to be built in areas of potential stray currents within 200 feet of the structure (for example: an electrified railroad), a corrosion expert shall evaluate the potential need for corrosion control requirements.

If the wall is required to be designed for seismic loads, it shall be stated in the notes for the wall.

WALLS > 5,000 FT^2 OF VERTICAL FACE AREA

For this situation, proprietary retaining walls will most likely be more economical; therefore, a cast-in-place wall design should generally not be done except where site conditions or soil constraints may require a cast-in-place wall. The Designer shall determine which proprietary retaining walls are appropriate for each site. The Designer shall also lay out the proprietary retaining walls in the contract with at least the following information:

- A list of the specific walls allowed for each site. For instance, the Designer may limit the selections based on the available right of way at a site.
- Retaining wall plan view with all required dimensions, offsets, contours, property lines, utilities, etc.
- Retaining wall elevation view showing top and bottom elevations, approximate footing step locations, existing and finished grade, etc. Where required, the

Designer shall also show the location of railings or fences, light standard and/or sign support anchorage locations, rigid metal conduit and junction boxes.

- Typical Sections (schematic) of the wall showing pay limits and minimum drainage requirements. Specific details are not required for each wall manufacturer, only for each wall type.
- All soils information normally used for the design of a cast-in-place wall shall be shown in the contract, including but not limited to borings and maximum factored bearing resistance.
- Temporary Sheeting required for excavation.
- If the wall is required to be designed for seismic loads, it shall be stated in the notes for the wall.
- For mechanically stabilized earth walls with metallic soil reinforcements that are to be built in areas of potential stray currents within 200 feet of the structure (for example: an electrified railroad), a corrosion expert shall evaluate the potential need for corrosion control requirements.

LARGE ANTICIPATED SETTLEMENTS AND LIQUEFACTION

If large settlements or liquefaction are anticipated that require a wall supported on piles, in general, proprietary retaining walls should not be used. Even though these walls can accommodate some settlement, the opening and closing of the joints would produce an undesirable appearance. For these situations, a cast-in-place wall should be designed supported on piles, or the proprietary retaining walls shall be detailed with pile supported full width footings.

WALLS SUPPORTING ROADWAYS

If the wall supports a roadway where there is a possibility of future underground utilities and drainage structures, mechanically stabilized earth walls should not be used. This would not be the case for walls supporting limited access highways. If the utilities are extensive or deep, it may not be possible to use the modular wall options either.

MULTIPLE WALLS IN SAME PROJECT

If there are several retaining walls within the same project, the Designer may wish to require that all walls selected by the Contractor for the project be manufactured by the same wall supplier. This is especially true for walls that are close together.

PRE-CONSTRUCTION PROCEDURES

The Designer should contact the wall companies for tall walls or walls with unusual geometry to be sure that the proprietary walls will function at each site. This should be done during the preliminary design phase of the project.

Prior to construction advertising, the Designer should inform in writing each proprietary wall company that they are listed as acceptable alternates in the contract. This will allow them to obtain the contract in order to accomplish preliminary design during advertising for the project. Part of this submission should include the anticipated advertising date.

REQUIREMENTS FOR FOUNDATIONS

SPREAD FOOTINGS ON SOIL

The contract shall show the following:

- The maximum design foundation pressure for the controlling Strength and Service Limit States.

  Maximum Design Foundation Pressure = X.X TSF (Strength I)

  X.X TSF (Service I)

- If applicable, also show the maximum design foundation pressure for the Extreme Event Limit State.

  Maximum Design Foundation Pressure = X.X TSF (Extreme Event II)

DRIVEN PILES

The contract shall specify that the Contractor is required to submit pile point reinforcement and splice details to the Engineer for review and approval.

This following note shall be included on the contract drawings:

Prior to driving the piles, the Contractor shall submit to the Engineer for review and acceptance their proposed method and sequence of pile driving.

The pile plan(s) included in the contract drawings shall show or note the following:

- A legend denoting vertical, battered and test piles.
- The number, location and length of test piles, if applicable.
- The location of load test pile(s), if applicable.
- The location of dynamic monitoring (pda) pile(s), if applicable.
- The number, location and estimated length for vertical and battered production piles. If no test pile is specified or if the estimated pile length is intended to be used as the order length, the following note shall be included:

The estimated pile length(s) is to be used as the pile order length.

- Material designation of piles, including pile point reinforcement and splices.
- The maximum design pile load for the controlling Strength and Service Limit States for each foundation section.

Maximum Design Pile Load = XX Tons (Strength I)
XX Tons (Service I)

If applicable, also show the maximum design pile load for the Extreme Event Limit State.

Maximum Design Pile Load = XX Tons (Extreme Event II)

- The ultimate pile capacity for each foundation section, as defined as:
Ultimate Pile Capacity = (Factored Design Load)/φ + Scour + Downdrag

Scour= The estimated skin friction resistance of the soil above the predicted scour depths.

Downdrag= The estimated side friction resistance of a compressible soil above the neutral point (determined when computing the downdrag load (DD) due to settlement).

φ = The resistance factor based on the design load limit state, resistance determination method, and required field testing.

Example:

ULTIMATE PILE CAPACITY	
Abutment 1	XX tons
Pier No. X	XX tons
Abutment 2	XX tons

The Scour and Downdrag resistance along with φ will be determined by the geotechnical engineer and included in the geotechnical report. The Contractor will use the Ultimate Pile Capacity to properly size pile driving equipment and load testing apparatus. The Engineer will use the Ultimate Pile Capacity to establish the required driving resistance and validate load test results.

EARTH RETAINING SYSTEMS AND COFFERDAMS

HIGHWAY APPLICATIONS

The location and limits, both horizontal and vertical, of all temporary and permanent earth retaining system Contract items shall be shown at each location.

PERMANENT STEEL SHEET PILING

Permanent Steel Sheet Piling is defined as a required and permanent structural element integral to the design of the structure. Permanent Steel Sheet Piling is designed and engineered by the Designer. To avoid unnecessary proprietary specificity, permanent steel sheet piling should be specified and designated by AASHTO or ASTM material classification and minimum required section modulus.

TEMPORARY EARTH RETAINING SYSTEMS

Temporary earth retaining system shall be any type of adequately braced temporary retaining wall which the Contractor elects to build to satisfy, and which does satisfy, the condition that existing facilities be properly retained during excavation or fill for the placement of substructure or other facilities. A Temporary Earth Retaining System shall be designated in the contract to be left in place only if its removal may be detrimental to the structure. The item “Earth Retaining System Left in Place” shall be used only for a Temporary Earth Retaining System designated by the Designer to be left in place. A Temporary Earth Retaining System requested by the Contractor to be left in place for their own convenience is not compensable for additional payment.

RAILROAD APPLICATIONS

The location and limits, both horizontal and vertical, of all temporary and permanent earth support systems necessary for the construction of railroad structures must be shown in the contract. All contract items for temporary and/or permanent earth support systems for railroad structures and facilities must be submitted to the affected railroad for review during the standard project design submission phases. Contract plan details for temporary earth support in railroad applications must be specific in name and include a railroad parenthetical in the contract item name. Typical items for such use are “Temporary Sheet Piling (Railroad)” and “Soldier Pile and Lagging (Railroad).” The Designer should select the most appropriate temporary retaining system type in consideration of existing soil conditions and construction access limitations.

PERMANENT STEEL SHEET PILING

Permanent Steel Sheet Piling adjacent to railroad tracks shall be designed for each specific location and shown in the Contract. It should be specified and designated as noted in the Section “Railroad Applications” above.

TEMPORARY EARTH SUPPORT SYSTEMS

Through communication with the affected railroad company during a project's scope development, a determination will be made whether or not a complete design for a temporary earth support system is required. When the temporary earth support system is fully designed by the Designer, the contract special provision for the subject item should allow for the Contractor to submit an alternate design.

The Designer shall evaluate the global stability of the railroad embankment for the temporary (and permanent) condition. In cases where the stability of the embankment may be compromised, the Designer shall either provide a complete design or include the necessary constraints for a Contractor to properly design the earth support system.

Working drawings and design calculations prepared by the Contractor shall be submitted to the **CTDOT** and affected railroad company for review.

Items, such as "Sheet Piling left in Place (Railroad)" and "Soldier Pile and Lagging Left in Place (Railroad)" shall be designated in the contract only if their removal may be detrimental to the structure, as determined by the Designer in concert with the affected railroad company. Any system requested to be left in place by the Contractor for their convenience is not compensable.

WATER-HANDLING-COFFERDAMS AND TEMPORARY WATER REDIRECTION

Water-Handling-Cofferdams and various methods to temporarily redirect water from the site are used so that construction can take place in the dry. Various methods and items may be used to achieve this end depending on the nature of the site, the nature of the construction, and the amount of water encountered. Because of the need to secure environmental permits, considerable detail of the dewatering plan intended may be required in the contract.

STRUCTURE EXCAVATION (COMPLETE)

The items "Structure Excavation-Earth (Complete)" or "Structure Excavation-Rock (Complete) are generally used where water intrusion into the excavation results from groundwater seepage or very minor stream or drainage flow. There is no additional payment for dewatering or temporarily diverting water since the work required to construct in the dry is considered to be of such a minor nature that it can be considered to be incidental to the excavation items. Any cofferdams, temporary redirection, pumping, or any other dewatering methodology is included in the cost of the work. Generally, since the impact on water resources is so small, very little detailing is required in the contract.

HANDLING WATER

If a structure cannot be constructed in the dry, a cofferdam is needed. The item "Handling Water" is generally used where a temporary redirection of a watercourse is required and is generally used for construction of culverts or retaining walls adjacent to the watercourse. This lump sum item includes any temporary water handling structures such as barriers, temporary pipes, or drainage channels, necessary to complete the work. Also included is any excavation required to accomplish the temporary redirection of surface water.

Any required excavation for the permanent construction will be paid for under the items “Structure Excavation-Earth (Complete).” “Structure Excavation-Rock (Complete)” or appropriate excavation items. A conceptual scheme showing all temporary water handling structures such as barriers, temporary pipes, and drainage channels, and a conceptual scheme for staging of construction for water handling must be shown in the contract and will usually be included in permit applications. The hydraulic design of the aforementioned temporary facilities is based on the watercourse hydrology and information contained in the **DRM** [6.15]. A temporary design water surface elevation associated with the proposed temporary hydraulic facility should be shown in the contract and the permit plates.

The contractor will be required to submit working drawings to detail the proposal shown in the contract. If the contractor’s working drawings differ from the proposal shown on the contract to the extent that a revision to the permit is required, the contractor will be required to prepare and apply for any revisions required to the permit.

COFFERDAM AND DEWATERING

The item “Cofferdam and Dewatering” is generally used where substructure elements are located partly or wholly in the streambed and where the bottom of footing is below water level, or on foundation work where considerable flow or concentration of water is present that cannot be conveniently temporarily redirected from the site. The extent of work involved in placing and dewatering the cofferdam is such that it is more than a minor part of the excavation items and is not considered incidental to those items. This item should be used in conjunction with the items “Structure Excavation-Earth (Excluding Cofferdam and Dewatering)” and “Structure Excavation-Rock (Excluding Cofferdam and Dewatering).”

A cofferdam forms an enclosure that may be completely dewatered to allow work in the dry. It may consist of steel sheet piling or any other material the contractor elects to use to satisfy this requirement. Some sides of the cofferdam may be formed by the existing stream bank or by existing structures such as abutments or retaining walls. However, any existing structures intended to be used must extend below the anticipated bottom of excavation and must be resistant to intrusion of significant amounts of water from below the footing.

Cofferdams that encroach into water channels must be hydraulically analyzed based on the watercourse hydrology and information contained in the **DRM** [6.15]. A temporary design water surface elevation associated with the proposed temporary hydraulic facility should be shown in the contract and the permit plates.

The horizontal and vertical limits of the cofferdam must be shown in the contract. Cofferdams are designed and detailed by the Contractor and submitted to the Engineer for review.

CONCRETE STRUCTURES

MATERIALS

CAST-IN-PLACE CONCRETE

Concrete shall conform to the **SSC** requirements of "Section 6.01 – Concrete for Structures" and "Section M.03 – Portland Cement Concrete," including modifications by Supplemental Specifications and Owned Special Provisions. The class of concrete specified for bridge components shall generally conform to the following guidelines:

- Substructure Components - Class PCC03340*
- Bridge Superstructures – Class PCC04462

  *If "Z" is other than "0," written permission shall be obtained from the Bridge Principal Engineer.

Occasionally, concrete classes that meet these guidelines are not appropriate for a specific use. In these instances and with permission from the **CTDOT**, Designers may modify the concrete class for the intended use as follows:

- For concrete components with extremely congested reinforcing, Designers shall consider specifying a concrete class with a smaller aggregate.
- Where higher 28-day strength or higher early strength is needed to resist applied loads, Designers may specify a class of concrete with a greater strength.

When the dimensions of a cast-in-place concrete component (not cast underwater) qualify it as "Mass Concrete," in accordance with the **SSC**, the PCC classification system will still be used to specify the compressive strength, aggregate size and Exposure Factor. Concrete suppliers may tailor the mix to address temperature and cracking in the concrete, but the Contractor shall prequalify the proposed mix with ample time to place the concrete. Designers shall verify that sufficient time is available in the contract for the Contractor to prequalify the mix before its intended use.

"Underwater Concrete" will also be specified using the PCC classification system. The Designer shall include a cofferdam in the contract to construct underwater concrete components and clearly designate the concrete as "Underwater Concrete."
The density of cast-in-place concrete shall be assumed to be as follows, unless proof of another value is available:

- Normal Weight Concrete of all PCC Classes: 150 pounds per cubic foot
- Lightweight Concrete: 125 pounds per cubic foot

ULTRA HIGH PERFORMANCE CONCRETE (UHPC)

Ultra High Performance Concrete (UHPC) shall conform to the requirements of the Owned Special Provision “Ultra High Performance Concrete.”

FABRICATION REQUIREMENTS

The prestressed concrete fabricator’s plant shall be certified by the Precast Prestressed Concrete Institute Plant Certification Program. The certification shall be as a minimum in the B3 Category, except for draped strand members, in which case a B4 Category certification is required. The certification requirements shall be shown on the plans.

TOLERANCES

Tolerances for prestressed members shall conform to the limits specified in the *Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products* (MNL-116).

DESIGN AND DETAILING REQUIREMENTS

CAST-IN-PLACE, REINFORCED CONCRETE MEMBERS

PAY ITEMS

Pay item names reflect the character of the bridge components that will be measured for payment under the item. To reduce the number of item names, similar components may be included together under the same item. See Table - Cast In Place Concrete Pay Items for a list of item names from which Designers may choose, and a list of components that may be considered similar enough to include with each item.

TABLE - CAST IN PLACE CONCRETE PAY ITEMS

<table>
<tr><th>ITEM NAME</th><th>COMPONENTS INCLUDED</th><th>Concrete Mix Class (PCCXXXYZ[1])</th></tr>
<tr><td>Footing Concrete</td><td>Footings, leveling pads, pile caps, cut-off and return walls</td><td rowspan="4">PCC0334Z</td></tr>
<tr><td>Footing Concrete (Mass)</td><td>Footings, pile caps</td></tr>
<tr><td>Abutment and Wall Concrete</td><td>Abutments, wingwalls, retaining walls, endwalls, headwalls, concrete bearing pedestals, cheekwalls, keeper blocks, curbs</td></tr>
<tr><td>Abutment and Wall Concrete (Mass)</td><td>Abutments, wingwalls, retaining walls, endwalls</td></tr>
<tr><td>Not Applicable</td><td>Steps, Copings</td><td>PCC0336Z</td></tr>
<tr><td>Surface Repair Concrete</td><td>Abutments, Walls, Columns, Caps, Parapets, Box Culverts</td><td rowspan="2">PCC04481, PCC05581</td></tr>
<tr><td>Structural Repair Concrete</td><td>Columns, Caps, Parapets, Box Culverts</td></tr>
</table>

Column and Cap Concrete	Pier columns, pier caps, concrete bearing pedestals, keeper blocks	PCC0446Z
Column and Cap Concrete (Mass)	Pier columns, pier caps	
Approach Slab Concrete	Approach slabs, concrete aprons on grade	
Barrier Wall Concrete	Barrier Walls (includes footing, stem and parapet)	
Bridge Deck Concrete	Bridge decks, haunches, backwalls and end diaphragms cast integral with the deck, concrete curbs	PCC04462
Bridge Deck Concrete (SIP-Forms)	Bridge decks, haunches, backwalls cast integral with the deck, concrete curbs	
Parapet Concrete	Parapets on bridge decks, parapets on wingwalls and parapets on retaining walls	
Bridge Sidewalk Concrete	Bridge sidewalks, curbs, raised medians	
[1]Exposure Factor, Z, shall be "0" unless another value is approved by the Bridge Principal Engineer		

When estimating the unit bid price of a cast-in-place concrete component, the largest contribution is not the cost of the concrete material. Additional factors that contribute to the cost of a concrete component are: complexity and congestion of reinforcing, forming and removal of forms, concrete placement and consolidation, sequence and timing of pours, finishing and access needs. When these factors are similar enough for different bridge components, those components may be included together in the same bid item.

Note that a component such as a concrete curb does not have its own pay item. This component may be measured for payment with the item in the contract whose character of work is most similar. The concrete curb may be included for measurement under the item, "Bridge Deck Concrete," but if a "Bridge Sidewalk Concrete" item were included in the contract, it would be the preferable item with which to include the concrete curb because the character of work is more similar. If the structure were a box culvert with shallow fill, the contract may not include an item for "Bridge Deck Concrete" or "Bridge Sidewalk Concrete." In such a case, if the item, "Abutment and Wall Concrete" is included for wingwalls, the curb could be measured for payment under that item. Note that in this situation, the character of work is not a close match, but the volume of concrete in the concrete curb may be small enough to have little effect on the unit bid price for "Abutment and Wall Concrete." The total cost of the concrete curb will also be affected insignificantly by applying the unit bid price for "Abutment and Wall Concrete." It would not be necessary to include an additional item exclusively to pay for the small volume of concrete curb.

New items may be created when, in the opinion of the **CTDOT**, no item exists in the Master Bid List that adequately describes the bridge component in question, or when the character of work of similar components is significantly different for a specific situation.

PLAN REQUIREMENTS

Concrete pay items shall be clearly listed in the General Notes within the plans. Adjacent to each pay item in the General Notes, list the cast-in-place concrete components to be measured for payment under that item. Following the list of components, specify the concrete mix class. Although components are grouped together under a pay item to reflect the character of work, the mix class may vary among components within the pay item. In such a case, separate and group the components in the note by mix class.

Listing pay items and components in the General Notes may not provide sufficient clarity for measuring quantities and distinguishing pay limits between items. To provide clarity, drawings with pay limits may be needed. One such example is the item, "Parapet Concrete." Where a parapet is constructed at the top of a wall, delineation between the items, "Parapet Concrete" and "Abutment and Wall Concrete" is needed. A convenient limit for this is the construction joint between the two components (see Figure – Concrete Wall with Parapet Pay Limits below).

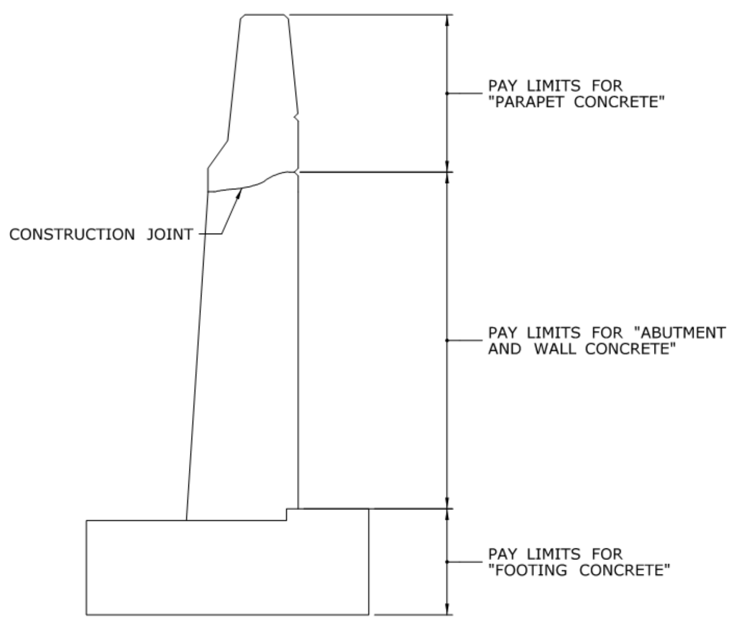


FIGURE – CONCRETE WALL WITH PARAPET PAY LIMITS

PRECAST, PRETENSIONED CONCRETE MEMBERS

OVERSIZED MEMBERS

For members in excess of 120 feet, the use of field spliced post-tensioned bulb tees should be considered.

COMPOSITE CONSTRUCTION DETAILING REQUIREMENTS

The following note, with a leader pointing to the top surface of the member, shall be shown on the plans:

Raked Finish

CAMBER

An acceptable method for estimating cambers and deflections in simple span members using multipliers can be found in the “PCI Design Handbook - Precast and Prestressed Concrete.”

POST-TENSIONED TRANSVERSE STRANDS

Structures composed of deck units placed butted to each other without a composite deck shall be post-tensioned transversely with prestressing strands.

The number and location of these transverse ties is dependent upon the following: the length of the member, the depth of the member, the skew angle of the structure, and stage construction. Based on the skew angle of the structure, the ties may be placed parallel to the skew of the structure or normal to the sides of the member.

The appropriate post-tensioning procedure along with the following note shall be shown on the plans:

No additional dead loads or live loads shall be applied to the butted deck units until the transverse ties have been fully tensioned and the grout in the longitudinal shear keys has reached a seven-day compressive strength of 4500 psi.

The transverse strands shall be post-tensioned in accordance with one of the following procedures:

For structures with skew angles less than or equal to 30 degrees:

TRANSVERSE STRAND POST-TENSIONING PROCEDURE
1. After erecting the prestressed deck units for the construction stage, install the transverse ties. 2. Tension each transverse tie to 5 kips. 3. Seal the bottom of the longitudinal shear keys with closed cell polyethylene foam backer rod and place non-shrink grout in the longitudinal shear keys and internal diaphragms. The grout shall be rodded or vibrated to ensure that all the voids in the shear keys are filled.

4. On shallow members with one row of ties, include the following note:

   When the grout has attained a compressive strength of 1500 psi, tension each transverse tie to 30 kips.

   On deep members with two rows of ties, include the following note:

   When the grout has attained a compressive strength of 1500 psi, at each transverse tie location tension the bottom tie to 15 kips, then the top tie to 15 kips. Repeat this tensioning sequence once more so that each tie is tensioned to 30 kips.

NOTE: Where the total initial post-tensioning force of all the transverse ties is sufficient to displace the exterior members, the designer shall modify the post-tensioning procedures to require placement of hardwood shims between the members. The Designer shall specify the number and location of these shims. The shims shall be placed between as many members as is required such that the total initial post-tensioning force does not displace any members.

For structures with skew angles greater than 30 degrees:

TRANSVERSE STRAND POST-TENSIONING PROCEDURE

1. As each member is being erected, install the transverse ties and place hardwood shims between the adjacent deck units at each transverse tie hole location on the top and bottom.
2. On shallow members with one row of ties, include the following note:

   Secure each member to the preceding member by tensioning each transverse tie to 30 kips before erecting the next member.

   On deep members with two rows of ties, include the following note:

   Secure each member to the preceding member by first tensioning the bottom tie at each transverse tie location to 15 kips, then the top tie to 15 kips. Repeat this tensioning sequence once more so that each tie is tensioned to 30 kips.
3. After all the members have been erected, seal the bottom of the longitudinal shear keys with closed cell polyethylene foam backer rod and place non-shrink grout in the longitudinal shear keys and internal diaphragms. The grout shall be rodded or vibrated to ensure that all the voids in the shear keys are filled.

4. When the grout has attained a compressive strength of 1500 psi, remove the hardwood shims. The voids left in the grout from the top shims shall be filled with grout. The voids in the grout from the bottom shims may be left unfilled.

DRILLING HOLES

The following note shall be shown on the plans:

The drilling of holes in (or the use of power actuated tools on) prestressed members will not be permitted.

STEEL STRUCTURES

MATERIALS AND FABRICATION

STRUCTURAL STEEL DESIGNATIONS

The structural steel designations shall be shown on the plans. The designations shall reference AASHTO material specifications and include the applicable suffix codes. The suffix "T" indicates a Non-Fracture Critical material whereas an "F" indicates a Fracture Critical material. The "T" or "F" is followed by the appropriate AASHTO temperature zone for Connecticut, which is "2."

Examples:

Non-Fracture Critical Bridge Members	AASHTO M270 Grade 50 T2 AASHTO M270 Grade 50 WT2
Fracture Critical Bridge Members	AASHTO M270 Grade 50 F2 AASHTO M270 Grade 50 WF2

FASTENERS

The high-strength bolt, nut and washer designations shall be shown on the plans. These designations shall reference ASTM Specifications, and include types and grades where applicable.

WELDING

Fillet weld sizes shall be shown on the plans. Flange to web, stiffener and connection plate welds shall be a minimum of 5/16.

Weld symbols for complete penetration groove welds shall be specified, without dimensions, by three capital letters, CJP. This allows the weld joint configuration and details to be determined by the fabricator.

Non-destructive testing (NDT) of welds shall be specified with symbols, combined with the welding symbols, for the welds requiring testing. The quantities of non-destructive testing methods required for field welds shall be shown in the "Inspection of Field Welds" block on the General Plan.

Multiple pass welds, inspected by the magnetic particle method, shall have each pass or layer inspected and accepted before proceeding to the next pass or layer.

The welding specifications shall be shown on the plans.

FABRICATION REQUIREMENTS

The structural steel fabricator's plant shall be certified by the AISC Quality Certification Program. The certification requirements depend on the category of structure being fabricated as follows:

For non-fracture critical members:

1. Bridge Fabricator Simple (SBR) or Bridge Component (CPT).

Typical work includes:

1. *Bridge cross frames for straight bridges with skew angles less than 30 degrees*
2. *Highway sign structures*
3. *Bridge inspection catwalks*
4. *Grid decks*
5. *Scuppers*
6. *Expansion joints*
7. *Bearings*

2. Bridge Fabricator Simple (SBR).

Typical work includes:

1. *Straight simple un-spliced rolled beams*

3. Bridge Fabricator Intermediate (IBR).

Typical work includes:

1. *Rolled beam with field or shop splices, straight or with radius over 500 feet*
2. *Built up I-shaped plate girder with constant depth except for dapped ends, with or without splices, either straight or with radius over 500 feet*
3. *Built up I-shaped plate girder with variable depth, either straight or with a radius over 1000 feet*
4. *Truss with a length 200 feet or less that is entirely pre-assembled at the verified facility and shipped in no more than three sub-assemblies*

4. Bridge Fabricator Advanced (ABR).

Typical work includes:

1. *Tub or trapezoidal box girders, closed box girder bridges*
2. *Curved girders with radius under 500 feet*
3. *Large or non-preassembled trusses, arches*
4. *Moveable bridges*
5. *Cable stayed bridges*

If the structure has fracture critical members or components, the fabricator's plant shall also be certified to produce fracture critical members in accordance with a fracture control plan as defined by the **AWS D1.5**. A fabricator with this endorsement will have a suffix "F" added to the above categories (Category IBR,F or Category ABR,F).

The certification requirements for specific components shall be shown on the plans.

GENERAL DESIGN REQUIREMENTS

COST EFFECTIVE SPAN LENGTHS

The following are appropriate ranges of cost effective span lengths for various steel bridges types:

TYPE OF BRIDGE	COST EFFECTIVE SPAN LENGTH (ft)
Rolled Beams	50 to 90
Plate Girders	80 to 250
Box Girders	150 to 250

The span lengths shown are for simple span bridges. For continuous bridges, these span lengths can be assumed to be measured from dead load inflection points.

For spans over 250 feet, special design studies must be done. Plate and box girders may still be the structure of choice since they provide redundancy. Other options are arches, trusses or cable stayed bridges, although these structure types should be limited to very long spans.

FRACTURE CRITICAL BRIDGE MEMBERS

DEFINITIONS

Fracture Control Plan (FCP) - The Fracture Control Plan is the materials testing and fabrication provisions for Fracture Critical Members as outlined in the **AWS D1.5**.

Fracture Critical Member (FCM) - Fracture Critical members or member components are tension members or tension components of bending members (including those subject to reversal of stress), the failure of which would be expected to result in collapse of the bridge. The designation "FCM" shall mean fracture critical member or member component. Members and components that are not subject to tensile stress under any condition of live load are not fracture critical.

Attachments - Any attachments welded to a tensile zone of a FCM member shall be considered a FCM when any dimension of the attachment exceeds 4 inches in the direction parallel to the calculated tensile stress in the FCM. Attachments shall meet all requirements of the Fracture Control Plan.

Welds - All welds to FCM's shall be considered fracture critical and shall conform to the requirements of the Fracture Control Plan. Welds in compression members or compression areas of bending members are not fracture critical.

GENERAL PROVISIONS

Each FCM shall be individually designated on the plans by three capital letters, FCM, enclosed in a diamond.

Based on the definitions above, the following guidelines shall be followed for designation of FCM's on plans:

I-Shaped Girder Bridges - For longitudinal girder bridges, FCM components of the beams shall be considered fracture critical if there are three or less girders in the bridge cross section. This requirement does not apply to temporary stages in construction.

Box Girder Bridges - For longitudinal box girder bridges, FCM components of the beams shall be considered fracture critical if there are two or less box girders in the bridge cross section. For the case of a two-box girder cross section, the top flanges and the welds of the webs to the top flanges shall not be considered fracture critical. This requirement does not apply to temporary stages in construction.

The list above is not all inclusive. Additional bridge types and components may require classification as FCM.

CONNECTIONS AND SPLICES

BOLTED SPLICES

DETAILING REQUIREMENTS

The bolt diameter, hole size, bolt spacing, end distances and edge distances shall be shown on the plans.

CROSS MEMBER CONNECTIONS

DETAILING REQUIREMENTS

When cross members are considered secondary members, the size, number and general layout of the bolts for bolted connections should be shown on the plans. Bolt hole spacing and edge distances should be left to the fabricator.

When cross members are considered primary members, connections shall be fully designed and all bolt hole and bolt sizes, spacings, end and edge distances, plate thicknesses, and any other pertinent information shall be shown on the plans.

Holes for end diaphragm connections shall be located parallel to the main member's web. Standard sized holes shall be used in the cross members while oversized holes, unless otherwise noted, shall be used in the stiffener or connection plates. At one side of a cross member, standard sized holes field drilled through the stiffener or connection plate may be used as an alternate method for erection.

Long slotted holes in the stiffener or connection plates shall be considered for erection of intermediate cross members for girders adjacent to a stage construction line.

For bridges with skews more than 20 degrees, when the differential dead load deflection of adjacent girders at any intermediate cross member connection is 3/4 inches or more, long slotted holes shall be detailed in the stiffener or connection plates attached to the girder with the larger deflection. The following note should appear on the plans when long slotted holes are used:

Bolts in long slotted holes shall only be finger-tightened prior to pouring the deck slab and then fully-tightened immediately after completing the pour.

Gusset plates shall be made rectangular to simplify fabrication.

Shop welds shall be made on one side, as much as practical, to avoid having to turn over the cross member assemblies in the fabricating shop.

COMPOSITE CONSTRUCTION

DETAILING REQUIREMENTS

The minimum height shear connector is 4 inches. The maximum height of unstacked shear connectors is 8 inches. Stacked shear connectors shall be used at the locations where the haunch depth exceeds 6 inches.

Shear connectors are typically welded to the members in the field. Field welding through a mist coat of up to 2 mils of zinc-rich primer is permissible.

Only the diameter of the shear connectors shall be shown on the plans. Shear connector heights shall not be shown on the plans. The heights shall be determined after the erected members have been surveyed and the haunch depths calculated.

On flange splice plates, one row of shear connectors shall be placed along the centerline of the splice plates.

DEAD LOAD DEFLECTION AND CAMBERS

SIMPLE SPAN BRIDGES

Dead load deflection and camber diagrams are not required for simple span bridges. Dead load deflections and cambers shall be calculated at the mid-span of the structure for the following listed items for each member and tabulated on the plans:

1. Structural Steel Deflections. Deflections due to the weight of the beams or girders, including the diaphragms and bracing and calculated using the moment of inertia of the steel section.

2. Additional Dead Load Deflections. Deflections due to the uncured concrete slab and haunches, utilities, and any other loads supported by the steel section alone. These deflections shall be calculated using the moment of inertia of the steel section.

3. Composite Dead Load Deflections. Deflections due to the parapets, curbs, sidewalks, railings, bituminous concrete overlay and any other loads that are placed after the slab has cured. This deflection shall be calculated using the moment of inertia of the composite section with a modular ratio equal to 3 times that of the final section as outlined in the **BDS**.

4. Total Dead Load Camber. Camber required to compensate for the summation of the structural steel, slab dead load and the composite dead load deflections listed above.

5. Vertical Curve Ordinate Camber. Camber required when the member falls within the limits of a summit vertical curve. When the member falls within the limits of a sag vertical curve, provisions for sag ordinates must be made within the concrete haunch and shall not be specified in the camber table.

6. Extra Camber. Extra camber shall be provided when the grade of the roadway is on a tangent grade or on a sag vertical curve and is computed as follows:

    - Extra Camber (inch) = L / 100, where: L = Span Length (feet)

When the roadway is on a crest vertical curve, the extra camber is to be specified only when it exceeds the vertical curve ordinate. In this case, the amount of extra camber to be tabulated shall be only that portion in excess of the vertical curve ordinate.

7. Total Camber. The Total Camber is equal to the summation of all calculated cambers and is that dimension to which the member is to be fabricated.

CONTINUOUS SPAN BRIDGES

Dead load deflections and cambers shall be tabulated for the following listed items for each member and shown on the plans. The locations tabulated shall be the member bearing points and points at equal spaces along the member at approximately 10 feet on center:

- Structural Steel Deflections: Same as for simple span bridges.
- Additional Dead Load Deflections: Same as for simple span bridges.
- Composite Dead Load Deflections: Same as for simple span bridges except that composite section properties should be used for both positive and negative moment regions.

- Total Dead Load Camber: Same as for simple span bridges but measured to a reference line, which is a theoretical straight line in each span connecting the points located at the top of the web at the centerlines of bearing.
- Vertical Curve Ordinate Camber: Same as for simple span bridges.
- Extra Camber: Extra camber shall not be provided for continuous bridges.
- Total Camber: The Total Camber is equal to the summation of all calculated cambers and is that dimension to which the member is to be fabricated.

A table for dead load deflections and cambers, and a diagram for total camber shall be shown on the plans. Refer to AASHTO/NSBA Steel Bridge Collaboration Guideline Document G1.2 for example presentation.

SUPERSTRUCTURE JACKING REQUIREMENTS

DETAILING REQUIREMENTS

Lift points shall be clearly identified on the plans. The dead and live loads required to jack the bearing shall also be shown on the plans. If there are more than five lines of girders, two sets of loads shall be shown. The loads shall be for simultaneous jacking of all girders, and for jacking of individual girders. Additional stiffeners or brackets, if required, shall be shown on the plans.

STRUCTURE TYPE SPECIFIC REQUIREMENTS

I-SHAPED AND BOX PLATE GIRDERS

WEB PLATES

In general, for plate girders with web depths less than 50 inches, unstiffened webs are more economical. For web depths greater than 50 inches, the following alternates shall be investigated for the web design to determine which is the most cost effective:

a. Fully stiffened web with minimum web plate thickness
b. Unstiffened Web
c. Partially stiffened web with only a few stiffeners near supports

In order to determine which of these alternates is most cost effective, the 1 to 4 rule should be used. That is, if the web and flange material costs \$1 per pound, then the connection plate material costs \$4 per pound.

FLANGE PLATES

The number and spacing of flange plate thickness transitions should be based on the total cost of the finished girder. While numerous flange transitions will produce the lightest girder, the fabrication costs for the splices may result in a higher total cost. The Designer should investigate eliminating flange transitions, especially where they are closely spaced. As a rule, the approximate weight of flange material that should be saved in order to justify the introduction of a flange transition is as follows:

M = 255 + 21A
M = Weight of steel, pounds
A = Cross sectional area of thinner flange plate, square inches

In order to eliminate shop welded butt splices, field splices should be located at flange plate transitions.

SHOP SPLICES

Shop flange splices shall be located a minimum of 6 inches from web splices.

Both web and flange splices shall be located a minimum of 6 inches from stiffeners and connection plates.

This information on web and flange plate shop splices shall be shown on the plans.

BOLTED FIELD SPLICES

For box girders, where bolted field splices are called for, the splice shall be detailed to provide adequate clearance for bolting the connections at the acute corners between the top flange and the web for both bolts and splice plates.

CURVED GIRDERS

The Designer is responsible for assuring that the structure is constructable and that it will be stable during all stages and under all loading conditions. To achieve this end, the Designer must supply basic erection data on the contract plans. This information must include, but is not limited to, the following:

- Pick points and reactions at pick points for all girder sections.
- Temporary support points to be used during all stages and loading conditions, and reactions for which support towers should be designed at all of these points.
- Deflections to be expected in all girders under all conditions of temporary support and under all anticipated loading conditions.
- Direction pertaining to the connection of diaphragms to assure stability during all temporary conditions.

Specifications prepared for this work must require the Contractor to submit full erection plans, prepared and stamped by a Professional Engineer registered in the State of Connecticut, for review by the **CTDOT**. These plans will be reviewed by the Designer as a working drawing and comments forwarded from the Office of Engineering to the District Engineering Manager having jurisdiction over the project for transmittal to the Contractor. The Designer's review must ensure that all information given on the Contract plans has been accurately accounted for in the Contractor's erection plans.

The Designer shall provide any such additional information, up to and including full erection plans in the Contract documents as directed by the **CTDOT**.

DECKS AND DECK PROTECTIVE SYSTEMS

CAST-IN-PLACE CONCRETE DECKS

DECK POURING SEQUENCE

For bridges with continuous members, cast-in-place concrete decks shall be placed in sequence. The sequence of pouring concrete shall be shown on the plans and include the following:

- sections in which the deck is to be poured,
- sequence in which the sections are to be poured,
- direction of pouring each section, and
- minimum compressive strength the concrete in each section must obtain prior to placing concrete in other sections.

Additionally, the following note shall be shown on the plans:

A deck pouring sequence different from that shown may not be used without the prior approval of the Engineer.

CONSTRUCTION JOINTS

Construction joints to facilitate deck construction are permitted. Transverse construction joints are typically required when a sequence of pours is necessary. Longitudinal construction joints may be required for stage construction.

Closure pours may be detailed for stage construction conditions where large differential deflection is anticipated.

FINISHED DECK AND GRADE ELEVATIONS

All elevations necessary for construction of the deck and placement of the bituminous concrete overlay shall be shown on the plans.

Bridges located at merging and diverging roadways shall be carefully detailed with the dimensions and elevations necessary for construction.

FINISHED DECK ELEVATIONS

For cast-in-place concrete decks, finished deck elevations and member deflections shall be tabulated at member bearing points and at points equally spaced along the members at approximately ten feet on center or at tenth points along the span, whichever is greater. The finished deck elevations are those elevations on the top of the concrete deck. The tabulated member deflections are those deflections due to all dead loads except the selfweight of the members and cross members. These elevations and deflections are to be used to calculate haunch depths.

For precast concrete deck panels, deck elevations shall be tabulated at edges of the panels at the panel joints. The deck elevations are those elevations on the top of the concrete panel.

FINISHED GRADE ELEVATIONS

Finished grade elevations are those elevations on top of the final riding surface (such as the bituminous concrete overlay). On all bridges, the finished grade elevations shall be shown at the following points:

- the intersection of the point of application of grade line with the deck joints and ends of slabs,
- the intersection of the gutter lines with the deck joints and ends of slabs, and
- the intersection of the cross slope breaks at the shoulders with the deck joints and ends of slabs.

STAY-IN-PLACE FORMWORK

STEEL FORMWORK

Welding of stay-in-place metal form supports to tension zones in girder top flanges is not permitted.

The Designer shall clearly identify on the structural steel plans all top flange tension zones where welding of stay-in-place form supports is not permitted.

DECK JOINTS

TRANSVERSE EXPANSION JOINTS

JOINTS WITH THERMAL MOVEMENT RANGE (TMR) BETWEEN ½ AND 1 ½ INCHES

Include a table on the plans specifying the thermal movement range at all asphaltic plug joint locations.

JOINTS WITH THERMAL MOVEMENT RANGE (TMR) BETWEEN 1 ½ AND 3 INCHES

Include a table on the plans specifying allowed products and corresponding installation information.

For each product, manufacturers typically identify the preformed joint seal by its nominal capacity (the manufacturer's recommended movement capacity of the seal in a joint installed perpendicular to the direction of movement). Designers shall select products appropriate for the site and list those products in a table on the contract plans using the manufacturers' designations. A template for this table is available for use under Guide Sheets. One table shall be included for each expansion joint location (for example, "Abutment No. 1," shall be included beside, "Description of Joint Location," at the top of the table for the Abutment No. 1 joint).

To assist Designers with selection of a preformed joint seal, an Excel spreadsheet is available under Guide Sheets. Notes below the spreadsheet are provided to assist Designers with the use of the program. Designers shall be responsible for selecting a properly designed joint seal for the specific joint conditions. The spreadsheet is only a guide. Note that the spreadsheet is formatted similar to the Table template. This is to facilitate the transfer of design information into the Table template for inclusion in the plans.

When Designers propose preformed joint seals in bridge deck joints that pass through sidewalks, only foam-supported silicone joint seals shall be specified for that deck joint and the sidewalk joint. It is preferable to use the same joint seal in the parapet as well to allow the same joint gap in the parapet, sidewalk and deck. See the Guide Sheets for typical details at sidewalks.

The foam-supported silicone joint seals are preferred at sidewalk joints because they meet ADA requirements when installed as shown in the Typical Details. Since the sidewalk curb height is too low to allow a V-shaped seal from the deck joint to turn up the curb and rise sufficiently beneath a foam-supported seal in the sidewalk, both the deck joint and the sidewalk preformed joint seal shall be the foam-supported type.

BEARINGS

SOLE PLATES

On bridges with a new superstructure or on bridges undergoing rehabilitation that will maintain the existing superstructure, modifications to existing substructure components that will remain in place, such as replacement of existing concrete bearing pads, may be required to meet the minimum thickness requirement.

For new steel superstructures, sole plates are part of the structural steel pay item. The sole plate shall be detailed by the designer with the thickness dimensioned at the intersection of the centerline of the bearing and the centerline of the supported member.

The steel fabricator is responsible for determining the sole plate bevel(s) based on site geometry and member camber. The designer shall check to ensure the minimum sole plate thickness requirement is met during the shop plan review.

The designer shall specify the condition of all surfaces, including coatings if applicable, of the sole plates consistent with the design parameters in the contract documents.

BRIDGE RAILS AND BARRIERS

TRAFFIC RAILS

For continuous construction, the pouring sequence for all parapets shall be identical to that of the slab.

All new and retrofitted traffic and combination railings on bridges and retaining walls shall have an overall minimum height of 42 inches measured from a roadway or sidewalk surface. MASH Test Levels (TL) shall be shown on the plans.

The single slope barrier is preferred over the F-shape parapet. However, the F-shape parapet may be used where tie-in to adjacent barriers are F-shape. If sidewalks are required on the bridge, they shall be topped with an appropriate pedestrian railing, bicycle railing, or fence system.

BURIED STRUCTURES

REINFORCED CONCRETE BOX CULVERTS AND FRAMES

BACKFILL REQUIREMENTS

Pervious Structure Backfill shall be placed above a slope line starting at the top of the heel and extending upward at a slope of 1:1.5 (rise to run) top the top of the structure's roof. The backfill from the top of the roof to the bottom of the subbase shall meet the recommendations in the geotechnical report.

In cut situations, the following note, with a leader pointing to the slope line, shall be placed on the plans:

Slope line except where undisturbed material obtrudes within this area.

Rock fill or boulders shall not be placed within 2 feet of the top of the structure's roof. The following note, with leaders pointing to the limits, shall be placed on the plans:

No rock fill or boulders shall be placed within these limits.

Backfill materials, limits, and requirements for the structure and wingwalls shall be clearly shown and noted on the plans.

PRECAST CONCRETE BOX CULVERTS

Precast concrete box culverts are items that are furnished and installed by the Contractor in accordance with the owned special provision "_x_ Precast Concrete Box Culvert." The Designer is responsible for reviewing the Contractor's shop and working drawing submittals for the box culverts.

The Designer is responsible for designing and detailing precast concrete box culverts including all other box culvert components, such as cutoff and return walls, nosings, sills, headwalls and wingwalls.

Precast concrete box sections shall be designed for all construction load effects that may be applied during all stages/phases of construction.

REINFORCEMENT DETAILS

The Designer shall determine the concrete cover requirements and note the requirements on the plans.

MINIMUM THICKNESS OF FLOOR, SIDES AND ROOF

The Designer shall note the minimum thickness requirements of the box culvert elements on the plans.

HEADWALLS

Headwalls are typically connected to box culvert roofs with dowel bar mechanical connections. The Designer is responsible for designing and detailing the headwall connection to the box culvert. The Designer shall check to ensure that the end sections of precast concrete box culverts adequately resist the load effects from the headwall.

CAST-IN-PLACE CONCRETE BOX CULVERTS

The Designer is responsible for designing and detailing cast-in-place concrete box culverts including all other box culvert components, such as cutoff and return walls, nosings, sills, headwalls and wingwalls.

EXPANSION, CONTRACTION AND CONSTRUCTION JOINTS

Expansion and contraction joints in the culvert floor, sides and roof shall be provided in accordance with **BDS**. Construction joints shall be placed as conditions warrant.

No reinforcement shall pass through expansion and contraction joints. Reinforcement shall pass through construction joints.

PRECAST CONCRETE FRAMES

The Designer is responsible for designing and detailing precast concrete frames including all other frame components, such as footings, nosings, headwalls and wingwalls.

Precast concrete frames are furnished and installed by the Contractor in accordance with the special provision “Precast Concrete Three-Sided Rigid Frame.” The Designer is responsible for reviewing the Contractor’s shop and working drawing submittals for the frames.

LOAD RATINGS

GENERAL

Bridge Load Rating criteria presented herein are not intended to imply that new or rehabilitated structures are to be designed with load rating methodologies, philosophies, methods and techniques.

During the design and construction phases of a project, load rating packages, as defined by **BLRM**, shall be submitted for permanent bridges to the **CTDOT** Load Rating Section (**LRS**) for review and action in accordance with the following:

- During the design phase (PE phase) of the project, concurrent with the 90% Design Submission, an As-Proposed Load Rating Package shall be submitted to the **LRS** for review. The As-Proposed Load Rating Package must be accepted by the **LRS** prior to advancing the project to Advertising, unless a waiver to advertise has been obtained.
- During the construction phase (CN phase) of the project, following the Semi-Final Inspection for a structure, the Project Manager shall coordinate with District Construction to confirm the as-built condition, schedule for the as-built plans, and prioritization of as-built sheets to ensure the As-Constructed Load Rating Package can be submitted as soon as practical.

  Once all work affecting the structure is completed and as-built plans or partial as-built plans are available, As-Constructed Load Rating Packages shall be submitted to the **LRS** for review.

It is the Project Manager's responsibility that the Minimum Acceptable Rating Factors are met with the inclusion of any changes to condition occurring or identified during construction that are not captured in the As-Proposed Load Rating.

Procedures should be established with the Designer to recompute rating factors as soon as practical following a request, need, or identification of a change in condition to the bridge, and establish expectations for notifying the Project Manager of unacceptable rating factors so that mitigation or remedial action can be incorporated into the project in a timely manner.

PERMANENT BRIDGES

Load rating requirements are based on the following general scope of work categories:

- New structure/superstructure replacement – includes new and replacement buried structures, superstructure replacements, new structures and full bridge replacements
- Major structure rehabilitation – includes deck replacement, structure widening, structural steel repair and modifications to buried structures
- Minor structure rehabilitation – includes deck patching, resurfacing and safety improvements

Design, legal, permit and emergency vehicle live load ratings shall be performed for all new/replacement bridges and buried structures, and existing bridges and buried structures where rehabilitation/repair of the structure will affect the live load rating in accordance with the **BLRM,** amended as follows:

- All existing bridges and buried structures undergoing minor structure rehabilitation need not be load rated provided a load rating, including the CT-TLC rating, is on file with **CTDOT** that reflects the final condition, including deterioration noted on the most recent Bridge Inspection Report, of the structure after completion of the minor rehabilitation and that meets the requirements of the **BLRM**.

Commentary: All existing bridges and buried structures undergoing minor structure rehabilitation with resurfacing shall be load-rated to determine if they are adequate for the construction equipment used to remove/place the HMA overlay. If resurfacing increases the overlay thickness on an existing bridge, or additional dead load is added to the bridge, a load rating is required.

TEMPORARY BRIDGES

Design, legal, permit and emergency vehicle live load ratings shall be performed for all temporary bridges in accordance with the **BLRM**. The design vehicle live load rating at the inventory level is not required for temporary bridges that will be in service less than 3 years. The permit vehicle live load rating for the CT-TLC vehicle is not required for temporary bridges which will not be subject to the CT-TLC load effects. The requirement for load rating of other permit vehicles shall be coordinated with the Oversize and Overweight unit and included in the special provision. The load rating package, comprised of the load rating report and the load rating references folder, shall meet the requirements of **BLRM** [12] amended as follows:

- **BLRM** [12.1(c)] shall be replaced with the following: The load rating package shall reflect either the as-inspected condition or the proposed condition of the structure.
- All requirements of **BLRM** [12.1] shall be met with the exception of **BLRM** [12.1.1.1] and [12.1.1.2].
- All requirements of **BLRM** [12.2] shall be met with the exception of **BLRM** [12.2.4].

The load rating package is part of the Working Drawings package for the temporary bridge. The load rating package shall be reviewed by the project's structural designer to confirm it meets the Contract requirements. The load rating package for a temporary bridge does not need to be reviewed by the **CTDOT** Load Rating Section (LRS). Once the project's structural designer has completed the review of the Working Drawings package and informed District Construction the package is available for their action, the designer shall notify the LRS via email the review of a load rating package for a temporary bridge has been completed. The subject of the email notification shall include the project number, town and the phrase "Temporary Bridge Working Drawing Review Completed." The email shall include a link to the Working Drawings package.

Commentary: In determining if a live load rating for the CT-TLC vehicle is not required, designers should check to ensure the temporary bridge will not be subject to CT-TLC load effects from other adjacent CTDOT construction projects or CTDOT maintenance VIP projects.

Per BLRM [12.1(c)], typical load rating packaged for bridges containing analysis for partial conditions or multi-conditions shall not be submitted to the LRS. Since a temporary bridge represents a temporary/stage/intermediate construction condition, load ratings of temporary bridges shall not be sent to the LRS for review. The project's structural designer is responsible for the review of load rating packages for temporary bridges.

CONDITION FACTOR

Condition factors shall be in accordance with the **BLRM,** amended as follows:

For new bridges, the value of the condition factor used in the rating analysis shall be 1.00.

For bridges undergoing a superstructure replacement or major structure rehabilitation, the existing members and component to be rated shall be rehabilitated to a good/satisfactory condition or better, allowing the use of a condition factor with a value of 1.00, unless otherwise indicated in **BLRM**.

For bridges undergoing minor structure rehabilitation and requiring a load rating, the value of the condition rating factor shall reflect the structural condition of the member. The **CTDOT** shall be notified if the value of the condition factor used in the rating is less than 0.95.

SYSTEM FACTOR

System factors shall be in accordance with the **BLRM,** amended as follows:

The use of system factors that correspond to the **BDS** load factor modifiers for load rating is not allowed.

AVERAGE DAILY TRUCK TRAFFIC

The average daily truck traffic shall be in accordance with **BLRM,** amended as follows:

For new bridges and bridges undergoing a superstructure replacement, load factors for legal and permit load ratings shall be based on average daily truck traffic (ADTT), in one direction, greater than 5000.

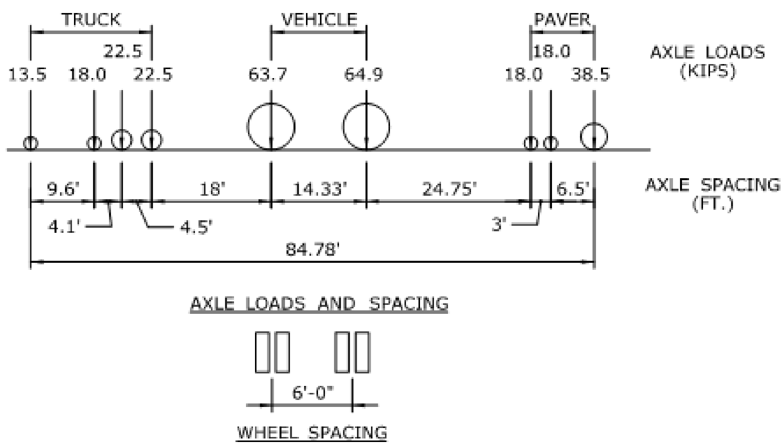
For bridges undergoing other major structure rehabilitation, the average daily truck traffic shall be in accordance with **BLRM** [4.1.1].

PERMIT LOAD RATING

Permit vehicle load ratings shall be performed for permit vehicles in accordance with the **BLRM**.

A load rating shall be performed, on all projects for which a load rating evaluation is required, for the following additional vehicle, load factor criteria and analysis parameters:

Permit load vehicle: CT-TLC (Paving Train)
Permit Type: Special (Limited Crossing)
Frequency: Single Trip
Loading Condition: Mix with traffic
Distribution Factor: One lane
Dynamic Load Allowance: 0.00



Note: TLC = tri-load combination of vehicles in paving train

CT-TLC PERMIT LIVE LOAD VEHICLE
279.6 kip on 9 axles

MINIMUM ACCEPTABLE RATING FACTORS

The minimum acceptable design, legal, permit and emergency vehicle rating factors for permanent bridges, are based on the general scope of work categories, and shall be no less than the values shown in the following table:

TABLE – RATING FACTOR REQUIREMENTS

Rating Procedure	**Minimum Acceptable Rating Factor (RF)**		
	New Structure/ Superstructure Replacement	**Major Structure Rehabilitation**	**Minor Structure Rehabilitation**
Design Load Rating, Evaluation Level – Inventory	1.20	1.20 See Note 1	Report value, may be less than 1.00
Design Load Rating, Evaluation Level – Operating	Report value	Report value	1.00 See Note 2
Legal Load Rating	1.20	1.20 See Note 2	1.00 See Note 2
Permit Load Rating	1.20	1.20 See Note 2	Report value, may be less than 1.00
Permit Load Rating, CT-TLC	1.10	1.10 See Note 2	Report value, may be less than 1.00
Emergency Vehicle Rating	1.20	1.20 See Note 2	1.00 See Note 2

Note 1: Rating factors less than the target minimum acceptable rating factor of 1.20 and greater than 1.00 require a bridge design variance. Rating factors less than 1.00 require a design exception.

Note 2: Rating factors less than the target minimum acceptable rating factor require a design variance.

Refer to **BRSDM** [V1A – Design Criteria] for information on design exceptions and design variances.

The above table applies to all limit states, i.e., Strength, Service and Fatigue. The Fatigue limit state shall be included under the “Design Load Rating Evaluation Level - Inventory” requirements.

The minimum acceptable design, legal, permit and emergency vehicle rating factors for temporary bridges shall be 1.00.

Commentary: For new structure, superstructure replacement and major bridge rehabilitation scope of work categories, target minimum acceptable rating factors of greater than 1.00 are specified to provide reserve capacity to account for any future changes to condition.

The target minimum acceptable rating factor for Permit Load Rating CT-TLC is lower than others since the live load is a result of work under CTDOT construction and maintenance projects and can be managed, controlled and adjusted before it crosses the structure.

NOTICE TO BRIDGE INSPECTORS

As a result of a recommendation in Administration Memorandum No. 80, the Designer shall note on the General Plan any members and components needing special attention, such as fracture critical members, during the inspection of the structure. This information shall be contained in the "Notice to Bridge Inspectors" block.

The "Notice to Bridge Inspectors" block shall be shown on the Semi-Final Design Plans and fully completed on the Final Plans for Review.

Bridge and Roadway Structures Design Manual

Release 1

Volume 2

Structure Design Requirements

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PREFACE AND ABBREVIATED TABLE OF CONTENTS

The ***BRSDM** Volume 2 - LRFD Bridge Design Specifications*, Release 1, (**BRSDM** [V2]) is a supplement to the *AASHTO LRFD Bridge Design Specifications 9th ed.* (**BDS**) and is intended to be used side by side.

BRSDM [V2] contains the following 13 sections:

Detailed Tables of Contents precede each section.

A two-column format is presented with commentary opposite the text it annotates. Sections and Articles mirror those contained in **BDS**.

Only Sections and Articles supplemented by this document are included. Sections and Articles not included within shall be assumed to contain no supplement or revision. Sections added by **CTDOT** contain the suffix CT (e.x. 14.5.2.3CT). **BRSDM** [V2] shall be considered a single document with **BDS**, as supplemented by **BRSDM** [V2], and all references to Sections and Articles herein shall be interpreted as reference to the respective Section or Article in both **BRSDM** [V2] and **BDS**.

BRSDM [V2] is intended for new construction. Designers may use this document for rehabilitation designs at their own discretion. Alternative design methodologies and codes may be required for reference in the rehabilitation of structures when certain limits presented herein are not met.

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SECTION 1

INTRODUCTION

Commentary is opposite the text it annotates.

1.1—SCOPE OF THE SPECIFICATIONS

This section shall be supplemented with the following:

Railway structures shall be designed using **American Railway Engineering and Maintenance-of-Way Association (AREMA)**. These Specifications are not to be used for railway structures.

1.3—DESIGN PHILOSOPHY

1.3.1—General

This section shall be replaced with the following:

This section shall be supplemented with the following:

Bridges shall be designed for specified limit states to achieve the objectives of constructability, safety, and serviceability, with due regard to issues of inspectability, economy, aesthetics, and bridge load rating, as specified in Article 2.5.

Regardless of the type of analysis used, Eq. 1.3.2.-1 shall be satisfied for all specified force effects and combinations thereof.

These requirements shall be met during all phases of construction for all structures affected by the Contract.

The design philosophy can be achieved during construction through iterating construction schemes, construction methods, oversizing components, strengthening existing components, establishing vehicular weight restrictions, specifying structure monitoring systems, or incorporating other design changes into the Contract with the approval of CTDOT.

Designs shall meet additional requirements, if requested by CTDOT, during construction which exceed the requirements of these specifications.

As an example, the Department may have a need to maintain conveyance of Oversize/Overweight vehicles through the work zone during construction.

1.3.3—Ductility

The third paragraph shall be replaced with the following:

For the strength limit state:

$\eta_D \geq$ 1.05 for nonductile components and connections

$=$ 1.00 for all other cases

1.3.4—Redundancy

C1.3.4

The second paragraph shall be replaced with the following:

For the strength limit state:

$\eta_R \geq$ 1.05 for nonredundant members

= 1.00 for foundation elements where ϕ already accounts for redundancy as specified in Article 10.5, and all other cases

This section shall be supplemented with the following:

The design of non-redundant members or components is not permitted, unless approved in writing by the **CTDOT**.

Single-cell box superstructures and single-column bents shall be considered non-redundant.

This section shall be supplemented with the following:

Non-redundant systems such as girder and floor beam bridges should be avoided even though they may have an initial lower cost. The reason for this is the lack of redundancy, fatigue problems, and difficulties involved with future widening associated with these types of structures.

The only situations where non-redundant bridges should be considered are in the case of through-girder or through-truss spans where the minimum depth of the superstructure is critical and the minimum depth of the superstructure cannot be met by an alternative structure type.

The redundancy of members and components is addressed by the system factor described in the **MBE** and included in the load rating of the bridge.

1.3.5—Operational Importance

This section shall be replaced in its entirety with the following:

For the strength limit state:

$\eta_I \geq$ 1.05 for critical or essential bridges

= 1.00 for all other bridges

Critical and Essential Bridges are defined as those bridges that are:

1. On or over the following National Highway Systems (NHS) routes:

a. Eisenhower Interstate System
b. Other NHS Routes
c. Strategic Highway Network (STRAHNET) Routes and Connectors

2. On designated emergency evacuation routes

CTDOT does not make any performance distinction between Critical and Essential bridges.

C1.3.5

This section shall be replaced in its entirety with the following:

CTDOT bridge inspection reports include the following fields:

- NBI 100 – indicates if the inventory route on the bridge is "on" or "not on" a STRAHNET route.
- NBI 104 – indicates if the inventory route on the bridge is "on" or "not on" the NHS.

Information on the STRAHNET system can be found on the following website:

https://www.fhwa.dot.gov/policy/2004cpr/chap18.cfm

A map of the National Highway System in Connecticut may be found on the following website:

https://www.fhwa.dot.gov/planning/national_highway_system/nhs_maps/connecticut/ct_connecticut.pdf

Additional information on the NHS can be found on the following website:

http://www.fhwa.dot.gov/planning/national_highway_system/nhs_maps/

For a list of Critical and Essential bridges, contact **CTDOT** Bridge Management group.

For all other limit states:

η_I = 1.00

SECTION 2: GENERAL DESIGN AND LOCATION FEATURES

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SECTION 2

GENERAL DESIGN AND LOCATION FEATURES

Commentary is opposite the text it annotates.

2.3—LOCATION FEATURES

This section shall be supplemented with the following:

All bridges and culverts shall have bridge identification placards with the five-digit structure number located on or near the structure. The purpose of these signs is for the immediate identification to any Department employees visiting the structure.

For bridges over roadways, an additional bridge identification sign shall be located on or near the substructure for the roadway below.

Per Department *Policy No. P&P-6*, structures over water, in addition to the bridge identification placards, shall include a tributary sign for watercourse identification.

2.3.3—Clearances

2.3.3.3—Highway Horizontal

This section shall be supplemented with the following after the first paragraph:

For highway bridges with exit or entrances ramps, the curb to curb width shall at a minimum match the geometry required for exit and entrance ramps as outlined in the **HDM**.

Larger bridge widths may be necessary to meet sight distance requirements, to facilitate the maintenance of traffic and stage construction requirements or to accommodate standard width structural members.

Any bridge constructed or reconstructed on a State maintained highway with two or more lanes shall have a clear width of roadway not less than 28 feet, exclusive of the width of any sidewalk, unless in the judgement of the Commissioner a lesser width is warranted.

Generally, the curb-to-curb width of pedestrian bridges shall match the approach pathway width.

C2.3.3.3

This section shall be supplemented with the following:

Per Section 13a-86 of the **CGS**. Designers shall coordinate with the Highway Design Engineer for approach roadway geometries as required by the **HDM** and for any potential for future linear projects that may warrant a larger bridge width.

Refer to Section 13 for sidewalk design requirements.

2.3.3.4—Railroad Overpass

This section shall be supplemented with the following:

The determination of the horizontal clearance shall also consider the economics and constructability of the structure, influence of railroad loads on the structure, site conditions, drainage and utility requirements, railroad access and future track expansion.

The minimum vertical clearance for any structure crossing over railroad tracks shall be 20.50 feet, measured from the top of rail to the bottom of structure.

C2.3.3.4

This section shall be supplemented with the following:

The minimum vertical clearance for any structure crossing over railroad tracks is limited by Section 13b-251 of the **CGS**. Designers shall also coordinate with the Highway Design Engineer for vertical clearance requirements based on the roadway type as required by the **HDM**. Early coordination with the Highway Design Engineer, the Railroad Company, and any other party relative to the minimum vertical clearance is strongly recommended.

The minimum vertical clearance for any structure over railroad tracks on which trains are operated by means of overhead electrical wires (electrified tracks) shall be 22.50 feet, measured from the top of the rail to the bottom of the structure.

If the construction work includes only deck replacement (the removal and replacement of the bridge deck and supporting members) or minor widening of the structure, and the existing piers or abutments remain in place, the minimum vertical clearance shall be the structure's existing overhead clearance or 18.50 feet, whichever is greater.

2.3.3.5CT—Structures Adjacent to or Crossing over Roadways

In addition to the requirements in Articles 2.3.3.2 and 2.3.3.3, the minimum horizontal and vertical clearance for any structure adjacent to or crossing over a roadway shall conform to the **HDM**. The provisions of **FHWA's** "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges" should be considered when the clearances specified in the **HDM** cannot be achieved, and a Design Exception is required to allow a lesser clearance.

The lowest portion of a structure mounted sign support shall be a minimum of 12 inches above the lowest component of the fascia member of the bridge to which it is attached.

Refer to **BRSDM** [V4] for vertical clearance requirements of overhead sign supports.

2.3.3.6CT—Structures Crossing over Waterways

Refer to Article 2.3.3.1 for U.S. Coast Guard permit requirements for navigational waterways.

The waterway opening shall be consistent with the hydraulic characteristics of the waterway.

2.3.5CT—Transportation of Structural Members and Components

2.3.5.1CT—General

In general, the length, width, height and weight of a prefabricated structural member or component for use in a highway, pedestrian or railway structure is limited by the ability to ship the item over State highways and bridges.

These physical properties are indirectly limited by the vehicle regulations in the **CGS**. The **CGS** include the following limitations on the dimensions of vehicles using State highways without the need for a permit:

- Vehicle Width (Section 14-262(a)(1)) – The width of a vehicle and combination vehicle and trailer, including its load, is limited to 8'-6", without a permit.
- Vehicle Length (Section 14-262(c)) – The length of the semi-trailer portion of a tractor-trailer unit, including its

load, is limited to 48 feet, without a permit.

- Vehicle Height (Section 14-264) – The height of a vehicle, with its load, is limited to 13’-6", without a permit.
- Vehicle Weight (Section 14-267a(b)(8)) – The gross vehicle weight (weight of vehicle including its load) is limited to 80,000 pounds, on vehicles with a 51 feet wheelbase, without a permit.
- Axle Weights of Vehicles (Section 14-267a) – The axle weights of vehicles vary and are determined by vehicle type and axle spacing.

Section 14-270 of the **CGS** assigns authority to the Commissioner of Transportation to grant permits for vehicles exceeding the limits of the vehicular regulations.

To facilitate construction of the **CTDOT** projects, **Policy Statement HO-10** was developed. It states that the **CTDOT** will grant a permit via an authorized permit route for the transportation of "any structural beam (member or component) that measures 120 feet or less and weighs no greater than 120,000 pounds provided the individual axle weights on the vehicle and trailer transporting the beam (member or component) do not exceed 20,000 pounds." The phrase "structural beam" may be interpreted to mean any structural member or component.

2.3.5.2CT—Design Requirements

The vehicle regulations of the **CGS** and **CTDOT Policy Statement HO-10** establish design guidelines for the length, width, height and weight of prefabricated structural members and components.

To avoid problems associated with transporting materials during construction, prefabricated structural members or components that will require a permit to be transported should be identified early in the design phase.

The maximum member or component shipping length, width, height and weight shall be shown on the Contract plans. For the preliminary submissions, the best available information should be shown on the Contract plans. The actual, as designed, shipping lengths, widths, heights and weights should be shown on the Contract plans for the final submission for review.

The shipping information will be reviewed by the **CTDOT** Oversize and Overweight Permits Section, which will determine if the members are transportable.

If a member exceeds the length and weight limits of **CTDOT Policy Statement HO-10**, the Designer must submit adequate justification with a preliminary submission to **CTDOT**. If sufficient justification exists, the **CTDOT** Office of Engineering will request a waiver of **HO-10** and confirmation that a permit will be granted to transport the member in accordance with Section 14-270 of the **CGS** from the **CTDOT** Oversize and Overweight Permits Section.

If a member, when transported, will exceed the height and width limits of the **CGS**, the Designer must submit adequate justification with a preliminary submission to **CTDOT**. If sufficient justification exists, the **CTDOT** Office of Engineering will request confirmation that a permit will be

C2.3.5.2CT

For additional information, refer to Article 2.3.5.1CT.

granted to transport the member in accordance with Section 14-270 of the **CGS** from the **CTDOT** Oversize and Overweight Permits Section.

The special provision entitled "Section 1.06 – Control of Materials" should be included in all projects. This special provision addresses the shipping of materials in accordance with the **CGS** and the **CTDOT Policy Statement HO-10**.

If a member exceeds the height and width limits of the **CGS** or the length and weight limits of **HO-10**, and the **CTDOT** Oversize and Overweight Permits Section confirms that a permit will be granted in accordance with Section 14-270 of the **CGS** to transport the member, the project's Contract documents should indicate that the **CTDOT** has confirmed with the Oversize/Overweight permit office that the proposed members are eligible to be "Permitted" in accordance with the **CTDOT** Permitting Regulations.

2.5—DESIGN OBJECTIVES

2.5.2—Serviceability

2.5.2.2—Inspectability

This section shall be supplemented with the following:

All bridges shall include features, both off and on the structure, that will make them accessible to bridge inspectors and facilitate the future inspection of the structure.

In addition to the features listed above, features may also include a shelf at the face of the abutment stem, ladder stops on slopes, hand rails and cables, electrical outlets, and any other facility necessary for inspection of the structure.

The features may also include the design and placement of structural members and components (such as generous bridge seats for box girder structures, internal cross frames and bracing in box girders) that allow access for bridge inspectors.

For bridges that are excessively wide, where normal inspection equipment cannot access the interior members, the bridge may require permanent moveable inspection platforms or permanent catwalks.

C2.5.2.2

This section shall be supplemented with the following:

Per Administration Memorandum No. 80.

The Designer shall coordinate with the Bridge Safety and Evaluation Unit for the need and placement of these features for the 60% design submission.

The need for and type of permanent inspection platforms shall be determined by the **CTDOT**.

2.5.2.3—Maintainability

This section shall be supplemented with the following:

Preference shall be given to integral abutments where feasible and jointless structures should be considered for all multiple span bridges.

Provisions for thermal movement of the bridge shall generally be made at or behind the abutments. For bridges on a grade, provisions for thermal movement of the bridge shall generally be made at the high end of the bridge.

Reducing the number of deck joints helps eliminate common areas of maintenance, reducing associated degradation and long-term costs.

2.5.2.5—Utilities

This section shall be supplemented with the following:

To ensure that the structure remains functional and aesthetically pleasing wherever possible, the following Sections will apply to the installation of utilities on structures.

2.5.2.5.1CT—Underground Facilities

Permanent underground installations, which are to be carried on and are parallel to the longitudinal axis of the structure, shall be placed in an out of sight location between the beams. No part of the utility or its supporting structure shall project below the bottom of the bridge superstructure.

In those instances where the proposed superstructure type is not adaptable to carrying utilities in an out-of-site location on the structure, the proposed utility installation shall be the subject of an individual study as to its disposition.

In those instances where an existing structure type is not adaptable to carrying utilities in an out-of-sight location on the structure, the proposed utility installation shall be the subject of an individual study as to its disposition.

Underground facilities shall not be suspended from or attached to the outside face of the superstructure, unless otherwise approved by the **CTDOT**.

Where aesthetics are a prime consideration, the utility shall be placed underground to the extent necessary to preserve the aesthetics of the structure and the surrounding area.

2.5.2.5.2CT—Aerial Facilities

2.5.2.5.2aCT—Aerial Facilities Passing Over Structure

Aerial facilities (telephone, electrical, cable television, etc.) located along a highway that continues onto a structure shall be made an underground installation at the ends of the structure and carried across the structure. These facilities shall be placed in an out of sight location either between the beams or inside of a sidewalk if available. Placing utilities inside of the sidewalk shall be used only for telephone or cable TV. Electric conduits shall in no case be cast inside of a sidewalk as excessive heat buildup may be detrimental to both the utility and the structure.

Where unnecessary expense would be incurred by going underground, facilities carried on support poles may be carried aerially alongside a structure if it is practical to span the entire crossing.

The determination to carry the utilities either aerially or underground shall be studied at an early stage of the design with regards to such factors as economy, aesthetics, safety, and maintaining the characteristics of the local environment.

2.5.2.5.2bCT—Aerial Facilities Passing Under Structure

Aerial facilities (telephone, electrical, cable television, etc.) located along a highway that passes under a structure shall in no instance be permitted to pass over the structure but shall be attached to the underside of it. An underground installation within the structure limits shall be considered. The underground portion of the installation shall extend a distance beyond the limits of the structure(s) required to retain the aesthetics of the structure.

Solutions to special or unusual conditions shall be determined at a field review with **CTDOT** and Utility Company representatives. The field review team shall include the Design Engineer and the Utilities Engineer. In the event that a mutually agreeable solution cannot be reached, the matter shall be forwarded through channels to the Transportation Chief Engineer for a ruling.

2.5.2.5.3CT—Utilities Adjacent to Structure

When underground utilities (existing or proposed) are located in the vicinity of structures, a review of the utility installation relative to the substructure design is required to determine if protection of the substructure is necessary.

The factors to be considered are the type, size, and location of the utility, the pressure in the line, the soil conditions, the material composition of the utility and the structure foundation.

The initial installation and future maintenance of the utility shall be investigated for their effects on the structure.

Utilities passing underneath a permanent structure foundation should be avoided unless approved by **CTDOT**.

If it is determined that protection of the utility is necessary, the following are variations that should be considered:

1. Relocation of the utility.
2. Relocation of the substructure unit.
3. Protection of the substructure unit with sheeting.
4. Sheeting the utility trench.
5. Placing the utility in adequate encasement (sleeves or deflectors).
6. Placing the substructure on piles.
7. Require material composition of the utility to be ductile iron, prestressed concrete or steel (desirable in all cases).
8. Use of shut-off valves on both sides of the bridge (desirable in all cases).

This does not preclude other possibilities, which the Designer or the utility engineer may have to offer.

2.5.2.5.4CT—Emergency Temporary Installations

Temporary installations of an emergency nature may be placed on the sidewalk of a structure, but such installation must be either removed or replaced by a permanent out-of-sight installation within one year of the date of the temporary installation. Where sidewalks are not available, special consideration and study will be required to insure a safe and acceptable placement of the temporary installation.

Upon completion of the temporary installation, immediate steps shall be initiated to ensure that the temporary installation is removed or replaced with the time limit above and in a manner acceptable to the **CTDOT**.

2.5.2.5.5CT—Highway Illumination Poles

If poles for highway illumination are needed within the non-access lines in the proximity of the structure, the location and type of poles shall be approved by the **CTDOT**.

Illumination poles routinely provided by manufacturers are intended to be mounted on fixed, ground mounted foundations. Mounting on non-fixed structures subject to deflection and vibration, such as bridge spans, may result in unacceptable movement or vibration of the pole, possibly resulting in structural failure of the pole or damage to lighting fixture. Therefore, mounting of illumination poles on bridge spans should be avoided whenever possible. If standard illumination poles must be mounted on the span, they should be mounted as near as possible to abutment or piers for spans up to 200 feet. For spans over 200 feet, they should not be mounted over 50 feet from abutment or pier locations. If illumination poles are required in areas outside these limits, they must meet one of the following criteria:

- Be of a non-standard design that has been specifically designed for placement on a moving structure, and be appropriate for the amplitude and frequency of the motion anticipated.
- Be of reduced height (under 30 feet mounting height) and certified by the manufacturer as appropriate for this application.
- Dampening devices may be included as necessary.

2.5.2.5.6CT—High Voltage Transmission Facilities

Long distance high voltage transmission facilities shall be the subject of a special study. Where aesthetics are a consideration, the placement of the facility underground should be considered. The final determination shall be weighed considering both the economics and aesthetics of the location in question. If required, the alternate proposals shall be submitted to **CTDOT** for final determination.

2.5.2.6—Deformations

2.5.2.6.2—Criteria for Deflection

This section shall be supplemented with the following:

For all highway and pedestrian bridges, the criteria for deflection in this section is mandatory.

2.5.2.6.3—Optional Criteria for Span-to-Depth Ratios

This section shall be supplemented with the following:

For all highway and pedestrian bridges, the criteria for span to depth ratios in this section is mandatory.

The span to depth ratio may be waived with written approval by **CTDOT**.

2.5.4—Economy

2.5.4.1—General

This section shall be supplemented with the following:

Member spacing should be maximized in order to reduce the number of members required, thereby reducing the costs for fabrication, shipping, erection and future maintenance. In order to provide redundancy, a minimum of four stringer lines should be used in a bridge cross section.

Preferably, all members in a span shall have the same dimensions to facilitate fabrication and minimize costs. Generally, on multi-span structures, the individual span lengths may vary but the member depth should be constant.

C2.5.4.1

This section shall be supplemented with the following:

Generally, the most economical spacing for rolled beams is between 8 feet and 9.5 feet. It is recommended that the minimum spacing for I-shaped plate girders and top flanges of box girders be kept at 9 feet.

Preferably, structures composed of butted deck units shall be designed with 3’-11½" wide members. Typically, the cost per square foot of deck surface is less for 3’-11½" wide members than it is for 2’-11½" wide members due to high fabrication costs.

2.5.6CT—Bridge Load Ratings

Designers shall take into account the requirements of **BRSDM** [V1B] – Load Rating in their design.

2.6—HYDROLOGY AND HYDRAULICS

2.6.4—Hydraulic Analysis

2.6.4.4—Bridge Foundations

2.6.4.4.1—General

This section shall be supplemented with the following:

Foundations, for both new and existing structures, adjacent to or within waterways subjected to scour shall be designed for changes in foundation conditions resulting from the scour design flood and the scour check flood. Structure foundations shall include bridge foundations, supporting intermediate piers and abutments, foundations for 3-sided frames supporting pedestals and frame legs, and foundations for walls retaining transportation facilities. The design of foundations for scour encompasses both the placement of the foundations and the evaluation of the foundations for changes in conditions due to the scour. The scour design flood, scour check flood, and the changes in foundation conditions resulting from scour shall be determined in accordance with the **DRM**.

C2.6.4.4.1

This section shall be supplemented with the following:

The terms “scour design flood” and “scour check flood” used in this practice are consistent with *Evaluating Scour at Bridges*, 5th Edition, dated April 2012, FHWA-HIF-12-003, Hydraulic Engineering Circular No. 18 (**HEC-18**). The terms “scour design flood” and “scour check flood” are equivalent to the **BDS** terms “design flood for bridge scour” and “check flood for bridge scour” respectively.

Since the terminology used by disciplines and documents varies, for clarification refer to *Technical Brief (TechBrief)* FHWA-HIF-19-060. The *TechBrief* defines terms, differentiates between hydraulic design and scour design and describes the interaction of limits states and scour depths in foundation design within the context of the **BDS**.

This practice separates the design of foundations for scour into 2 parts, foundation support and placement, and foundation evaluation for limit states, to differentiate and clarify requirements.

Scour is determined in accordance with the **DRM**. The manual refers to the use of **HEC-18** for scour design.

Per **DRM** [9.5], scour information is presented in a scour evaluation report (see Section 9, Appendix C). The report may include, but not be limited to, the following: scour depths and limits, flow elevations and flood velocities for each flood event; recommended foundation placement elevations; limits and details for scour countermeasures; and hydraulic analysis assumptions for existing substructure components that remain in place.

Designers should understand that designing structures for scour requires coordination and collaboration between hydraulics, geotechnical and structural engineers. Multiple design iterations may be required before a final solution is developed that meets each discipline's requirements.

Regarding scour at bridges, the bridge abutment shall be defined to include the structure(s) fully or partially supported by the abutment foundation, such as the bridge end, abutment wall, curtain and wing walls, and approach slab, and walls critical to bridge stability. Retaining walls supporting the approach fill side slopes that are not critical to the bridge stability are not considered part of the bridge abutment for scour considerations.

2.6.4.4.2—Bridge Scour

This section, including the title, shall be deleted and replaced with the following:

2.6.4.4.2—Foundation Scour

The greatest scour depth may not occur at the least frequent flood (i.e., greatest discharge) event selected for the scour design flood frequency and scour check flood frequency. Flood events associated with low tail water, occurrence of ice or debris dams, overtopping conditions, waterway confluences, changes in the angle of approach flow due to movement of the channel or any other flood events that can adversely affect the scour depth shall be considered when determining the governing the flood event for the scour design flood and the scour check flood. The term scour design flood is used to designate a flood event, with a magnitude that is less than or equal to the discharge selected for the scour design flood frequency, that will cause the worst-case scour. Similarly, the term scour check flood is used to designate a flood event, with a magnitude that is less than or equal to the discharge selected for the scour check flood frequency, that will cause the worst-case scour.

The potential for stream migration and its effect on scour depth outside of the main channel shall be considered when determining the potential depth of scour at the bridge abutment and interior piers outside of the main channel.

The potential for scour to undermine the channel slope, resulting in a slope failure shall be considered when assessing the potential scour depth, including the potential of the scour-induced slope failure to cause lateral loading on the bridge foundations for the abutment as well as the nearby intermediate pier foundations.

For deep foundations, the effect of the foundation components, such as footing/pile caps, piles, etc., above the

C2.6.4.4.2

This section shall be supplemented with the following:

For additional information for determining scour depth, refer to the **DRM**, **HEC-18** and *TechBrief* FHWA-HIF-19-060.

Low tail water conditions, occurrence of ice or debris dams, overtopping flood conditions, or changes in the angle of approach flow due to movement of the channel tend to be the most problematic with regard to scour depth.

To eliminate the potential errors, the terms scour design flood and scour check flood should be used to designate the governing flood events that will cause the worst-case scour.

Designing foundations scour does not preclude potential damage to highway approaches from flood events. For additional information, refer to **HEC-18**, Section 2.1, Item 4.

If scour induced slope failure of the channel bank and approach fill slope is possible, refer to the **DRM**. The manual refers to the use of the document *Stream Stability at Highway Structures*, 4th edition, *dated 2012,* Hydraulic Engineering Circular (**HEC-20**), FHWA-HIF-12-004, for scour design.

total scour depth shall be included in determining the total scour depth.

For new structures, changes in foundation conditions resulting from scour shall be determined without the benefits provided by scour countermeasures.

For additional information on scour countermeasures, see Article 2.6.4.4.5CT.

The provisions for tsunami-induced scour are contained in the AASHTO Guide Specifications for Bridges Subject to Tsunami Effects.

2.6.4.4.3CT—Foundation Support and Placement

Foundations adjacent to or within waterways subjected to scour shall be supported on piles or drilled shafts, scour resistant rock, or on spread footings founded below (outside) the scour limits. Supporting foundations subjected to scour on piles, drilled shafts or on scour resistant rock is preferred. The use of structural tremies (concrete placed under water) to directly support a foundation without piles is not permitted.

The use of scour countermeasures shall meet Article 2.6.4.4.5CT.

The placement of foundations subject to scour shall consider the type of foundation support, the location of the foundation within the channel, the migration of the thalweg, and the scour due to the scour design flood and scour check flood.

HEC-18, Article 2.3.2, notes that the thalweg of channels can migrate within the bridge opening. Foundation placement shall consider this condition.

The potential for lateral channel migration shall also be considered in determining foundation placement. For abutment and pier foundations located outside the main channel, where there is potential for lateral channel migration and the foundation could end up in the migrated channel, the foundations shall be placed using the scour depth determined for the main channel.

Foundation placement shall meet the requirements of Articles 10.6 and 10.7, as applicable.

Foundation placement shall be adjusted to avoid effects of potential channel slope failure due to scour and to ensure proper embedment of scour countermeasures.

Recommendations for the placement of foundations subject to scour are provided in the scour evaluation report described in the **DRM**.

If the potential effects of lateral channel migration are not specifically addressed by the scour evaluation report, the Designer should confirm with the hydraulics engineer that potential lateral channel migration will not affect the placement of the foundations.

For deep foundations, such as foundations on piles or drilled shafts with and without projecting footings/piles caps, subjected to scour, the bottom of abutment/pier stems (top of footing/pile cap, as applicable) shall be placed below (outside) the scour due to the scour design flood. The scour, in this case, shall be no less than the summation of the long-term degradation and the contraction scour, and include the effect of lateral channel migration, as applicable.

The placement of foundations on piles or drilled shafts (deep foundations) with footings/piles caps is consistent with Section 2.2, Page 2.6, Item 6 entitled “For Deep Foundations (Drilled Shaft and Driven Piles) With Footings or Caps” of **HEC-18**.

The practice also addresses foundations on piles or drilled shafts (deep foundations) without projecting footings/piles caps, such as integral abutments. This foundation type is not specifically addressed by **HEC-18**. The placement of the bottom of the stem is consistent with foundations on piles or drilled shafts with footings/piles caps.

For shallow foundations on spread footings, supported on soil or erodible rock, subjected to scour, the top of the footing shall be placed below the total scour due to the scour design flood and the scour check flood.

The placement of shallow foundations on spread footings is more conservative than the requirements in Section 2.2, Page 2.4, Item 1 entitled “Spread Footings on Soils – Piers” and Item 2 “Spread Footings on Soils – Abutments” in **HEC-18**.

Shallow foundations supported on scour resistant rock shall be designed, detailed, and constructed to maintain the integrity of the supporting rock. The bottom of the foundation shall be at or below the top of scour resistant material.

For bridge abutments supported on a spread footings that are reliant for support on a retaining wall subject to scour, the bottom of retaining wall stem (top of footing/pile cap, as applicable) shall be placed below (outside) the total scour due to the scour design flood and the scour check flood regardless of whether the foundation is a deep foundation or a shallow foundation.

For recommendations on the placement of foundations for abutments and piers entirely outside the total scour limits, coordinate with hydraulic and geotechnical engineers.

The final foundation placement and details shall be determined in coordination with a multi-discipline group of structural, hydraulic, and geotechnical engineers. The foundation placement and details shall be clearly specified in the contract documents and shall meet the recommendations included the final scour evaluation report, the final hydraulics report and final geotechnical report.

Figures 2.6.4.4.3-1CT, 2.6.4.4.3-2CT, 2.6.4.4.3-3CT, and 2.6.4.4.3-4CT provide guidance on the bottom of abutment/pier stems (top of footing/pile cap, as applicable) placement.

For additional information for foundations supported on scour resistant rock, refer to **HEC-18**, Section 2.2, Page 2.5, Item 3 entitled “Spread Footings on Rock Highly Resistant to Scour”.

Placing the bottom of retaining wall stem (top of footing/pile cap, as applicable) below (outside) the total scour ensures that the soil behind the stem will not be scoured and unavailable to support the abutment.

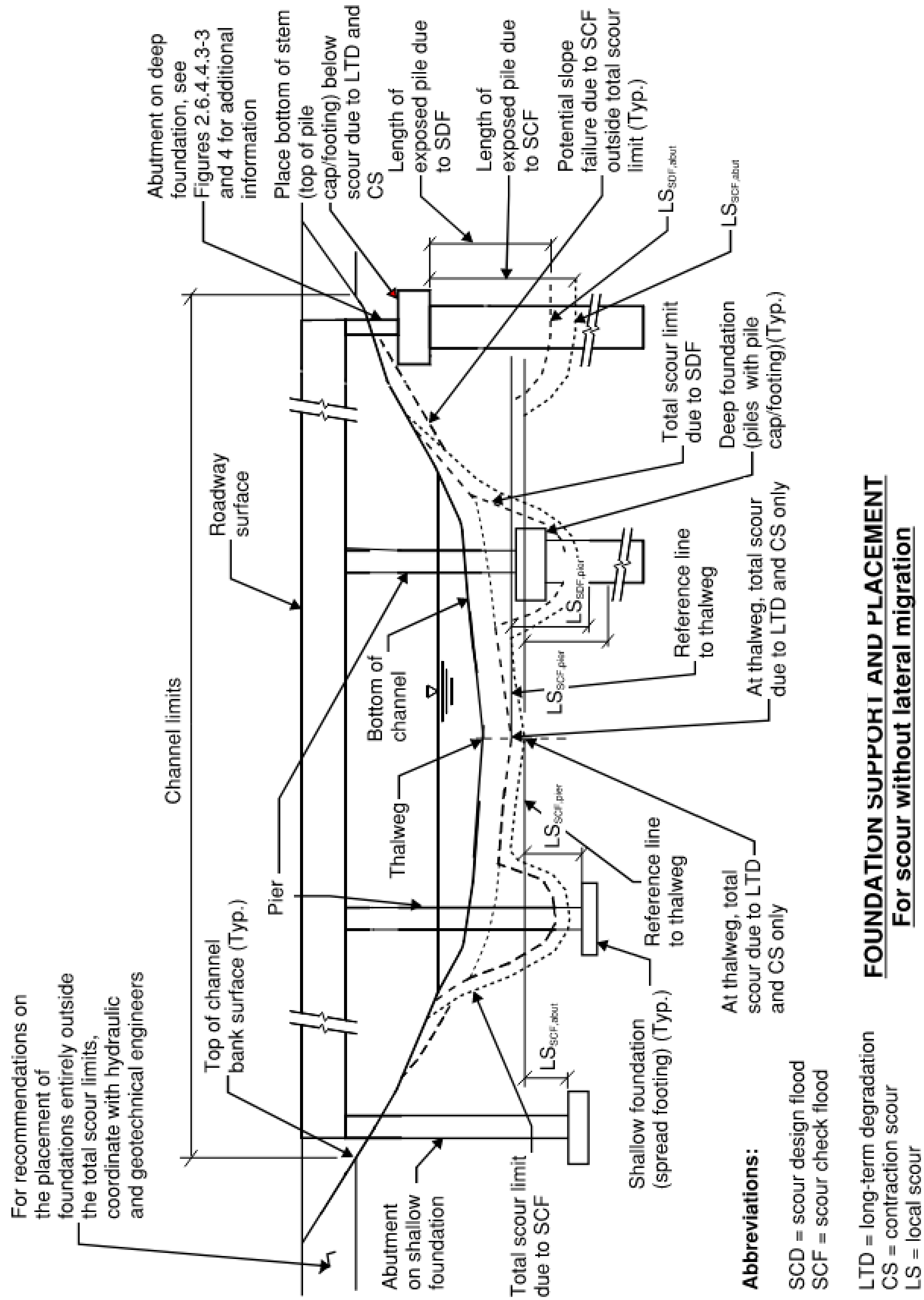

Figure 2.6.4.4.3-1CT—Foundation Support and Placement for Scour Without Lateral Migration

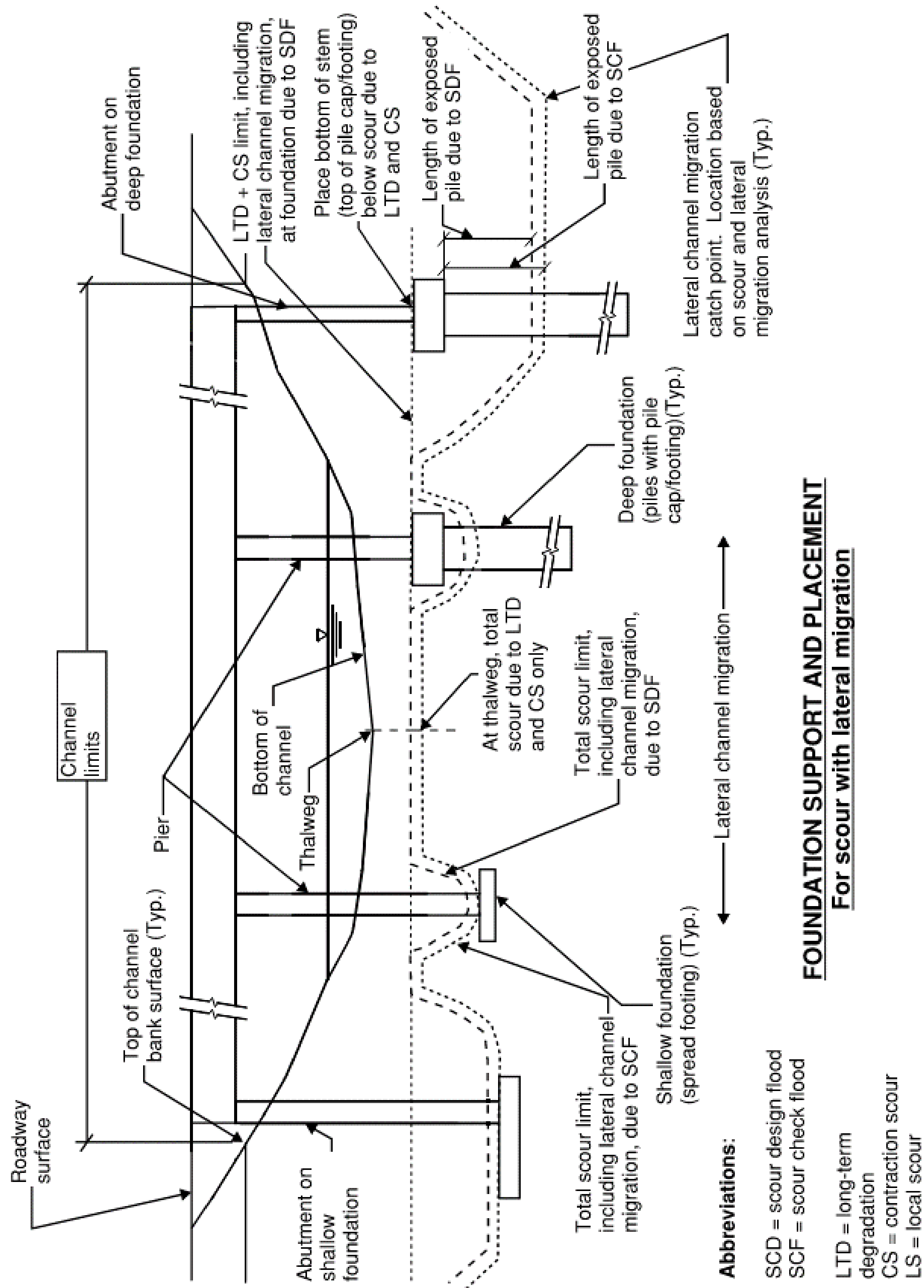

Figure 2.6.4.4.3-2CT—Foundation Support and Placement for Scour With Lateral Migration

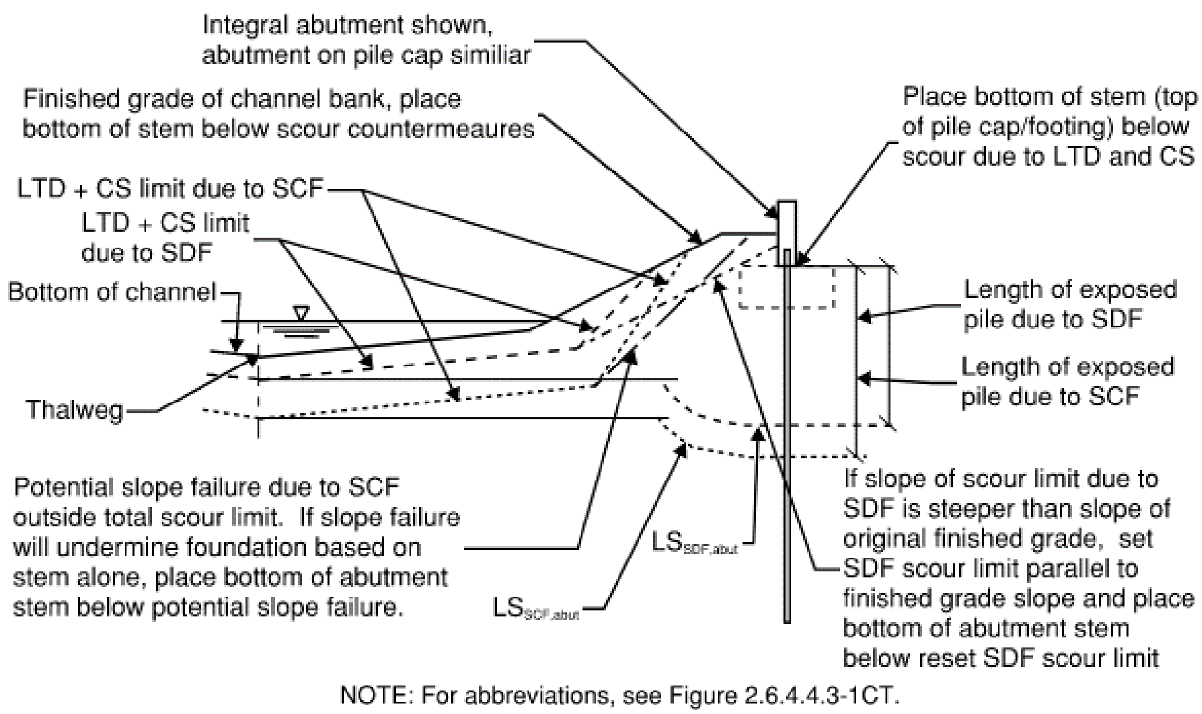

Figure 2.6.4.4.3-3CT—Foundation Support and Placement for Embankment Abutment on Deep Foundation and Scour Without Lateral Migration

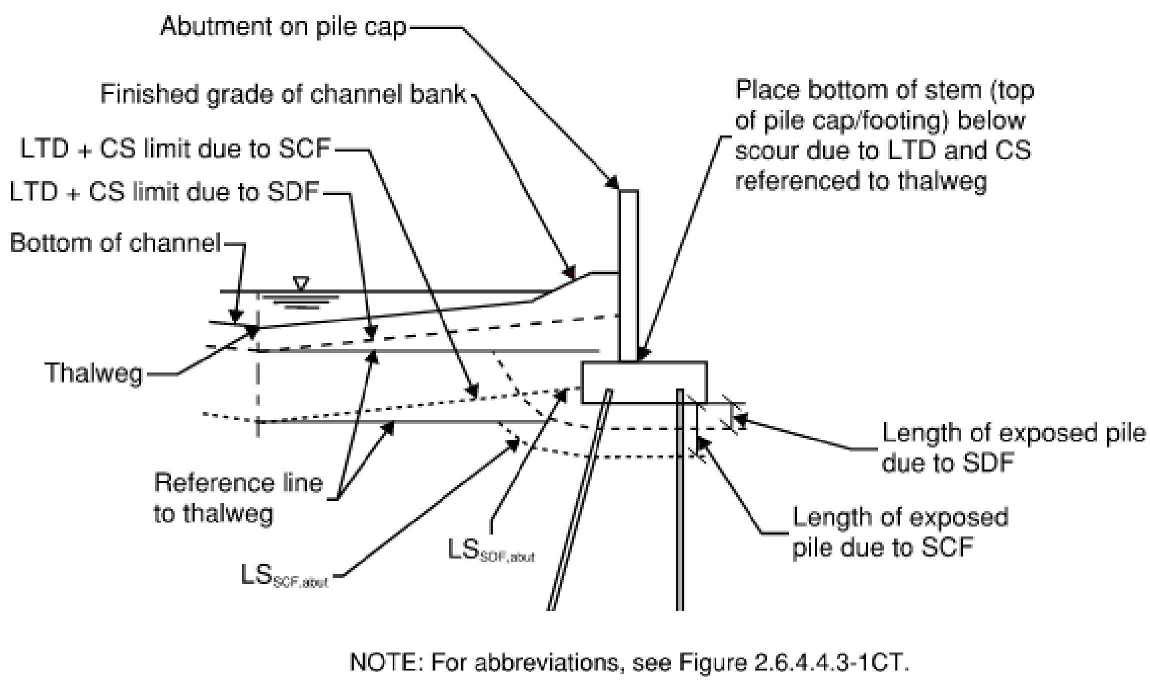

Figure 2.6.4.4.3-4CT—Foundation Support and Placement for Full Height Abutment on Deep Foundation and Scour Without Lateral Migration

2.6.4.4.4CT—Foundation Evaluation for Limit States

Foundations subject to scour shall meet and satisfy the applicable limit states for conditions with and without scour, and with aggradation.

Foundations shall be evaluated for the following conditions and satisfy the following limit states:

- For changes in foundation conditions resulting from a scour design flood, both during and after the flood event, bridges and walls shall be evaluated and satisfy the Strength, Service and Extreme Event limit states in Table 2.6.4.4.4-1CT assuming all the streambed material above the combined component scour has been removed and is unavailable for foundation support.
- For changes in foundation conditions resulting from a scour check flood, both during and after the flood event, bridges and walls shall be evaluated and satisfy the Extreme Event III limit state in Table 2.6.4.4.4-1CT assuming all the streambed material above the combined component scour has been removed and is unavailable for foundation support.
- For changes in foundation conditions resulting the yearly mean discharge flood event, bridges and walls shall be evaluated and satisfy the Extreme Event II limit state in Table 2.6.4.4.4-1CT assuming all the streambed material above the combined component scour has been removed and is unavailable for foundation support.

C2.6.4.4.4CT

For Extreme Event I limit state, the 100%-100%-0% contribution from long-term degradation, contraction scour, and local scour, respectively addresses the following:

- A conservative assumption that a 975-year seismic event can occur near the end of an anticipated 75-year service life of a bridge when the full magnitude of long-term degradation is attained.
- The full magnitude of contraction scour would occur under the first 100-year flood event and retain the calculated value thereafter.
- 0% local scour assumes the holes refill shortly after the scour design flood event and is not considered probable in conjunction with 100% long-term degradation, 100% contraction scour, and a design seismic event.

The scour combination for Extreme Event II limit state for CV has been adapted from the *AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges* (2009).

Table 2.6.4.4.4-1CT – Combination of Scour Components for Different Limit States (in %)

Limit State	Long-term Degradation	Contraction Scour	Local Scour	Flood
Service	100	100	100	Scour Design Flood
Strength	100	100	100	Scour Design Flood
Extreme Event I	100	100	0	Scour Design Flood
Extreme Event II, Case A1 for CV	50	50	50	Scour Design Flood
Extreme Event II, Case A2 for IC	50	50	50	Scour Design Flood
Extreme Event II, Case B for CV	50	0	0	Yearly mean discharge
Extreme Event III	100	100	100	Scour Check Flood

For the design of bridges for scour conditions, the operational importance shall be independent of the bridge's classification (critical/essential/typical) for the strength limit state. The factor related to operational importance, η_I , shall be taken as 1.05.

For the evaluation with scour conditions, at abutments, walls, and other structures subject to earth load effects, designers shall assume the earth may both remain and not remain in contact with and act upon the rear face of the

Since the revised factor related to operational importance affects the strength limit state, it only applies to changes in foundation conditions resulting from a scour design flood.

Scour conditions may result in the removal of only a portion of the streambed or adjacent embankment/backfill at a foundation. Load effects on foundations from all possible scour conditions shall be investigated. At the drilled stems, footings/pile caps and drilled shafts/piles. Load cases shall include both balanced and unbalanced loading conditions.

shafts/piles soil arching should be considered.

The design vehicle live load shall include dynamic load allowance for load cases where the foundation components are no longer surrounded by soil because of scour.

The design vehicle live load is defined by Article 3.6.1.2.

The requirement for applying the dynamic load allowance to the design vehicle live load assumes that foundation components may no longer be surrounded by soil because of scour.

The load effects of water shall be based on the elevations and velocities associated with the scour design flood and scour check flood.

Deep foundation design, for assessing overburden stress used for bearing resistance calculations, shall consider the effect of the lost soil due to scour as shown in Figure 2.6.4.4.4-1CT. Similarly, for shallow foundations (i.e., spread footings) located below Point C in the figure, the overburden stress used for assessing bearing resistance after scour should be calculated as shown in this figure.

Shown in Figure 2.6.4.4.4-1CT is a simplification that can be used to calculate the overburden stress needed for foundation bearing resistance calculations for Service, Strength, and Extreme Event limit states. If a more accurate estimate of overburden stresses is needed, complex three-dimensional modeling would be required, and such modeling may be considered for use in foundation design subject to owner approval.

Additional design requirements for deep foundations with regard to scour are provided in Articles 10.5.5.3.2 and 10.7.3.6, plus commentary.

Whenever total scour depth exposes deep foundation elements, the foundation evaluation shall also consider the potential for damage due to erosion, debris impacts, wood borers, corrosion from exposure to stream currents, or other environmental effects.

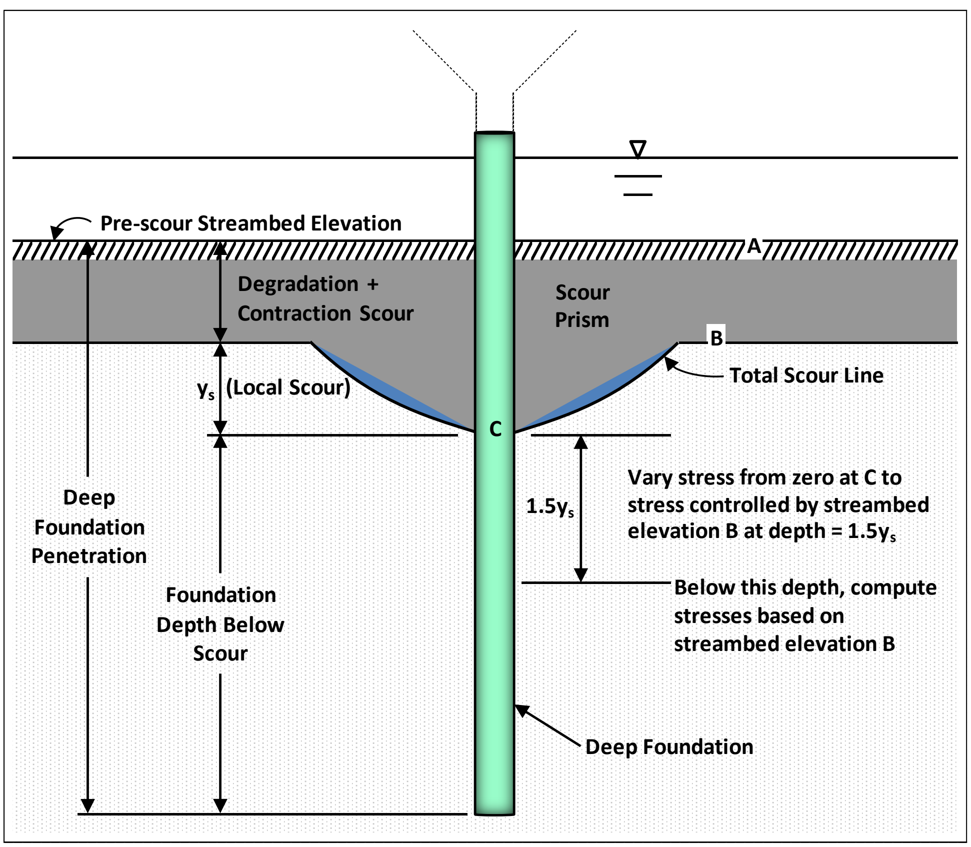

Figure 2.6.4.4.4-1CT Illustration of Scour Prism and its Effects on Deep Foundations (adapted FHWA-NHI-18-024, GEC10 – Drilled Shaft Manual)

2.6.4.4.5CT—Scour Countermeasures

Scour countermeasures shall meet the requirements of the **DRM**.

C2.6.4.4.4CT

The limits, placement and details of scour countermeasures shall meet the requirements of the **DRM**. The manual refers to the use of the document *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance*, 3rd Edition, dated September 2009, FHWA-NHI-09-112, Hydraulic Engineering Circular No. 23 (**HEC-23**) for additional guidance on countermeasure selection and design. For additional information is also included in the *Technical Brief (TechBrief)* FHWA-HIF-19-007.

For new bridges and new walls retaining highways, changes in foundation conditions resulting from a scour design flood and scour check flood shall be determined without the benefits provided by scour countermeasures.

For new bridges and new walls retaining highways, the use of revetments, armoring, permanent steel sheet piling, or permanent cofferdams, to reduce scour impact and foundation depths may only be permitted if the placement of the foundations to meet Article 2.6.4.4.3CT is proven not constructable due to the project constraints.

For existing bridges and walls retaining highways undergoing rehabilitation where the existing substructure and walls will be retained, if the existing foundations do not meet the design criteria for new bridges and new walls retaining highways, scour countermeasures may be used to mitigate scour.

When fendering or other pier protection systems are used, their effect on pier scour and collection of debris shall be taken into consideration in the design. Since scour prediction equations are not available for this scenario, the structural, hydraulic, and geotechnical aspects of the design, based on advanced modeling, local experience and engineering judgment, shall be coordinated and differences resolved prior to implementation of fendering and other pier protection methodologies.

Advanced three-dimensional modeling may be needed to assess the effect of fendering or other pier protection systems on scour.

2.6.4.4.6CT—Existing Substructure Components

For bridge replacement projects with foundations adjacent to or within waterways, the extent of the removal of existing substructure components and elements, either wholly or partially, must be addressed during the design phase. The existing substructure components and elements may include abutment stems, wall stems, pier stems, footings, pile caps, and piles.

Considerations for leaving existing substructure components and elements, either wholly or partially, in place include efforts to minimize construction costs, facilitate water handling, simplify construction, reduce construction duration, limit hydraulic affects, or reduce environmental impacts. These efforts are constrained by the need to ensure that leaving existing substructure components and elements in place can be accurately reflected in both the hydraulic and scour analysis, will be environmentally permittable, and any future changes in the position of the components will not exacerbate conditions due to any event that result in an unacceptable hydraulic condition, scour conditions worse than the original design, or have a negative environmental impact.

The removal limits of existing substructure components and elements shall be coordinated, developed, and justified with a multi-discipline group of structural, hydraulic, geotechnical and environmental engineers along with representatives from the **CTDOT** Office of Environmental Planning. The removal limits shall be clearly specified in the contract documents and shall agree with the final scour evaluation report, the final hydraulics report, and the environmental permits.

New work shall be independent of the existing component parts that remain in place and shall not rely on existing component parts to provide any structural benefit.

2.6.4.5—Roadway Approaches to Bridge

This section shall be supplemented with the following:

Retaining walls shall be designed for scour as specified in Articles 11.6.3.4, 11.7.2.3, 11.10.1, and 11.10.2.2.

C2.6.4.5

This section shall be deleted and replaced with the following:

Highway embankments on floodplains serve to redirect overbank flow, causing it to flow generally parallel to the embankment and return to the main channel at the bridge. Roadway embankment and retaining wall designs should include countermeasures where necessary to limit damage caused by overbank flow parallel to the embankment. Such countermeasures may include:

- relief bridges, culverts, or other structural openings,
- retarding the velocity of the overbank flow by promoting growth of trees and shrubs on the floodplain and highway embankment within the highway right-of-way or constructing small dikes along the highway embankment,
- protecting fill slopes subject to erosive velocities by use of riprap or other erosion protection materials on highway fills and spill-through abutments, and
- where overbank flow is large, utilize guide banks to protect abutments of main channel and relief bridges from turbulence and resulting scour.

Additional information and design guidelines on scour countermeasures are provided in **HEC-23**.

Although overtopping may result in failure of the embankment, this consequence is preferred to failure of the bridge. The low point of the overtopping section should not be located immediately adjacent to the bridge, because its failure at this location could cause damage to the bridge abutment. If the low point of the overtopping section must be located close to the abutment, due to geometric constraints, the scouring effect of the overtopping flow should be considered in the design of the abutment. Design studies for overtopping should also include evaluation of any flood hazards created by changes to existing flood flow patterns or by flow concentrations in the vicinity of developed properties.

Bridge approach embankment slopes exposed to scour should be protected with properly designed scour countermeasures designed in accordance with **HEC-23** where possible, considering any regulatory requirements.

The risk of bridge approach fill failure due to scour may be an acceptable risk as the approach fill typically can be replaced quickly to restore access to the bridge crossing. The impact of such approach fill loss to bridge approach structures such as wing walls, bridge approach slabs, and small (i.e., short in height and length) retaining walls that support the approach embankment side slopes will need to be considered. This is especially important if significant stream channel migration risk is not low, as much more of the embankment could be affected, or, as illustrated in **HEC-18**, Figure 8.7(c), the bridge abutment could become like an intermediate bridge pier with regard to increased scour depth due to local and contraction scour.

The length of bridge approach embankment or wall relative to the bridge abutment location that can be affected by scour, and how deep the scour is likely to occur, will depend on several factors, including the length of the approach embankment within the floodplain and the potential for stream migration. For the portion of the approach retaining wall up on the flood plain (i.e., outside the main channel), scour due to long-term degradation is no longer applicable, and only contraction scour and local scour should be considered to locate the wall footing or wall base.

2.6.6—Roadway Drainage

2.6.6.1—General

This section shall be supplemented with the following after the second sentence of the first paragraph:

Generally, deck cross slopes in both the travel lanes and the shoulders of highway bridges shall conform to the roadway cross slopes found in **HDM**. Mechanical screeds, used when placing cast-in-place concrete decks, can accommodate multiple cross slope breaks. On bridges with precast components, such as precast adjacent box beams, the structural slab, or bituminous concrete overlay, may be placed to match the approach roadway cross section.

This section shall be supplemented with the following:

The profile for highway bridges shall match the approach roadway.

Wherever possible, surface drainage should be handled with roadway catch basins located before and after the bridge. When it is not possible to handle all surface drainage off the bridge, the entire deck drainage system shall be designed to be as maintenance free as possible.

Wherever possible, drainage routes shall be short and direct, and abrupt changes in direction shall be avoided.

Pipes and troughs shall be sized to handle design flows, and slopes shall be maximized. Clean-outs shall be provided at strategic locations to simplify maintenance of the system.

Where pollution of streams, ponds and water supply areas may be a factor, further consideration is required.

2.6.6.3—Type, Size, and Number of Drains

This section shall be supplemented with the following:

Adequate facilities shall be provided to collect and dispose of the surface water on the structure by means of suitable scuppers, troughs and leaders where required by drainage requirements.

The FHWA document **HEC-21** "Design of Bridge Deck Drainage" or its successor shall be used.

Direct discharge from scuppers, etc., is preferable over waterways, and all other areas where the discharge will not be detrimental to the area below.

2.6.6.3.1CT—Scuppers

Scuppers are to be located to avoid long runs of pipe/trough and shall be designed to satisfy the structural and drainage requirements.

Fiberglass or polyethylene are the preferred materials to be used for scuppers and downspouts.

Scuppers shall be designed in accordance with the provisions of the latest edition of the **DRM**.

2.6.6.3.2CT—Drainage Piping

Drainage piping shall have a minimum diameter of 8 inches.

For aesthetic reason, leaders shall be located on the inside face of the fascia girders, on the rear face of piers, and recessed into the front face of abutments.

Piping shall not be located within concrete pier columns.

Piping should not be used where runs are required with an angle of less than 30 degrees to the horizontal.

2.6.6.3.3CT—Open Troughs

Where piping is not appropriate, open troughs shall be used.

Troughs shall be of adequate depth and shall have a self-cleaning pitch of 3% or greater to carry the discharge and minimize the possibility of spilling or clogging. Provisions must be made to contain the splashing where scuppers spill into the troughs.

2.6.6.3.4CT—Underground Pipe

Underground drainage beyond the bridge piping must conform to the requirements of the **DRM** and shall be included in the roadway items.

2.8—REFERENCES

This section shall be supplemented with the following:

AASHTO. *Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges,* 2nd ed. American Association of State Highway and Transportation Officials, Washington, DC, 2009.

AASHTO. *Guide Specifications for Bridges Subject to Tsunami Effects*. American Association of State Highway and Transportation Officials, Washington, DC, 2022.

CGA. *Connecticut General Statutes*. Connecticut General Assembly, Hartford, CT, 2024.

CTDOT. *CTDOT Drainage Manual*. Connecticut Department of Transportation, Newington, CT, 2000.

CTDOT. *Connecticut Highway Design Manual*. Connecticut Department of Transportation, Newington, CT, 2023.

CTDOT. *Policy on Structural Beams*. Policy Statement HO-10. Connecticut Department of Transportation, Newington, CT, 2019.

FHWA. *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance,* 3rd ed. Hydraulic Engineering Circular No. 23. FHWA-NHI-09-111. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2009.

FHWA. *Design of Bridge Deck Drainage*. Hydraulic Engineering Circular No. 21. FHWA-SA-92-010. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1992.

FHWA. *Hydraulic Considerations for Shallow Abutment Foundations*, Technical Brief. FHWA-HIF-19-007. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2018.

FHWA. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*. FHWA-PD-96-001. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1995.

FHWA. *Scour Considerations withing AASHTO LRFD Design Specifications*, Technical Brief. FHWA-HIF-19-060. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2021

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TABLE OF CONTENTS

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SECTION 3

LOADS AND LOAD FACTORS

Commentary is opposite the text it annotates.

3.4—LOAD FACTORS AND COMBINATIONS

3.4.1—Load Factors and Load Combinations

Revise the 7th bullet and insert a new 8th bullet as follows:

- Extreme Event II—Load combination relating to ice load, blast load, and collision by vessels and vehicles.
- Extreme Event III- Load combination for design considering the effects of check flood scour condition.

C3.4.1

Revise the 4th and 5th bullets as follows:

- Although these limit states include water loads, *WA*, the effects due to *WA* are considerably less significant than the effects of changes to foundation condition due to scour. Article 2.6.4.4.4CT addresses the effects of scour combined with extreme event limit states.
- The joint probability of *BL*, *EQ*, *CT*, *CV*, and *IC* is extremely low, and, therefore, the events are specified to be applied separately. Under these extreme conditions, the structure may undergo considerable inelastic deformation by which locked-in force effects due to *TU*, *TG*, *CR*, *SH*, and *SE* are expected to be relieved.

Supplement Extreme Event Limit State Commentary as follows:

Design for the scour check flood has been included in the Extreme Event III limit state to highlight the loads that will act on the bridge during such events. Furthermore, conditions of the foundations under scour check flood are evaluated to consider any reduction in geotechnical resistance and stiffness due to scour.

Revise Table 3.4.1-1 as follows:

Table 3.4.1-1 – Load Combination and Load Factors – Revised Extreme Event Limit States for Scour

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time				
										EQ	*BL*	*IC*	*CT*	*CV*
Extreme Event I [1]	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II [1]	1.00	0.50/1.00	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Extreme Event II, Case A1 [1]	1.00	1.00	1.00	—	—	1.00	—	—	—	—	—	—	—	1.00
Extreme Event II, Case A2 [1]	1.00	1.00	1.00	—	—	1.00	—	—	—	—	—	1.00	—	—
Extreme Event II, Case B [2]	1.00	1.00	1.00	—	—	1.00	—	—	—	—	—	—	—	1.00
Extreme Event III [3]	1.00	1.00	1.00	—	—	1.00	—	—	—	—	—	—	—	—

Notes: [1] - Flood event = Scour Design Flood
[2] - Flood event = Yearly mean discharge
[3] - Flood event = Scour Check Flood

This section shall be supplemented with the following:

This section shall be supplemented with the following:

Changes in foundation conditions resulting from a scour check flood shall be considered at the Extreme Event III limit state. The load factor *LL*, *IM*, *CE*, *BR*, *PL*, and *LS* is specified as 1.00 to ensure that the bridge can remain operational until the extent of any damage can be evaluated and repaired. PennDOT has a similar requirement. Consider a case where a flood event damages the highway approach to a multi-span bridge and scours material around abutment and pier foundations to a depth greater than the depth calculated for a scour design flood but less than a depth calculated for a scour check flood. By quickly rebuilding the highway approach the bridge can be opened for at least limited use by a design vehicle live load until the extent of any damage at the abutments and piers can be evaluated and repaired.

For permanent bridges, the load factor for design vehicle live load for the Extreme Event I limit state (γ_{EQ}) shall be 0.50.

For temporary works, such as jacking devices, falsework, and shoring, the load factor for design vehicle live load for the Strength I limit state shall be 1.35.

For temporary bridges that will be in service less than 3 years, the load factor for design vehicle live load for the Strength I limit state shall be 1.35. For temporary bridges that will be in service less than 3 years, the load factor for design vehicle live load for the Extreme Event I limit state shall be 0.0.

Temporary bridges shall be designed at the operating level for the design vehicle live load. The Designer shall contact the **CTDOT** Oversize and Overweight Permits Section for the use of permits on temporary bridges.

3.4.2—Load Factors for Construction Loads

3.4.2.1—Evaluation at the Strength Limit State

Paragraph four of this section shall be supplemented with the following:

The load factor for wind (*WS*) for the Strength Load Combination III during construction shall not be less than 1.0.

3.5—PERMANENT LOADS

3.5.1—Dead Loads: *DC*, *DW*, and *EV*

This section shall be supplemented with the following:

Concrete unit weights shall be increased by 0.005 kcf for calculation of dead loads when reinforced with mild steel or prestressing.

Table 3.5.1-1 shall be amended as follows:

Material		Unit Weight (kcf)
Compacted Sand, Gravel, Silt and Clay		0.125
Bituminous Concrete		0.155
Concrete	Lightweight	0.120

This section shall be supplemented with the following:

All vehicular bridges shall be designed to account for the load effects due to a bituminous concrete/HMA overlay, including bridges detailed to be constructed without an initial overlay. Designers should estimate pavement thicknesses on the structure and design per estimation.

C3.5.1

This section shall be supplemented with the following:

The unit weight for "Compacted sand, gravel, silt and clay" includes typical items like Pervious Structure Backfill, Granular Fill, Subbase, and in-situ soils.

"Bituminous concrete overlay, HMA overlays" unit weight accounts for increased weight of aggregate typically used in CT.

For minimum overlay thickness, refer to Article 9.7.7.2CT. For additional overlay thickness information, refer to Article 9.7.7CT.

3.6—LIVE LOADS

3.6.1—Gravity Loads: *LL* and *PL*

3.6.1.2—Design Vehicular Live Load

3.6.1.2.1—General

This section shall be supplemented with the following:

The design vehicular live load, during all phases of construction for both new and existing permanent bridges, as well as temporary bridges, shall be in accordance with this section.

Live load effects shall be considered in the design of temporary works, such as jacking devices, falsework and shoring, as required for construction.

Adjustments to the live load requirements for bridges under construction and for temporary bridges shall be coordinated with both the Principal Engineer in the Division of Bridges and the Oversize/Overweight (OS/OW) Permit Office within the Bureau of Highway Operations.

C3.6.1.2.1

This section shall be supplemented with the following:

The **BDS** and the LRFR are design specifications and rating specifications respectively. Since not all bridge components are subject to load rating, vehicular bridges should be designed by the **BDS** and evaluated in accordance with **BRSDM** [V1B-Load Ratings], dependent on the type of work. The final load rating shall be in accordance with **BLRM**.

Adjustments to live load requirements for bridges under construction must be coordinated with multiple offices within **CTDOT** to ensure structural adequacy of the bridges for the vehicles required to use them.

3.6.1.3—Application of Design Vehicular Live Loads

3.6.1.3.2—Loading for Optional Live Load Deflection Evaluation

This section shall be supplemented with the following:

For all highway and pedestrian bridges, the criteria for deflection of this section is mandatory.

3.7—WATER LOADS: *WA*

3.7.5—Change in Foundations Due to Limit State for Scour

This section shall be deleted in its entirety.

C3.7.5

This section shall be deleted in its entirety.

3.8—WIND LOADS: *WL* AND *WS*

3.8.1—Horizontal Wind Load

3.8.1.2—Wind Load on Structures: *WS*

3.8.1.2.5CT—Wind Loads During Construction

All bridges, both new and existing, shall be evaluated for wind loads during all phases of construction.

Evaluations shall meet the requirements of the AASHTO Guide Specifications for Wind Loads on Bridges During Construction (**GSWLB**) for the following:

C3.8.1.2.5CT

The requirements have been provided to highlight the applicable specifications for designers and mandate the use of the guide specifications. The determination and application of wind loads conforms to the **BDS**, **GSWLB** and the **GSBTW**

- For new bridges and the rehabilitation of existing bridges requiring new superstructures, evaluations shall consider construction that occurs from the initial erection of structural members up to the time the deck is completely placed and fully cured
- For existing bridges undergoing rehabilitation, evaluations shall consider construction that occurs as various structural components and elements are wholly or partially removed resulting in changes to the load effects on the remaining components and ability of the existing components, with a condition other than new, to resist the applied load effects, including removed materials and construction equipment.

GSWLB Article C1 describes wind loads that act during construction as those that act "during the erection of the girders up to the time of placement of the deck". The requirement provided clarifies the state of bridge construction.

Evaluations for wind loads that act on bridges with decks fully placed and completely cured shall be determined in accordance with the **BDS**.

For temporary bridge works, such as temporary support towers, used during construction, evaluations for wind loads transmitted from the superstructure to the temporary components shall be determined in accordance with the preceding requirements.

Evaluations for wind loads on temporary bridge works shall meet the requirements of the AASHTO Guide Design Specifications for Bridge Temporary Works (**GSBTW**).

Wind speeds during construction to be used in the evaluations shall be determined by the designer.

The requirement assigns designers the responsibility for determining the wind speed. This requirement differs from that used for new traffic structures. Since new traffic structures may be constructed under district wide or statewide projects in various locations, one wind speed is specified for all new sign structures throughout the state.

Wind speeds during construction are based on the work zone classifications: active work zones and inactive work zones. The **GSWLB** defines active and inactive work zones relative to whether or not construction activities being performed during a time period. Active work zones include the periods of time when workers are on site and performing construction activities for beam/girder erection (from the picking of the first member to the completion of the framing for the construction stage/phase), form and work platform placement, placement of reinforcement and deck concrete placement, and placement of prefabricated deck panels. Inactive work zones include the periods of time from the erection and setting of the first member until the deck is completely placed and fully cured when no construction is being performed.

The requirements clarify that state of the bridge for active and inactive work zones.

For evaluations of structures during active work zone time periods the wind speeds are lower since higher winds may preclude the ability of workers to perform construction activities safely and successfully.

Wind speeds during inactive work zone time periods are greater than those during active work zone periods.

The wind speed for active work zones shall be no less than the value specified in the **GSWLB** unless a higher wind speed is determined by the designer. Designers shall obtain the concurrence from the Construction District administering the project to use a higher wind speed.

The requirement makes the designer responsible for revising the wind speed in active work zones.

In determining the wind speed reduction factor, for inactive work zones the construction duration shall be the time period from the erection and setting of the first member until all members in the stage/phase are composite with the deck (i.e. the deck concrete has fully cured). The minimum construction duration used for determining the wind speed reduction factor shall be greater than 1 year.

A minimum construction duration has been specified to account for potential construction delays.

Components and elements requiring evaluation for wind loads during construction may include structural members, bracing and cross frames both temporary and permanent, and the permanent substructure. All evaluations shall conservatively consider worst case load effects and the least structural resistance offered by the partially completed structure until the deck is fully placed and completely cured. Evaluations of active work zones shall include the load effects from worker live load, materials, stored materials and debris, material placement and equipment associated with the construction activities. Evaluations of inactive work zones shall include the load effects from materials, including stored materials and debris, and equipment in place during that time period.

Designers are responsible for evaluating both new and existing bridges for wind loads for all applicable limit states during all stages/phases of construction to ensure structural adequacy and the feasibility of construction. Superstructure framing members erected in the stages/phases as shown on the plans shall be evaluated for inactive work zone time periods. The permanent modifications of the member components and cross frame spacing is the preferred method be to meet applicable limit states and lateral deflection limits. Lateral bracing shall only be used where necessary. The feasibility of construction shall be documented on the plans with details and narratives of a workable erection scheme and construction sequence, and a viable demolition and removal procedure as required by the scope of the work. Maximum wind loads and wind load reduction factors for active and inactive work zones, assumed construction dead loads, and assumed construction live loads shall be shown on the plans.

Contractors are responsible for evaluating all fabricated members during handling, storage, shipping, and erection. Contractor shall prepare and submit working erection plans for review. The submittal shall include all details (including temporary bracing), instructions, calculations, and equipment and product data to document all work meets all applicable limit states and lateral deflection limits. Wind loads and wind load reduction factors for active and inactive work zones, construction dead loads, and construction live loads shall be documented in the submittal. The design and detailing of temporary bracing shall be included in the submittal.

For steel I-shaped members, the lateral deflection limit is L/150, where L equals the span length in feet.

3.10—EARTHQUAKE EFFECTS: *EQ*

3.10.1—General

This section shall be supplemented with the following:

All conventional bridges, new and existing bridges being rehabilitated, shall be designed for earthquake load effects in accordance with this section, except as noted below.

C3.10.1

This section shall be supplemented with the following:

Conventional bridges have slab, beam, girder, box-girder, deck unit, and truss superstructures; have single or multiple column piers, wall type piers or pile bents; and are founded on shallow or piled footings or shafts.

- All existing conventional bridges in Seismic Performance Zone 1 and undergoing major rehabilitation shall be analyzed and designed for earthquake load effects, unless specifically waived by **CTDOT**.
- All existing conventional bridges in other than Seismic Performance Zone 1 and undergoing major rehabilitation shall be analyzed for earthquake load effects. Bridge specific direction on how to address the results of the analysis will be provided by **CTDOT**.
- All existing bridges undergoing minor rehabilitation need not be analyzed or designed for earthquake load effects.

Refer to **BRSDM** [V1] for the limits of major rehabilitation.

Refer to **BRSDM** [V1] for the limits of minor rehabilitation.

All non-conventional bridges, new and existing bridges being rehabilitated, shall be designed for earthquake load effects as directed by **CTDOT**.

The need to design retaining walls for the Extreme Event I limit state shall be determined in accordance with Article 11.5.4.2.

The fourth paragraph shall be supplemented with the following:

All buried structures described and listed in Article C12.5.1, including end walls, wing walls and head walls, associated with box culverts and buried structures, need not be analyzed or designed for earthquake load effects, except where they cross active faults.

Non-conventional bridges include bridges with cable stayed, cable suspended superstructures, bridges with truss towers or hollow piers for substructures and arch bridges.

3.11—EARTH PRESSURE: *EH*, *ES*, *LS*, AND *DD*

3.11.1—General

This section shall be supplemented with the following:

For Pervious Structure Backfill, the effective angle of internal friction shall be taken as equal to 35 degrees.

3.11.6—Surcharge Loads: *ES* and *LS*

3.11.6.4—Live Load Surcharge (*LS*)

This section shall be supplemented with the following:

All permanent earth retaining structures shall be designed for a minimum surcharge loading equivalent to 2 feet of soil.

3.11.9CT—Unbalanced Loads

Tunnels, integral abutments, frames, arches, and culverts will require special consideration in the design and sequence of backfilling in order to prevent cracking, instability , and deflections due to unbalanced loading.

Backfilling requirements shall be shown on the plans to mitigate the load effects due to unbalanced backfilling.

C3.11.9CT

This list of structures is not a fully inclusive list. The Designer shall use reasonable judgement when determining the need for analysis of unbalanced backfill loading for structures not included within this list.

For permanent structures designed and built during construction, backfill requirements should be included in the contract documents.

3.12—FORCE EFFECTS DUE TO SUPERIMPOSED DEFORMATIONS: *TU, TG, SH, CR, SE, PS*

3.12.2—Uniform Temperature

3.12.2.1—Temperature Range for Procedure A

Table 3.12.2.1-1 shall be replaced with the following:

The temperature range used for the calculation of thermal movement for integral abutments shall be 150ºF. This range is based on a mean low temperature of -30ºF and a mean high temperature of +120ºF.

The temperature range used for the calculation of thermal movement for all other structures shall be 120ºF. This range is based on a mean low temperature of -10ºF and a mean high temperature of +110ºF. The median temperature for design of bearings and bridge joints shall be +50ºF.

3.12.2.3—Design Thermal Movement

This section shall be supplemented with the following:

The coefficient for thermal expansion (α) shall be taken as $6.4x10^{-6}$ in./in/ºF.

C3.12.2.3

This section shall be supplemented with the following:

This equates to approximately 2.75-inch total movement for a 300 foot long bridge.

3.14—VESSEL COLLISION: *CV*

3.14.1—General

This section shall be supplemented with the following:

Evaluation of the following vessel collision events shall be combined with foundation conditions due to scour.

- Case A - A drifting empty barge breaking loose from its moorings and striking the bridge.
- Case B - A ship or barge tow striking the bridge while transiting the navigation channel under typical waterway conditions.

C3.14.1

The 13^{th} paragraph and bulleted list that follows shall be deleted in their entirety.

3.17CT—CONSTRUCTION LOADS

This section shall be supplemented with the following:

All bridges shall be designed to account for construction loads and their effects to ensure the adequacy of the structure during all phases of construction. Construction loads, including dynamic effects, assumed in the design shall be shown on the contract documents. These loads include but are not limited to crane loads, storage loads, and paving and milling trains.

Construction live load shall be no less than 0.050 ksf.

Refer to **SSC** for limitations on construction vehicles and additional loads.

3.16—REFERENCES

This section shall be supplemented with the following:

CTDOT. *Bridge Load Rating Manual*, Version 2018.1.0. Connecticut Department of Transportation, Newington, CT, 2018.

CTDOT. Standard Specifications for Roads, Bridges, Facilities and Incidental Construction Form 818. Connecticut Department of Transportation, Newington, CT, 2020.

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STRUCTURAL ANALYSIS AND EVALUATION

Commentary is opposite the text it annotates.

4.4—ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS

This section shall be supplemented with the following:

All new, existing and temporary bridges, both final and during all phases of construction, shall be analyzed in accordance with this section.

Bridges with straight members and one or more supports skewed greater than 30 degrees, bridges with horizontally curved girders, and other complex framing shall be analyzed using refined methods in accordance with Article 4.6.3.

These requirements for the methods of analysis are consistent with the **BLRM**.

4.6—STATIC ANALYSIS

4.6.2—Approximate Methods of Analysis

4.6.2.2—Beam-Slab Bridges

4.6.2.2.1—Application

This section shall be supplemented with the following:

Members should be laid out parallel, include the same section properties and be uniformly spaced as much as practical. If this is unavoidable, the live load distributions factors outlined in this section shall not be used. The Designer should carefully investigate these situations to account for the variation in live load and member stiffness.

4.6.2.2.1aCT—Distribution of Dead Loads on Butted Prestressed Box Members with Varying I

If prestressed box members with different moments of inertia are used in the same superstructure, the dead loads and pedestrian live load applied after the members are interconnected, shall be distributed in proportion to each member's moment of inertia according to the following:

$$DL_k = DL_{Total}\left(\frac{I_k}{\sum_{i=1}^{n} I_i}\right) \qquad (4.6.2.2.1aCT\text{-}1)$$

where:

DL_k = dead load of member "k"
DL_{Total} = total dead loads, excluding member weight, applied the superstructure, such as deck, sidewalks, railings, parapets, overlays, etc.
I_k = moment of inertia of member "k"
$\sum_{i=1}^{n} I_i$ = total moment of inertia of all members

4.6.3—Refined Methods of Analysis

4.6.3.3—Beam-Slab Bridges

4.6.3.3.3—Curved Steel Bridges

This section shall be supplemented with the following:

Designers must investigate all temporary and permanent loading conditions, including load from wet concrete in the deck pour, including all stages in the deck pouring sequence, for all stages of construction. Future re-decking must also be considered as a separate loading condition.

Diaphragms and cross-frames must be designed as full load carrying members.

C4.6.3.3.3

This section shall be supplemented with the following:

A three-dimensional analysis representing the structure as a whole and as it will exist during all intermediate stages and under all loading conditions is essential to accurately predict stresses and deflections in all girders and diaphragms and must be performed by the Designer.

Further design information for curved structures is contained in the AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges and the FHWA Manual for Refined Analysis in Bridge Design and Evaluation.

4.9—REFERENCES

This section shall be supplemented with the following:

AASHTO. *AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges*. American Association of State Highway and Transportation Officials, Washington, DC, 2003.

CTDOT. *Bridge Load Rating Manual*, Version 2018.1.0. Connecticut Department of Transportation, Newington, CT, 2018.

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SECTION 5

CONCRETE STRUCTURES

Commentary is opposite the text it annotates.

5.1—SCOPE

This section shall be supplemented by the following:

Cast-in-place, non-reinforced concrete members and components are not permitted, except for the use of underwater concrete for cofferdam seals. A cofferdam seal shall not be considered a structural member or component.

5.4—MATERIAL PROPERTIES

5.4.2—Normal Weight and Lightweight Concrete

5.4.2.1—Compressive Strength

The fourth paragraph shall be supplemented with the following:

The use of lightweight concrete shall be on a case by case basis, but its use shall generally be avoided.

This section shall be supplemented with the following:

Cast-in-place footings, abutments, walls, steps and copings shall have a minimum concrete compressive strength of 3.0 ksi.

Cast-in-place pier columns, pier caps and their associated keeper blocks and bearing pedestals, slabs, barrier walls including footings, parapets and sidewalks shall have a minimum concrete compressive strength of 4.0 ksi.

The design of precast, non-prestressed members shall be based on a minimum concrete compressive strength (f'c) of not less than 5.0 ksi.

C5.4.2.1

This section shall be supplemented with the following:

These categories of components also include leveling pads, pile caps, wing walls, retaining walls, endwalls, sills, nosings, concrete bearing pedestals, cheekwalls, keeper blocks and curbs. Refer to **BRSDM** [V1B] and **CTDOT Bridge Plan Notes Guide** for PCC classifications.

Slabs include approach slabs, bridge deck slabs including haunches, backwalls integral with deck, and concrete aprons on grade. Refer to **BRSDM** [V1B] and **CTDOT Bridge Plan Notes Guide** for PCC classifications.

5.4.2.4—Modulus of Elasticity

The definition of w_c shall be supplemented with the following:

For reinforced concrete or prestressed concrete, the unit weight of normal weight concrete for the calculation of the Modulus of Elasticity shall be 0.145kcf.

C5.4.2.4

For dead load calculations, the unit weight for reinforced concrete or prestressed concrete shall still use the values in Table 3.5.1-1.

5.4.3—Reinforcing Steel

5.4.3.1—General

The first paragraph shall be replaced by the following:

All reinforcing bars shall conform to the requirements of ASTM A615, Grade 60.

All reinforcing bars be shall galvanized. Galvanized reinforcing bars shall be galvanized, after fabrication, to the requirements of ASTM A767, Class 1, including supplemental requirements.

Other reinforcement material and coatings may only be specified with the approval of **CTDOT**.

Epoxy Coated reinforcing bars shall be epoxy coated to the requirements of ASTM D3963.

Stainless steel bar reinforcement shall conform to the requirements of ASTM A955.

Wire and welded plain steel wire fabric shall conform to the requirements of the **SSC**.

Deformed steel wire and welded deformed steel wire fabric shall conform to the requirements of the **SSC**.

Other types of reinforcing bars, deformed wire, cold-drawn wire, and deformed or plain welded wire reinforcement shall conform to the material standards as specified in **CON** [9.2].

5.4.7CT—Ultra High Performance Concrete (UHPC)

Ultra High Performance Concrete (UHPC) shall conform to the requirements of the Owned Special Provision "Ultra High Performance Concrete" or other applicable special provisions on a per project basis as approved by the Department. The use of UHPC on State projects for non-prequalified uses shall be approved by the Department.

5.5—LIMIT STATES AND DESIGN METHODOLOGIES

5.5.1—General

5.5.1.2—Design Methodologies

This section shall be supplemented with the following:

The composite section used for computing live load stresses shall also be used for computing stresses induced by composite dead loads.

The modular ratio for composite design shall be computed based on the modulus of elasticity of the concrete deck and the modulus of elasticity of the prestressed concrete member.

5.6—DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS—B REGIONS

5.6.3—Flexural Members

5.6.3.5—Deformations

5.6.3.5.2—Deflection and Camber

The first paragraph shall be supplemented with the following:

Growth in the modulus of elasticity shall also be included.

Geometry of members needs to be considered. NEXT Beams shall use a time-dependent analysis.

The *PCI Bridge Design Manual* does not recommend the use of the multipliers specified in Section 5.6.3.5.2 when using the multiplier method for NEXT Beams.

Designers are cautioned that some software calculates camber for prestressed concrete members using the multiplier method only.

5.6.7 – Control of Cracking by Distribution of Reinforcement

The following shall be included after paragraph 1:

The reinforcement spacing to control cracking shall be based on Class 2 exposure.

5.9—PRESTRESSING

5.9.1—General Design Considerations

5.9.1.1—General

The following shall be included after paragraph 1:

Prestressing strands may be either draped or de-bonded to reduce the tensile stresses at the member ends. Mixing draped and debonded strands in a member is permitted.

For main members, the strand diameter shall be 0.6 inches. Smaller strand diameters may be used for smaller components such as precast deck panels or other applicable Special Provisions on a per project basis as approved by the Department.

5.9.1.2—Design Concrete Strengths

This section shall be supplemented with the following:

The design concrete compressive strength for prestressed concrete members shall be between 6.0 ksi to 10.0 ksi.

The design concrete compressive strength at release, f'_{ci}, shall be between 0.70 f'_c to 0.80 f'_c but no greater than 6.5 ksi.

C5.9.1.2

This section shall be supplemented with the following:

The design concrete compressive strength limits are based on the range of values used by PCI Northeast for the NEXT Beam span table.

The design concrete compressive strength limits and maximum limits are similar to values specified by surrounding Northeast states.

5.9.1.6—Tendons with Angle Points or Curves

This section shall be supplemented with the following:

The total hold down force of all draped strands for each member shall not exceed 75% of the total weight of the member.

5.9.2—Stress Limitations

5.9.2.3—Stress Limits for Concrete

5.9.2.3.1—For Temporary Stresses before Losses

5.9.2.3.1b—Tensile Stresses

The following shall be included after paragraph 1:

The tensile stress limits for concrete in prestressed superstructure members at the Service III limit state after losses, in other than segmentally constructed bridges, shall be based on the members being subjected to severe corrosive conditions.

C5.9.2.3.1b

Connecticut bridges are subjected to de-icing materials. The tensile stress limits associated with severe corrosive conditions have been selected with the goal of maximizing the service life of prestressed structural members.

The rating factors at the Service III limit state, for design and legal vehicles specified by the **BLRM**, shall meet the requirements of **BRSDM** [V1B].

5.9.4—Details for Pretensioning

This section shall be supplemented by the following:

Prestressing strands shall be tensioned to the stresses listed in Table 5.9.2.2-1.

When possible, all members in a span shall have the same number of strands, prestressing force and distance to the center of gravity of the strands to facilitate fabrication and minimize costs.

The following information shall be shown on the plans:

- The ultimate tensile strength of the strands
- The jacking force per strand
- The number of strands and location
- The center of gravity of strands
- The strand diameter
- De-bonding locations (if required), and
- The approximate location of drape points (if required)

5.9.4.3—Development of Pretensioning Strand

5.9.4.3.3—Debonded Strands

Paragraph A of this section shall be supplemented with the following:

No more than 25% of the total number of strands may be debonded.

Paragraph E of this section shall be supplemented with the following:

Diagonally adjacent strands may be debonded.

5.10—REINFORCEMENT

This section shall be supplemented with the following:

The minimum size bar shall be #4, unless otherwise authorized. For compressive and tension forces, the minimum reinforcement bar size shall be #5 with a maximum spacing of 12 inches.

The Designer shall reduce bar spacing to increase bar area before increasing bar size.

5.10.1—Concrete Cover

This section shall be supplemented with the following:

Requirements of this section shall apply to mechanical couplers.

For precast concrete box culverts, the minimum concrete cover over all reinforcement on any surface of the roof, floor and walls shall be 2 inches. In structures exposed to salt water, the minimum concrete cover over all reinforcement on any surface of the roof, floor and walls shall be 3 inches.

For cast-in-place concrete box culverts, the minimum cover over all reinforcement shall be 2 inches, except the cover over the outer reinforcement in the floor slab shall be 3 inches. The cover over all reinforcement in structures exposed to salt water shall be 4 inches.

Cover shall be measured from the inside of the relief when form liners or surface treatments are used.

5.10.3—Spacing of Reinforcement

This section shall be supplemented with the following:

Spacing shall be equally spaced, or in whole inch increments. Designers shall take into account ease of placement when specifying rebar placement. Bar spacing shall be placed to better align with temperature and shrinkage reinforcement.

Bar lengths shall be specified in 1-inch increments.

C5.10.3

This section shall be supplemented with the following:

It is desirable for the spacing of bar reinforcement to be in 6-inch increments for ease of placement and inspection. For bridge decks, columns, pier caps, and other similar components, it may not be practical for such spacing.

5.10.8—Development and Splices of Reinforcement

5.10.8.3—Development by Mechanical Anchorages

This section shall be supplemented with the following:

Mechanical connectors and anchorages used to splice galvanized reinforcement shall be galvanized.

C5.10.8.3

This section shall be supplemented with the following:

Designers shall pay careful attention to cover for mechanical connectors as they are larger than the rebar. Mechanical connectors and anchorages shall be measured for payment by the number of connectors or anchorages installed and accepted.

5.10.8.4—Splices of Bar Reinforcement

5.10.8.4.1—Detailing

The following shall be inserted prior to paragraph 1:

The maximum length of reinforcing bars shall be 40 feet. Where longer bars are required, splices shall be detailed.

5.10.9CT—Shear Reinforcement for Composite Action

The shear reinforcement used in the design of the members should be used to achieve composite action with the deck. Additional reinforcement may be added if the area of shear reinforcement is not sufficient to produce composite action. There is no need to extend all shear reinforcement into the deck if it is not required for composite action.

Shear reinforcement used for composite action shall be extended into the concrete deck. In deck unit members, the reinforcement shall be fabricated from one bar and have two loops that extend into the deck. In bulb tee members, the reinforcement shall be terminated with a 90 degree hook.

The top surface of the members shall be roughened with a raked finish to assist in composite action. The finish shall be shown on the Contract plans.

For reinforcement in deck haunches, refer to Article 9.7.1.1.1CT.

5.11—SEISMIC DESIGN AND DETAILS

5.11.1—General

This section shall be supplemented with the following:

All structures shall include restraint devices or connections, such as keeper blocks, bearings or dowels, designed to transfer seismic forces from the superstructure to the substructure.

The design and detailing of the restraint devices or connections shall account for thermal movement of the structure.

5.11.1.1CT—Transverse Seismic Restraint

On structures composed of prestressed concrete beams, supported by seat type abutments, the superstructure shall be restrained transversely by a keeper block placed between the center members at abutments. If necessary, multiple keeper blocks may be used at each abutment to resist the forces.

At piers supporting members with a continuous deck, the superstructure shall be restrained with dowels projecting from the pier into the full height diaphragm.

At piers supporting members with a discontinuous deck, the superstructure shall be restrained transversely by a keeper block placed between the center members at abutments.

On structures composed of butted deck units, the superstructure shall be restrained transversely by cheekwalls located at each end of the abutments and piers. Semi-integral backwall behind the abutment or box beams extending into keys in the backwall may be used in lieu of cheekwalls.

5.11.1.2CT—Longitudinal Seismic Restraint

On structures composed of bulb tee girders or spread deck units, the superstructure shall be restrained longitudinally by keeper blocks placed behind the end of each member at abutments after their erection.

On structures composed of butted deck units, the superstructure shall be restrained longitudinally by a backwall placed behind the ends of the members at the abutments after their erection.

5.12—PROVISIONS FOR STRUCTURE COMPONENTS AND TYPES

This section shall be supplemented with the following:

Cast-in-place, non-reinforced concrete members and components are not permitted.

The use of cast-in-place reinforced concrete is acceptable for all types of members and components. Generally, cast-in-place concrete is used for substructure components, bridge decks and parapets. Cast-in-place concrete may be used for superstructures when it is found to be economical and feasible.

5.12.1—Deck Slabs

This section shall be supplemented with the following:

Deck joints should be eliminated wherever possible. The number of deck joints over piers should be minimized on multiple span structures by using continuous decks.

5.12.1.1CT—Continuous Decks Supported by Simple Spans

On multi-span structures composed of simple spans, the decks shall be made continuous over the piers with no positive moment connection, wherever practical. The supporting members shall be designed as simple spans.

On structures composed of bulb tee girders or spread deck units, the deck shall be placed continuous over a full height diaphragm. The diaphragm shall be placed at piers between the ends of the members in adjacent spans and extend transversely between the parallel members.

On structures composed of butted deck units, the ends of the members shall be connected with a "T-shaped" closure pour.

C5.12.1.1CT

National Cooperative Highway Research Program (NCHRP) Report Number 322 "Design of Simple-Span Precast Prestressed Bridge Girders Made Continuous" suggests that consideration should be given to the design of jointless bridges (that is, members with a continuous slab with no moment connection), since there is little or no structural advantage to designing for live load continuity.

5.12.2—Slab Superstructures

5.12.2.3—Precast Deck Bridges

5.12.2.3.1—General

This section shall be supplemented with the following:

Structures composed of spread deck units require a composite concrete deck. The deck shall be detailed to match the roadway cross section. The members shall be placed plumb.

On structures composed of butted deck units, the members may be placed on either a straight (level or sloped) or broken cross section alignment. The alignment of the members need not match roadway cross section. The bituminous concrete overlay shall be placed to match the roadway cross section.

The maximum allowable skew angle for butted deck units is 45 degrees.

On structures composed of butted deck units, the members shall be placed at a nominal spacing to provide a gap between the adjacent members that accommodates the sweep of the members. The 2’-11½" wide members should be nominally spaced at 3 feet. The 3’-11½ " wide members should be nominally spaced at 4 feet.

In addition to the other requirements specified in Article 2.5.2.5, the following shall apply:

- On structures composed of spread deck units, utilities may be placed between adjacent members.
- On structures composed of butted deck units, utilities may be placed between two members in a utility bay located under a sidewalk. Under no circumstances will utilities be permitted to be located inside deck units.

Precast, non-prestressed superstructure members supporting vehicular traffic are not permitted.

C5.12.2.3.1

This section shall be supplemented with the following:

The nominal spacings were determined by increasing the actual member width to a convenient value. The spacings have not been set at the maximum allowable sweep since it varies with the span length. If the actual sweep of the members will not allow the members to be placed at the nominal spacing shown, the members should be butted up to and placed parallel with the adjacent member.

5.12.2.3.1aCT—Standard Members

Precast, prestressed concrete deck units used in superstructures are generally limited to those available from area fabricators. Available member types include solid slabs, voided slabs and box beams.

Precast, prestressed concrete deck units shall be detailed without holes of post-tensioning.

C5.12.2.3.1aCT

Prestressed deck units are precast pretensioned rectangular sections with or without voids. Sections with circular voids are referred to as voided slabs and sections with rectangular voids are referred to as box beams. Sections without voids are referred to as solid slabs.

5.12.2.3.1bCT—Modifications to Standard Members

Prestressed deck units may be modified to facilitate the placement of reinforcement that extends from the tops of members for components such as parapets and sidewalks or to accommodate anchors for temporary traffic barrier.

The circular and rectangular voids in deck units may be reduced in size or removed for placement of the reinforcement. Generally, the voids shall be placed symmetrically about the vertical axis of the member. The Designer shall calculate the section properties for the modified sections.

Spread deck units shall be detailed without shear keys and holes for post-tensioned transverse strands.

The fascia members of structures of butted deck units shall be detailed without a shear key at the outside face, unless provisions are being made for a future widening.

5.12.2.3.3—Shear-Flexure Transfer Joints

5.12.2.3.3a—General

This section shall be supplemented with the following:

Post-tensioning is prohibited for joining precast longitudinal components.

5.12.2.3.3c—Post-Tensioning

This section shall be replaced with the following:

Post-tensioning is prohibited for joining precast longitudinal components.

5.12.2.3.3f—Structural Overlay

This section shall be supplemented with the following:

Where the roadway vertical geometry will allow it, a 6 inch minimum thickness concrete deck slab shall be cast on top of the prestressed deck units, designed to provide shear transfer between beams, and eliminate the need for transverse post-tensioning.

Shear keys of adequate width shall be shown between adjacent members. Transverse reinforcement from the deck unit shall protrude from the side of the deck unit and be developed within the shear key to assist in distributing loads between beams. An attempt shall be made to design and detail the deck unit width and protruding transverse reinforcement to fit within a form width of 4 feet.

5.12.3—Beams and Girders

5.12.3.1—General

This section shall be supplemented with the following:

Structures composed of bulb tee girders require a composite concrete deck. The deck shall be detailed to match the roadway cross section. The members shall be placed plumb.

Utilities may be placed between adjacent members.

Precast, non-prestressed superstructure members supporting vehicular traffic are not permitted.

All structural members in contact with and supporting a concrete deck shall be designed for composite action unless otherwise noted in these specifications. The members shall be designed assuming construction without shoring (unshored construction).

5.12.3.1.1CT—Standard Members

Precast, prestressed concrete beams and girders used in superstructures are generally limited to those available from area fabricators. Available member types include Northeast Bulbtee (NEBT) girders, Northeast Deck Bulb Tee (NEDBT) girders, Northeast Extreme Tee (NEXT) beams and Precast Concrete Economical Fabrication (PCEF) girders.

Prestressed concrete bulb tee superstructure designs should consider both the Northeast Bulbtee (NEBT) and the Precast Concrete Economical Fabrication (PCEF) beams with dimensions nominally equivalent to the NEBT dimensions. The prestressing shall be designed based on the NEBT dimensions and section properties.

C5.12.3.1.1CT

For the latest listing of area fabricators and the bridge members produced, refer to the PCI Northeast website, www.pcine.org.

5.12.3.1.2CT—Modifications to Standard Members

NEBT girders shall not be altered or modified, except as follows:

- The top flange at the girder ends may be clipped to minimize the bridge seat widths at abutments.
- Minor variations of NEBT girders are allowed for equivalent PCEF sections.

5.12.3.2—Precast Beams

5.12.3.2.1—Preservice Conditions

This section shall be supplemented with the following:

Multi-span structures composed of continuous spans shall be designed with field spliced post-tensioned bulb tee girders. The bulb tee girders shall be pretensioned to control cracking during shipping and handling. The pretensioning of the girders shall be accounted for in the final design. Field splices in the members should be made near points of low dead load moment.

5.12.4—Diaphragms

This section shall be supplemented with the following:

In this article, the term “diaphragm” is used throughout for simplicity. Steel diaphragms may be either a single member or consist of multiple members, i.e., a cross frame. Cross frames are typically used with deep members or used to accommodate utilities.

Structures composed of discretely spaced pretensioned concrete members with a concrete deck shall have external end diaphragms and may require external intermediate diaphragms.

C5.12.4

This section shall be supplemented with the following:

External end diaphragms are always required and shall be placed between members at all abutments and piers. End diaphragms shall be placed over and aligned with the centerline of each bearing line.

External end diaphragms are required to provide alignment and stability for construction, support discontinuous decks, resist and transfer transverse loads effects, and encase the ends of members to provide corrosion protection.

External intermediate diaphragms shall be placed as follows, unless analysis indicates a more frequent placement (smaller spacing) is required:

- For spread box beam deck units
  - No intermediate diaphragms are required for spans up to 65ft.
  - One intermediate diaphragm is required at midspan in spans over 65ft and up to 100ft.
  - In pretensioned box beam members an internal diaphragm shall be provided at each external diaphragm connection.

Since it is not anticipated that spread box beam members will be used in spans over 100 ft., requirements for placement of external intermediate diaphragms are not provided.

Internal diaphragms allow the use of dowels bars for the bolting of rolled sections or weldments to the web of the member.

- NEXT beams
  - External intermediate diaphragms are not required.

Provision matches PCINE recommendations.

- Bulb-tee girders
  - Intermediate diaphragms are required at midspan for span lengths less than 80ft, at third points for span lengths between 80ft and 120ft, and quarter points for span lengths greater than 120ft.

The requirements meet past **CTDOT** practice and match PCINE recommendations for deck bulb tee girders.

- Deck bulb tee girders –
  - Intermediate diaphragms are required at midspan for span lengths less than 80ft, at third points for span lengths between 80ft and 120ft, and quarter points for span lengths greater than 120ft.

The requirements match PCINE recommendations.

On bridges skewed less than or equal to 20 degrees, the intermediate diaphragms shall be placed in line along the skew. On bridges skewed more than 20 degrees, intermediate diaphragms shall be placed normal to the main members and be staggered, not placed in a line, across the width of the bridge.

The skew angle requirements are similar to the **BDS** requirements for steel I-girder members.

The need for additional external intermediate diaphragms shall be investigated for all stages of construction and the final condition.

External end diaphragms shall be comprised of structural steel encased in reinforced concrete, except for the end diaphragms of NEXT beams which shall be entirely reinforced concrete. The end diaphragm concrete shall be structurally monolithic with the deck and encase the end of the pretensioned concrete members. A construction joint between the diaphragm and bottom of the deck is optional. The diaphragm concrete shall extend to the bottom of the bottom flange of the member. At semi-integral abutments, conventional abutments and piers, the bottom of the diaphragm shall be shaped to allow for jacking the structure for bearing replacement. Horizontal reinforcement shall be placed continuous through the webs at the ends of the members. Horizontal reinforcement shall also be placed continuous across the ends of the members. Vertical stirrups shall be placed to contain the horizontal reinforcement. At piers, the use of continuity diaphragms to eliminate deck joints is preferred.

The structural steel member in the end diaphragm provides for alignment and stability for construction.

The concrete encasement of the end diaphragm provides support of the discontinuous deck above and additional corrosion protection of the end diaphragms. The concrete encasement of the member end provides corrosion protection of the prestressing strand ends.

The concrete encasement requirement is automatically met at fully integral abutments and semi-integral abutments.

Where continuity diaphragms are used, the structure may be designed as continuous or a series of simple spans. Refer to the Article 5.12.3.3 for additional information.

External intermediate diaphragms shall be comprised of structural steel. The diaphragms shall be as deep as practical. The use of external intermediate diaphragms comprised entirely of reinforced concrete are not permitted unless allowed by an approved design variance.

To facilitate erection, external intermediate diaphragms of structural steel are preferred.

Single member diaphragms shall consist of structural steel W shapes, MC shapes, or bent plates. Diaphragms with multiple members shall consist of structural steel W or MC shapes with angles, or all angles. The diaphragms shall be connected to WT shape rolled sections or T-shape weldments that are attached to the webs of the pretensioned concrete members with threaded anchorages and headed high-strength bolts. Threaded anchorages shall consist of dowels bars or plates with threaded couplers and welded studs. Anchorages shall be adjusted to avoid prestressing strands. No less than 2 vertical rows of fasteners shall be used in the diaphragm to pretensioned concrete member connection. Additional bent plates to connect the diaphragm to the WT may be used for skews greater than 20 degrees. All diaphragms and connection materials shall be galvanized after fabrication.

Since the attachment of the diaphragm to pretensioned concrete member is a structural connection, dowels bars, plates with threaded couplers and welded studs, and high-strength bolts are specified.

Anchorage components shall not be visible on the member webs after installation of the diaphragms.

Oversized holes or field drilled holes are recommended for the connection of the WT shape rolled sections or T-shape weldments to the member webs. Galvanized coatings, damaged by the field drilling operations, shall be repaired prior to making final bolted connections.

These provisions have been provided to account for member camber, member sweep and other fabrication/construction, and to facilitate construction. The recommendation for holes should be shown on the Plans.

The Designer is responsible for designing and detailing the external end and intermediate diaphragms and the connections, including anchorages.

The end and intermediate diaphragms shall be detailed to accommodate proposed project utilities where required. Utility supports shall not be anchored into or suspended from the underside of the concrete deck.

5.12.10CT—Superstructure Jacking Requirements

Provisions for jacking of the superstructure shall be provided at all bearing locations.

Lift points shall be located adjacent to the bearings and may be on main or secondary members. Preferably, lift points shall be over the bridge seats of abutments and the tops of piers so that jacks may be founded on these components minimizing the need for extensive temporary structures.

Jacking lift points shall be designed per Article 3.4.3.1.

Jacks shall be sized to lift 150% of the value used to design the jacking lift point. Space shall be provided on the bearing area to fit the jack size necessary.

Superstructure and substructure members and components shall be strengthened as required to support the jacking loads.

5.13—ANCHORS

5.13.1—General

This section shall be supplemented with the following:

The drilling of holes in (or the use of power actuated tools on) prestressed members shall not be permitted. However, inserts for attachments may be placed in the members during fabrication. A note shall be included on the Contract plans.

C5.13.1

This section shall be supplemented with the following:

Refer to **BRSDM** [V1B] for notation requirements.

The use of later editions of the ACI 318 for the design, testing, evaluation and installation of post-installed adhesive bonded anchors is acceptable.

5.13.2—General Strength Requirements

5.13.2.1—Failure Modes to be Considered

This section shall be supplemented with the following:

Due to loading conditions, adhesive bonded anchors and dowels may be subject to sustained tension loads. Designers shall consider the time frame an anchor is subjected to a tension load to determine if the load is sustained. A load that remains constant over a period of time greater than 7 days shall be considered a sustained load. All sustained tension loads shall be considered significant.

C5.13.2.1

This section shall be supplemented with the following:

The term “sustained load” is meant to represent loads that remain static and do no change significantly with time.

For additional information on sustained tension, refer to *FHWA Technical Advisory T5140.34* dated January 16, 2018.

5.13.2.3—Determination of Anchor Resistance

This section shall be supplemented with the following:

The resistance factors for each adhesive bonded anchor failure mode shall be based on the following:

- The steel for the anchors shall be ductile
- Anchor Category 3 shall be assumed for anchors in sustained tension load. Otherwise Anchor Category 2 shall be assumed
- Supplemental reinforcement shall be assumed to be not present

When adhesive bonded anchors are subject to sustained tension loads, the factored bond resistance shall be determined using ACI 318 as modified herein.

C5.13.2.3

This section shall be supplemented with the following:

Resistance factors in ACI are based on anchor ductility, anchor category and the presence or absence of supplemental reinforcement.

Since adhesive bonded anchors in sustained tension typically are installed in locations that may be difficult to install and inspect, a lower resistance factor associated with Category 3 is specified.

Since it is difficult to determine if the reinforcement in existing concrete meets the ACI requirements for supplemental reinforcement, adhesive bonded anchors shall be designed assuming that supplemental reinforcement is not present.

The design provisions of ACI 318 as modified by Article 5.13 assume a 100 year life at a moderate temperature. Anchors expected to sustain loads for longer than 100 years or for more severe temperatures require more stringent loading criteria than those modified by **BDS**.

5.13.5CT—Post-Installed Adhesive Bonded Anchors and Dowels

5.13.5.1CT—General

The installation of adhesive bonded anchors and dowels into rock is not addressed by this practice.

C5.13.5.1CT

The installation of anchors and dowels using materials other than adhesive bonding materials, such as grout, is not addressed in this practice.

Post-installed adhesive bonded anchors and dowels are used to connect new construction to existing structurally sound concrete.

Two methods are recognized to design post-installed embedments using adhesive bonding material, anchor design methodology (ACI 318, Chapter 17) and reinforced concrete design methodology (ACI 318, Chapter 25). To differentiate between the 2 methods, the post-installed embedments are referred to as adhesive bonded anchors and adhesive bonded dowels, respectively.

Adhesive bonded anchors are composed of adhesive bonding material and steel anchors installed in holes drilled into existing concrete. The anchors may be threaded rods or deformed reinforcing bars with an embedment no greater than 20 times the anchor diameter.

Adhesive bonded dowels are composed of adhesive bonding material and deformed steel reinforcing bars installed in holes drilled into existing concrete. The dowels shall be embedded no less than the tension development length of the bar calculated using its full yield strength and no greater than 60 times the bar diameter.

Adhesive bonded anchors and dowels shall be installed in clean, dry holes drilled into existing structurally sound concrete.

Structurally sound concrete is solid when sounded with a hammer, uncracked, greater than 21 days old, and has a compressive strength no less than its design strength when it was originally placed.

Adhesive bonding materials shall be installed when the temperature of the surrounding air, adhesive bonding material, steel anchors, dowels and the concrete is 40 degrees F and rising.

5.13.5.2CT—Limitations

C5.13.5.2CT

The use of adhesive bonded anchors and dowels to attach to new construction is not permitted. Cast-in-place anchor rods, dowels or mechanical connectors (such as dowel bar splicers) shall be used to make connections between recently constructed structural components.

The installation of adhesive bonded anchors and dowels into delaminated or structurally unsound concrete is not permitted.

The installation of adhesive bonded anchors and dowels into lightweight concrete is not permitted.

The use of adhesive bonding material to structurally connect an epoxy coated anchors or dowels is not permitted. The use of hot-dip galvanized or stainless-steel anchors or dowels with adhesive bonding material is acceptable provided they meet the requirements in the material's ICC-ES Report.

The use of adhesive bonded anchors and dowels in connections that lack structural redundancy is not permitted. To ensure structural redundancy, increase the number of anchors and dowels and/or connection locations.

Adhesive bonded anchors and dowels shall only be installed in holes drilled vertically down, downwardly inclined or horizontally. The installation of adhesive bonded anchors and dowels in holes drilled with upwardly inclined or overhead orientations is prohibited.

Designers shall consider the cure time of adhesive bonding material when specifying construction sequences and construction time.

The use of adhesive bonded anchors and dowels to resist sustained tensile loads is prohibited, for both permanent and temporary construction, in the following applications:

- Pier cap retrofit, support or repairs
- Primary reinforcement for abutment and wall stems
- Primary reinforcement for deck overhang replacement and repairs
- Supports for utilities, drainage systems and catwalks

The use of adhesive bonded anchors and dowels to resist high cycle fatigue or impact loads is prohibited, except as follows:

- The connection metal railing systems to existing curbs
- The connection of concrete parapets to existing wall stems
- The connection of temporary traffic barrier to existing bridge decks

The use of adhesive bonded dowels to connect concrete parapets to existing decks may not be feasible due to minimal deck thickness resulting in insufficient embedment to fully develop the reinforcement.

The connection of railings, parapets and barriers must be **MASH** compliant.

The use of adhesive bonded anchors to connect sign, traffic control (mast arm and span pole), and lighting/luminaire structures to foundations is prohibited except as follows:

- To connect small single post regulatory sign supports to a concrete structure provided the support is not located over or can fall onto a travelway. A minimum of 4 anchors shall be used in the connection.
- To connect structure mounted sign supports to the outside face of concrete parapets provided the structural frame is attached to the concrete at no less than 3 locations with a minimum of 4 anchors at each location.

The use of adhesive bonded anchors to connect structure mounted sign supports to the outside face of concrete parapets will typically require anchor in sustained tension to be installed with a horizontal orientation. The installation of these anchors requires certified installers and inspectors.

- To connect luminaires to an existing concrete parapet. A minimum of 4 anchors shall be used in the connection.

5.13.5.3CT—Design of Adhesive Bonded Anchors

C5.13.5.3CT

Anchors installed with adhesive bonding material shall be designed, tested, evaluated and installed in accordance with the requirements of Article 5.13 supplemented herein.

All fixture to concrete connections using adhesive bonded anchors shall be designed and detailed as rigid fixture connections. The rigid fixture assumption shall be validated by the Designer.

Anchor design for fixture to concrete connections, such as baseplates, per ACI 318 is predicated on a rigid fixture assumption. For example, rigid baseplate design assumes the plate cross-section remains plane under loading and the plate does not undergo any significant deformation, allowing simplified linear-elastic calculations. Designers shall ensure that the rigid fixture assumption is valid for all fixture to concrete connections using adhesive bonded anchors.

The compressive strength of concrete used in the design of adhesive bonded anchors shall be no greater than the documented strength used in the existing concrete component.

When the strength of the existing concrete is unknown, the compressive strength used in the design of adhesive bonded dowels shall be 2.5 ksi.

The design of adhesive bonded anchors shall be based on a cracked concrete condition.

The design of the adhesive bonded anchor shall assume that supplemental reinforcement is not present.

The characteristic bond stress for the adhesive material used in the design of the adhesive bonded anchors shall be 1.00 ksi in dry, cracked concrete at temperature range A. The anchor category of the adhesive bonding material shall be at least Category 2.

The minimum characteristic bond stress for adhesive material required by the contract documents shall be 1.00 ksi or greater in dry, cracked concrete at Temperature Range A (ACI 355.4, Section 8.5). Manufacturer's tabulated bond strength values are typically based on normal weight concrete of a specified strength. Based on NCHRP Report 757, since the adhesive bonding material is not expected to be at or above 120ºF for a significant portion of its service life Temperature Range A was selected.

The Designer is responsible for the review of the adhesive bonding material product information submitted by the Contractor to ensure that product meets or exceeds the values used in the design and is acceptable for use as required by the contract documents. The specifications require the adhesive bonding material used with anchors meet assessment requirements for use under sustained tension loads and installation in holes drilled horizontally.

5.13.5.4CT—Design of Adhesive Bonded Dowels

Dowels installed with adhesive bonding material meeting the assessment requirements of ACI 355.4-11, or latest edition; and the ICC-ES Acceptance Criteria for Post-Installed Adhesive Anchors in Concrete Elements (AC308) for post-installed reinforcing bars shall be designed in accordance with Article 5.10.8 and other applicable provisions of Section 5. The design is subject to the adhesive bonding material product specific Evaluation Service Report (ESR). The installation shall conform to the conditions of the adhesive bonding material manufacturer's printed installation instructions (MPII).

The compressive strength of concrete used in the design of adhesive bonded dowels shall be no greater than the documented strength using in the existing concrete component.

When the strength of the existing concrete is unknown, the compressive strength used in the design of adhesive bonded dowels shall be 2.5 ksi.

C5.13.5.4CT

The Designer is responsible to review the product information submitted by the Contractor to ensure that product has been assessed for use under sustained tension loads, installation in holes drilled horizontally and with post-installed reinforcing bars designed in accordance with Article 5.10.8 and the applicable provisions of Section 5 and is acceptable for use as required by the contract documents.

5.14—DURABILITY

5.14.4—Corrosion-Resistant Reinforcement

Paragraph 2 shall be replaced with the following:

Reinforcing steel shall be galvanized.

C5.14.4

Differing steel protection of steel systems may be warranted for rehabilitation projects to match existing reinforcement coatings.

5.14.7CT—Protective Coatings

In prestressed concrete members, the non-prestressed steel, including reinforcement extending out of the units, shall be galvanized.

C5.14.7CT

All concrete surfaces subjected to salt spray from marine environments, or spray from de-icing chemicals, shall be sealed with a penetrating sealer provided on the **CTDOT** Qualified Product List.

The sealer shall be applied in accordance with the specification, "Penetrating Sealer Protective Compound."

It is anticipated that silanes and siloxanes will protect concrete for approximately 7-12 years, after which time, they should be re-applied. Designers shall specify the application of a penetrating sealer in rehabilitation projects – including bridge preservation projects to ensure the continued protection of concrete surfaces.

The use of colored sealers is permitted only with the written approval of the **CTDOT**.

Colored stains for concrete shall not be considered to protect concrete from de-icing chemicals.

Colored sealers may experience blistering or peeling over time, creating an undesirable appearance.

Colored stains may be incompatible with the penetrating sealer. Should colored concrete be desired, consideration shall be given to applying the pigment to the mix design so a penetrating sealer may be applied to the finished concrete.

5.15—REFERENCES

This section shall be supplemented with the following:

AASHTO. *LRFD Bridge Construction Specifications*, 4th ed. American Association of State Highway Transportation Officials, Washington, DC, 2017

ACI. *Building Code Requirements for Structural Concrete and Commentary.* ACI CODE-318-19. American Concrete Institute, Farmington Hills, MI, 2019.

ACI. *Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary*. ACI CODE-355.4-19. American Concrete Institute, Farmington Hills, MI, 2019.

CTDOT. *Bridge Load Rating Manual*, Version 2018.1.0. Connecticut Department of Transportation, Newington, CT, 2018.

FHWA. *Use and Inspection of Adhesive Anchors in Federal-Aid Projects.* Technical Advisory T5140.34. Federal Highway Administration, U.S. Department of Transportation, Washing, DC, 2018.

ICC-ES. *Acceptance Criteria for Post-installed Adhesive Anchors in Concrete Elements*. AC308. International Code Council Evaluation Services, Washington, DC, 2013

NCHRP. Report Number 322 *Design of Simple-Span Precast Prestressed Bridge Girders Made Continuous*. National Cooperative Highway Research Program, Washington, DC, 1989.

NCHRP. Report Number 757 *Long-Term Performance of Epoxy Adhesive Anchor Systems*. National Cooperative Highway Research Program, Washing, DC, 2017.

PCI. *PCI Bridge Design Manual 3rd Edition, Second Release, August 2014*, Precast/Prestressed Concrete Institute, Chicago, IL. 2014.

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SECTION 6

STEEL STRUCTURES

Commentary is opposite the text it annotates.

6.4—MATERIALS

6.4.1—Structural Steels

This section shall be supplemented with the following:

Round hollow structural sections (HSS) shall be specified based on the need for the material to meet Charpy-V notch (CVN) requirements in accordance with the following table:

Round HSS			
CVN Requirements	Designation	Fy, ksi	Comments
Required	ASTM A1085, Grade A	50	
Not Required	ASTM A500, Grade C	46	Preferred over ASTM A53, Grade B
	ASTM A53, Grade B	35	

AASHTO M270 Grade 50 shall generally be used for all structural steel. If the structure is to remain uncoated and allowed to weather, AASHTO M270 Grade 50W shall be used.

6.4.1.1CT—Coated and Uncoated Structural Steel

New structural steel bridges may be either coated or uncoated.

- •Uncoated steel shall be weathering steel.
- •Coated steel shall be either shop galvanized or shop metallized. Metallizing shall be sealed and top coated.

Uncoated weathering steel is the preferred system for new structural steel bridges should the criteria for its use be met.

Field metallizing is the preferred coating system for existing structural steel bridges.

A paint only system shall only be used for existing structural steel bridges and shall not be used to coat structural steel of new superstructures.

C6.4.1.1CT

In order to reduce future maintenance, the use of coated steel should be minimized. Uncoated weathering steel should be the first choice for structural steel bridges with life-cycle cost as a consideration. The use of galvanizing or metallizing and top coating should also be considered where the look of weathering steel is objectionable.

Weathering steel should be the first consideration for most bridges, especially those in rural areas. The use of weathering steel in urban areas or where the bridge will be highly visible shall be discussed with the Municipal Officials prior to its use. Weathering steel shall be designated for all structural steel bridges over railroads.

Where the use of weathering steel is not appropriate, such as bridges subject to vehicular salt spray, near a salt water environment, or a heavy industrial area, the use of galvanized steel should be considered. Where the length of the structural steel members precludes use of galvanized steel, shop metallizing should be used. Shop metallizing shall include a colored urethane topcoat.

6.4.1.1.1CT—Uncoated Weathering Steel

Where weathering steel has been found to be appropriate in accordance with **CTDOT** guidelines, its use should conform to the **FHWA** Technical Advisory T5140.22, "Uncoated Weathering Steel in Structures," dated Oct. 3, 1989, **AASHTO**/NSBA "Uncoated Weather Steel Reference Guide" and amended as follows:

a. The design of weathering steel for bridges subject to vehicular salt spray, near a salt water environment, or a heavy industrial area should incorporate modest increases in flange plate thicknesses to allow for some minor section loss in the future.
b. The interior surfaces of box girders, including all structural steel components within the box girders (such as diaphragms, cross-frames, connection plates, etc.) shall be painted in accordance with the special provision, entitled "Structural Steel (Site No. X)." The intermediate coat shall be white (Aerospace Material Specification 595 Color No. 27925) in order to facilitate bridge inspection.
c. Whenever possible, unpainted weathering steel bridges must be designed to eliminate deck joints. If deck joints cannot be eliminated, the areas adjacent to the joints shall be protected from leakage. Generally, the ends of the beams directly under joints shall be metallized or painted for protection. For bridge decks that extend past the backwall and integral abutments, beam ends need not be painted. The topcoat shall be Brown, Aerospace Material Specification 595 Color No. 20062. The steel should be metallized or painted for a distance approximately equal to one and one half times the depth of the girder on either side of the joint. All structural steel components within this distance (such as diaphragms, cross-frames, connection plates, stiffeners, etc.) shall also be painted.
d. Proper precautions should also be taken to minimize substructure staining for construction conditions and the service life of the bridge. In general, this will include providing catchments and diversion bars at all bearings and ensuring that the Contractor adequately protects the substructure during construction.
e. Provisions should also be included to control vegetation growth under the structure to reduce the moisture in the air that could have a detrimental effect on the structure.

C6.4.1.1.1CT

The limits of the structural steel requiring painting shall be delineated on the Contract plans.

6.4.1.1.2CT—Coated Structural Steel

In general, coated structural steel bridges shall be galvanized or metallized and top coated. For existing bridges, when required, structural steel shall be prepared and coated in accordance with the special provision, entitled "Structural Steel (Site No. X)."

With the exception of major structures or architecturally or historically significant structures, the choice of color for shop and field painting the topcoat of steel, shall be limited to the following:

a. Green - Aerospace Material Specifications 595, Color No. 24172
b. Green - Aerospace Material Specifications 595, Color No. 24277
c. Blue - Aerospace Material Specifications 595, Color No. 26329

Blue shall be used for bridges that span over waterways. Green shall be used for bridges that span over land or roadways.

The use of galvanized steel or metallizing should be considered in order to reduce future maintenance obligations.

6.4.3 Bolts, Nuts and Washers

6.4.3.1—High-Strength Structural Fasteners

This section shall be supplemented with the following:

High-strength bolt, nut and washer designations shall be shown on the Contract plans.

6.4.3.1.1—High-Strength Bolts

This section shall be supplemented with the following:

All bridge fasteners shall be high-strength bolts conforming to the requirements of ASTM F3125 Grade A325 or F3125 Grade A490.

ASTM F3125 Grade A325 Type 3 is preferred in conjunction with weathering steel.

On coated structures, high-strength bolts shall conform to ASTM F3125 Grade A325 Type I and be hot-dipped galvanized in accordance with ASTM F2329 or mechanically galvanized in accordance with ASTM B695, Class 55.

6.4.3.1.2—Nuts Used with ASTM F3125 Bolts

This section shall be supplemented with the following:

Nuts for ASTM F3125 Grade A325 bolts shall meet the requirements of ASTM A563, Grade DH, DH3, C, C3 and D. Where Type 3 bolts are used, the nuts shall be Grade C3 or DH3.

Nuts for ASTM F3125 Grade A490 bolts shall meet the requirements of ASTM A563, Grade DH. Where Type 3 bolts are used, the nuts shall be Grade DH3.

6.6—FATIGUE AND FRACTURE CONSIDERATIONS

6.6.1—Fatigue

6.6.1.2—Load-Induced Fatigue

6.6.1.2.6CT—Flanges

For all types of steel bridges, the design of the flanges should be based on Category C in order to allow the welding of diaphragm connection plates to the flanges.

If a preliminary design does not satisfy the requirements of Category C, then one of the following options should be followed:

1. The flange can be increased in size to reduce the live load stress range.
2. The location of flange transitions can be changed to reduce the live load stress range.

6.6.2—Fracture

6.6.2.2—Fracture-Critical Members

This section shall be supplemented with the following:

All fracture critical members shall be identified on the Contract plans. Each portion of a bending member that is fracture critical including welds shall be clearly described giving the limits of the FCM.

The design of pin and hanger structures is not allowed.

C6.6.2.2

This section shall be supplemented with the following:

Refer to **CTDOT Bridge Plan Notes Guide** for additional notes.

6.7—GENERAL DESIGN AND DETAILING REQUIREMENTS

6.7.2—Dead Load Camber and Detailing of Structural Components

This section shall be supplemented with the following:

Dead load deflection and camber diagrams are not required for simple span bridges. Dead load deflections and cambers shall be calculated at the mid-span of each member and tabulated on the Contract plans.

For continuous span bridges, dead load deflections and cambers shall be tabulated for each member at each bearing point and equal spaces along the member at approximately 10 feet on center.

C6.7.2

This section shall be supplemented with the following:

Refer to **BRSDM** [V1B] for additional guidance for tabulating dead load deflections and camber.

6.7.4—Diaphragms and Cross-Frames

6.7.4.1—General

The following shall be included after the second paragraph:

Intermediate and end bearing diaphragms and cross frames (cross members) shall be provided for rolled beams, plate girders and box girders.

6.7.4.2—I-Section Members

This section shall be supplemented with the following:

Intermediate diaphragms and cross-frames shall preferably be placed at the 0.4 point from end supports of end spans of continuous bridges and at the center of interior spans. If practical, they should also be placed adjacent to a field splice. Diaphragms and cross-frames shall be spaced as far apart as possible to limit the overall number of diaphragms and cross-frames.

The need for diaphragms and cross-frames shall be investigated for all stages of construction.

The 4th and 5th paragraphs of this section shall be replaced with the following:

Where support lines are not skewed more than 20 degrees from normal, intermediate diaphragms or cross-frames shall be placed in line along the skew.

Where support lines are skewed more than 20 degrees from normal, intermediate diaphragms or cross-frames shall be normal to the girders and shall be placed in discontinuous lines (staggered) across the width of the bridge.

6.7.4.2.1CT—I-Section Members with Web Depth ≤ 4 feet

Channel members shall be typically used for end and intermediate diaphragms.

6.7.4.2.1aCT—Intermediate Diaphragms

For intermediate diaphragms, the channel size shall be dependent upon the main member's depth. Refer to Table 6.7.4.2-1 for use in bridge skews less than 30 degrees. For bridge skews greater than 30 degrees, the members shall be designed.

Table 6.7.4.2-1 – Typical Channel Diaphragms

DEPTH (in)	CHANNEL SIZE
21 - 24	C12 x 20.7
27 - 30	C15 x 33.9
33 - 36	MC18 x 42.7

C6.7.4.2.1aCT

Depth in Table 6.7.4.2-1 refers to total beam depth for rolled beams and actual web depth for plate girders.

6.7.4.2.1bCT—End Bearing Diaphragms

End bearing diaphragms shall preferably be channel sections and should be designed as simple span members with vertical dead loads and live loads plus impact applied.

For bridges with severe skew angles or wider girder spacings, wide flange sections or K-frames may be used in lieu of channels.

End bearing diaphragms shall be placed along the centerline of bearings and be set on a sloped line. A minimum clear distance of 12 inches shall be provided between end bearing diaphragms and front face of backwall.

The bridge skew angle shall be considered in determining the length of the end bearing diaphragm. Consideration shall be given to composite action in the design of all end diaphragms. For both non-composite and composite end bearing diaphragms, 7/8 inch diameter shear connectors with a maximum spacing of 12 inches shall be welded to the top flange of the end diaphragms.

C6.7.4.2.1bCT

The preferred Channel size shall be C15 x 33.9 and MC18 x 42.7.

6.7.4.2.2CT—I-Section Members with Web Depth ≥ 4 feet

6.7.4.2.2aCT—Intermediate Diaphragms

Cross-frames shall be used for intermediate bracing.

C6.7.4.2.2aCT

The most economical intermediate cross-frame considered for use shall be the X-type. When additional bracing is required, K-type frames should also be considered.

Cross-frame depths shall be constant to facilitate fabrication. All members shall be fabricated from equal leg angles or WT sections.

6.7.4.2.2bCT—End Bearing Diaphragms

End bearing cross-frames shall have a K-type configuration with a channel member used at the top. All other members shall be equal leg angles or WT sections.

The design of the top member shall be in accordance with Article 6.7.4.2.1bCT.

The size of the end bearing cross frame's bottom chord may be increased to provide for future jacking of the girder ends.

6.7.4.3—Composite Box-Section Members

This section shall be supplemented with the following:

If plate diaphragms are used, they shall be connected to the webs and flanges of the section.

Intermediate cross-frames that are not required for the completed bridge may be required for construction purposes and shall be located and spaced as a matter of engineering judgement. They may be installed as temporary or left-in-place as permanent members.

Consideration shall be given to locate, at a minimum, intermediate cross-frames at the lifting points of each shipping piece, on each side of a field splice, and at maximum positive moment sections.

Intermediate cross-frames shall be designed to satisfy the construction load stresses and slenderness ratio requirements and shall be fabricated from equal leg angles.

C6.7.4.3

This section shall be supplemented with the following:

Typical cross-frame configurations shall be the X and K-types.

6.7.5—Lateral Bracing

6.7.5.2—I-Section Members

This section shall be supplemented with the following:

Lateral Bracing should be avoided whenever possible. The need to temporarily brace the compression flange for stability during erection shall be investigated. Reducing the cross-frame spacing or modifying flange plate dimensions shall be considered when attempting to eliminate the bracing.

When lateral bracing is required, it shall be designed to satisfy Article 6.7.5.1 and the slenderness ratio requirements.

The allowable fatigue stress ranges shall not be exceeded at the connections.

Lateral bracing members shall typically consist of equal leg angles or WT sections attached to the flange or web via gusset plates, clip angles or WT sections. Gusset plates shall be bent to accommodate the difference in elevation between girders. If it is not practical to make connections to the flange, then connections shall be made to the web. Flange connections shall not interfere with the web to flange welds.

The minimum thickness of gusset plates shall be ½ inches. The minimum size angle used as a connecting or lateral bracing

C6.7.5.2

This section shall be supplemented with the following:

Warren type patterns with single members is recommended.

member shall be L4x4x5/16. Angles with unequal legs should not be used.

6.7.5.3—Tub Section Members

This section shall be supplemented with the following:

To increase the torsional stiffness of an individual box to tub section during fabrication, erection and placement of the slab, permanent internal lateral bracing, either full or partial length, shall be placed at or near the plane of the top flanges.

Lateral bracing members and their connections shall be similar to those for I-Shaped members specified in Article 6.7.5.2.

Allowable fatigue stress ranges shall not be exceeded where the gusset plate attaches to the flange or web.

C6.7.5.3

This section shall be supplemented with the following:

Warren type bracing, without transverse members, should be considered for efficiency. X-bracing patterns should be avoided for economy.

6.7.6—Pins

This section shall be supplemented with the following:

The design of pin and hanger structures is not allowed.

6.7.9CT—Superstructure Jacking Requirements

Provisions for jacking of the superstructure shall be provided at all bearing locations.

Lift points shall be located adjacent to the bearings and may be on main or secondary members. Preferably, lift points shall be over the bridge seats of abutments and the tops of piers so that jacks may be founded on these components minimizing the need for extensive temporary structures.

Jacking lift points shall be designed per Article 3.4.3.1.

Jacks shall be sized to lift 150% of the value used to design the jacking lift points. Space shall be provided on the bearing area to fit the jack size necessary.

6.7.9.1CT—New Construction

At abutments, preference shall be given to widening of the bridge seat and providing auxiliary jacking stiffeners so that jacks may be placed in front of the bearing to jack under than beam. Provisions for massive diaphragms, which restrict access to the ends of the beam and backwalls should be avoided.

At piers with continuous caps, preference shall be given to designing diaphragms for jacking forces and providing auxiliary pads on pier caps.

Other unusual situations will require special study and may require provisions for jacking from ground level.

C6.7.9.1CT

An example of an unusual situation would be piers consisting of individual columns under each girder.

6.7.9.2CT—Rehabilitation Projects

For superstructure replacements, jacking provisions shall be provided only if economically viable.

Superstructure and substructure members and components shall be strengthened as required to support jacking loads.

C6.7.9.2CT

Jacking requirements are not reason to justify major substructure modifications where the substructures are otherwise adequate.

6.7.10CT—Inspection Hand Rails

When girders are 5 feet or more in depth, a safety hand bar shall be provided 42 inches above the bottom flange for inspection access on both sides of all girders except the outside face of fascia girders.

The bar shall have a minimum diameter of 1 inch and shall be designed for a minimum point load of 270 pounds.

6.10—I-SECTION FLEXURAL MEMBERS

6.10.1—General

This section shall be supplemented with the following:

The use of rolled beams should be investigated for appropriate span lengths.

C6.10.1

This section shall be supplemented with the following:

The cost of fabricating rolled beam sections is significantly lower than equivalent I-shaped plate girders.

6.10.1.3—Hybrid Sections

This section shall be supplemented with the following:

The design of hybrid I-shaped girders should be avoided.

C6.10.1.3

This section shall be supplemented with the following:

Hybrid girders may be used with the approval of **CTDOT**.

6.10.2—Cross-Section Proportion Limits

6.10.2.1—Web Proportions

This section shall be supplemented with the following:

The minimum thickness of web plates shall be 3/8 inches. Web plate depths shall be specified in 2 inch increments.

C6.10.2.1

This section shall be supplemented with the following:

Refer to **BRSDM** [V1] for guidance on economic designs.

6.10.2.2—Flange Proportions

This section shall be supplemented with the following:

The minimum thickness of flange plates shall be ¾ inches.

The minimum width of flange plates shall be the greater of either the length of the unsupported field piece divided by 85 or 12 inches. The flange plate widths shall be specified in 2 inch increments.

Flange width transitions shall generally be avoided. The Designer shall make every attempt to vary flange thickness instead to satisfy design considerations. At flange plate transitions, the thickness of the thinner plate shall not be less than ½ the thickness of the thicker plate.

C6.10.2.2

This section shall be supplemented with the following:

This minimum thickness is required to eliminate warping of the plates when they are welded to the web.

The minimum width is to minimize potential stability problems during various phases of construction.

Refer to **BRSDM** [V1] for guidance on economic designs.

Flange width transitions shall not be located within a shipping unit.

6.10.10—Shear Connectors

6.10.10.1—General

This section shall be supplemented with the following:

All structural members in contact with and supporting a concrete deck shall be designed for composite action.

In general, 7/8 inch diameter stud type shear connectors shall be used for composite construction. Spirals, angles or channel shear connectors are not permitted.

6.10.11—Web Stiffeners

6.10.11.1—Web Transverse Stiffeners

6.10.11.1.1—General

The 1st paragraph shall be replaced with the following:

Transverse stiffeners shall consist of plates fillet welded to both sides of the web. For exterior girders, transverse stiffeners shall be fillet welded to the inside face only. Installation of transverse stiffeners on the outside face of exterior girders is not acceptable.

The 3rd paragraph shall be replaced with the following:

Stiffeners used as connecting plates for diaphragms or cross-frames shall be attach to both flanges with fillet welds.

This section shall be supplemented with the following:

The minimum fillet weld size for connection of transverse stiffeners to the web and flanges shall be 5/16 inches.

Transverse stiffener plates on any one structure should have the same width and thickness. The minimum thickness of a stiffener plate shall be ½ inches.

C6.10.11.1.1

The limit of stiffeners located on the inside face of exterior girders is for aesthetic reasons.

Using like plates for the entirety of one structure simplifies fabrication.

6.10.11.2—Bearing Stiffeners

6.10.11.2.1—General

This section shall be replaced with the following:

Bearing stiffeners shall be placed on the webs of built-up sections and rolled beams at all bearing locations. At other locations on built-up sections or rolled shapes subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, either bearing stiffeners shall be provided or the web shall satisfy the provisions of Article D6.5.

Bearing stiffeners shall consist of plates fillet welded to both sides of the web. The stiffeners shall extend the full depth of the web and as closely as practical to the outer edges of the flanges. Bearing stiffeners serving as connection plates shall be connected to each flange with fillet welds.

The minimum fillet weld size for connection of bearing stiffeners to the web and flanges shall be 5/16 inches.

Bearing stiffener plates on any one structure should have the same width and thickness. The minimum thickness of a bearing stiffener plate shall be ¾ inches.

When bearing stiffeners consist of two pairs of plates, the pairs shall be placed symmetrically over the bearings and offset sufficiently to permit welding.

C6.10.11.2.1

The third paragraph shall be replaced with the following:

For skewed plates, the **AWS D1.5** design requirements for skewed joints should be considered when sizing the welds.

Using like plates for the entirety of one structure simplifies fabrication.

Bearing stiffener plates shall be vertical after the application of full dead loads.

A note shall be included on the Contract plans indicating plumb under full dead load.

6.10.11.3—Web Longitudinal Stiffeners

6.10.11.3.1—General

This section shall be supplemented with the following:

The use of longitudinal stiffeners is discouraged.
If longitudinal stiffeners are required:

- They shall be evaluated for fatigue.
- They shall consist of plates fillet welded to one side of the web. The minimum fillet weld size shall be 5/16 inches.
- The use of them on the outside face of exterior girders is acceptable.
- They shall be discontinued at bolted field splices.
- Complete joint penetration groove weld splices in longitudinal stiffeners shall be made before attachment to the web, and non-destructively tested by the ultrasonic method.

6.10.12—Cover Plates

6.10.12.1—General

This section shall be supplemented with the following:

Cover plates shall be narrower than the flange to which they are attached.

The minimum thickness of a cover plate shall be 1/2 inches.

The attachment of cover plates shall be made with fillet welds. Cover plate attachments shall be designed for fatigue.

All fillet welds connecting the cover plate to the beam shall be non-destructively tested by the magnetic particle method.

The Contract plans shall clearly state that, if the cover plate is fabricated by butt welding two or more plates together, the butt welds shall be non-destructively tested by ultrasonic testing prior to attaching the cover plate to the beam.

6.10.12.2—End Requirements

6.10.12.2.1—General

This section shall be supplemented with the following:

For simple span rolled beams with cover plates, the cover plates shall be extended approximately full length. Cover plates shall be fillet welded across the ends.

For continuous span rolled beams with cover plates, the cover plates shall be terminated with end welds in non-fatigue regions.

The ends of cover plates shall be rectangular in shape with rounded corners. The minimum radius on the corners shall be 3 inches. Tapered cover plate ends are not permitted.

C6.10.12.2.1

This section shall be supplemented with the following:

For the bottom flange, the regions are near the interior supports and, for the top flange, the regions are near the middle of the spans.

6.11—COMPOSITE BOX-SECTION FLEXURAL MEMBERS

6.11.1—General

This section shall be supplemented with the following:

Generally, box girders should be considered only for very long spans. They should also be investigated for use on curved roadways where torsional rigidity is required.

The design of hybrid box girders should be avoided.

In addition to the other requirements specified in Article 2.5.2.5, the following shall apply:

- Gas, water and sewer lines are prohibited from being located within box girders. Electric, telephone and cable companies are discouraged from locating their lines within the boxes. All utilities can generally be accommodated outside of and between box girders.

Box girders shall be designed for the additional weight of stay-in-place forms placed within the boxes to form the deck slab.

C6.11.1

Hybrid girders may be used with the approval of **CTDOT**.

6.11.1.3—Flange-to-Web Connections

This section shall be supplemented with the following:

A minimum distance of 1 inch shall be provided between the outside face of the web and the edge of the bottom flange.

At web stiffeners, provide a ½ inch clearance above a line 60 degrees from the bottom flange.

C6.11.1.3

This section shall be supplemented with the following:

This is to act as a holding shelf for the flux deposited by the welding machine.

This is to accommodate a traveling welding machine.

6.11.1.4—Access and Drainage

This section shall be supplemented with the following:

Access manholes shall be provided in the end or bottom flange of box girders. The distance between the end diaphragm and the backwall should be increased to a minimum of 2 feet when access is provided in the end diaphragms. For access through the bottom flange, ladder supports shall be incorporated.

The manholes shall have rounded corners fitted with a hinged cover and provided with an appropriate locking system and all access doors shall open inward.

When access is provided through the end diaphragms, the access door should be covered with a steel wire mesh to allow ventilation. If access manholes are provided through the bottom flange, the access doors should be designed to be light weight.

Access holes shall be provided through all solid diaphragms.

Stresses resulting from the introduction of access holes in steel members shall be investigated and kept within all allowable limits.

In order to provide drainage of the inside of the box girder, 2 inch minimum diameter drains shall be provided at the low

C6.11.1.4

This section shall be supplemented with the following:

These manholes shall be located and detailed such that bridge inspectors can gain access without the need for special equipment.

The preferred location for access is through the ends of the boxes.

end of the girder. The corners of all plates should be clipped so as not to trap moisture inside the girder.

Bridge deck drainage may extend vertically through the girder but shall not be carrier longitudinally within it.

6.11.2—Cross-Section Proportion Limits

6.11.2.1—Web Proportions

6.11.2.1.1—General

This section shall be supplemented with the following:

Box girder cross-sections shall be a trapezoidal shape with webs sloped equally out from the bottom flange.

Webs shall be the same depth. The minimum web depth shall be 78 inches.

The minimum thickness of web plates shall be 3/8 inches. Web plate depths shall be specified in 6 inch increments.

In general, box girders shall be rotated so that the top and bottom flanges are parallel with the deck cross slope.

C6.11.2.1.1

This section shall be supplemented with the following:

This minimum depth is to allow for inspection and maintenance inside the box girders.

6.11.2.2—Flange Properties

This section shall be supplemented with the following:

The minimum thickness of flange plates shall be ¾ inches. The maximum flange plate thickness shall be 3 inches.

Flange plate widths shall be specified in 2 inch increments. Flange width transitions shall generally be avoided. The Designer shall make every attempt to vary flange thickness instead to satisfy design considerations. At flange plate transitions, the thickness of the thinner plate shall not be less than half the thickness of the thicker flange.

C6.11.2.2

This section shall be supplemented with the following:

This minimum thickness is required to eliminate warping of the plates when they are welded to the web.

Refer to **BRSDM** [V1] for guidance on economic designs.

6.11.10—Shear Connectors

This section shall be supplemented with the following:

Refer to Article 6.10.10 for shear connector requirements for box girders.

6.13—CONNECTIONS AND SPLICES

6.13.1—General

This section shall be supplemented with the following:

Shop connections may be made by either bolting or welding. Generally, all field connections should be made with high strength bolts.

Welded field splices are not allowed.

The design of pin and hanger structures is not allowed.

C6.13.1

This section shall be supplemented with the following:

The use of field welding is discouraged due to difficulties with achieving proper coatings in the field.

6.13.2—Bolted Connections

6.13.2.1—General

This section shall be supplemented with the following:

All bolted connections shall be designed as slip critical connections.

In general, connections shall be designed with 7/8 inch diameter ASTM F3125 Grade A325 high strength bolts.

To facilitate steel erection, only one type and diameter of bolt should be specified on any one bridge.

Bolt diameter, hole size, bolt spacing and edge distances shall be shown on the Contract plans, in addition to the types of connections and class of fraying surfaces.

C6.13.2.1

This section shall be supplemented with the following:

Refer to **CTDOT** Guide Sheets for detailing requirements.

6.13.2.4—Holes

6.13.2.4.1—Type

6.13.2.4.1a—General

This section shall be supplemented with the following:

When standard holes are to be detailed, and design considerations permit, connections should be designed to accommodate oversized holes to allow for potential enlargement of holes in the field where necessary to facilitate field erection.

6.13.2.8—Slip Resistance

This section shall be supplemented with the following:

Connections on uncoated bridges and coated bridges shall be designed with Class B surface conditions. Connections on metallized bridges shall be designed with Class B surface conditions and shall be unsealed only at the connection faying surface. Connections on galvanized bridges shall be designed with Class C surface conditions.

For connections with surfaces with different coatings, the connection shall be designed with the controlling surface condition factors.

6.13.3—Welded Connections

6.13.3.1—General

This section shall be supplemented with the following:

Welding of structural steel members or components for inspection platforms shall conform to **AWS D1.1**.

C6.13.3.1

This section shall be supplemented with the following:

Welding for structural steel members or components for sign supports shall conform to **AWS D1.1**. Refer to **LTS** for sign support welding requirements.

6.13.3.4—Size of Fillet Welds

This section shall be supplemented with the following:

Fillet weld sizes shall be shown on the Contract plans.

6.13.3.6—Fillet Weld End Returns

The second paragraph shall be supplemented with the following:

Connections made with fillet welds placed on opposite sides of a common plane of contact shall not be detailed with the weld-all-around symbol. These welds should terminate 0.5in short of corners or ends.

6.13.5—Connection Elements

6.13.5.1—General

6.13.5.1.1CT—Cross Member Connections

The design of the connection of cross members shall be consistent with the design of the members being attached.

Provisions for future jacking of the superstructure shall be provided per Article 6.7.9CT.

The connections for the end bearing cross members shall be designed for the shear due to dead and live loads plus impact imparted by the deck directly to the cross members.

For intermediate diaphragm connections, the number of bolts should be a minimum of 4 bolts on each side of the diaphragm. In all cases, the number of bolts should be kept to a minimum.

6.13.6—Splices

This section shall be supplemented with the following:

Location of shop and field splices is dependent upon such factors as design criteria, available length of plates and members, transportation of members, erection and site limits, etc..

C6.13.6

This section shall be supplemented with the following:

Refer to Article 2.3.5CT on the transportation of members for additional information and guidance.

6.13.6.1—Bolted Splices

This section shall be supplemented with the following:

Standard holes shall be specified for bolted splices. However, bolted splices shall be designed as if oversized holes were to be used.

The minimum thickness of web and flange splice plates shall be ½ inches. Splices shall be detailed with a minimum edge distance of 2 inches. The maximum distance between the ends of the members being spliced shall be 1 inch.

C6.13.6.1

This section shall be supplemented with the following:

This is to allow for reaming of holes in the field to facilitate fit-up.

6.17—REFERENCES

This section shall be supplemented with the following:

AASHTO/NSBA Steel Bridge Collaboration. *Uncoated Weathering Steel Reference Guide*. American Association of State Highway and Transportation Officials, Washington, DC, 2022

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SECTION 9

DECKS AND DECK SYSTEMS

Commentary is opposite the text it annotates.

9.4—GENERAL DESIGN REQUIREMENTS

9.4.6CT—Construction Joints

Decks adjacent to construction joints shall be properly designed and supported for all loading conditions. The members supporting the deck adjacent to the construction joints shall also be investigated to ensure that they are adequate for all loading conditions.

9.7—CONCRETE DECK SLABS

9.7.1—General

9.7.1.1—Minimum Depth and Cover

This section shall be supplemented with the following:

The minimum thickness of a cast-in-place concrete deck slab shall be 8.5 inches. The top 0.5-inch shall be included in calculations as dead load only. It should be assumed to be sacrificial and not included as a permanent part of the deck for design purposes.

The minimum cover to the top mat of reinforcement shall be 2.5 inches.

The minimum cover to the bottom mat of reinforcement shall be 1 inch.

9.7.1.1.1CT—Haunches

The minimum haunch depth shall be 2 inches for prestressed bulb tee, and 1 inch for all other structure types, measured from the top of the top flange of the member to the bottom of the slab. For members with splices, the top of the member shall be the top of the splice plate.

A deeper haunch may be required when the width of the top flange of a member exceeds 1.33 feet due to the cross slope of the slab. Changes in flange plate thicknesses and allowable camber tolerances shall be taken into consideration when determining the haunch depth to use in calculations.

Haunches with thicknesses less than 4 inches do not require reinforcement. Haunches with thicknesses from 4 inches to 6 inches shall be reinforce. Reinforcement for haunches with thicknesses greater than 6 inches shall be detailed by the Designer.

9.7.1.2—Composite Action

This section shall be supplemented with the following:

All structural members in contact with and supporting a concrete deck shall be designed for composite action, unless otherwise noted in these specifications. The members shall be designed assuming construction without shoring (unshored construction).

9.7.1.5—Design of Cantilever Slabs

This section shall be supplemented with the following:

The concrete deck overhang, measured from the centerline of the fascia member to the outside edge of the deck, should be limited to four feet or the depth of the member, whichever is less.

For deck overhangs greater than 4 feet, the design shall include requirements in the contract documents for special forming requirements needed to prevent torsional rotation of the fascia member during concrete placement. This rotation is caused by the effect of the typical forming brackets used in construction.

Cantilever deck overhangs shall be designed in accordance with Appendix A13.

9.7.1.6CT—Detailing Practices

9.7.1.6.1CT—Finished Deck Elevations

For cast-in-place concrete decks, finished deck elevations and member deflections shall be tabulated at member bearing points and at points equally spaced along the members at approximately ten feet on center or at tenth points along the span, whichever is greater.

The finished deck elevations are those elevations on the top of the concrete deck. The tabulated member deflections are those deflections due to dead loads except the self-weight of the members and cross members. These elevations and deflections are to be used to calculate girder camber and haunch depths.

For precast concrete deck panels, deck elevations shall be tabulated at edges of the panels at the panel joints. The deck elevations are those elevations on the top of the concrete panel.

9.7.2—Empirical Design

This section and all subarticles in this section shall be replaced with the following:

The design of cast-in-place concrete decks using the empirical design method is not permitted.

9.7.3—Traditional Design

9.7.3.2—Distribution Reinforcement

This section shall be supplemented with the following:

At acute corners of the deck, when the skew angle exceeds 20 degrees, additional reinforcement shall be placed parallel to the end of the slab with appropriate increase in slab thickness.

Additional distribution reinforcement shall be placed midway between the top and bottom longitudinal bars at the end of decks. The bar size and length shall be as follows:

Span Length	Bar Size and Length
≤ 50ft	#5 x 5ft
> 50 ft and ≤ 80 ft	#5 x 8ft
> 80 ft	#5 x 10ft

9.7.4—Stay-in-Place Formwork

9.7.4.1—General

This section shall be supplemented with the following:

Typically, stay-in-place formwork shall be metal. If deck reinforcing bars are galvanized, metal stay-in-place forms shall be galvanized or other non-conductive material.

Prestressed concrete stay-in-place formwork may be used only with the written approval of the **CTDOT**.

C9.7.4.1

This section shall be supplemented with the following:

Forms for the construction of cast-in-place concrete bridge decks may be removable.

9.7.4.2—Steel Formwork

The first paragraph shall be replaced with the following:

Panels shall be specified to be tied together mechanically at their common edges and fastened to their support. Welding of stay-in-place metal form supports to tension zones in girder top flanges is not permitted.

This section shall be supplemented with the following:

The use of stay-in-place metal forms is permitted in all but the following locations:

- Under cantilever slabs such as the overhang outside of fascia girders
- Under longitudinal deck joints between median girders
- A bridge less than 15 feet above mean high water level of a salt-laden body of water.

For all bridges where stay-in-place metal forms are permitted, the Designer shall include the provision in the design calculations for the stay-in-place metal forms.

All affected members shall be designed to carry the additional dead load of the stay-in-place forms. Lightweight foam filler shall be used to fill valleys of the stay-in-place forms.

The cover of the bottom reinforcement shall be measured from the top of the stay-in-place metal forms.

For all bridges for which stay-in-place metal forms are permitted, girder deck load deflection and camber calculations shall include the estimated weight of stay-in-place forms with foam valley fillers. Where stay-in-place metal forms are provided, the Designer must note the assumed uniform weight of the stay-in-place metal forms on the bridge plans.

9.7.7CT—Concrete Deck Protective Systems

9.7.7.1CT—General

The decks of all bridges, both new and rehabilitated, shall be protected from damage, deterioration and corrosion due to deicing salts.

Concrete decks shall be protected by membrane waterproofing and bituminous concrete overlay.

Other methods to protect concrete decks, such as using a cathodic protection system or latex modified concrete, may only be used with the written approval of the **CTDOT**.

9.7.7.2CT—Membrane Waterproofing

The preferred method to protect cast-in-place and precast concrete decks consists of using galvanized reinforcement and a membrane waterproofing protected with a bituminous concrete overlay.

The standard membrane waterproofing shall be "Membrane Waterproofing (Cold Liquid Elastomeric)." “Membrane Waterproofing (Woven Glass Fabric)” may be used when the life of cold liquid elastomeric is anticipated to exceed the useful service life of the structure.

If during construction, a temporary condition is required to restore traffic, the following shall apply:

- Temporary pavement is required:
  - Not intended for winter shutdown
    - Apply bond breaker to deck and apply temporary pavement
    - For final pavement, remove temporary pavement and apply "Membrane Waterproofing (Cold Liquid Elastomeric)."
  - Intended for winter shutdown
    - Apply "Penetrating Sealer Protective Compound" to concrete, bond breaker and temporarily pave.
    - In spring, remove temporary overlay, apply "Membrane Waterproofing (Cold Liquid Elastomeric)."
- Temporary pavement is not required
  - Allow traffic to ride on bare deck and/or aggregated "Membrane Waterproofing (Cold Liquid Elastomeric)." Pave as soon as possible.

C9.7.7.2CT

Designers shall contact the **CTDOT** Pavement Design Unit for min/max lift thickness for each type of HMA and include on the Contract plans.

The Designer shall take into consideration time frames and schedule for projects using accelerated bridge construction techniques when selecting the membrane waterproofing system.

Torch applied membranes should be avoided.

9.7.7.3CT—Bituminous Concrete Overlay

The membrane waterproofing shall be protected by a bituminous concrete overlay. The minimum thickness of the bituminous overlay atop the membrane waterproofing shall be 3 inches on all new bridges as well as all existing bridges that have adequate load carrying capacity. The Designer shall consult with the **CTDOT** Pavement Design Unit for the pavement structure.

Existing bridges that do not have adequate load carrying capacity for a 3-inch overlay shall receive a 2.5-inch thick bituminous concrete overlay. The Designer shall consult with the **CTDOT** Pavement Design Unit for the pavement structure.

All bridge decks receiving a bituminous concrete overlay, excluding culverts, pipes and buried structures, shall have gutter lines sealed.

Bituminous concrete overlays are difficult to fully compact along gutter lines and are prone to leakage and porosity. Sealing along the gutter line of bridges is to reduce infiltration of water and deicing chemicals.

Newly built bridges, bridges being milled and overlaid, and bridges being rehabilitated such as in a strip, patch, membrane and overlay project.

Bridge plans shall clearly identify the location and length along the parapet, curb, or barrier where the gutter line sealing shall be performed.

The pay item "Gutter Line Sealing for Bridges" shall be used. Other pavement sawcut or sealing items shall not be used for this application.

9.7.7.4CT—Weepholes

Weepholes shall be provided in cast-in-place concrete decks to drain the membrane and overlay interface. Weepholes shall be placed along gutterlines adjacent to deck joints at the low end of spans on the low side of cross slopes. Weepholes shall outlet on the inside of fascia members. The outlet pipe of the weephole shall be extended as required so as not to drain onto the superstructure members and components. Weepholes shall not be located over travel lanes, shoulders, sidewalks, parking areas, or in spans over railroad tracks. Where easily achieved during rehabilitation projects, existing weepholes should be plugged and paved over when not in accordance with these requirements.

9.10—REFERENCES

This section shall be supplemented with the following:

CTDOT. *Connecticut Highway Design Manual*. Connecticut Department of Transportation, Newington, CT, 2023.

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SECTION 10

FOUNDATIONS

Commentary is opposite the text it annotates.

10.4—SOIL AND ROCK PROPERTIES

10.4.6—Selection of Design Properties

10.4.6.6—Erodibility of Rock

This section shall be replaced with the following:

Consideration should be given to the physical characteristics of the rock and the condition of the rock mass when determining a rock's susceptibility to erosion in the vicinity of bridge foundations. Physical characteristics that should be considered in the assessment of erodibility include cementing agents, mineralogy, unconfined compressive strength, rock quality designation (RQD), joint spacing and orientation, joint roughness and alteration, and weathering.

C10.4.6.6

This section shall be replaced with the following:

There is no consensus on how to determine erodibility of rock masses near bridge foundations. Refer to HEC-18 (FHWA, 2012) and Arneson et al. (2012) when determining the potential for a rock mass to scour.

10.5—LIMIT STATES AND RESISTANCE FACTORS

10.5.1—General

This section shall be supplemented with the following:

Changes in foundation conditions resulting from scour, as specified in Article 2.6.4.4.4CT, shall be considered.

10.5.2—Service Limit States

10.5.2.1—General

Revise the 3rd bullet as follows:

- scour design flood

C10.5.2.1

The 3rd paragraph shall be revised as follows:

The scour design flood is defined in Article 2.6.4.4 and its consideration under the service limit state specified in Article 2.6.4.4.4CT.

10.5.3—Strength Limit States

10.5.3.1—General

Revise the 2nd bullet as follows:

- loss of lateral and vertical support due to scour, and

C10.5.3.1

The 4th paragraph shall be revised as follows:

The scour design flood is defined in Article 2.6.4.4. and its consideration under the strength limit state is specified in Article 2.6.4.4.4CT.

10.5.4—Extreme Event Limit States

10.5.4.1—Extreme Events Design

C10.5.4.1

This section shall be replaced with the following:

Extreme events include scour check flood, vessel and vehicle collision, seismic loading, and other site-specific situations that the Engineer determines should be included.

Appendix A10 gives additional guidance regarding seismic analysis and design.

10.5.5—Resistance Factors

10.5.5.1—Service Limit States

The 2nd paragraph shall be replaced with the following:

A resistance factor of 1.0 shall be used to assess the ability of the foundation to meet the specified deflection criteria after soil removal due to the scour design flood.

10.5.5.2—Strength Limit States

10.5.5.2.1—General

The 3rd paragraph shall be revised as follows:

The foundation resistance after the scour design flood shall provide adequate foundation resistance using the resistance factors given in this Article. The resistance factors shall be those used in the Strength Limit State, without scour.

C10.5.5.2.1

The 7th paragraph shall be revised as follows:

Design for the scour design flood must satisfy the requirement that the factored foundation resistance is greater than the factored load determined with the scoured soil removed.

10.5.5.3—Extreme Event Limit States

10.5.5.3.2—Scour

The 1st paragraph shall be replaced with the following:

The provisions of Article 2.6.4.4.4CT shall apply to the changed foundation conditions resulting from scour. Resistance factors at the extreme event shall be taken as 1.0 except that for uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

C10.5.5.3.2

This section shall be replaced with the following:

The specified resistance factors should be used provided that the method used to compute the nominal resistance does not exhibit bias that is unconservative. See Article C10.7.3.6 for guidance regarding bias values for pile resistance prediction methods.

Design for scour is discussed in Hannigan et al. (2016).

10.6—SPREAD FOOTINGS

10.6.1—General Considerations

10.6.1.1—General

This section shall be supplemented with the following:

Generally, for cast-in-place construction, gravity-type or semi-gravity L-type sections shall be used for abutments, wingwalls and retaining walls founded on rock. Generally, structural underwater concrete is not permitted.

10.6.1.2—Bearing Depth

The 1st paragraph shall be replaced with the following:

Where the potential for scour, erosion or undermining exists, shallow foundations (i.e., spread footings) shall be located such that the top of the footing is located at or below

C10.6.1.2

The 2nd paragraph shall be replaced with the following:

For spread footings founded on excavated or blasted rock, attention should be paid to the effect of excavation and/or blasting. Blasting of highly resistant competent rock formations may result in overbreak and fracturing of the

the maximum anticipated depth of scour, erosion, or undermining as specified in Article 2.6.4.4.3CT.

rock to some depth below the bearing elevation. Blasting may reduce the resistance to scour within the zone of overbreak or fracturing. See Article 10.4.6.6 regarding factors affecting erodibility of rock.

This section shall be supplemented with the following:

The top of all footings in soil should be a minimum of 12 inches below the finished grade. The bottom of all footings in soil shall not be less than 4 feet below, measured normal to the finished grade.

There is no minimum embedment for footings placed on competent rock.

10.6.3—Strength Limit State Design

10.6.3.4—Failure by Sliding

This section shall be supplemented with the following:

Generally, the use of footing keys to develop passive pressure against sliding is not allowed. The use of passive earth pressures along the sides of foundations to prevent sliding is also not allowed. Resistance from sliding shall be attained through friction between the foundation and the supporting material.

10.7—DRIVEN PILES

10.7.1—General

10.7.1.2—Minimum Pile Spacing, Clearance, and Embedment into Cap

This section shall be supplemented with the following:

Pile foundations supporting abutments shall have a minimum of two rows of piles, unless the piles are incorporated into a fully integral abutment. Piles shall be anchored to and embedded in the footings a minimum of 12 inches.

The top of all footings should be a minimum of 12 inches below the finished grade. The bottom of all pile caps shall not be less than 4 feet below, measured normal to the finished grade.

10.7.1.4—Batter Piles

This section shall be supplemented with the following before the first paragraph:

Piles may be installed vertical or battered. The path of battered piles should be checked to ensure the piles remain within the right of way and do not interfere with piles from adjacent and existing substructure units, nor conflict with temporary sheeting, cofferdams or utilities.

10.7.1.5—Pile Design Requirements

This section shall be supplemented with the following:

Piles may be either end bearing or friction or a combination of the two. Piles end bearing on bedrock or dense hardpan typically are steel H-piles. Piles driven through a high compacted fill or native soil containing numerous boulders and cobbles shall be steel H-piles. H-piles conforming to the requirements of ASTM A709 Grade 50 is preferred. Friction piles shall be used for most other cases. Generally, friction piles are precast concrete, cast-in-place concrete or prestressed concrete. Timber piles are not permitted. For H-piles, pile point reinforcement shall be prefabricated.

Maximum pile spacing and maximum nominal resistance per pile should be utilized to minimize the number of piles. The lateral resistance of a pile pattern is the combination of the lateral component of the force acting on the battered piles and the lateral resistance of each pile, vertical and battered, in the pattern.

10.7.3—Strength Limit State Design

10.7.3.6—Scour

The 1st and 2nd paragraphs shall be replaced with the following:

The effect of scour shall be considered in determining the minimum pile embedment and the required nominal driving resistance, R_{ndr}. The pile foundation shall be designed so that the pile penetration after soil has been removed due to the scour design flood satisfies the required nominal axial and lateral resistance.

The resistance factors shall be those used in the design without scour. The side resistance of the material lost due to scour should be determined using a static analysis and it should not be factored, but consideration should be given to the bias of the static analysis method used to predict resistance.

C10.7.3.6

The section shall be replaced with the following:

The piles will need to be driven to the required nominal bearing resistance plus the side resistance that will be lost due to scour. The nominal resistance of the remaining soil is determined through field verification. The pile is driven to the required nominal bearing resistance plus the magnitude of the side resistance lost as a result of scour, considering the prediction method bias. Bias is defined as the measured/predicted value of resistance, in this case pile side resistance. Typically, the average method bias, based on available databases should be used. This bias can be based on a national database, or based on a local database, if enough measurements are available to reliably establish an average value. Example bias values for various pile static resistance prediction methods based on a national database are provided in Paikowsky et al. (2004) and Allen (2005).

To use bias values to adjust pile resistance predictions, since a bias greater than 1.0 means the method predicts less resistance than is actually present and a bias less than 1.0 means that the method predicts more resistance than is actually present, the bias adjusted resistance is determined by multiplying the resistance lost due to scour by the bias value. Since in this case the goal is to estimate lost resistance due to scour, a conservative estimate is obtained when the method predicts more resistance than is actually present.

Another approach that may be used takes advantage of dynamic measurements. In this case, the static analysis method is used to determine an estimated length. During the driving of test piles, the side resistance component of the bearing resistance of pile in the scourable material may be determined by a signal matching analysis of the restrike dynamic measurements obtained when the pile tip is below the scour elevation. The material below the scour elevation must provide the required nominal resistance after scour occurs.

In some cases, the flooding stream will carry debris such as wood or ice that will induce horizontal loads on the piles.

Additional information regarding pile design for scour is provided in Hannigan et al. (2016).

10.7.4—Extreme Event Limit State

The 4th paragraph shall be revised as follows:

When designing for scour, the pile foundation design shall be conducted as described in Article 10.7.3.6, except that the scour resulting from the check flood and resistance factors consistent with Article 10.5.5.3.2 shall be used.

10.7.7—Determination of R_{ndr} Used to Establish Contract Driving Criteria for Nominal Bearing Resistance

The last bullet shall be replaced with the following:

- extreme event limit state nominal bearing resistance considering scour as specified in Article 10.7.4.

10.7.9—Probe Piles

This section shall be supplemented with the following before the first paragraph:

Test piles are typically required to establish pile order lengths and pile capacity for friction piles. If pile driving records and pile load test data are available for a site (e.g. a bridge widening where the same pile type is proposed), specifying the pile order length in the contract may be considered. Test piles with dynamic monitoring will still be required to establish the ultimate pile capacity. Static pile load tests may also be considered.

This section shall be supplemented with the following after the last paragraph:

Pile order lengths for end bearing piles on bedrock should be provided in the Contract when there is confidence in the subsurface profile. If a test pile(s) is being considered to establish order lengths for end bearing piles on bedrock, the benefit of the refined pile order length shall be weighed against the time required to obtain the production piles. Test piles with dynamic monitoring and/or static load tests may still be required to establish the ultimate pile capacity. The need for test piles is evaluated by the geotechnical engineer and should be included in the Geotechnical Report.

Readily available pile types should be used whenever possible; especially if the order length cannot be established until after test piles are driven. The location of test piles and load tests should be in areas that are readily accessible, and allow enough room for the Contractor to perform the work. A preconstruction test pile program may be considered on large projects where a benefit can be realized by establishing pile type(s) and pile capacity during the design phase.

If no test piles are specified for a given substructure, the estimated pile length shall be used as the pile order length. For these cases, the estimated pile length should be increased slightly to insure there is sufficient length.

10.9—MICROPILES

10.9.1—General

This section shall be supplemented with the following:

For conventional and semi-integral abutments, micropiles shall be designed for lateral loads by the Designer in coordination with the geotechnical engineer. Minimum requirements for the micropiles to meet lateral loading demands shall be shown on the plans. Axial loads shall be determined by the Designer and shall be shown on the plans.

For fully integral abutment bridges, the bond zone diameter and bond zone length is determined by the Contractor to meet the axial loading demands shown on the plans. All other aspects of the micropiles shall be fully designed and detailed for all loading conditions in accordance with these provisions and Appendix CT11.1.1 in coordination with the geotechnical engineer.

C10.9.1—General

This section shall be supplemented with the following:

Axial loads, including Ultimate Pile Capacity, shall be provided on the Contract Plans for the Strength I and Service I limit states.

Fully integral abutment bridges behave as a system with the superstructure and are sensitive to the pile foundation stiffness. Should changes be proposed to the micropile design, other aspects of the structure will be affected and may require redesign of components of the bridge above the foundation. Due to this, the Designer is required to perform a full design of the micropiles during the Design phase and proposed changes to the micropiles during construction shall be reviewed with extreme scrutiny.

The axial resistance of the micropile bond zone is highly dependent upon the means and methods used to install the micropile, so the final length and diameter of the bond zone is determined by the Contractor’s micropile designer to meet the controlling Factored Pile Loads (STL, SVC and UPC) shown on the plans. Determination of minimum micropile bond zone length and diameter is determined by the Designer.

10.9.3—Strength Limit State Design

10.9.3.3—Scour

This section shall be replaced with the following:

The provisions of Article 10.7.3.6 shall apply.

C10.9.3.3

This section shall be replaced with the following:

See Article C10.7.3.6.

10.10—REFERENCES

This section shall be supplemented with the following:

Hannigan, P.J., Rausche, F., Likins, G.E., Robinson, B.R., and Becker, M.L. *Design and Construction of Driven Pile Foundations*. Geotechnical Engineering Circular No. 12, Vol. I and II. Federal Highway Administration, U.S. Department of Transportation, Washing, DC, 2016

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SECTION 11

WALLS, ABUTMENTS, AND PIERS

Commentary is opposite the text it annotates.

11.2—DEFINITIONS

The definition for Abutment shall be supplemented with the following:

Semi-Integral Abutment—Semi-Integral Abutments are integral from the superstructure through a portion of the abutment stem.

This section shall be supplemented with the following:

Proprietary Systems—Wall systems that are on a list of approved retaining walls by **CTDOT**. No other proprietary retaining walls will be allowed. The following wall systems are defined as proprietary systems:

- Mechanically Stabilized Earth Walls
- Prefabricated Modular Walls

Non-proprietary Systems—Wall systems not defined as proprietary systems.

11.4—SOIL PROPERTIES AND MATERIALS

11.4.1—General

This section shall be supplemented with the following:

The effective angle of internal friction shall be taken equal to 35 degrees.

C11.4.1

This section shall be supplemented with the following:

The effective angle of internal friction specified herein is to coincide with the use of Pervious Structure Backfill.

11.6—ABUTMENTS AND CONVENTIONAL RETAINING WALLS

11.6.1—General Considerations

11.6.1.1—General

This section shall be supplemented with the following before the first paragraph:

Abutment types shall be selected considering structure aesthetics, foundation recommendations, structure location, and the loads it must transmit to the foundation. For structures over waterways, the abutment type and location should also be specified with consideration to hydraulic conditions at the site. Wherever possible, use stub (embankment) abutments for structures over waterways.

The acceptable abutment types include non-proprietary systems such as gravity walls, cantilever walls, counterfort walls and integral abutments. Abutments shall not be placed on fill supported by mechanically stabilized earth walls or prefabricated modular walls, except for Geosynthetic Reinforced Soil-Integrated Bridge Systems. Generally, for abutments and wingwalls founded on rock, where the footings are exposed, the abutment and wingwalls shall be designed without a toe.

Generally, abutments shall be constructed of reinforced concrete. Cast-in-place footings and stems shall be constructed in accordance with Section 5.

C11.6.1.1

Preference shall be given to integral abutments.

For the concrete compressive strengths of structures covered under this section, refer to Article 5.4.2.1.

Walls should be used where the construction of a roadway or facility cannot be accomplished with slopes. Walls can be classified as either retaining walls, or wingwalls. Retaining walls may be non-proprietary systems or proprietary systems.

The tops of retaining walls shall not be determined by the exact fill slope, but shall follow a smooth unbroken line for a more pleasing appearance. This may require the use of vertical curves, in which case elevations shall be given at 5 foot intervals.

Abutments and walls shall receive surface treatments in accordance with **BRSDM** [V1B].

Retaining walls shall be numbered in accordance with **BRSDM** [V1B].

11.6.1.3—Integral Abutments

This section shall be supplemented with the following:

For guidance on fully integral abutments, refer to Appendix CT11.1.

11.6.1.3.1CT—Semi-Integral Abutments

Typically, a joint will be detailed in the abutment stem. For design purposes, the connection of the superstructure to the substructure shall be modeled as a pinned connection. The lower portion of a semi-integral abutment shall be designed as a standard cantilever abutment with all vertical forces from the superstructure transmitted to lower portion of the abutment.

11.6.1.4—Wingwalls

This section shall be supplemented with the following before the first paragraph:

Wingwalls are used to provide lateral support for the bridge approach roadway embankment. For bridges with long wingwalls that are parallel to the roadway, the wingwall shall be referred to as a retaining wall.

Wingwalls shall preferably be U-type (parallel to the roadway). Flared wingwalls are permitted where conditions warrant such as for hydraulic performance of waterway crossings. Wingwall types shall be non-proprietary systems.

C11.6.1.4

Wingwalls shall be numbered in accordance with **BRSDM** [V1B].

11.6.1.4.1CT—Flared Type Wingwalls and Retaining Walls

The stems of flared type wingwalls shall be 1.33 feet wide at the top, with the rear face battered. The minimum batter shall be 10 :1.

11.6.1.4.2CT—U-Type Wingwalls with Sidewalks

The top of the wingwall section shall conform to the parapet width for the full length. If a batter is required, the rear face shall be vertical to approximately 12 inches below the sidewalk.

11.6.1.4.3CT—U-Type Wingwalls with Sloped Curb

The top of the wingwall section shall conform to the parapet width for the full length. If a batter is required, the rear face shall be vertical to approximately 12 inches below the bottom of subbase.

11.6.1.6—Expansion and Contraction Joints

This section shall be supplemented with the following:

Construction joints shall be placed as conditions warrant. Construction joints other than those shown in the Contract require prior approval from the Engineer. Expansion or contraction joints shall not be provided in footings. Footings for abutments and walls should be continuous including any steps provided. No reinforcement shall pass through expansion and contraction joints. Reinforcement shall pass through construction joints.

11.6.3—Bearing Resistance and Stability at the Strength Limit State

11.6.3.4—Subsurface Erosion

The 1st paragraph shall be replaced with the following:

For walls constructed along rivers and streams, scour of foundation materials shall be evaluated during design, as specified in Article 2.6. Where potential problem conditions are anticipated, adequate protective measures shall be incorporated in the design, including, but not limited to, locating the top of the wall footing below the scour depth determined in accordance with Articles 2.6.4.4.3CT and 2.6.4.5 or adding scour countermeasures to protect the wall footing.

11.6.7CT—Dampproofing

The rear face of cast-in-place and precast abutments and wall stems shall be damp-proofed.

11.6.8CT—Gravity and Counterfort Abutments

11.6.8.1CT—Steel Girder and Concrete Bulb Tee and Box Girder Bridges

Gravity, cantilever, and counterfort walls, with bridge seats, may be used for abutments.

Bridge seats shall be sloped with a minimum 2 inch draw from the front face of the backwall and closed at the ends. When determining bridge seat widths, consideration shall be given to superstructure jacking requirements as given in Article 6.7.9CT. On bridges constructed with box girders, the clear distance from the end of the box girder to the face of the backwall should be no less than two feet.

At the elevation of the bridge seat, the minimum dimension from the front face of the abutment stem to the centerline of the bearings shall be 1.25 feet. The minimum backwall thickness shall be 1.25 feet. Stem thicknesses may be less than the combined dimensions of the bridge seat and backwall.

11.6.8.2CT—Butted Deck Unit and Box Beam

Gravity, cantilever, and counterfort walls, with bridge seats, may be used for abutments.

Bridge seats shall be sloped to match the grade of beams. Provisions should be provided on the Contract plans to provide drainage at the low end of the span.

At the elevation of the bridge seat, the minimum dimension from the front face of the abutment stem to the centerline of the bearings shall be 9 inches. The minimum backwall thickness shall be 1.25 feet. Stem thickness may be less than the combined dimensions of the bridge seat and backwall.

11.7—PIERS

This section shall be supplemented with the following:

Pier types shall be selected considering structure aesthetics, foundation recommendations, structure location, and the loads it must transmit to the foundation. The acceptable concrete pier types include wall piers, open column bents, multiple column piers, and single column piers. The use of permanent steel piers and pier bents is discouraged due to future maintenance.

Generally, piers shall be constructed of reinforced concrete. Piers may be made integral with the superstructure.

Footings, concrete pier stems, columns, and pier caps shall be constructed in accordance with Section 5. Post-tensioned concrete pier caps may require concrete with greater compressive strengths.

All reinforcement in piers shall conform to Section 5. The concrete over the reinforcement in pier footings, stems, columns, and pier caps shall be 3 inches.

Circular concrete columns are preferred over rectangular concrete columns. With circular columns, spiral reinforcement is preferred over ties.

Cantilever concrete pier caps shall be post tensioned in order to eliminate cracking. The design shall be based on zero tension in the top of the cap after all losses have occurred under all loads.

The top surfaces of concrete piers and concrete pier caps shall have a transverse slope of 1 :10 (rise to run). The slope shall be in both directions from the centerline to the face of the pier with a minimum draw of 2 inches.

Drilling holes for anchor bolts will not be permitted in concrete pier caps for new structures. Anchor bolts installed before the concrete is placed shall be set and held accurately by a template. Anchor bolts to be set after the concrete is poured shall be set in forms that shall be placed before the concrete is poured. The Designer shall indicate in the Contract which method of setting anchor bolts is to be used.

For structures over waterways, the following criteria applies:

C11.7

This section shall be supplemented with the following:

If possible, on large projects with many piers, the type of pier shall be consistent throughout the entire project for reasons of economy.

While the design of steel pier caps is allowed, they are discouraged. For additional information, refer to Section 7.

For the concrete compressive strengths of structures covered under this section, refer to Article 5.4.2.1.

- Pier foundations on floodplains should be designed to the same elevation as pier foundations in the stream channel if there is likelihood that the stream channel will shift its location over the life of the bridge.
- Align piers with the direction of flood flows. Assess the hydraulic advantages of round piers, particularly where there are complex flow patterns during flood events.
- Streamline piers to decrease scour and minimize the potential for the buildup of ice and debris. Use ice and debris deflectors where appropriate.

Piers shall receive surface treatments in accordance with **BRSDM** [V1B].

11.7.2—Pier Protection

11.7.2.2—Collision Walls

This section shall be supplemented with the following:

To limit damage to piers by railroad equipment, collision walls shall be provided in accordance with **AREMA**. Extensions to collision walls may be required to satisfy site conditions. The top surface of the collision wall shall have a transverse slope of 12 :1.

To limit damage to piers by vehicular traffic, collision walls shall be provided. The minimum height of the wall shall be 42 inches, and shall be placed a minimum of 6 inches from the face of the pier.

11.7.2.3—Scour

This section shall be replaced with the following:

The scour potential shall be determined, and the design shall be developed to prevent failure from this condition as specified in Articles 2.6.4.4.3CT and 2.6.4.5.

11.7.3CT—Pier Types

11.7.3.1CT—Wall Piers

A wall pier consists of a solid wall that extends up from its foundation. Generally, wall piers or wall piers combined with open bents should be considered at water crossings. Wall piers offer minimal resistance to water and ice flows.

11.7.3.2CT—Open Column Bents

An open column bent consists of a pier cap beam and supporting columns in a frame-type structure. Open column bents should be considered for wide overpasses at low skews.

Open column bents founded on rock shall generally be designed with isolated footings while open column bents founded on soil shall generally be designed with combined footings. When these piers are founded on piles, they may be designed with either isolated or combined footings.

11.7.3.3CT—Multiple Column Piers

A multiple column pier consists of an individual column supporting each beam or girder. Multiple column piers should be considered for wide overpasses at low skews.

11.7.3.4CT—Single Column Piers

Single column piers are simple, easy to construct, require minimum space, and provide open appearance to traffic. Single column piers may have a hammer head pier cap. Hammer head piers should be considered for overpasses at high skews with tight alignment constraints. This type of pier provides open appearance when supporting structures with long spans.

11.10—MECHANICALLY STABILIZED EARTH WALLS

11.10.1—General

This section shall be supplemented with the following:

- If the wall supports a roadway where there is a possibility of future underground utilities and drainage structures.

C11.10.1

The 4th paragraph shall be replaced with the following:

The potential for catastrophic failure due to scour is high for MSE walls if the reinforced fill is lost during a scour occurrence. Consideration should be given to lowering the base of the wall or to alternative methods of scour protection, such as sheet pile walls and/or riprap of sufficient size, placed to a sufficient depth to preclude scour.

11.10.2—Structure Dimensions

11.10.2.2—Minimum Front Face Embedment

The 3rd paragraph shall be replaced with the following:

For walls constructed along rivers and streams, Article 11.6.3.4 applies, except that the embedment depth shall be established at a minimum of 2.0 ft below potential scour depth.

11.10.11—MSE Abutments

C11.10.11

This section shall be supplemented with the following:

Designers should reference **FHWA**'s *Design and Construction Guidelines for Geosynthetic Reinforced Soil Abutments and Integrated Bridge Systems* when designing GRS-IBS abutments.

11.10.12CT—Embankment Walls

Embankment walls are proprietary wall systems. Embankment walls are mechanically stabilized earth structures

C11.10.12CT

There are several approved manufacturers of these types of walls.

faced with dry cast concrete block that are less than 8 feet high and support an embankment. Embankment walls are typically used to support earth only, not roadways or where there is a potential for future underground utilities or drainage structures. The mechanical strength of the wall comes from the soil reinforcements comprised of either geogrids or welded wire mesh.

11.11—PREFABRICATED MODULAR WALLS

11.11.4—Safety Against Soil Failure

11.11.4.1—General

The 2nd paragraph shall be replaced with the following:

Passive pressures shall be neglected in stability computations, unless the base of the wall extends below the scour depth as specified in Article 11.6.3.4, freeze–thaw, or other disturbance. For these cases only, the embedment below the greater of these depths may be considered effective in providing passive resistance.

11.11.4.5—Subsurface Erosion

This section shall be replaced with the following:

Bin walls may be used in scour-sensitive areas only where their suitability has been established. The provisions of Article 11.6.3.4 shall apply.

11.11.9CT—Inverted Wall Systems

Inverted wall systems are modular block walls with a modified design methodology where smaller modular units are at the bottom of the wall and larger units at the top.

Due to the current sole source requirement, inverted wall systems can only be used where site conditions restrict the use of all other retaining wall systems. Inverted wall systems are well-suited for the specific scenario in which ground conditions restrict the use of temporary earth retaining systems (such as where ledge prohibits driven or drilled piles ; adjacent structures may be damaged due to vibrations) and open excavation is restricted (e.g. – undermining of adjacent structures, utilities, etc.; Rights-of-Way constraints).

The minimum length of the bottom unit shall be the lesser of 40% of the height of the exposed face of the wall and 6 feet.

For T-Wall modular blocks, top unit faces may extend to a maximum height of 10 feet to accommodate earth cover. Maximum stem lengths may be 32 feet.

C11.11.9CT

The only current inverted wall system is the T-Wall modular block wall provided by Reinforced Earth.

The minimum bottom length is as recommended by Reinforced Earth.

Maximum dimensions were provided by Reinforced Earth.

11.14CT—APPROACH SLABS

11.14.1CT—General

Approach slabs shall be provided on all bridges carrying State highways. Approach slabs shall be strongly considered on all bridges undergoing superstructure replacement and local road bridges.

C11.14.1CT

Approach slabs should extend the full width of the roadway, including shoulders and sidewalks, have a standard length of 16 feet and be 1.25 feet thick. Generally, approach slabs should follow the skew of the bridge for skew angles up to 35 degrees. For skew angles greater than 35 degrees, the ends of the approach slabs should be square to the roadway with a minimum length of 15 feet. Acute corners of approach slabs and approach pavement should be squared off for a distance of five feet from the gutter line. Approach slabs should be anchored to the bridge abutment.

Approach slabs shall be constructed in accordance with Section 5. Approach slabs shall be covered with a waterproofing membrane and a bituminous concrete overlay. All the material items used in the construction of the approach slabs, including the overlay, shall be included in the structure items and quantities.

All elevations necessary for the construction of the approach slabs shall be shown in the Contract. These elevations shall include the elevations at the point of application of grade line, the gutter lines and at shoulder break lines at both ends of the approach slabs.

For the concrete compressive strengths of structures covered under this section, refer to Article 5.4.2.1.

11.13—REFERENCES

This section shall be supplemented with the following:

AREMA. *Manual for Railway Engineering*. American Railway Engineering and Maintenance-of-Way Association, Lanham, MD, 2019.

FHWA. *Stream Stability at Highway* Structures, 4th ed. Hydraulic Engineering Circular No. 18. FHWA-1P-90-017. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2012.

FHWA. *Urban Drainage Design Manual*, 3rd ed. Hydraulic Engineering Circular No. 22. FHWA-NHI-10-009. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2013.

FHWA. *Design and Construction Guidelines for Geosynthetic Reinforced Soil Abutments and Integrated Bridge Systems*. FHWA-HRT-017-080. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2018.

APPENDIX CT11.1—INTEGRAL ABUTMENTS

CT11.1.1—Fully Integral Abutment

CT11.1.1.1—General

Fully integral abutments are defined as abutments that are integral from the superstructure through to the piles so that all thermal movements and girder end rotations are transferred from the superstructure through the abutment to the supporting piles. Fully integral abutments can be considered for single-span or multi-span continuous bridge structures.

The use of fully integral abutment bridges is generally a preferred structure alternative and shall be considered when the site geotechnical conditions and structure geometry are suitable, and when the criteria to specify such structures described in this section are met.

Limitations to the use of fully integral abutment bridges:

- Skew shall be less than 30 degrees.
- Maximum abutment height shall be 12 feet (to reduce passive earth pressure) from finished roadway grade to bottom of abutment stem/pile cap.
- The abutments at each end of the bridge shall have similar configuration and geometry. The difference between overall abutment heights at each end of the structure shall not exceed one (1) foot.
- Maximum total bridge length shall be 150 feet for steel bridges and 200 feet for concrete bridges.
- Maximum single span length limit is 150 feet.
- All girders shall be straight and girder depth shall not be less than the span-to-depth ratios specified in **BDS.**
- Piles shall be Grade 50 steel H-piles with minimum flange width of 10 inches and weak axis oriented (web is oriented perpendicular to centerline of girders) or concrete-filled pipe piles. The piles shall meet the pile ductility requirements.
- Minimum pile embedment length below abutment stem is 10 feet in soil. The designer must verify that pile fixity will be achieved. Fixity is determined during lateral pile analysis (P-Δ analysis) and defined in Article C6.15.2.
- Wingwalls shall be monolithic cantilevered U-type wingwalls parallel to the girders with a minimum length of 5 feet and a maximum length of 10 feet, as measured from the rear face of the abutment.

Based on the parametric study performed by New England Transportation Consortium Project No. 19-1, U-type wingwall orientation, in comparison to inline orientation, substantially reduces the transverse moments at the pile head. The soil mass contained between the wingwalls provides lateral stability to the abutment and thus significantly reduces unfavorable abutment movements. Any wingwall length beyond 10 feet shall be designed as a freestanding retaining wall with an expansion joint between the freestanding abutment stem and the wingwall capable of accommodating the full abutment movement in the longitudinal and lateral directions.

CT11.1.1.2—Analysis and Design Requirements

Approximate methods of structural analysis and the guidelines described in this section can be used to determine all the load effects to be used for the design of fully integral abutment bridges that satisfy the previously listed limitations. The use of fully integral abutment bridges that does not meet the "limitations to the use of fully integral abutment bridges" shall require a design variance justified by the designer and approval from the Transportation Division Chief of Bridges.

The design loads of fully integral abutment bridges that do not meet one or more of the criteria specified under the "limitations to the use of fully integral abutment bridges" shall be determined via refined analysis. For refined analysis, designers may use the flow charts included at the end of this section to determine the substructure design loads.

Abutments and piles shall be designed considering the combined load effects for all phases of construction.

All fully integral abutment bridge components shall be designed in accordance with these guidelines and in conjunction with the applicable sections of **BRSDM** and **BDS**.

CT11.1.1.3—Loads, Load Factors and Load Combinations

Loads: All permanent, construction and live loads shall be as specified in Section 3 except as otherwise noted in this section.

Load Combinations: Load combinations shall be as specified in Section 3 except as otherwise indicated in this section.

Dynamic Load Allowance: Dynamic load allowance shall be considered in the design of abutment and supporting piles.

Live load surcharge: Live load surcharge may be neglected in the presence of approach slabs in the final service state of the bridge. For the construction phase, live load surcharge loading shall be considered to account for construction loading.

Braking Force: Braking force can be neglected in the design of abutment components.

Earth Pressures: Substructures shall be designed for passive lateral earth pressure under thermal expansion. Design parameters for backfill material shall be in accordance with Article 3.11.1 or as per Geotechnical Report.

Passive pressure coefficient shall be estimated using Rankine Theory:

$$K_p = \frac{1+\sin(\emptyset)}{1-\sin(\emptyset)} = \tan^2\left(45+\frac{\emptyset}{2}\right) \quad \text{(CT11.1.1.3-1)}$$

Uniform Temperature: Thermal force due to only uniform temperature differentials on fully integral abutment bridges shall be considered in design.

Based on a study by Purdue University on behalf of FHWA (FHWA/IN/ JTRP-2011/16), "it was determined that the primary loading of integral abutment bridges is a result of temperature and shrinkage strains that occur in the superstructure over its life. Length and skew, therefore, are primary factors controlling the movement of these structures." According to the study, the maximum lateral demand of the pile can be estimated by converting the shrinkage and temperature strains into maximum longitudinal and transverse displacements which is shown to be experienced by the pile at the bridge's acute corner.

The following equations may be used to determine the pile longitudinal displacements for different skew angles:

For skews $0° \le \theta \le 30°$:

$$\Delta_L = F(\varepsilon_{\Delta T}+\varepsilon_S)\frac{L}{2}+0.006\theta \quad \text{(CT11.1.1.3-2)}$$

where:

Δ_L = Longitudinal deflection of supporting pile (in)
F = Restraint reduction factor, 0.6
$\varepsilon_{\Delta T}$ = Strain due to temperature differential (in/in), $\alpha\Delta T$
α = Coefficient of thermal expansion (1/°F)
ΔT = Maximum negative temperature differential (as a positive number) (°F)
ε_s = Shrinkage strain (in/in), 500×10^{-6} (recommended estimate)
L = Total structural length (in)
θ = Skew angle (degrees)

It was determined from the parametric analysis (FHWA JTRP-2011) that the displacement demand of the pile in the longitudinal direction (along the length of the structure) can be determined by multiplying the displacement due to unrestrained thermal and shrinkage strains by a constant reduction factor, as well as adding additional displacement to account for the rotation of the abutment as caused by the skew. The displacement is characterized by a bilinear curve given by the following equations. The reduction factor was determined to be approximately 0.6.

Secondary Load Effects: The secondary load effects on substructures due to thermal gradient, creep, shrinkage and differential settlements shall only be considered for bridges exceeding the total bridge lengths specified under limitations to the use of fully integral abutment bridges.

CT11.1.1.4—Design Methodology

CT11.1.1.4.1—Superstructure Design

Boundary conditions for superstructure to substructure connection shall be assumed to be pinned-roller connection for the design of positive moments of the superstructure.

Forces induced by thermal movement due to uniform temperature changes shall be equally distributed to each integral abutment. Total horizontal displacement shall be calculated considering the total length of the bridge and the displacement shall be apportioned to each of the abutments.

A deck concrete placement sequence shall be indicated on the plans such that all the dead load girder rotations will take place at the girder ends without transferring any rotational forces to the piles prior to attaining fully integral abutment behavior.

In order to prevent the transfer of rotational forces to the piles before the integral behavior, the deck end concrete shall be cast monolithically with abutment end diaphragms last after the rest of the deck concrete is placed. A single continuous deck concrete placement operation from the end of an abutment to another shall not be allowed. The deck concrete shall be placed simultaneously with abutment end diaphragms such that no cold or construction joint shall be formed within the deck.

A non-compressible material shall be placed under the girders as temporary bearings. The temporary bearings may only be designed for non-composite superstructure dead and construction loads.

Unreinforced neoprene elastomeric bearing pads alone, or in combination with precast concrete pads to accommodate different girder bearing elevations, may be considered for integral abutment bridge construction. These temporary bearings have been successfully built for integral abutment construction in the past.

CT11.1.1.4.2—Substructure Design

CT11.1.1.4.2a—Abutments

The integral abutment wall stems (the abutment portion below girder seat) and abutment end diaphragms (the abutment portion above girder seat) shall be designed to resist all dead and live loads transmitted from the superstructure, and the passive earth pressure of the backfill material placed behind the abutment.

Unbalanced loading conditions during all phases of construction shall be considered as indicated in Article 3.11.9CT.

The overall abutment height shall be kept as short as possible to minimize the effect of earth pressure. The bottom elevation of abutment stems shall be determined in accordance with Article 2.6.4.4 but shall not be less than 4 feet from the finished grade measured normal at the front face of the abutment.

The abutment width shall not be less than 3.5 feet to accommodate abutment end diaphragm construction. The end diaphragm shall have the same width and length as the abutment stem.

The abutment seats shall be roughened with 1/8-inch amplitude except under the bearings. Trowel smooth finish shall be provided under the bearings.

The bottom of girder bottom flange shall be located at least 4 inches from the abutment seat to facilitate concrete consolidation of abutment end diaphragm.

Integral abutments shall be designed considering the following Design Cases:

- **Design Case 1:** The abutment wall stem shall be designed for all vertical dead and live loads assuming that it is acting as a continuous beam supported by the piles. Horizontal reinforcement at the top and bottom faces of stem shall be provided for flexural resistance.

  Vertical shear reinforcement shall be provided for the stem shear resistance. Wall stem shall be considered to act as a continuous beam over the piles as supports.

  Abutment stem rear face vertical reinforcement shall be designed for passive earth pressure at thermal expansion. The abutment shall be assumed to act a cantilever fixed at the abutment seat and free at the bottom of the wall stem at thermal expansion. The abutment wall stem front face reinforcement shall be designed to resist load effects at thermal contraction without soil pressure. The front face vertical reinforcement may be considered the same amount as the rear face reinforcement conservatively for simplicity.

  The abutment stem horizontal bars at the front and rear faces shall be designed to resist the moment from passive earth pressure at thermal expansion. For front face positive moment reinforcement, the stem may be assumed to be a simply supported beam supported between two piles for conservatism. For rear face negative moment reinforcement, the stem may be assumed to be a continuous beam supported by the piles.

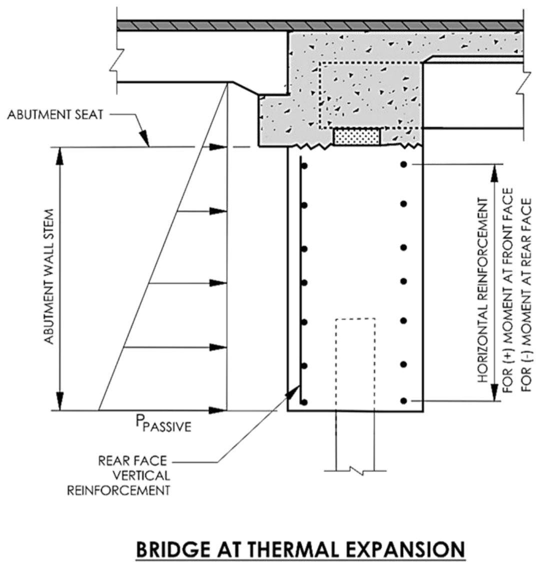


Figure CT11.1.1.4.2a-1

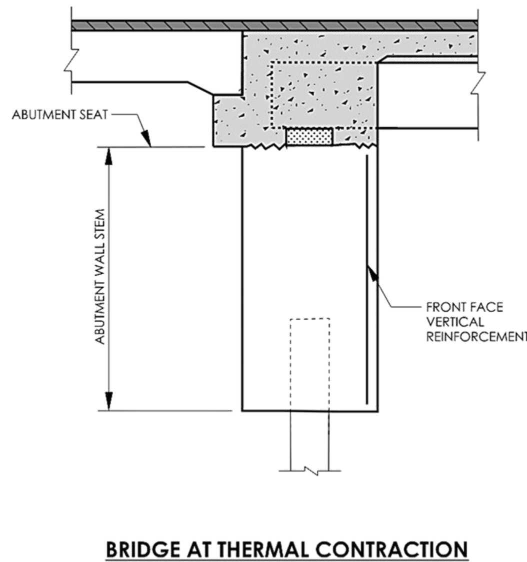


Figure CT11.1.1.4.2a-2

Where the pile spacing to stem height ratio meets the requirements of concrete design for D-Regions or deep beam analysis, a strut and tie analysis may be considered per Article 5.8 for additional reinforcement in the abutment stem.

- **Design Case 2:** The abutment end diaphragm front and rear face horizontal reinforcement shall be designed to resist passive earth pressure, assuming the diaphragm acts as a continuous beam spaning over the girders, which can be considered as supports. The span lengths shall be equal to the girder spacings along the bearing line.

  According to FHWA's recommendation, the continuous beam analysis may be simplified by assuming the beam acting as a simple span between piles and then taking 80% of the simple span moment to account for the continuity.

CT11.1.1.4.2b—Wingwalls

Each U-type cantilevered monolithic wingwall shall be designed to provide primary flexural resistance to moments about an axis parallel to its length and about its vertical axis. The shear, torsion, tension, and bi-axial bending moments about centroidal axes of abutment shall also be considered in design for adequate structural capacity at the connection between the wingwall and abutment. All horizontal wingwall reinforcement shall be doweled into the abutment for proper anchorage to transfer all the connection forces between the wingwall and the abutment.

A 1 foot chamfer shall be used between the abutment and wingwalls. The 1-foot shall be measured from the point of intersection of the rear face of the abutment and the wingwall.

Wingwalls can be rectangular or bottom tapered based on the design.

Designers may consider tapering the bottom of the wingwalls to reduce lateral earth pressures when needed. A tapered wingwall causes difficulty in compaction of backfill along the inclined surface underneath the wingwall taper during construction. On the other hand, a rectangular wingwall is simple to construct however it is subjected to high lateral earth pressure. A rectangular wingwall is also prone to concrete cracking at the bottom corner of the free end during thermal expansion.

U-type cantilevered monolithic wingwalls shall be assumed to be fully supported by the abutments without relying on the underlaying soil for bearing.

Wingwalls shall be designed considering the following Design Cases:

- **Design Case 1:** The wingwalls shall be designed for at-rest pressure when the bridge moves laterally and pushes the wingwall against the backfill.

- **Design Case 2:** The wingwalls shall also be designed for active pressure in conjunction with a vehicular impact force on the parapet. In this case, the wingwall moves laterally away from the backfill.

Additional reinforcement for integral abutments and wingwalls shall be included for temperature and shrinkage reinforcement to prevent surface cracking of the concrete.

CT11.1.1.4.2c—Piles

Single row layout of piles shall be considered to support fully integral abutment bridges. If H-piles are used, the piles shall be oriented such that the weak axes are perpendicular to the longitudinal axis of the bridge (in the direction of thermal movement) regardless of the skew. This reduces the flexural resistance of piles against thermal movements and provides resistance to lateral movements. Other pile types such as steel-encased concrete piles or micropiles may be considered as per the recommendation of the geotechnical engineer.

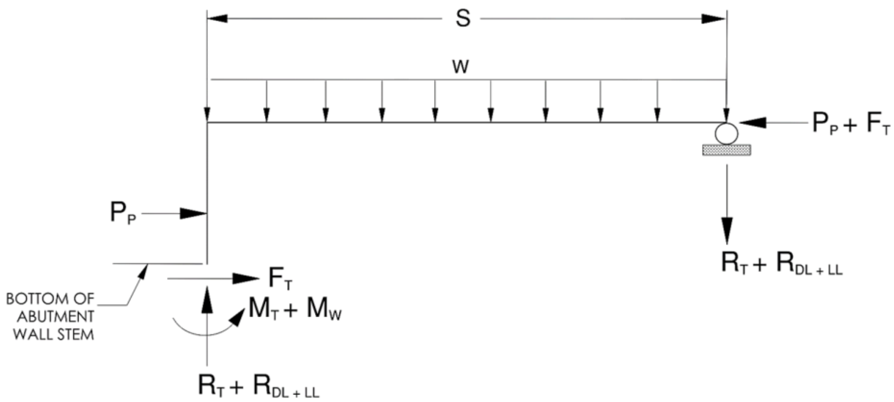
All piles shall extend to the location of point of fixity at which the pile is held firmly (the location at which zero deflection occurs in the pile) or deeper to achieve the intended integral abutment behavior.

All piles shall be placed such that the centerline of piles shall match the centerline of girder bearing to eliminate any additional moments induced on the pile due to the eccentricity of the superstructure gravity loads.

A pile is not required under each girder therefore the quantity of piles may be different than the girder quantity.

The pile design shall satisfy both geotechnical and structural capacity. The piles for abutments that are not scour susceptible shall be assumed fully braced. For scour susceptible abutments, the exposed portion of pile from the bottom of abutment to the first point of fixity shall be considered unbraced.

Piles shall be designed to resist all dead and live loads as well as all horizontal loads and movements. All piles shall be embedded a minimum of 2 feet into the integral abutment wall stems. Pile design forces shall be determined considering the idealized free body diagram shown below:



Idealized free body diagram

Figure CT11.1.1.4.2c-1

Pile dead load reactions, R_{DL}, shall be determined assuming that all dead loads are uniformly distributed to all piles.

Pile live load reactions, R_{LL}, shall be determined considering pile group action:

$$R_{LL} = \frac{W_{LL}}{N} + \frac{W_{LL}eX}{I} \qquad \text{(CT11.1.1.4.2c-1)}$$

where:

R_{LL} = Pile live load reaction (kips)
W_{LL} = Total live load at the abutment (kips)
N = Total number of piles in the abutment
e = Eccentriticy of total live load to center of pile group (kips)
X = Distance from a given pile to center of pile group (kips)
I = Moment of inertia of pile group (ft^2)

Girder end rotation due to all dead and live loads, M_W, shall be determined at thermal expansion. The moment induced by the end rotation shall be applied to each pile.

$$M_W = \frac{4EI}{L_E}\theta_W \qquad \text{(CT11.1.1.4.2c-2)}$$

$$\theta_W = \frac{WS^2}{24E_G I_G} \qquad \text{(CT11.1.1.4.2c-3)}$$

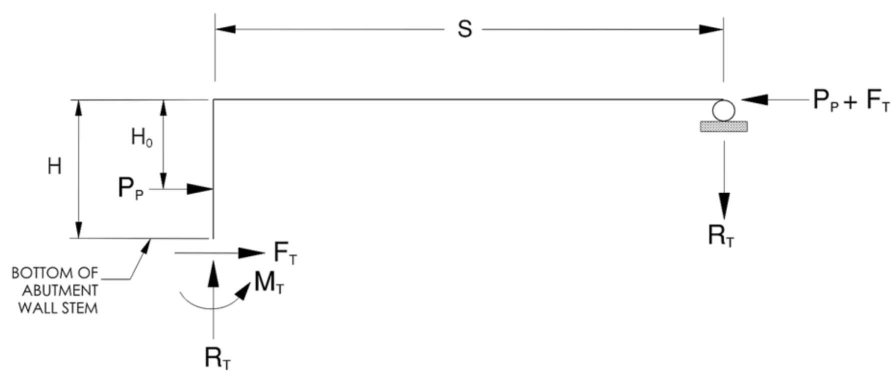
$$W = \frac{2\ N_P\ R_{DL+LL}}{N_G} \qquad \text{(CT11.1.1.4.2c-4)}$$

where:

M_W = Girder end moment induced by girder end rotation at thermal expansion (k-in)
θ_W = Girder end rotation at thermal expansion (rad)
W = Total dead and live loads acting on full span (kips)
R_{DL+LL} = Vertical Pile reaction due to dead and live loads (kips)
N_P = Number of piles
N_G = Number of girders
E = Modulus of elasticity of pile material (ksi)
I_G = Moment of inertia of composite girder (in^4)
E_G = Modulus of elasticity of girder material (ksi)
I = Moment of inertia of the pile with respect to the plane of bending (in^4)
L_E = Equivalent cantilever length, from bottom of abutment wall stem to fixed base of equivalent cantilever (in)
S = Bearing to bearing span length (in)

The axial force in each pile due to thermal load effects, R_T, shall be determined in accordance with the idealized free body diagram for thermal loading shown below:

$$R_T = \frac{P_P H_0 + F_T H + M_T}{S} \quad \text{(kips)} \qquad \text{(CT11.1.1.4.2c-5)}$$



Idealized free body diagram for thermal loading

Figure CT11.1.1.4.2c-2

where:

P_P = Passive earth pressure at thermal expansion (kips)
M_T = Pile bending moment due to thermal expansion (k-in)
F_T = Horizontal force induced due to thermal expansion (kips)
H = Total height of backfill (in)
H_0 = $\frac{2H}{3}$ (in)
S = Bearing to bearing span length (in)

The thermal load effects on each pile shall be determined using equivalent cantilever idealization method with either fixed-head or pinned-head boundary condition at the top of the pile as per the recommendation of the geotechnical engineer.

Piles that are subjected to factored design moments that are less than or equal to their plastic moment capacity shall be designed considering elastic behavior. For elastic design, pile design moment shall be limited to the plastic moment capacity of pile, the failure shall be assumed at yielding and redistribution of internal forces shall be neglected at ultimate capacity.

According to Transportation Research Record 1223 (Rational Design Approach for Integral Abutment Bridge Piles) a pile embedded in soil can be analytically modeled as an equivalent beam-column without transverse loads between the member ends and with a base fixed at a specific soil depth. Either a fixed head or pinned head for the beam-column approximates the actual rotational restraint at the pile head.

The pile bending moment, M_T, and horizontal force, F_T, induced by thermal movement for different boundary conditions shall be calculated as followings:

For a fixed-head boundary condition:

$$M_T = \mathrm{Min}\left(M_P, \frac{6EI\Delta}{L_E^2}\right) \quad \text{(CT11.1.1.4.2c-6)}$$

$$F_T = \frac{12EI\Delta}{L_E^3} \quad \text{(CT11.1.1.4.2c-7)}$$

For a pinned-head boundary condition:

$$M_T = \mathrm{Min}\left(M_P, \frac{3EI\Delta}{L_E^2}\right) \quad \text{(CT11.1.1.4.2c-8)}$$

$$F_T = \frac{3EI\Delta}{L_E^3} \quad \text{(CT11.1.1.4.2c-9)}$$

where:

Δ = Displacement (in)
E = Modulus of elasticity of pile material (ksi)
I = Moment of inertia of the pile with respect to the plane of bending (in^4)
L_E = Equivalent cantilever length, from bottom of abutment wall stem to fixed base of equivalent cantilever (in)
M_p = Plastic moment capacity of pile (k-in)

The piles shall be designed as a structural member subjected to both axial load and flexure. The design shall satisfy the **BDS** requirements for combined axial compression and flexural bending interaction relationship.

All piles shall meet width-to-thickness or slenderness ratio limits specified in **BDS** to fully develop their yield strength and to prevent local buckling prior to achieving their design yield strength.

CT11.1.1.4.3—Refined Analysis

Boundary conditions at the abutments, girder ends and piers shall be modeled to account for translational and rotational restraints, releases and stiffnesses.

All backfill soil springs shall be nonlinear and shall disengage for movement away from the backfill. Designers shall coordinate with a geotechnical engineer for appropriate spring type to consider site specific conditions.

Either direct soil-structure interaction or equivalent cantilever methods may be used to design the piles. If equivalent cantilever method is elected, the following deflection equation may be used to determine the equivalent cantilever length for piles that are fully fixed at the base and free to translate at the top:

$$L = \sqrt{\frac{n_{piles}12EI}{K}} \quad \text{(CT11.1.1.4.3-1)}$$

where:

n = number of piles
E = modulus of elasticity of pile material (ksi)
I = moment of inertia of the pile with respect to the plane of bending (in^4)
K = transverse pile stiffness (k/in)

CT11.1.1.5 – Figures and Details

Direct Soil-Structure Interaction Model:

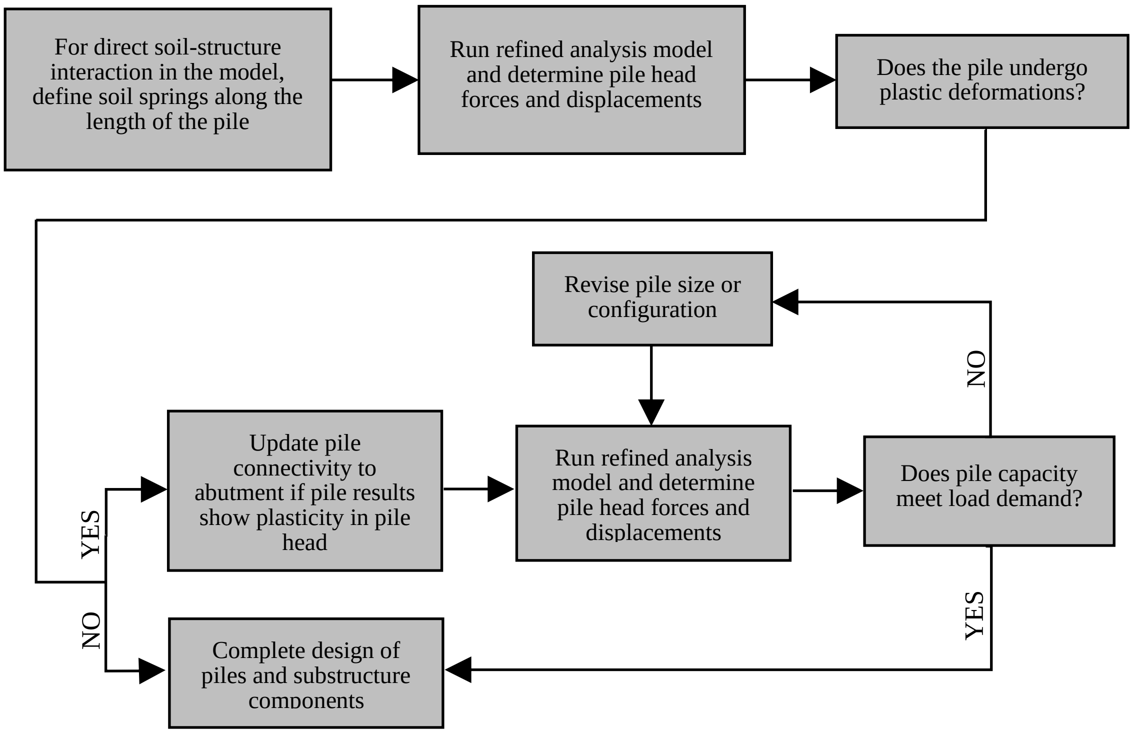


Figure CT11.1.1.4-1

Equivalent Cantilever Length Model:

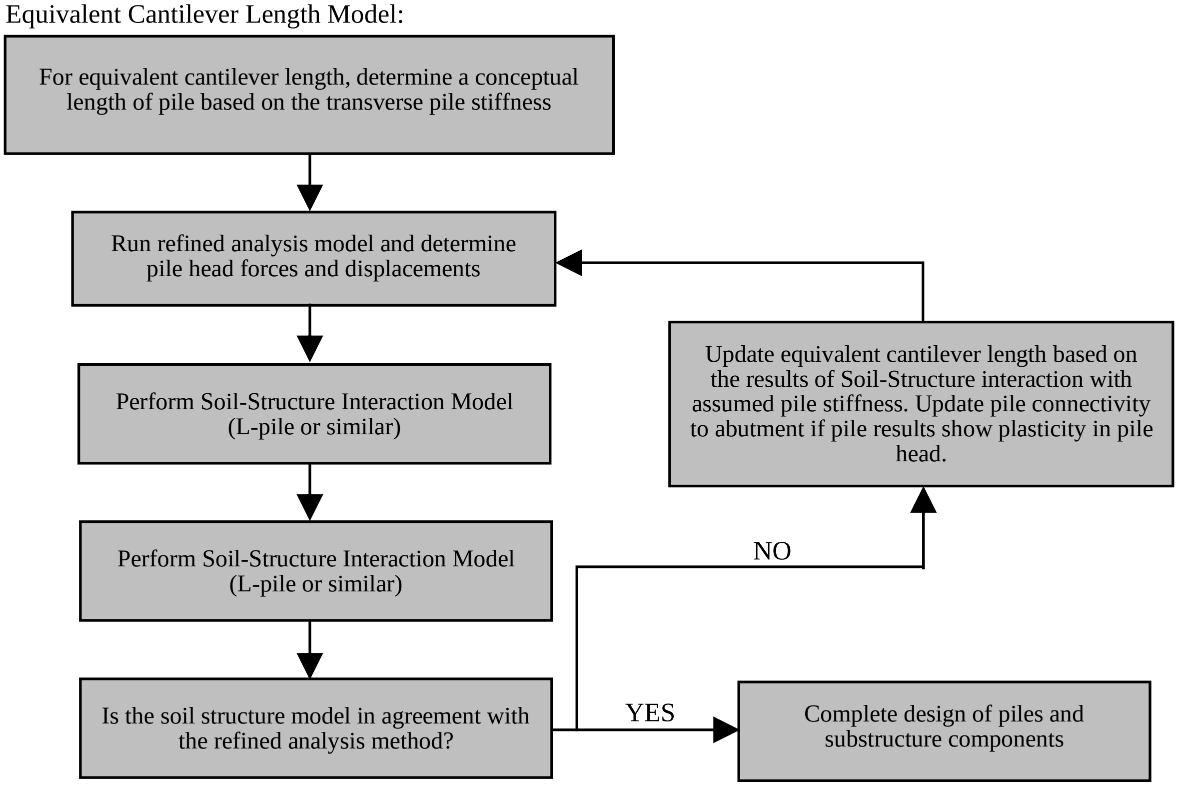
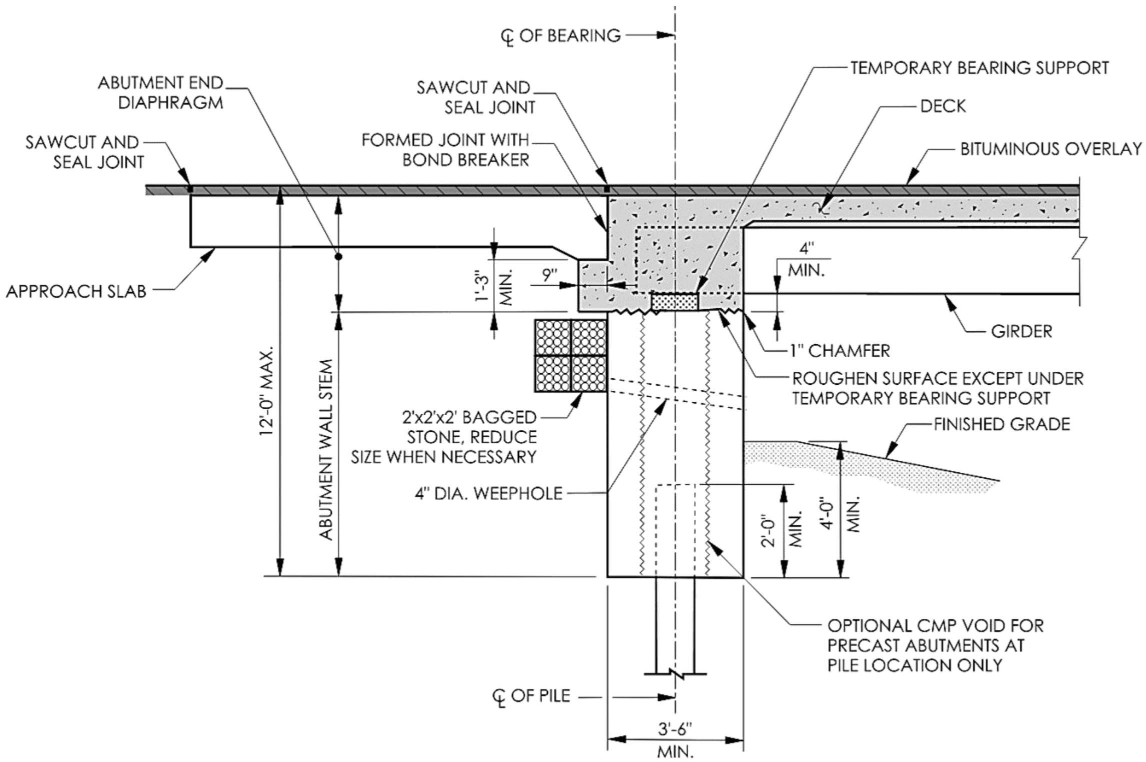


Figure CT11.1.1.5-2



FULLY INTEGRAL ABUTMENT SECTION
SCALE: NOT TO SCALE

Figure CT11.1.1.5-3

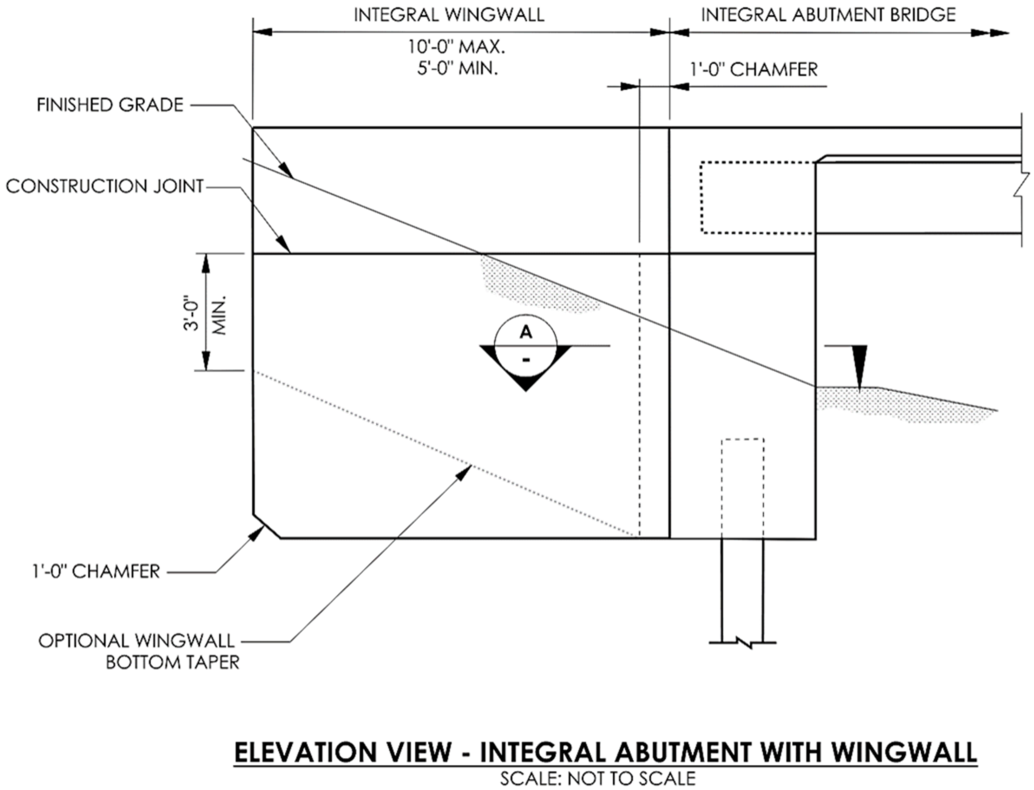


Figure CT11.1.1.5-4

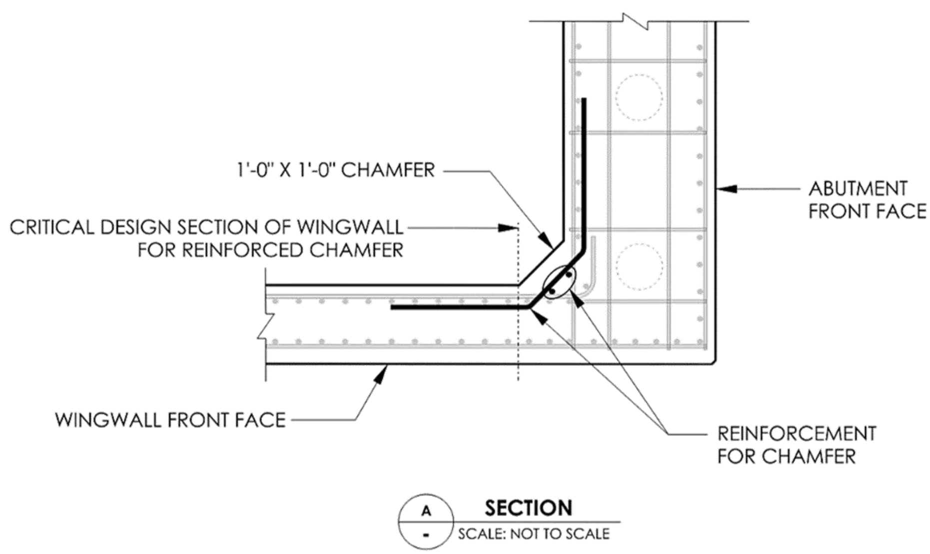


Figure CT11.1.1.5-5

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SECTION 12:

BURIED STRUCTURES AND TUNNEL LINERS

Commentary is opposite the text it annotates.

12.4—SOIL AND MATERIAL PROPERTIES

12.4.2—Materials

12.4.2.2—Concrete

This section shall be supplemented with the following:

The compressive strength of concrete, f'_c, cast-in-place box culverts and three-sided structures shall be no less than 4 ksi or the corresponding concrete strength of the adjacent concrete, whichever is greater.

Concrete for cast-in-place pile caps, wall stems, cut-off walls, return walls, sills, baffles, and headwalls shall meet the requirements of Class PCC04462.

Concrete for closure joints between precast concrete components shall meet the requirements of Class PCCXXX82, matching the compressive strength of the adjacent concrete members, or be Ultra High-Performance Concrete (UHPC).

The use of non-shrink grout or structural non-shrink grout in closure pours between precast concrete components is not permitted.

C12.4.2.2

This section shall be supplemented with the following:

Refer to **BRSDM** [V1B] for additional cast-in-place concrete requirements.

The use of cast-in-place concrete for entire new box culverts and three-sided frames is rare. Cast-in-place concrete is primarily used for closure pours to connect adjacent precast sections or to connect existing structures to new precast structures, and for short sections of structures used to extend existing structures.

Refer to the **SSC** for concrete classes.

Refer to Article 5.12.2.3.3e.

For Section 12, the term closure joint is used instead of the terms shear key or closure pour to describe a connection between precast sections. Closure joint is a term used by PCINE. Refer to Article 12.14.5.4 for additional requirements.

For Section 12, the terms conventional concrete and UHPC shall be used to differentiate between the two cast-in-place concrete materials.

The aggregate size of the concrete class specified was selected to facilitate concrete placement with limited distances between existing concrete surfaces, reinforcement, and formwork. The exposure factor of the concrete class was specified to match that required for the precast concrete.

12.4.2.4—Precast Concrete Structures

This section shall be supplemented with the following:

The concrete for the precast box culverts shall meet the requirements of Class PRC05062.

The concrete for the precast three-sided frames shall meet the requirements of Class PRC05062, Class PRC06062 or Class PRC08062, as required by the design.

C12.4.2.4

This section shall be supplemented with the following:

Class PRC05062 has traditionally been specified for precast concrete box culverts of all sizes.

The precast concrete classes selected match those tabulated in the **SSC**.

The use of a concrete class other than Class PRC05062 will impact the type of concrete used in the cast-in-place closure joint since the concrete is required to a compressive strength comparable to that of the adjacent concrete members. See Article 12.4.2.2. The maximum compressive strength of conventional cast-in-place concrete is generally 5 ksi. See the **SSC**.

Substitutions or changes to the precast concrete class designation shown on the plans that revise the compressive concrete strength shall be justified by the Contractor and submitted in a request for Request for Change (RFC) to the **CTDOT** for review and action. The use of a precast concrete class that is different than that shown on the plans shall require a revised load rating by the designer to document and reflect the material property change.

Concrete for precast cut-off walls, return walls, sills, baffles, and headwalls shall meet the requirements of Class PRC05062.

The use of dry cast concrete for the fabrication of precast box culverts, precast three-sided frames and other associated precast components is not permitted.

Class PRC05062 has traditionally been specified for miscellaneous precast concrete elements and components.

12.4.2.7—Steel Reinforcement

This section shall be supplemented with the following:

Refer to Article 5.4.3 for reinforcing steel requirements.
The use of welded wire fabric is not permitted.

12.4.2.11CT—Protection of Miscellaneous Steel Components

All steel components and hardware permanently embedded in precast concrete components, for either temporary or permanent use, shall be galvanized in accordance with ASTM A153 or ASTM B695, Grade 50, or be stainless steel.

Embedded steel components may be for either temporary or permanent use and may include but not be limited to lifting fixtures, keys, threaded inserts, bolts, devices, attachments, miscellaneous hardware, and form ties cast into precast concrete components.

12.11—REINFORCED CONCRETE CAST-IN-PLACE AND PRECAST BOX CULVERTS AND REINFORCED CAST-IN-PLACE ARCHES

12.11.1—General

This section shall be supplemented with the following:

Reinforced concrete box culverts may be made of either precast or cast-in-place concrete. Generally, when conditions warrant a box culvert, for reasons of economy, it shall be made of precast concrete. Full-length cast-in-place concrete box culverts shall only be used with the approval of **CTDOT**. Sections of box culverts may be cast-in-place when required by site conditions such as transitions between different size culverts, transitions between new and existing box culverts, adjacent utilities cannot be relocated, or highly skewed culvert ends.

The culvert dimensions shall be consistent with the hydraulic characteristics of the waterway. Preferably, the height of the box culvert (the dimension from the top of the floor (invert) to the bottom of the roof) should be a minimum of 6 feet to facilitate its maintenance and inspection. For culverts that are designed to “silt in” with soil, the height should be measured from the invert of the channel.

For precast culverts, the size selection should be coordinated with the manufacturers to be consistent with standard sizes that are readily available. On projects requiring more than one culvert, of different size openings, an economic study should be conducted to determine if it is possible to use the same size opening for more than one structure.

Box culverts do not need to be analyzed for scour. However, erosion countermeasures may be required if recommended by the Hydraulics Report.

Generally, box culverts shall be founded on 12 inches of "Granular Fill" to provide slightly yielding uniformly distributed support over the bottom width of the box section. The fill shall extend 2 feet beyond the sidewalls of the box culvert.

Box culverts founded on unyielding foundations, such as rock or piles, are not permitted.

Joints between precast units shall be grouted and covered with geotextile fabric.

12.11.1.1CT—Cutoff and Return Walls

The inlet and outlet ends of box culverts shall rest on cutoff walls. The cutoff walls shall have return walls below the outside walls that extend a minimum of 4 feet from the rear face of the cutoff wall. These walls shall be embedded a minimum of 4 feet below the finished elevation of the bottom of the channel. The walls shall have a minimum thickness of 12 inches. The floor of the box culverts shall be connected to the cutoff walls with dowels.

12.11.1.2CT—Nosings between Adjacent Parallel Multicell Box Culverts

The inlet and outlet ends of the walls between adjacent parallel multicell box culverts shall be protected with nosings. The nosings may be either cast-in-place or precast. The nosings shall be founded on the cutoff wall and connected to the walls.

The maximum allowable joint width between adjacent parallel units shall be one inch. In order to provide a positive means of lateral bearing between parallel units, after placing the nosing, the joint shall be filled with sand made flowable by mixing it with water.

12.11.1.3CT—Sills

Sills shall be provided at the inlet and outlet ends of box culverts when warranted by hydraulic or environmental conditions. The dimensions shall be as recommended by the Hydraulic Report. The sills shall have a minimum thickness of 12 inches and shall be connected to the floor of the box culverts with dowels.

12.11.1.4CT—Headwalls

Headwalls at the inlet and outlet shall be provided to satisfy the site grading conditions. The headwalls shall have a minimum thickness of 1.25 feet at the top. For precast box culverts, dowel bar mechanical connectors shall be used to connect headwall stems to the roof of the structure. The rear face of headwalls shall be dampproofed. Railings or fences

C12.11.1.4CT

The box culvert roof thickness may be governed by the development of the reinforcement required to connect the headwall to the box culvert.

shall be placed on all headwalls in accordance with the requirements of Article 13.13CT.

Where headwalls lie within the deflection zone of guiderail, the headwall projection above grade shall be limited.

For projection limits, refer to **CTDOT** *Highway Standard Sheets* for guidance.

12.11.1.5CT—Wingwalls

Generally, cast-in-place concrete wingwalls shall be provided at the inlet and outlet of all box culverts. The Designer should coordinate with the hydraulic engineer as to the appropriate angles for the flared wingwalls. The wingwalls should abut the ends of the outside walls of the box culvert. Wingwall stems and footings shall be made independent of the culvert walls, cutoff and return walls. The elevation of the bottom of the wingwall footings shall match the cutoff, return walls, and frame foundation. The minimum thickness at the top of the wingwall stems shall be 1.25 feet. The rear face of wingwalls shall be dampproofed. Railings or fences shall be placed on all wingwalls in accordance with the requirements of Article 13.13CT.

Wingwalls shall preferably be U-type (parallel to the roadway). Flared wingwalls are permitted where conditions warrant such as for hydraulic performance of waterway crossings. The acceptable wingwall types shall only include non-proprietary systems.

12.11.1.6CT—Dampproofing

Exterior faces of side walls shall be dampproofed.

When the distance from the top of the buried structure to top of the roadway is greater than 2 feet, the exterior face of the top slab shall also be dampproofed.

When the distance to top of the buried structure from the top of the roadway is less than 2 feet, refer to Article 12.11.1.9CT.

12.11.1.7CT—Subsurface Drainage

Provisions for subsurface drainage are not required for the culvert and frame backfill.

12.11.1.8CT—Backfill Requirements

Unless otherwise directed, all box culverts and their associated wingwalls shall be backfilled with "Pervious Structure Backfill" in accordance with the requirements of **BRSDM** [V1B].

Place a wedge of Pervious Structure Backfill above a slope line starting at the top of the heel and extending upward at a slope of 1 :1.5 (rise to run) to the bottom of the subbase.

Rock fill or boulders shall not be placed within two feet of top of box culverts.

In cut situations, if the material is soft silt or clay, the backfill limits shall be determined by the Designer and submitted for review and approval with the Geotechnical Report.

12.11.1.9CT—Membrane Waterproofing

Membrane waterproofing shall be applied to box culverts and precast concrete three-sided rigid frames. The membrane shall cover the entire exterior surface of the roof and extend 12 inches down the sidewalls.

When the distance from the top of the buried structure to top of the roadway surface is less than 2 feet, the preferred membrane waterproofing shall be "Membrane Waterproofing (Cold Liquid Elastomeric)."

When this distance is greater than 2 feet, the membrane waterproofing may be either Cold Liquid Elastomeric or Woven Glass Fabric.

12.11.1.10CT—Railing and Fences

For railing and fence requirements, refer to Article 13.13CT.

12.11.1.11CT—Reinforcement Details

The reinforcement shall conform to the requirements of Article 5.10.

Reinforcement shall be provided in haunches.

12.11.1.12CT—Minimum Thickness of Floor, Sides and Roof

The minimum thickness of precast concrete box culvert floor, sides and roof shall be 8 inches.

The minimum thickness of cast-in-place concrete box culvert floor, sides and roof shall be 12 inches.

C12.11.1.12CT

The thickness of the roof of the box culvert sections at the inlet and outlet may be governed by the development of the reinforcement required to connect the headwall to the box culvert.

12.14—PRECAST REINFORCED CONCRETE THREE-SIDED STRUCTURES

12.14.1—General

This section shall be supplemented with the following:

Precast reinforced concrete three-sided structures may be either a rectangular frame (with vertical walls and a horizontal roof), an arch-topped structure (with vertical walls) or an arch (with no walls). These structures have separate foundations with footings/pile caps, with or without a vertical stem (pedestal walls). Foundations adjacent to or within waterways subjected to scour shall meet the requirements of Article 2.6.4.

C12.14.1

This section shall be supplemented with the following:

The **CTDOT** has coordinated fabricators of precast reinforced concrete three-sided rectangular frame structures in determining the supplemental requirements in Article 12.14. Not all fabricators can meet every requirement. For conditions not addressed, designers are directed to coordinate with potential fabricators to be consistent with fabrication standards, practices, and limits.

Rectangular frames are also referred to as flat top frames.

Precast reinforced concrete arch-top and arch structures are proprietary products that are traditionally designed by a fabricator. The use of these structure types requires a design-build item in the construction contract. The construction specification requirements shall require working drawings, drawing submittals for design and fabrication drawings, design calculations and load ratings. Construction schedules shall include the time required to prepare and review the working drawing submittals for the design and load ratings in advance of fabrication. Load ratings must be reviewed by the **LRS** before the working drawing submittal can be returned to the Contractor.

Precast reinforced concrete three-sided structures typically provide clear spans from approximately 10 feet to approximately 50 feet. The maximum clear span for flat-top structures is approximately 40 feet. Three-sided structures are generally specified for clear spans greater than 20 feet. See Article 12.14.4 for span limits of flat top frames.

For foundations adjacent to or within waterways placed on spread footings may consider the use of an invert slab to mitigate the impacts of scour. The use of a short-span bridge should be investigated before a final determination is made to use a precast reinforced concrete three-sided structure.

Refer to the **SSC** for pay item names.

For structures over watercourses, three-sided flat-top structures are preferred over arch top structures because they provide a hydraulic opening greater than arches of equivalent clear span and maximum clear height when flowing full.

The use of multi-barrel three-sided structures is discouraged due to potential for the buildup of debris and ice at the adjacent walls at the inlet.

12.14.2—Materials

12.14.2.1—Concrete

This section shall be supplemented with the following:

Refer to Article 12.4.2.2 for additional requirements.

12.14.2.2—Reinforcement

This section shall be supplemented with the following:

Refer to Article 12.4.2.7 for additional requirements.

12.14.2.2.1CT—Protection of Miscellaneous Steel Components

Refer to Article 12.4.2.11 for additional requirements.

12.14.3—Concrete Cover for Reinforcement

This section shall be supplemented with the following:

Concrete cover for reinforcement shall meet the requirements of Table 12.14.3-1CT. This requirement is not applicable to reinforcement projecting from the precast sections that will be subsequently embedded in cast-in-place concrete closure joints, decks, or headwall stems, railings or curbs.

C12.14.3

This section shall be supplemented with the following:

The concrete cover for reinforcement on the exterior (exterior) face of the roof is independent of subsequent treatment on that surface, such as the placement of a waterproofing membrane or a cast-in-place concrete deck.

Table 12.14.3-1CT

Environmental exposure conditions	Location	Concrete Cover (in.)
Precast – structures not exposed to sea water	Top surface of roof	2.5
	All other surfaces of roof and walls	2
Precast - structures exposed to sea water	All surfaces of roof and walls	3

12.14.4—Geometric Properties

This section shall be supplemented with the following:

The clear span and clear height of the structure opening shall meet the requirements of the Hydraulic report.

The maximum design span of the precast sections for flat top frames shall be limited to 35 feet.

The maximum rise of the precast sections for flat top frames shall be limited to 10 feet to facilitate shipping and handling. Wall heights shall be equal. Pedestal walls shall be used to support the precast section walls to obtain greater clear heights to meet site constraints.

Preferably, the clear height of the structure shall be a minimum of 6 feet to facilitate maintenance and inspection.

C12.14.4

This section shall be supplemented with the following:

The term clear span is used to describe a hydraulic feature The clear span is the perpendicular distance between the interior face of the walls.

The clear height is the dimension from the top of the channel invert to the bottom of the roof. The clear height may vary depending upon the shape of the channel, grade of the channel, and the interior shape of the roof.

Designers shall coordinate with the hydraulics engineer on the structure dimensions to ensure the requirements of each discipline are met. The haunches on the interior surfaces of the wall to roof connection and the shape of the roof, flat top or arch, shall be reflected in the structural and hydraulic models.

The term design span is used to describe a structural variable. The design span is the distance measured between the centerlines of the walls at the mid-width of each wall. The design span is always greater than the clear span. For non-skewed sections, the clear span and design span are perpendicular to the wall face and parallel to the precast section ends. Since skewed section may be skewed at either one or both ends and the skewed ends may or may not be parallel, the design span is only parallel to the precast section ends when the ends are skewed equally.

Although some northeast area fabricators can fabricate precast sections with spans up to 40 feet, **CTDOT** requirements for minimum element thicknesses, design span to roof thickness ratio, and precast section weights combine to restrict the spans for flat top frames.

Fabricator literature generally lists clear spans of three-sided structures in 1-foot increments. However, site constraints may require spans in other than whole foot increments.

The rise of a flat top structure is the dimension from the bottom of the wall to the bottom of the roof.

The 10-foot rise limit does not include the roof thickness.

This requirement is consistent with the CTDOT requirements for box culverts.

The roof shall have a uniform thickness of not less than 14 inches.

Walls shall have a uniform thickness of not less than 12 inches. The use or substitution of tapered walls is not permitted.

The minimum roof thickness is based on a design span of 20 feet. See Article 12.14.5.1 for additional requirements.

The **CTDOT** has chosen to specify only uniformly thick walls. This condition is reflected in both the hydraulic and structural design of the structure and reflected in the environmental permitting documents.

Precast reinforced concrete frames shall have equal leg 45-degree reinforced concrete haunches.

Flat top frames with equal leg 45-degree haunches are non-proprietary structures. Flat top frames with unequal legs (the longer leg supports the roof) can be fabricated but are not preferred since they are proprietary structures.

See Article 12.14.5.1 for additional requirements.

The haunch leg dimensions, H, as shown in Figure 12.14.4-1 shall be sized to meet the following requirement:

To ensure that the bottom main reinforcement in the roof is adequately extended and developed within the limits the wall and the haunch, the haunch dimensions are based on meeting the requirements of Article 5.10.8.1.2a. The combined wall and haunch dimensions enable the reinforcement to meet the requirements of Article 5.10.8.1.2b.

The first 3 requirements address the extension of the reinforcement for the condition where the reinforcement is no longer required to resist flexure at the critical section shown in Figure 12.14.4-1.

The last requirement addresses the development of the required reinforcement for the condition where the reinforcement is required to resist flexure at the critical section.

- The value of [(Tw – concrete cover) + H] shall be no less than the following:
  - d , the effective depth of the member, the distance from the top of the roof to the centroid of the main reinforcing in the bottom of the roof, (in.)
  - 15 times d_b , where d_b is the diameter of the main reinforcing in the bottom of the roof, (in.)
  - 1/20 times the clear span, (in.)
  - l_d , the tensile development length based on the area of steel required for the main reinforcing in the bottom of the roof at the critical section, (in.)
- The minimum value of H shall be no less than 1.25 times t_{wall}.

The **CTDOT** has established a minimum haunch dimension.

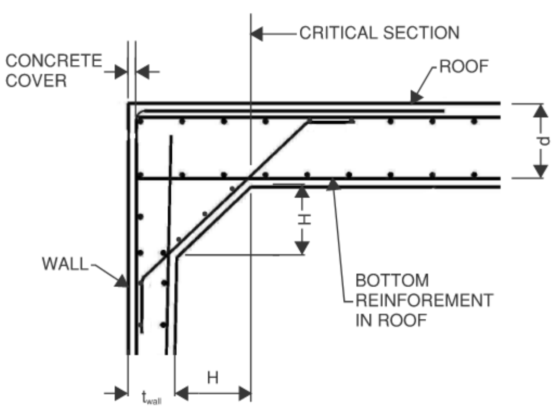

Figure 12.14.4-1

The minimum lay length/width of precast sections shall be no less than 4 feet. The precast section width (lay length) shall be selected to meet the structure length required. The width of the precast sections in a structure shall be constant to

Article 12.14.5.2 refers to Article 12.11.2.1 for load distribution. Article 12.11.2.1 includes both a span-to-roof thickness ratio and a lay length/width requirements for box culverts. **CTDOT** applies these same requirements to flat top frames to avoid the need for edge beams.

facilitate fabrication. A layout plan of the precast sections shall be shown on the plans.

See Article 12.14.6CT for additional layout plan requirements.

Precast sections with square ends are preferable. Skewed sections are acceptable if required to satisfy right-of-way constraints, staged construction requirements, utilities or other site constraints requiring a skewed end. The ability to skew only one end of a section may be limited to small skew angles.

See Article 12.14.5.1 for additional requirements for precast sections with one end skewed.

The maximum skew angle of precast sections shall be no greater than 45 degrees. The skew angle shall be rounded to the nearest 1 degree.

Note, a structure's orientation to the centerline of the highway may be at a skew greater than 45 degrees.

Precast three-sided structures may occasionally need to be placed on moderate or steep grades to match the grade of a watercourse or crossing. Regardless of grade, the top interior surface of the roof of precast sections shall not be stepped between adjacent sections. The placement of precast sections on sloped support structures that result in non-vertical wall joints shall be limited to support structures with a maximum slope/grade of 2%. If matching a steeper slope is necessary, the ends of the precast sections must be beveled to create vertical joints and the footings may be stepped or the length of the wall varied.

Precast fabricators should be contacted for the maximum grade that can be fabricated if a grade larger than 2% is proposed.

The roof and walls of the precast sections shall be constructed monolithically. Construction joints in the walls, roof or the wall-roof intersection of precast flat top three-sided structures are not permitted.

Fabricators fabricate precast sections for flat top three-sided structures oriented as follows:

- By casting the section on the wall/roof ends with 1 concrete placement
- By casting the section in an as erected orientation ("tabletop") with temporary shoring to support the roof with 1 concrete placement
- By casting the section inverted, on the roof with the walls projecting up, with 1 concrete placement
- By casting the section inverted, on the roof followed by the casting of the walls on the inverted roof, with more than 1 concrete placement

Since construction joints in the walls or roof are not permitted, the casting the section inverted, on the roof followed by the casting of the walls on the inverted roof, with more than 1 concrete placement is not an acceptable fabrication method.

Fabrication tolerances for the precast sections shall meet the requirements of Figures 12.14.4-2 and 12.14.4-3 and Tables 12.14.4-1 and 12.14.4-2. Fabrication tolerances shall be shown on the plans.

The fabrication tolerances in Figure 12.14.4-1 were developed based on the fabrication tolerances for prefabricated substructure elements shown in the PCINE Guidelines for Precast Substructures Used in ABC, NCHRP Web-Only Document 243 Recommended Guidelines for Prefabricated Bridge Element and Systems and Recommended Guidelines for Dynamic Effects for Bridge Systems (NCHRP Project 12-98), and FIU ABC-UTC Webinar on Tolerances for Prefabricated Bridge Elements and Systems (PBES) on June 29, 2017.

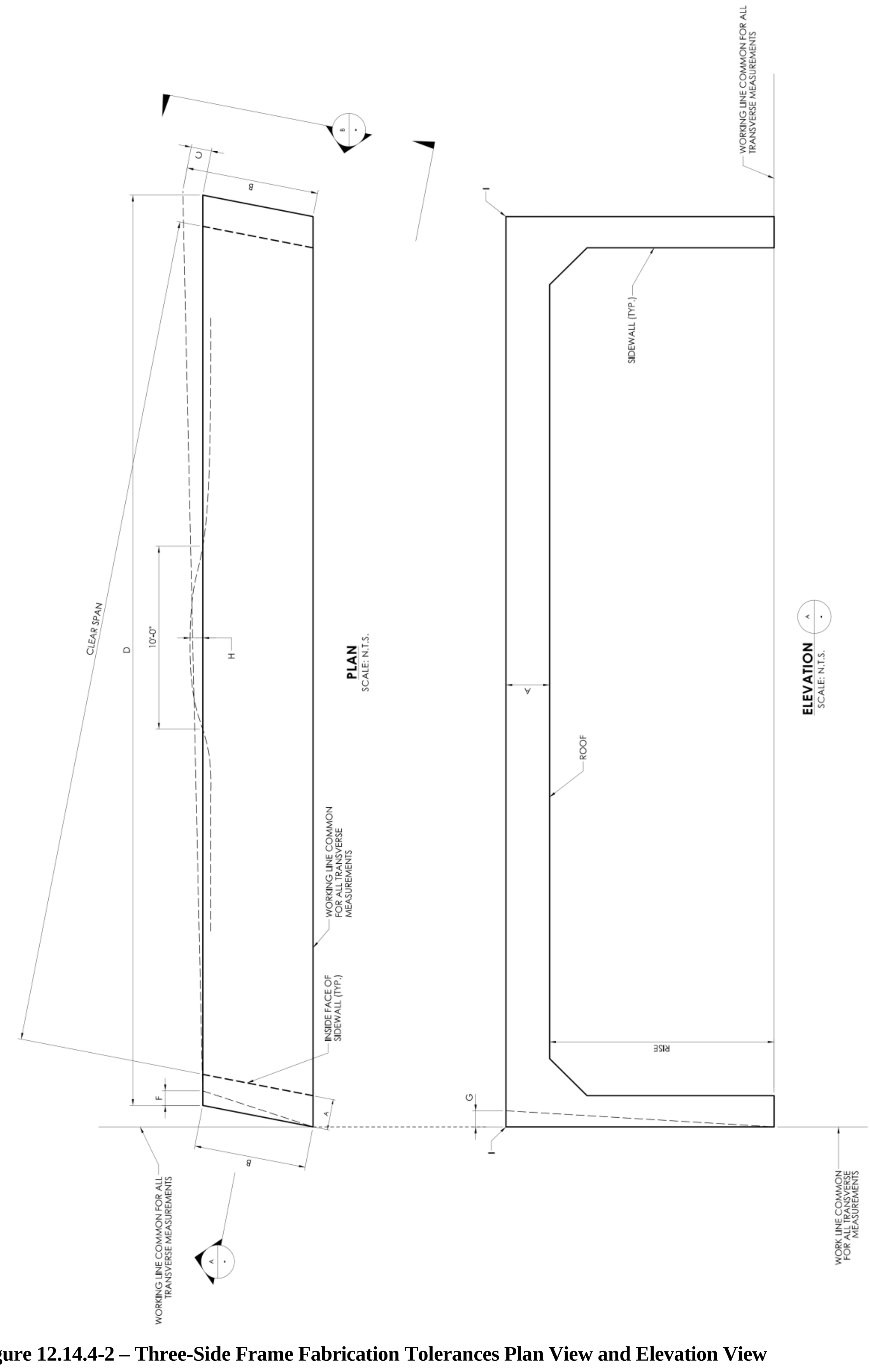

Figure 12.14.4-2 – Three-Side Frame Fabrication Tolerances Plan View and Elevation View

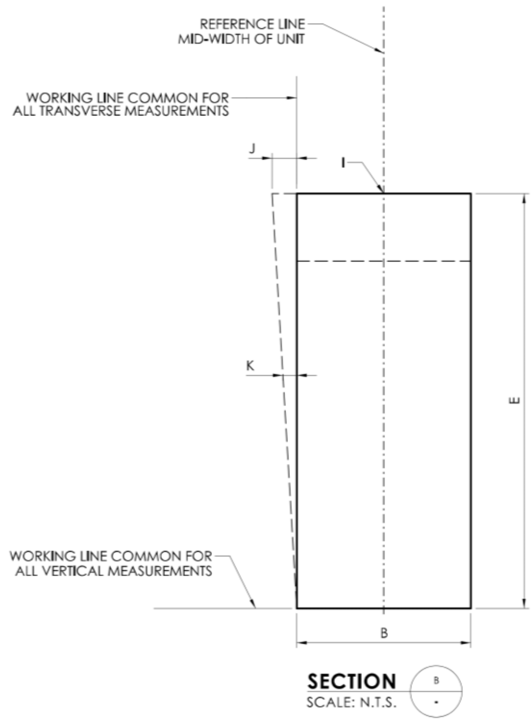

Figure 12.14.4-3 – Three-Side Frame Fabrication Tolerances Section

Table 12.14.4-1 – Three-Sided Frame Fabrication Tolerances

A	ROOF AND WALL THICKNESS	±¼"
B	WIDTH, LAYING LENGTH	±¼"
C	LAYING LENGTH OF OPPOSITE SURFACES	< ±⅛" PER FOOT OF INTERNAL SPAN OR ±¾" MAXIMUM
D	LENGTH	±½"
E	HEIGHT (OVERALL)	±⅜"
F	VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	± ⅛" PER 12 INCH WIDTH ± ½" MAXIMUM
G	VARIATION FROM SPECIFIED ELEVATION END SQUARENESS OR SKEW	± ⅛" PER 12 INCH HEIGHT ± ½" MAXIMUM
H	LOCAL SMOOTHNESS OF ANY SURFACE	±¼" IN 10 FEET

Table 12.14.4-2 – Three-Sided Frame Erection Tolerances

I	TOP ELEVATION FROM NOMINAL TOP ELEVATION MAXIMUM LOW MAXIMUM HIGH	- ¼" ¼"
J	MAXIMUM PLUMB VARIATION OVER HEIGHT	½"
K	PLUMB IN ANY 10 FEET OF HEIGHT	¼"

12.14.5—Design

12.14.5.1—General

This section shall be supplemented with the following:

Three-sided structures shall be analyzed using elastic methods and the cross section modeled as a plane frame (2D) using gross section properties or using a more rigorous analysis method.

Precast three-sided structures, including foundations, shall be designed for an envelope of the governing load effects, independent of the actual foundation support on rock, piles or a spread footing, for the following support conditions:

- a fully pinned support condition
- a pin-roller support condition

A more rigorous analysis of the pin-roller support condition is permitted if soil springs (linear-elastic or non-linear) are substituted for the horizontal supports allowing for movement at the maximum horizontal reaction for the governing factored load case. The determination of the maximum allowable horizontal movement shall be coordinated between the structural engineer and the geotechnical engineer.

The design span to rise ratio of flat top structures shall be no greater than 4 to 1.

The design span-to-roof thickness ratio shall be no greater than 18. Combining the precast section roof thickness with a cast-in-place deck thickness to meet this requirement is not permitted.

C12.14.5.1

This section shall be supplemented with the following:

Conservatively, an envelope of the governing load effects resulting from the noted support conditions is used for the design of the precast frame portion of the structure since the actual support conditions may not be accurately reflected by any one of the idealized modeled support conditions.

Fully pinned support conditions require site conditions and construction details to be able to prevent any horizontal displacement of the walls at the supports. Such a boundary condition may exist if the wall base is connected to a pedestal wall or footing, and the footing is on rock or pile supported with adequate details to resist displacement.

Fixed-fixed support conditions require site conditions and construction details to be able to prevent any horizontal displacement and rotation of the wall base at the supports. However, since Article 12.14.5.14CT requires an unreinforced, grouted keyway connection at the wall base which would allow for rotation, the fixed-fixed support condition is not considered for precast three-sided structures.

The span to rise ratio is an empirical requirement.

As span to rise ratios approach 4 to 1, frame moment distribution is more sensitive to support conditions, and positive moments at midspan can significantly exceed computed values even with relatively small horizontal displacement of the frame legs supports. At sites where the three-sided structure span to rise ratio would be exceeded, other bridge options should be investigated.

This requirement provides a minimum roof thickness for a given design span independent of applied loads to provide a roof with a greater cross section and structural resistance to minimizes the potential for structural cracks in partially and fully cured precast concrete sections.

See Article 12.14.4 for additional commentary.

The **CTDOT** has chosen to be more restrictive by using the design span instead of the clear span.

This requirement is more conservative than the minimum roof (slab) thickness for non-prestressed slabs required by Article 12.14.5.4, Eq. 12.14.5.4-2.

Table C12.14.5.1-1 lists minimum roof thicknesses to meet this requirement.

Table C12.14.5.1-1	
Design Span	$T_{roof, min.}$
ft.	in.
20	13.33
21	14.00
22	14.67
23	15.33
24	16.00
25	16.67
26	17.33
27	18.00
28	18.67
29	19.33
30	20.00
31	20.67
32	21.33
33	22.00
34	22.67
35	23.33

Designers should understand that precast fabricators may have limits on the roof thickness of precast sections.

The composite action resulting from placement of a deck on the roof of a structure shall be neglected.

See Article 12.14.5.4 for additional requirements.

The weight of any precast section shall be no greater than 60 kips.

As with any prefabricated section or section, the weight of a piece is bound by equipment limitations, both at the fabrication plant and in the field, and shipping limitations such as state vehicle regulations and existing bridge load capacities. The weight limit was selected to be consistent with PCINE Guidelines for Precast Substructures used in ABC.

Precast sections shall be evaluated for all permanent and temporary loads effects imposed on the components during the fabrication, handling, shipping, and erection as well as from all construction work at the site. Load effects may result from form removal, handling at the fabrication plant, storage, transportation to the site, erection handling, and field construction activities. The evaluation shall account for changes in concrete strength and load/support conditions of the precast sections.

The goal of the evaluation is to ensure the feasibility of fabrication and constructability of the precast sections while minimizing potential structural cracks in partially and fully cured precast concrete sections that may compromise the structural adequacy and service life of the final structure.

Form removal includes the removal of breakaway forms (forms removed by workers by hand), the removal of the precast section from a form (requires lifting) or a combination of both.

The evaluation of the precast sections shall meet the requirements of **BDS** and **ABC** for all applicable load combinations and load cases. Any fabrication or construction limitations and requirements resulting from the evaluation of the precast sections shall be shown on the working/shop drawing and the contact plans.

More than 1 load case per a load combination may have to be evaluated to address all possible loading conditions.

Designer and Contractor responsibilities for the evaluation of the precast sections shall meet the requirements of **ABC**.

See **ABC** [1.4].

Designers shall evaluate the precast sections to meet the requirements of the **BDS** after the breakaway forms are removed and before the section is lifted; and for field construction activities once the sections have been erected at the site and all temporary supports have been removed. Designer evaluations of precast section account for all possible casting orientations.

To ensure the feasibility and construction of the precast sections, the designer is required to analyze the precast sections before the sections are lifted and after the sections are erected. **ABC** [1.4.1] does not specifically address these requirements.

Although precast sections may be cast in multiple orientations, sections should be evaluated assuming the sections are cast in an as erected orientation (“tabletop”) with

Contractors shall evaluate the precast sections to meet the requirements of **ABC** [8.4] for removal of the sections from forms, handling, shipping, and erection.

The Contractor is responsible for determining all pick points required for lifting and re-orienting/rotating the precast sections at the fabrication plant or at the project site. Pick points shall not be located in concrete block outs for partial depth closure joints.

The roof and walls shall have 2 mats of reinforcement.

The top main reinforcement in the roof shall be full-length bars. The exterior face main vertical reinforcement in the walls shall be either full-length bars with a 90-degree hook at the top or full-length bars with additional L-shaped corner reinforcement at the top. The bottom main reinforcement in the roof shall be full-length bars. Interior face main vertical reinforcement in the walls shall extend from the base of the wall to within 4” of the top surface of the roof. At the base of the walls, U-shaped reinforcement shall be provided and lapped with the vertical reinforcement on the interior face and the vertical reinforcement on the exterior face.

The exterior face corner reinforcement shall be spliced with the exterior face top main reinforcement in the roof and the exterior face main reinforcement in the walls, as applicable. The corner reinforcement shall be adequately developed beyond the critical section for flexure. Additionally, the corner reinforcement shall be extended to meet the requirements of Article 5.10.8.1.2a.

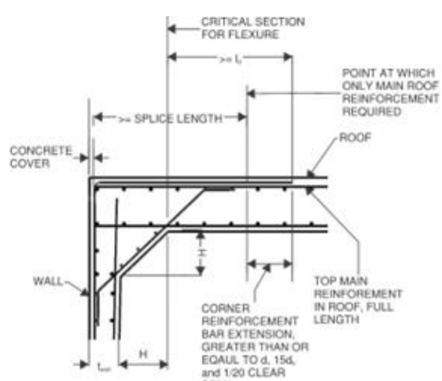

Figure 12.14.5.1-1CT

breakaway forms (except for the base on which it is cast) and the breakaway forms are removed before the concrete is fully cured leaving the roof unsupported.

The load factors in **ABC** [2.4.1.1] address the removal of the precast elements <u>from</u> formwork, handling, shipping, and erection.

Construction specifications include this requirement.

Refer to the PCI Design Handbook, Chapter 8 for additional information.

Construction specifications include this requirement.

When lifting precast sections oriented in an as erected orientation (“tabletop”) with 2 pick points on the roof located closer to the walls, the roof of the section tends to experience positive bending resulting in the walls of the section rotating and displacing outward. When 4 pick points on the roof are used and distributed along the same alignment, the roof tends experience negative bending resulting in the walls of the section rotating and displacing inward. Unsymmetrical precast sections may require pick points along more than 1 alignment due to the location of the center of gravity of the section.

Each reinforcement mat includes main reinforcement and transverse reinforcement.

Transverse reinforcement shall be placed on each face of the roof and walls within the main reinforcement. In the roof, the bottom transverse reinforcement shall be designed as distribution reinforcement meeting the requirements of Article 5.11.2.1. The spacing of transverse reinforcement shall be 6 inches. The top transverse reinforcement shall match the size and spacing of the bottom transverse reinforcement. In each face of the walls, the bar size of the transverse reinforcement shall match the bar size of the reinforcement in the roof. The reinforcement spacing shall not be greater than 12 inches.

Transverse reinforcement is placed within the main reinforcement to facilitate placement of main reinforcement in closure joints.

The reinforcement in the haunches shall meet the following requirements:

- The haunch reinforcement, including bends, shall be fabricated in 1 plane.
- Reinforcement shall be placed parallel to the top main reinforcement (in plan view) in the roof.
- The reinforcement shall extend to the exterior face transverse reinforcement in the wall and to just below the top transverse reinforcement.
- Reinforcement shall be placed on the interior face of the haunch with concrete cover meeting the requirements of Article 12.14.3.
- The haunch reinforcement leg extensions shall extend no less than 18 inches from the center of the bend radius.
- At rectangular partial depth closure joints, the reinforcement shall extend to within 1 in. of the bottom of the blockout and be fully embedded in the precast concrete.
- Reinforcement size and spacing shall be no less than the size and spacing of the interior face main reinforcing in the wall.

By placing the reinforcement within the transverse reinforcement, conflicts between bars are avoided and the placement of all reinforcement is simplified.

The minimum reinforcing bar size shall be #4. The maximum reinforcing bar spacing shall be 12 inches.

In precast sections with one end skewed, the main reinforcement must be splayed to fit the geometry of the skew. Splaying the reinforcement will require increased length of the main reinforcement and, more importantly, increase the design span of the section. For small skews, the splayed reinforcement may be adequate. However, large skews may require more reinforcement and can increase the design span to the point where increased slab thickness may be necessary. In precast sections with one end skewed, the length of the walls will vary. The designer shall ensure that the required reinforcement can be placed within the length of the walls and meet all applicable design and construction requirements.

Design span for non-skewed culverts is the perpendicular distance between the centerline of the sidewall interior surfaces. For culvert sections with skewed ends, the design span of end sections is the distance between the centerlines of the sidewall interior surfaces measured parallel to the skewed end.

In precast sections with both ends parallel and skewed, the top and bottom main reinforcement in the roof shall be designed for the design span parallel to the skew and the reinforcement shall be placed parallel to the skew.

The roof and walls of structures shall be proportioned and designed to eliminate the need for shear reinforcement.

The need for supplemental temporary bracing (i.e. tension ties, tendons, or rods; struts; angled braces; strong-back beams) to minimize and mitigate load effects on the roof and walls or to prevent movement (rotation and translation) of

The Contractor is responsible for the design, installation, and removal of all supplemental temporary bracing.

Construction specifications include this requirement.

walls relative to the roof while forms are removed from the precast sections or the precast sections are being rotated, lifted, moved, relocated, or transported at the fabrication facility, being transported from the fabrication facility to an intermediate site or the final project site, or being rotated, lifted, or erected at the project site shall be investigated by the Contractor. The evaluation performed to determine whether temporary bracing is required shall be documented in the working drawing submittal. Justification shall be provided when no temporary bracing is required. The temporary bracing shall be designed by the Contractor and located so that it will not impact field construction. The temporary bracing shall be installed and removed by the Contractor in accordance with requirements shown on the working/shop drawings. The working/shop drawings shall provide instructions/directions on the conditions that must be present before the temporary bracing is removed.

A precast section cast inverted, on the roof with the walls projecting up, may require the use of strong back beam(s) attached to the underside of the roof with embedded hardware, to remove the partially cured precast section from the forms, re-orient the section, and lift and relocate the section at the fabrication plant.

The placement of through holes in the walls and hardware permanently cast in the roof and walls to facilitate the following temporary work is permitted:

- Lifting and re-orienting/rotating the precast sections at the fabrication plant or at the project site
- Anchoring supplemental temporary bracing
- Positioning/pulling adjacent precast sections together
- Anchoring formwork for cast-in-place concrete

The need for hardware should be determined by the Contractor prior to submitting working and shop drawings.

The use of post installed anchorages in the precast sections is not permitted.

Holes through precast section components shall be sized and located by the Contractor and formed with non-metallic pipe that can remain in place. The holes shall be located to not adversely affect the fabrication of the design shown on the plans or field construction. All holes shall be through the precast sections and shall be shown on the shop drawings.

All cast-in place hardware shall be designed and located by the Contractor. The type, number, alignment of the hardware shall be chosen by the Contractor and located to not adversely affect the fabrication of the design shown on the plans or field construction. The use of supplemental reinforcement with the hardware is permitted. All hardware shall be recessed below adjacent concrete surfaces. Keyways that recess the hardware to facilitate installation are permitted. The keyways shall have roughened, exposed aggregate surfaces. Hardware used to position/pull adjacent precast sections together shall be located on exterior surfaces. Prior to applying load to the embedded hardware, the concrete shall attain the minimum concrete compressive strength associated with the safe working load of the hardware. The embedded hardware shall meet the requirements of Article 12.4.2.11CT. All cast-in-place hardware in the precast sections shall be shown on the shop drawings.

At the project site, after the work requiring the through holes and hardware is completed, all holes and hardware voids, that will not be otherwise encased in cast-in-place concrete, shall be filled with non-shrink grout flush with the adjacent concrete surface.

Regardless of the orientation of the section for concrete casting, forms shall not be removed from a precast section, nor shall the precast section be rotated, lifted, moved, relocated, or transported at the fabrication facility, until the concrete obtains a compressive strength no less than 70 percent of the specified 28-day compressive strength shown on the plans based on concrete cylinder compressive tests. Any reductions to this percentage shall be justified by the Contractor and submitted in a request for Request for Change (RFC) to the CTDOT for review and action.

The minimum concrete strength requirement establishes a consistent value be used for the evaluation of partially cured precast concrete sections.

This concrete strength is commonly referred to as the stripping concrete strength.

Construction specifications include this requirement.

The precast sections shall not be transported from the fabrication facility to an intermediate site or final project site until the concrete obtains 100 percent of the specified 28-day compressive strength shown on the plans based on concrete cylinder compressive tests.

Construction specifications include this requirement.

12.14.5.4—Shear Transfer in Transverse Joints between Culvert Sections

This section shall be supplemented with the following:

Precast, flat top, three-sided structures shall be designed with connections between the roof and the wall elements of adjacent sections, regardless of roof thickness, fill/ballast height, or the presence of a concrete roadway/relief slab over the structure. The following connections are permitted:

- Concrete Deck - Place a composite, cast-in-place reinforced concrete deck across the roofs of all precast sections and install partial depth reinforced concrete closure joint between walls of the adjacent sections.
- Closure Joints – Install reinforced concrete closure joints, either partial or full depth, between the roof and the walls of the adjacent precast sections.

The use of an open joint without a deck; a ship lapped joint; a non-reinforced concrete filled shear key; a non-shrink grout filled shear key, with or without post tensioning; and external or internal welded or bolted plate connections between adjacent precast sections, regardless of roof thickness, fill/ballast height, or the presence of a concrete roadway/relief slab over the structure are not permitted.

The designer shall select and detail the type of connection, concrete deck or closure joint, including all concrete materials, between precast sections.

At sites with less than 2 feet of ballast and a full depth closure pour is not specified, the precast sections shall use a concrete deck to connect the roof elements and partial depth closure joints to connect the wall elements.

At sites with less than 2 feet of ballast, the use of partial depth closure joints to connect the roof elements of precast sections is not permitted.

For partial depth closure joints, since the cross section of the closure joints are minimized, designers shall consider specifying conventional concrete with a smaller coarse

C12.14.5.4

This section shall be supplemented with the following:

By inter-connecting precast sections between the roof and the wall elements, the resulting structure can resist vertical and lateral load effects such as overturning effects from earth and collision loads, relative displacement between sections, and provide uniform surface for waterproofing membrane and dampproofing.

Refer to Article 12.4.2.2.

aggregate and better placement characteristics in confined areas.

The type of connection between precast sections shall meet the requirements of Table 12.14.5.4-1CT. The depth of ballast over the roof of the structure shall be taken as the minimum distance from the finished grade to the top of the roof within the roadbed.

In Table 12.14.5.4-1CT for structures with less than 2 feet of ballast, connecting the roof and wall elements using a concrete deck with a partial depth closure joint or using a full depth closure joint meets the requirements of Article 12.14.5.4 without considering the roof thickness. For structures with greater than or equal to 2 feet of ballast, providing a partial depth closure joint in the roof and walls will better distribute load effects and minimize potential leakage of backfill and water.

In Table 12.14.5.4-1 the materials referenced include both conventional concrete and UHPC to meet the matching compressive strength material requirement of Article 12.4.2.2.1CT. The maximum compressive strength of conventional cast-in-place concrete is generally 5 ksi.

The roadbed is the graded portion of a highway including portions within the top and side slopes, that has been prepared as a foundation for the pavement structure and shoulders.

Table 12.14.5.4-1CT

Depth of ballast	General description of connection	Material	Location of connection	Specified joint width between precast sections
Less than 2 feet	Concrete deck with partial depth closure joint in walls	Conventional concrete	Roof	Narrow
			Walls	Narrow
	Full depth closure joint	Conventional concrete	Roof	Wide
			Walls	Wide
	Full depth closure joint	UHPC	Roof	Wide
			Walls	Wide
Greater than or equal to 2 feet	Partial depth closure joint	Conventional concrete	Roof	Narrow
			Walls	Narrow
	Partial depth closure joint	UHPC	Roof	Narrow
			Walls	Narrow

Where narrow joints are required between precast sections for the concrete deck and partial depth closure joint connections, the joint shall be detailed with a specified joint width and a plus/minus tolerance. The combined dimensions are referred to as the “detail call out". The detail call out for narrow joints is fixed and independent of the concrete used in the closure joint. The detail call out for the narrow joint has been standardized and is shown in Table 12.14.5.4-2CT. Since the joint width is fixed, designers shall only vary the precast section width to obtain the structure length required. The width of the precast sections in a structure shall be constant to facilitate fabrication.

See Article 12.14.4 for additional requirements.

Table 12.14.5.4-2CT	
Minimum tolerable width for closed cell foam backer rod or closed cell expandible foam	3/8 inch
Joint width tolerance	5/8 inch
Summation of above = Specified joint width =	1 inch
Detail call out =	1” +/- 5/8 “

The value of the joint width tolerance matches that shown for “all elements” in the Table 4.5.2.1 of the NCHRP Web-Only Document 243 Recommended Guidelines for Prefabricated Bridge Element and Systems and Recommended Guidelines for Dynamic Effects for Bridge Systems (NCHRP Project 12-98)

Where wide joints are required between precast sections for full depth closure joint connections, the joint shall be detailed with a specified joint width and a plus/minus tolerance. The designer shall vary the specified joint width and along with the precast section width to obtain the structure width required. Full depth closure joints have minimum and maximum specified joint width dimensions dependent on the concrete and reinforcement used in the joint. Table 12.14.5.4-3CT lists minimum and maximum specified joint width for full depth closure joint connections. The width of the joints and the precast sections in a structure shall be constant to facilitate fabrication. The designer is responsible for ensuring adequate reinforcement clearances and splices are provided within the joint.

See Article 12.14.4 for additional requirements.

Table 12.14.5.4-3CT

Material	Bar Size	Joint width tolerance	Specified joint width	Detail call out
Conventional Concrete, f’c = 5 ksi	#5	5/8 inch	14 inch min.	14" ± 5/8"
			18 inch max.	18" ± 5/8"
	#4	5/8 inch	12 inch	12" ± 5/8"
			16 inch max.	16" ± 5/8"
UHPC	#5	5/8 inch	7 ½ inch min.	7 ½" ± 5/8"
			12 inch max.	12" ± 5/8"
	#4	5/8 inch	6 ½ inch	6 ½" ± 5/8"
			11 inch max.	11" ± 5/8"

For structures with a concrete deck and partial depth closure joint in the walls, the supporting precast sections shall have vertically faced ends and shall be erected with narrow joints. The deck shall be cast-in-place reinforced concrete and be connected to the roof with vertical projecting reinforcement. The top of the roof of the precast sections shall have a raked surface finish. The deck shall have a minimum thickness of 6 inches and 1 mat of reinforcement. The deck reinforcement shall meet the requirements of Article 5.12.2.3.3f and the following:

Since structures with a deck require reinforcing projecting from the top of the roof into the deck, the precast sections are typically fabricated in an as erected orientation (“tabletop”).

Partial thickness decks are referred to as structural concrete overlay in Article 5.12.2.3.3f.

- The projecting hooks shall be 90 degree hooks oriented transverse to the span of the section. The hooks shall project a minimum of 2.5 inches above the top of the roof of the precast section. The bar size of the hook shall be no less than a #4 bar. 1 - #5 bar shall be placed in each hook.
- Minimum top reinforcement cover shall be 2.5 inches
- Main reinforcement shall be placed transverse to the span of the precast sections and shall be no less than #5 at 6 inches
- Temperature and shrinkage reinforcement shall be no less than #5 at 12 inches.

The minimum deck thickness allows for tolerance in placing projecting hooks.

The walls of adjacent sections shall be connected with a partial depth reinforced concrete closure joint.

For structures with full depth closure joints, the adjacent sections shall be erected with a wide specified joint width plus or minus tolerances between adjacent sections. Full depth closure joints with a multi-sided cross section shall be formed in both the roof and walls on the ends of the precast sections. The use of cast-in-place concrete may be either conventional concrete or UHPC is permitted. For conventional concrete, the reinforcement in the closure joint shall be projecting U-shaped bars with the legs embedded in the ends of the precast section. For UHPC, the reinforcement in the closure joint shall be projecting straight bars embedded in the ends of the precast elements. The reinforcement in adjacent precast sections shall be offset along the closure joints to avoid conflicts and provide a non-contact lap splice. Longitudinal reinforcement shall be field placed in the corners of the overlapping U-shaped reinforcement and at each end of the straight bar lap splice. Additional main reinforcement shall be field placed at the surface of each element (roof and wall). The transverse closure joint reinforcement shall meet the design requirements for the transverse reinforcement in the precast sections. The transverse reinforcement in the roof full depth closure joints shall be no less than #5 at 6 inches.

The full depth closure joint described is similar to the full depth closure joints used with Northeast Solid Deck Beams developed by PCINE.

For structures with rectangular partial depth closure joints the adjacent sections shall be erected with a narrow specified joint width plus or minus tolerances between adjacent sections. Rectangular partial depth closure joints shall be formed on the outer surfaces in both the roof and walls on the ends of the precast sections. The dimensions (width and depth) of closure joints shall be based on the thickness of the structural element (roof or wall), reinforcing requirements, the material used to fill the key, either conventional concrete or UHPC, and the formwork requirements. The depth of a rectangular partial depth closure joint shall not exceed 35% of the roof thickness. The depth of a rectangular partial depth closure joint shall not exceed 50% of the wall thickness. A continuation of the transverse reinforcement shall project into the closure joint and be embedded into each precast surface with a 90-degree hook. Field placed transverse reinforcement shall be either doubly 180-degree hooked or straight, as required for the lap splices. Longitudinal reinforcement shall be field placed in the corners of all hooks and at each end of

Per ASTM A767, for #4 or #5 galvanized reinforcement, the minimum finished bend diameter for a 90-degree or 180-degree hook is $6d_b$. For #4 reinforcement, the diameter is 3.00 inches. For #5 reinforcement the diameter is 3.75 inches. To minimize the depth of the rectangular partial depth closure joints, the hooked reinforcement may be rotated no more than 60 degrees from vertical.

The depth of a partial depth closure joint is partly governed by the concrete material used to fill the void. For conventional concrete the depth of the closure joint in the roof can be determined assuming a 60-degree rotation of the field placed transverse reinforcement as follows:

2.5 in.	top of roof concrete cover
1.0 in.	assumed #8 main reinforcement required
0.625 in.	d_b of assumed #5 field placed transverse reinforcement required

the straight bar lap splice. Additional main reinforcement shall be field placed at the surface of each element (roof and wall). The field placed transverse closure joint reinforcement shall meet the design requirements for the transverse reinforcement in the precast sections. The transverse reinforcement in the roof partial depth closure joints shall be spaced no greater than 6 inches apart. The impacts of the concrete block outs on the reduction in the section properties of the precast sections shall be included in any structural evaluation of the section.

1.875 in.	(cos 60) * (min. finished bend diam.) = 1.875 in
0.625 in.	d_b of return for assumed #5 field placed transverse reinforcement required
6.625 in.	Total
Use partial depth closure joint with a depth no less than 7.0 inches to allow concrete paste/mortar to encase the hook.	

Since the depth of the partial depth closure joint cannot exceed 35% of the roof thickness, a closure joint depth of 7 inches would require a 20-inch-thick roof.

Similarly, for conventional concrete, the depth of a partial depth closure joint in a wall shall be no less than 6.5 inches. Since the depth of the partial depth closure joint cannot exceed 50% of the wall thickness, a closure joint depth of 6.5 inches would require a 13-inch-thick wall.

For UHPC the depth of the closure joint in the roof can be reduced since the field place transverse reinforcement is a straight bar. The minimum shall be 5 inches. Since the depth of the partial depth closure joint cannot exceed 35% of the roof thickness, a closure joint depth of 5 inches would require a 15-inch-thick roof.

The construction joint surfaces of the closure joints on the precast sections shall be roughened, exposed aggregate surfaces.

The exposed aggregate surface has been specified to improve the bond of cast-in-place cementitious materials to the precast sections and minimize potential leakage at the interface of the construction joint.

On the roof of structures, the top surface of the closure joints using UHPC shall be ground smooth and flush with the adjacent roof surfaces.

Construction activities after the placement of conventional concrete or UHPC in the closure pours that result in a change of load effects on the structure, such as backfilling, are not permitted until the concrete obtains the specified 28-day compressive strength shown on the plans based on concrete cylinder compressive tests.

12.14.5.7—Crack Control

This section shall be supplemented with the following:

The requirements of Article 5.6.7 shall be met based on an exposure factor for a Class 2 exposure condition.

12.14.5.9—Deflection Control at the Service Limit State

This section shall be supplemented with the following:

The deflection of the roof of all precast three-sided structures shall meet the requirements for structures used by pedestrians.

C12.14.5.9

This section shall be supplemented with the following:

Since structures, that initially may only carry vehicles may have to carry both pedestrian and vehicles in the future, all structures shall be initially designed assuming they will be used by pedestrians.

12.14.5.10—Footing Design

This section shall be supplemented with the following:

The footings for three-sided structures subject to scour shall be supported on rock or piles foundations. Spread footings may only be used for structures not subject to scour, such as multi-use trails, animal passes, etc.

Footings/pile caps supported by piles shall have no less than 2 rows of piles. Potential conflicts between battered piles supporting sidewall foundations shall be considered.

The use of a single row of piles, with or without alternating pile batters, is not permitted.

Footing to rock connection design and details shall meet the geotechnical engineer's requirements for design, detailing and materials.

Designers shall coordinate with geotechnical engineers to ensure that the differential footing movements and rotations are consistent with the boundary conditions used in the analysis

C12.14.5.10

This section shall be supplemented with the following:

Designers should coordinate with the geotechnical engineer to obtain input on rock competency, rock slope both longitudinal and transverse to foundations, rock removal and benching requirements, anchor hole diameter, anchor embedment depth, anchor spacing and grout materials.

The embedment depth of an anchor into rock should be determined based on the resistance of the anchor, grout and the rock.

The use of adhesive bonding material to anchor dowels into rock is not permitted since it is not addressed by **BDS**.

12.14.5.11—Structural Backfill

This section shall be supplemented with the following:

The backfill of all structures, including associated wingwalls, shall meet the requirements of **BRSDM** [V1B – Buried Structures].

Backfilling limits and requirements shall be shown on the plans to mitigate the load effects due to unbalanced backfill.

12.14.5.13CT—Subsurface Drainage

Provisions for subsurface drainage of the structure backfill shall meet the recommendations provided in the geotechnical report and the requirements of **BRSDM** [V1B].

12.14.5.14CT—Wall Base Connection

The designer shall determine the load effects from the structure at the wall base to supporting structure connection and design and detail a connection to accommodate the effects.

The minimum width of the keyway shall be the precast section wall thickness plus a minimum 6 inches. The keyway walls shall be no less than 9 inches thick and reinforced with no less than #5 U-type or hair pin shaped bars projecting downward (legs down) at 12 inches. The keyway shall provide no less than 6 inches of wall embedment.

C12.14.5.14CT

The keyway width shall provide at least 3 inches, each way, for adjustment, construction tolerance, and structural non-shrink grout placement.

On pile caps, since no less than 2 rows of pile are required, the keyway wall thickness will generally be greater than the minimum requirement.

Based on the minimum precast section wall thickness, the additional 6-inch keyway width and the 9 inch minimum keyway wall thickness requirements, the minimum pedestal wall thickness shall be no less than 2.5 feet.

The precast sections shall be leveled with durable, uncompressible, man-made, non-metallic shims placed in the base of the keyway below the wall. At a minimum, the base of the wall shall be embedded into the keyway and grouted with at least 0.5 inch of grout below the bottom of the wall. Additional durable, uncompressible, man-made, non-metallic shims and wedges shall be placed between the wall faces and the keyway to prevent displacement of the wall after any temporary bracing is removed. The grout shall be a structural non-shrink grout with a minimum compressive strength of 5 ksi, exhibit no shrinkage, and be suitable for use, after placement and curing in the dry, in underwater (submerged) conditions. The structural non-shrink grout shall be documented as suitable for placement in joints with the minimum width/thickness shown on the plans. The grout shall obtain a compressive strength no less than 5 ksi before any temporary bracing is removed and concrete is placed for the closure joints or the cast-in-place deck.

12.14.5.15CT—Dampproofing

Exterior surfaces of walls and the rear face of pedestal wall stems shall be dampproofed. On walls the dampproofing shall be placed to within 1 foot of the top of the roof. Dampproofing shall be applied to the exterior top finished grout surface of the wall base connection.

12.14.5.16CT—Membrane Waterproofing Requirements

Unless a concrete element or component (such as a full or partial thickness deck, headwall, footing for a railing moment slab) is placed directly on the top of the structure’s roof, a membrane waterproofing shall be placed on the top of the structure’s roof, regardless of the thickness of the ballast/fill on the roof. The membrane waterproofing shall cover the entire exterior top surface of the roof, extend 12 inches down the walls, and extend 12 inches up the side of headwalls and footings. A membrane waterproofing shall be placed on the top full and partial thickness decks.

The standard membrane waterproofing shall be Membrane Waterproofing (Cold Liquid Elastomeric).

The use of Membrane Waterproofing (Woven Glass Fabric) is permitted under the following conditions:

- On existing structures with an anticipated remaining service life of less than 20 years
- On new or existing structures in construction projects that include a complete road closure with a duration no greater than 14 days and require the finished the bituminous overlay to be placed during the road closure.

C12.14.5.16CT

Extending the membrane waterproofing up the sides of the headwalls and footings seals the construction joint at the top of the roof.

12.14.5.17CT—Penetrating Sealer Protective Compound

The interior concrete surfaces of walls and the underside of the roof shall be sealed with field applied penetrating sealer protective compound. The penetrating sealer protective compound shall be applied to the interior top finished grout surface of the wall base connection.

C12.14.5.17CT

Penetrating sealer protective compound is required on all structures regardless of environmental exposure conditions (exposure to de-icing materials or marine environments) to extend the protection of the concrete surfaces to ensure a long service life.

12.14.5.18CT—Approach Slabs

Three-sided flat-top structures do not require approach slabs.

C12.14.5.18CT

Although the paved approach may settle over time, the **CTDOT** has determined any pavement deficiency can be addressed by the addition of HMA.

12.14.5.19CT—Headwalls

Headwalls shall be provided at the structure inlet and structure outlet to satisfy the site grading conditions. Headwalls may be constructed with either cast-in-place or precast concrete. Headwalls shall have a minimum thickness of 1.25 feet at the top. Headwalls shall be connected to the roof of the structure with bar reinforcement. Dowel bar mechanical connectors may be used. The use of concrete inserts is not permitted. The rear face of headwalls shall be dampproofed. Railings or fences shall be placed on headwalls in accordance with the requirements of Article 12.14.5.21CT. Headwalls shall be designed to meet all applicable limit states.

Where headwalls lie within the deflection zone of guiderail, the headwall projection above grade shall be limited.

C12.14.5.19CT

The roof thickness may be governed by the reinforcement development length required to connect the headwall to the roof.

For projection limits, refer to **CTDOT** Highway Standard Sheets for guidance.

12.14.5.20CT—Wingwalls

Generally, wingwalls shall be provided at the structure’s inlet and outlet to retain fill and to direct water. Wingwalls may be constructed with either cast-in-place or precast concrete.

For structures over waterways, the designer should coordinate with the hydraulic engineer to determine acceptable wingwall orientations (U-type or flared) that meet hydraulic requirements and site conditions. The angle(s) of the wingwalls at the inlet should direct the water into the structure.

Wingwall stems and footings shall be made independent of the structure’s sidewalls, pedestal walls and footings/pile caps. The elevation of the bottom of the wingwall footings shall match the structure’s foundation. The elevation of the top of the end of the wingwall at the structure shall satisfy grading conditions but be no less than the elevation of the top of the structure’s roof.

The wingwalls should abut the ends of the sidewalls. Due to the skew and/or grade differences between the precast structure section and precast wingwalls it is necessary to do a cast-in-place closure pour between the end precast structure section and the wingwalls. A closure pour is not required if cast-in-place wingwalls are used.

C12.14.5.20CT

When precast concrete wingwalls are used, non-proprietary precast concrete wingwalls are preferred.

Wingwall stems shall have a minimum thickness of 1.25 feet at the top.

The rear face of wingwall stems shall be dampproofed.

Railings or fences shall be placed on wingwalls in accordance with the requirements of Article 12.14.5.21CT.

Wingwalls shall be designed to meet all applicable limit states.

12.14.5.21CT—Railing and Fences

For railing and fence requirements, refer to Section 13.

Railings with discrete posts attached directly to the roof precast sections are not permitted. Connecting railings with discrete posts to curbs or headwalls that are connected to the precast sections is acceptable. The use of a supplemental footing or moment slab is also acceptable.

C12.14.5.21CT

Connections of railing posts directly to precast sections is not permitted to avoid potential damage to the roof if the railing is struck by a vehicle.

12.14.5.22CT—Loads

12.14.5.22.1CT—Load Modifiers

Load modifiers shall meet the requirements of Article 12.5.4 for buried structures.

C12.14.5.22.1CT

For the strength limit states, buried structure redundant under earth fill.

12.14.5.22.2CT—Earth Pressures

The calculation of earth pressures and loads shall be based on a unit weight of 0.125 kcf.

C12.14.22.2CT

The unit weight shown is consistent with Article 3.5.1

12.14.5.22.3CT—Vertical Earth Pressure (EV)

The vertical earth loads from the fill on the roof shall be modified for soils-structure interaction for an embankment installation in accordance with Article 12.11.2.2.

12.14.5.22.4CT—Horizontal Earth Pressure (EH)

The calculation of earth pressures and loads shall be based on an at-rest condition for normally consolidated soils and an effective friction angle of 30º. The coefficient of lateral earth pressure at-rest, k_o, shall be 0.5.

Lateral earth pressure from the backfill against the walls shall be determined in accordance with Article 3.11.5.1.

A load condition where the earth pressure may reduce effects caused by other loads shall be investigated in accordance with Article 3.11.7. The earth pressure shall be reduced by 50%. The reduced earth pressure shall not be combined with the minimum load factor in Table 3.4.1-2.

C12.14.5.22.4CT

The coefficient of lateral earth pressure at-rest for normally consolidated is calculated using Equation 3.11.5.2-1.

The requirement is consistent with the **MBE** [6A10.10.2b].

Based on BDS [3.11.5.1], the equation for the lateral earth pressure becomes:

$$p = k_o * \gamma_s * z$$

with $k_o = 0.5$ and $\gamma_s = 0.125$ ksf

$$p = (0.5) * (0.125\ kcf) * z = (0.0625\ kcf) * z$$

The calculated lateral earth pressure may not agree with the default value used in some software programs.

Based on this requirement, the minimum lateral earth pressure is:

$$p = 0.5 * [(0.5) * (0.125\ kcf) * z] = (0.03125\ kcf) * z$$

The requirement is consistent with the **MBE** [6A10.10.2b].

The calculated lateral earth pressure may not agree with the default value used in some software programs.

12.14.5.22.5CT—Live Loads

Live loads, the application of live loads, the distribution of live loads and dynamic load allowance shall meet the requirements of **MBE** [6A.10.10.3, 6A.10.10.3a, and 6A.10.10.3b].

12.14.5.22.6CT—Approaching Wheel Load (AW)

The application of a live load surcharge in accordance with Article 3.11.6.4 is not applicable to flat top, three-sided structures.

C12.14.5.22.6CT

The requirement is consistent with the **MBE** [6A.10.10.3c].

Live load surcharge, in accordance with Article 3.11.6.4, is applicable to elements and components supporting the three-sided structure, such as pedestal walls and footings or pile caps.

Culverts have traditionally been evaluated for a live load surcharge that is appropriate for earth retaining structures. The live load surcharge is not appropriate for flat top, three-sided structures for the following reasons:

- Unlike retaining walls, where a vehicle load near a wall increases the overturning moment, a vehicle approaching a culvert produces a small lateral pressure that is resisted by the soil on the far side of the culvert.
- Lateral pressure near the mid-height of the wall will result in an increase in positive moments in the sidewall and negative moments at the corners and a decrease in positive moments in the slabs. Lateral pressure near the top of a shallow culvert primarily results in a thrust in the top slab which has almost no effect on the moments, and hence the reinforcement requirements

Flat top, three-sided structures with less than or equal to 2 ft of cover shall be loaded with a lateral pressure distribution to produce the effects of a truck axle just before going over the culvert. This pressure shall be applied to both sides of the culvert (see Figure 12.14.5.22.6-1CT) and computed using the following equation:

The requirement is consistent with **MBE** [6A.10.10.3c].

This approaching wheel load has been used in AASHTO and ASTM standards for precast concrete box culverts for over 40 years. It was first proposed by Heger, F.J. and Long, K.N. (1976) Structural Design of Precast Concrete Box Sections for Zero to Deep Cover Earth Cover Conditions and Surface Wheel Loads, Concrete Pipe and the Soil-Structure System, ASTM STP 630.

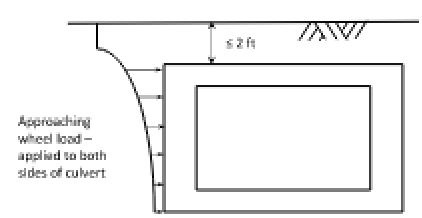

Figure C12.14.5.22.6-1CT

$$p_{lat(h_d)} = \frac{700}{h_d} \leq 800\ psf \qquad (12.14.5.22.6CT\text{-}1)$$

The units in Equation 12.14.5.22.6CT-1 are not typical of **BDS** and must be converted to ksf to be consistent with other LRFD calculations.

where:

$P_{lat(hd)}$ = lateral soil pressure resulting from an approaching wheel at depth h_d, psf
h_d = depth of fill to depth where pressure is calculated, ft

The approaching wheel load need not be considered for structures with more than 2 ft. of fill from the top of the roof to the top of the pavement.

12.14.5.22.7CT—Pavements

The use of pavements to distribute wheel loads and reduce load effects on new flat top, three-sided structures is not permitted.

C12.14.5.22.7CT

The use of the pavements to distribute wheel loads shall be limited to load rating evaluations of existing structures. Refer to **MBE** [6A.10.10.3d].

12.14.6CT—Constructability

Designers are responsible to ensure that the work required by the contract documents is constructable. A feasible sequence of construction work shall be shown on the plans. The plans shall include details for not only the permanent construction but also the stages of construction and temporary construction required to complete the work.

The dimensions of the precast section walls and the supporting structure shall be selected to allow for grading and placement of materials in the channel prior to the erection of the precast sections. Supporting structures and wall stems shall be designed to allow partial backfilling to facilitate channel construction.

Article 12.14.5.1 requires the Contractor investigate the need for temporary bracing (i.e. tension ties, tendons, or rods; struts; angled braces; strong-back beams) to minimize and mitigate load effects on the roof and walls for the time period from casting until the precast sections are supported by the permanent foundation. Designers shall assume temporary bracing will be necessary and be required through the erection of the precast sections. The plans shall address potential conflicts between the temporary bracing and both permanent and temporary work within the limits of the structure and provide a feasible construction methodology and sequence.

The layout plan of the precast sections shall be shown on the plans. The placement of sections shall be based on offset dimensions from a common working point or line. The placement of sections based on a center to center spacing or use of a spacer the thickness of the specified joint width is not permitted. The offset dimensions shall account for element tolerances, erection tolerances and joint width tolerances. Offset dimensions and joint width dimensions shall be shown with a plus/minus tolerance.

If stage construction is required, adequate work areas shall be provided for temporary earth retaining systems and

C12.14.6CT

Designers shall ensure the geometry of the precast structure will not conflict with the constructability of work at the site.

Permanent work may include the placement of material for the channel. Temporary work may include temporary facilities such as a water handling pipe or earth retaining system.

Erection of elements based on center of precast section to center of precast section spacing, or the use of a spacer the thickness of the joint width, shall not be used as this could lead to build up of tolerances.

Guidance for fabrication/element tolerances, construction/erection tolerances and specified joint widths can be found in PCINE Guidelines for Precast Substructures Used in ABC, NCHRP Web-Only Document 243 Recommended Guidelines for Prefabricated Bridge Element and Systems and Recommended Guidelines for Dynamic Effects for Bridge Systems (NCHRP Project 12-98), and FIU ABC-UTC Webinar on Tolerances for Prefabricated Bridge Elements and Systems (PBES).

temporary traffic barrier used to retain errant vehicles, and pedestrian access, if required.

Since the dimensions and weights of three-sided structures is substantial, cranes are required to place the sections. Adequate work areas must be provided for a crane (for outriggers, counterweight, boom, etc,), crane picks and movements/swings (overhead utility clearances, ROW arial trespass/encroachment) and material delivery at the site.

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SECTION 13

RAILINGS

Commentary is opposite the text it annotates.

13.7—TRAFFIC RAILING

13.7.1—Railing System

13.7.1.1—General

This section shall be supplemented with the following:

Traffic or combination rails are required for all structures carrying vehicular traffic. Railings may be solid concrete parapets or an open rail system.

All traffic or combination rails shall pass the crash testing requirements specified in **MASH** and shall be traffic railings on the Department Qualified Product List or the **CTDOT** Guide Sheets.

For continuous construction, the pouring sequence for all parapets shall be identical to that of the slab.

First preference shall be for solid concrete parapets.

When required by geometric or roadway design requirements, a concrete barrier wall should be detailed.

C13.7.1.1

This section shall be supplemented with the following:

Exceptions may be allowed for structures on non-NHS highways and must be approved on a case-by-case basis by **CTDOT**.

Refer to **CTDOT** Guide Sheets for typical details of **MASH**-compliant railings. The Designer is responsible for modifying these sheets as applicable for each project, but no modifications to the details shall be permitted that adversely impact the crash worthiness of the selected rail system unless approved by the Division Chief of Bridges.

For bridges on non-limited access highways where there is a strong need to provide a scenic view, an open bridge rail system approved by the **CTDOT** should be used in place of a solid concrete parapet. The use of this system should be limited to very sensitive areas.

13.7.1.1.1CT—Box Culverts and Short Span Bridges

Whenever possible, a bridge rail shall not be used and a rail system shall be as specified in **HDM** to span over short bridges and culverts.

The structure should be extended far enough behind the rail to provide the required deflection distance.

If the structure is beyond the limits of these rail systems, a solid concrete parapet or open bridge rail system with end blocks shall be used with a continuous approach rail element attached.

Should a barrier be attached to the culvert, the culvert sections must be checked for stability from collision forces and additional provisions may be required.

C13.7.1.1.1CT

On box culverts and very short span bridges, short runs of concrete parapet (less than 30 feet long) are visually disruptive and difficult to provide with an appropriate approach rail anchorage system.

One accepted alternative is nested W-beam rail systems to span over the structure by leaving out one, two or three of the rail posts. A drawing detailing these rail systems is available from the **CTDOT**. For design requirements, refer to **HDM.**

On very short structures with low drop-off heights, the **CTDOT** may waive the pedestrian and/or bicycle railing requirements on a case-by-case basis. Where pedestrian or bicycle requirements are not waived, the **CTDOT**'s Pedestrian Railing may be used, refer to Article 13.8.

13.7.1.1.2CT—Retaining Walls

On retaining walls adjacent to traffic, traffic rails shall be solid concrete parapets, 42 inches high and topped with a fence system, as applicable.

If the retaining wall is adjacent to a sidewalk, the parapet height above the top of the sidewalk shall be 36 inches and shall be topped with an appropriate pedestrian railing, bicycle railing, or fence system.

An open bridge rail system should be used in place of a solid concrete parapet where the resulting solid concrete parapet would be less than 30 feet long.

13.7.2—Test Level Selection Criteria

This section shall be supplemented with the following:

With the exception of the Merritt and Wilbur Cross parkways, a minimum of TL-4 shall be used for all interstate highways, freeways and expressways.

A minimum of TL-3 shall be used on all other state and local roadways.

The 42 inch high single slope and F-shape parapets may also be used on highways other than Interstates, expressways and freeways.

A concrete parapet with metal handrail adjacent to a sidewalk shall satisfy TL-3 crash test criteria.

MASH Test Levels (TL) shall be shown on the plans.

C13.7.2

This section shall be supplemented with the following:

Unfavorable geometric or other site conditions where vehicular rollover or barrier penetration could result in severe consequences may warrant a higher **MASH** TL as determined on a case-by-case basis.

13.7.3—Railing Design

13.7.3.1—General

This section shall be supplemented with the following:

The use of parapet end blocks above the top of the parapet shall be at the discretion of the Designer. In areas, involving sight distance problems, the parapet end blocks should not be used.

The end height of these blocks shall match the approach railing height. Where parapet end blocks are not provided, exposed rail ends and sharp changes in rail geometry shall be avoided.

The parapet details shall accommodate the lighting and signing standard anchorages outside of railing or fence. The lighting or signing shall not generally be located on a span over a railroad electrified zone.

Permanent median barriers on bridges shall be concrete and shall match the height and width on the roadway approaches. They may be either cast-in-place or precast concrete.

When required to meet architectural, historical, context sensitive design, or right-of-way settlement requirements and stipulations, aesthetic surface treatments cast on the traffic face of solid concrete traffic railings are acceptable provided the geometry of the surface asperities fall within acceptable limits for the shape of the railing. Aesthetic surface treatments referred to in this article are shapes, patterns and textures that are inset and formed into a concrete surface. Designers shall coordinate with stakeholders, both inside and outside of the CTDOT, as applicable, to determine

C13.7.3.1

This section shall be supplemented with the following:

mutually acceptable aesthetic surface treatments that meet all requirements without compromising the crashworthiness of the railing. Refer to **BRSDM** [V1B-Surface Treatments] for additional information on aesthetic surface treatments and warrants for their use.

The use of veneers or facings of stone masonry, both mortared and unmortared, on the traffic face or top of solid concrete railings is not permitted. The use of form liners that replicate the appearance of stone masonry on the traffic face or top of solid concrete railings is acceptable.

To minimize future maintenance requirements, the use of veneers or facings of stone masonry is not permitted. Railings formed entirely of concrete are preferred since they withstand minor vehicle impacts without damage, can be constructed with concrete and sealers that aid in preventing deterioration from deicing materials, and are expected to require less maintenance over their service life. Additionally, any future repairs can be performed with common materials typically used in **CTDOT** projects and by **CTDOT** maintenance forces.

Except for railings that include aesthetic surface treatments which are approved by the CTDOT as being MASH compliant, aesthetic surface treatments shall not exceed the limits described in the guidelines presented in the appendix of NCHRP Report 554, Aesthetic Concrete Barrier Design. Geometric shapes, patterns and textures inset and formed into the traffic face surface of railings shall conform to the guidelines for safety shape barriers and the guidelines for single-slope and vertical face barriers, as applicable. The surfaces created using form liners that replicate the appearance of stone masonry shall conform to the guidelines for stone masonry guardwalls.

Aesthetic surface treatments shall not exceed the guideline limits so that the safety and performance level of the railing will not be adversely affected if subject to a vehicle collision.

On railings with aesthetic surface treatments on the traffic face, the detailed concrete cover on the traffic face of the railing shall be increased, equal to the depth of the deepest depression or joint, resulting in an increase in the overall thickness of the railing, so that the minimum concrete cover is maintained at any location. The contours of the traffic face surface of the railing shall be maintained. No adjustments shall be made to the railing reinforcement.

By increasing the overall thickness of the concrete cover on the traffic face of the railing the structural and geometric crashworthiness of the railing will not be compromised.

13.7.3.2—Height of Traffic Parapet or Railing

This section shall be supplemented with the following:

All new and retrofitted traffic and combination railings on bridges and retaining wall shall have an overall minimum height of 42 inches measured from a roadway or sidewalk surface.

13.7.4CT—Temporary Traffic Barriers

Temporary barriers used to protect the traveling public during the construction of bridges shall be precast concrete and shall conform to the **CTDOT**’s standardized details.

In all cases, if the distance from the backside of the barrier to the edge of deck drop off is less than 6 feet, the barrier shall be rigidly attached to the deck unless otherwise approved by **CTDOT**. In cases where this distance is greater than 6 feet, factors such as type of road, speed, volume and composition of traffic, and the need to protect work areas with limited escape routes shall be taken into account and the barrier rigidly attached if appropriate.

Lines of barrier used strictly to separate opposing traffic

C13.7.4CT

Refer to **CTDOT** Highway Standard Drawings for typical details of temporary traffic barriers.

need not be rigidly attached to the deck and shall be paid for as a roadway item.

13.8—PEDESTRIAN RAILINGS

This section shall be supplemented with the following:

When a traffic or combination rail is not required, a railing is required when the vertical drop off is greater than 30 inches, as measured from the top of the adjacent sidewalk, roadway, or ground elevation to the lower elevation. The railing shall be a pedestrian railing, bicycle railing or fence.

When lighting or signing standards are located on structures, the railing shall be continuous at these locations. The lighting or signing shall be located outside of the continuous railing, between the railing and the outside face of parapet.

13.8.1—Geometry

This section shall be supplemented with the following:

A pedestrian rail is required on parapets less than 42 inches in height, and where a fence is not warranted.

Alternative pedestrian railings shall be designed in accordance with these specifications. The top of rail members shall be at least 42 inches above the top of the sidewalk or roadway.

C13.8.1

This section shall be supplemented with the following:

If a pedestrian rail is required, it is recommended to use the rail on both parapets, even if one of the parapets is 42 inches or greater in height.

Refer to **CTDOT** Guide Sheets "Metal Bridge Rail – Handrail" for details.

Refer to Article 13.13 for warrants for placement of fencing at bridges.

13.9—BICYCLE RAILINGS

13.9.1—General

This section shall be supplemented with the following:

When a traffic or combination rail is not required, a railing is required when the vertical drop off is greater than 30 inches, as measured from the top of the adjacent sidewalk, roadway, or ground elevation to the lower elevation.

A bicycle railing shall be designated for bridges on designated bicycle routes.

When lighting or signing standards are located on structures, the railing shall be continuous at these locations. The lighting or signing shall be located outside of the continuous railing, between the railing and the outside face of parapet.

C13.9.1

This section shall be supplemented with the following:

A map depicting designated bicycle routes in the State of Connecticut is available from the **CTDOT**.

13.11—CURBS AND SIDEWALKS

13.11.2—Sidewalks

This section shall be supplemented with the following:

Sidewalks shall be provided on bridges in accordance

C13.11.2

This section shall be supplemented with the following:

with **CTDOT** *Policy Statement E&C-19*.

The minimum sidewalk width shall be 5 feet.

Sidewalk widths may be increased in areas of heavy pedestrian traffic, on designated bike routes, or at locations requiring additional sight distance. Refer to the **Americans with Disabilities Act (ADA)** for additional requirements.

Sidewalks should be carried across a bridge if the approach roadway has sidewalks or sidewalk areas. Elsewhere, one or two sidewalks may be provided as warranted by current developments, anticipated area growth, traffic or pedestrian studies, etc.

Sidewalk curb heights on structures shall match the exposed height of the approach curbing. Where curbs are not provided on the approaches, the exposed curb height on the structure shall be 6 inches.

13.13CT—FENCING

13.13.1CT—General

A fence, if used, satisfies the requirements for either a pedestrian or bicycle railing.

For guidance on the placement of fencing at bridges, refer to Table 13.13.1-1CT.

C13.13.1CT

Under certain circumstances, fences are required by law as specified in **Public Act No. 00-184**. No waivers to these requirements that conflict with **Public Act No. 00-184** will be granted under any circumstances.

Table 13.13.1-1CT

Feature Crossed (Beneath Bridge)		**Feature Carried (Above Feature Crossed)**				
		Freeway		Non-freeway		Non-motorized user facility
		With Sidewalk	Without Sidewalk	With Sidewalk	Without Sidewalk	
Freeway		Required (1)	Not generally required	Required (1)	Required (1)	Required (1)(2)
Non-freeway		Required (1)	Not generally required	Required (1)	Required (1)	Required (1)(2)
Water		Not generally required	Not generally required	Not generally required	Not generally required	Case by case
Rail	Non-electrified	Required (1)(3)	Required (1)(3)	Required (1)(3)	Required (1)(3)	Required (1)(2)(3)
	Electrified	Required (1)(4)	Required (5)	Required (4)	Required (5)	Required (2)(4)

(1) Top of fence shall be 8 feet (96 in) above surface including any roadside barrier
(2) For rehabilitation of an existing structure with complete enclosure, consider retention of complete enclosure
(3) Maximum opening of 0.5 inch within 25 feet of tracks
(4) Top of solid barrier with height 9 feet (108 in) above sidewalk
(5) Top of solid barrier with height 8 feet (96 in) above road

When lighting or signing structures are located on structures, the fence shall be continuous at these locations. The lighting or signing shall be located outside of the continuous fence, between the fence and outside face of parapet. Fencing shall be designed with removable panels or other means to provide access to the handhole locations.

For bridges that cross multiple features, the most restrictive requirements (height, material, and configuration) apply to the entire bridge length unless a Department approved analysis indicates a transition to an adequate and less restrictive design for part of the length is cost effective.

13.13.2CT—Design Requirements

The height of the fencing above the top of the sidewalk or roadway surface shall be a minimum of 8 feet. Curved top fencing is not required.

The maximum size of the opening in the fence shall be 2 inches.

All fences shall minimize the use of horizontal rails. Fence fabric shall be installed on the pedestrian side of posts. On parapets adjacent to sidewalks, the face of the fence shall be flush with the parapet face adjacent to the sidewalk. On parapets not adjacent to sidewalks, the fence shall be set back as far as practical from the traffic face of the parapets.

Fencing should satisfy the aesthetic considerations of the structure and be designed in accordance with Article 13.8.

Where fencing is provided, it shall consist of black PVC coated fabric with galvanized steel posts and rails.

C13.13.2CT

Additional requirements for specific cases are included in the footnotes for Table 13.13.1-1CT. Refer to Article 13.13.3CT for railroad overpasses.

Exceptions will only be allowed for showcase bridges or bridges with historical significance.

13.13.3CT—Railroad Overpasses

Fencing is required on all structures over railroads. It shall be placed on both sides on the span over the railroad tracks. A solid barrier fence is required over electrified railroads.

On structures over non-electrified railroads, the maximum size of the opening within 25 feet of the tracks shall be 0.5 inches. A larger opening may be used outside of these limits.

The Designer shall coordinate with the Railroad on the requirement of curved top fencing.

13.12—REFERENCES

This section shall be supplemented with the following:

CGA. *An Act Concerning Pedestrian Walkways On Bridges And The Naming of Route 17 In Middletown*. Public Act No. 00-184. Connecticut General Assembly, Hartford, CT, 2000.

CTDOT. *Policy on Sidewalks*. Policy Statement E&C-19. Connecticut Department of Transportation, Newington, CT, 2011.

TxDOT. *Roadside Safety Device Crash Testing Program*, Research Project 9-1002-15. Texas Department of Transportation, Austin, TX.

APPENDIX A13

A13.4—DECK OVERHANG DESIGN

A13.4.1—Design Cases

This section shall be supplemented with the following:

On new bridge decks, all deck overhangs shall be designed for a minimum of TL-4 load effects.

A13.4.2—Decks Supporting Concrete Parapet Railings

This section shall be supplemented with the following:

The deck overhang shall be designed to resist the lesser of the resistance, M_c and T, or the vehicular impact moment, M_{CT}, and the coincidental axial tension force, T_{CT} calculated as follows:

For impacts within a concrete railing segment:

$$M_{CT,int} = \frac{\gamma_r F_t H_e}{L_{c,int} + (2X)} \quad \text{(A13.4.2-2CT)}$$

$$T_{CT,int} = \frac{\gamma_r F_t}{\left(L_{c,int} + (2X)\right) + 2H} \quad \text{(A13.4.2-3CT)}$$

For impacts at end of a concrete railing segment or at a joint where longitudinal rebar is discontinued:

$$M_{CT,end} = \frac{\gamma_r F_t H_e}{L_{c,end} + X} \quad \text{(A13.4.2-4CT)}$$

$$T_{CT,end} = \frac{\gamma_r F_t}{L_{c,end} + X + 2H} \quad \text{(A13.4.2-5CT)}$$

where:

γ_r = 1.2 for new or modified rails, 1.0 for analysis of existing rails
F_t = transverse vehicle impact force (kip) as specified in Table A13.4.2-1CT
H_e = effective height of vehicle rollover force (ft)
X = distribution length increase at overhang deck section being designed (ft) based on 30° angle from traffic face of barrier

CA13.4.2

This section shall be supplemented with the following:

The design of a deck overhang for Design Case 1 is also based on F_t corresponding to the test level as shown in Table A13.4.2-1CT, not the capacity of the barrier rail. To account for uncertainties in the load and mechanisms of failure, and to provide an adequate safety margin, the value F_t has been increased by 20%.

The value of L_c at the end of a concrete railing segment or at a joint is typically less than the value within a concrete railing segment. The top reinforcement in the overhang should be designed to accommodate this increased demand in this region.

Table A13.4.2-1CT—TL-4 Railing Design Forces and Geometric Criteria

H, Rail height (in)	36	42	>42
F_t, Transverse vehicle impact force (kips)	67.2	79.1	93.3
F_L, Longitudinal friction force (kips)	21.6	26.8	27.5
F_v, Vertical force of vehicle (kips)	37.8	22	NA
L_t and L_L (ft)	4	5	14
H_e, Effective height of vehicle rollover force (in)	25.1	30.2	45.5

Under *TxDOT Research Project 9-1002 "Roadside*

Safety Device Testing Program," Texas A&M Transportation Institute (TTI) researchers investigated the minimum height and lateral design load for **MASH** TL-4 bridge rails.

Researchers used impact simulations to calculate lateral impact loads imparted by the SUT (Single Unit Truck) based on **MASH** TL-4 impact conditions for a rigid single slope barrier with various heights.

Results indicated that the lateral loads for **MASH** TL-4 were significantly greater than those specified for *NCHRP Report 350* TL-4 impact conditions. Under **MASH**, the severity of TL-4 impacts increased 56% compared to *NCHRP Report 350*. Consequently, 32 inch tall barriers that met TL-4 requirements under *NCHRP Report 350* do not satisfy **MASH**. The minimum rail height for **MASH** TL-4 was determined to be 36 inches.

Further, the lateral load was determined to be approximately 68 kips. As the height of the barrier increases, more of the cargo box of the single unit truck is engaged and the lateral load on the barrier increases. For a barrier height of 42 inches, the lateral design impact load increases to approximately 80 kips. The 36 inch single slope bridge rail that was tested has a calculated capacity of approximately 70 kips. The continuous concrete rail performed well without any significant damage to the rail or deck.

The values in Table A13.4.2-1CT include the design impact loads in the lateral, longitudinal, and vertical direction, and the longitudinal distribution and height of the resultant lateral load were recommended for **MASH** TL-4 impacts.

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SECTION 14

JOINTS AND BEARINGS

Commentary is opposite the text it annotates.

14.2—DEFINITIONS

This section shall be supplemented with the following:

Bridge Deck Gap—The gap between two concrete deck ends or between an approach slab and a deck end, measured perpendicular to the joint at the top of concrete deck.

Combined Movement Range, CMR—Movement range at a deck joint gap caused by the Thermal Movement Range, using the design installation temperature as the upper limit, in combination with beam end rotation and other contributing longitudinal movements at the top of deck.

Deck Joint Gap—The perpendicular width of an opening identified as a bridge joint, measured at the wearing surface level between headers at two adjacent deck ends or between an approach slab and a bridge deck end. If no headers are constructed, but an asphaltic plug joint is installed, the deck joint gap is the perpendicular width of the opening measured at the level of the concrete bridge deck. A deck joint gap may be different than a bridge deck gap to accommodate installation of a bridge joint and to ensure adherence with manufacturer recommended minimum and maximum joint openings.

Effective Span Length—The sum of all lengths of spans contributing to the thermal movement at a bearing or joint.

Thermal Movement Range, TMR—Dimensional changes of a superstructure, typically measured at a bearing, bridge joint or deck end, due to changes in ambient temperature between a specified high and low. Dimension changes occur in all directions within the plane of the deck, but for the design of bridge bearings and joints, thermal movements are typically separated into the longitudinal and transverse components.

14.4—MOVEMENTS AND LOADS

14.4.2—Design Requirements

This section shall be supplemented with the following:

Thermal movements and rotations for all bearings, asphaltic plug joints, and preformed joint seals shall be designed using Service-I limit state with γ_{TU} = 1. All other joints shall be designed using Service-I limit state with γ_{TU} = 1.2.

14.4.2.2—High Load Multirotational (HLMR) Bearings

This section shall be supplemented with the following:

High Load Multirotational (HLMR) bearings should be considered for locations with moderate to high loads combined with moderate to high movement. The Designer should not completely design HLMR bearings for each location; however, a preliminary design should be done to determine the rough overall dimensions of the bearing.

C14.4.2.2

This section shall be supplemented with the following:

The specifications for HLMR bearings require the Contractor or their Fabricator to design the specific bearings based on the type of bearing that is supplied.

14.5—BRIDGE JOINTS

14.5.1—Requirements

14.5.1.1—General

This fourth paragraph shall be supplemented with the following:

Longitudinal deck joints should be avoided wherever possible due to problems with motorcycle safety and difficulties associated with the intersection of the transverse deck joints. If longitudinal joints are unavoidable, they shall be located out of the traveled way. Since differential vertical movement is common in longitudinal joints, the only joints that should be considered are elastomeric concrete headers with a preformed joint seal or concrete headers with a neoprene strip seal.

C14.5.1.1

This section shall be supplemented with the following:

A preformed joint seal with elastomeric concrete headers is preferred.

14.5.1.2—Structural Design

This section shall be supplemented with the following:

If the bridge is designed for seismic events where significant movement is important to the proper function of bridge elements (such as isolation bearings), the movement due to seismic forces shall be accommodated in the design of the joints.

For other bridges, the joint need not be designed for seismic movement and should not be designed to survive the seismic event undamaged.

14.5.1.3—Geometry

This section shall be supplemented with the following:

Refer to Article 14.7.9.1 for direction of movement of the superstructure.

14.5.2—Selection

This section shall be supplemented with the following:

It is not recommended to install transverse joints in the pavement for buried structures.

14.5.2.2—Location of Joints

This section shall be supplemented with the following:

Generally, open finger joints shall only be used behind the abutment backwall where the water and debris can be intercepted by a concrete drainage structure.

14.5.2.3CT—Thermal Movement Range (TMR)

The following are recommended expansion joint systems for various thermal movement ranges. Designers shall consider skew of the joint relative to the direction and

C14.5.2.3CT

The Designer shall investigate if the recommended expansion joint system for a given thermal movement range is the best choice for a given site. Site conditions may warrant magnitude of thermal expansion when selecting and sizing the joint.

alternative expansion joint systems. Alternative expansion joint systems may be used with the approval of **CTDOT**.

Each product manufacturer provides guidance as to how skew affects the way that product functions. Designers should become familiar with how each type of joint functions to ensure that the joints are properly designed.

For thermal movement ranges up to 0.5 inches, it is not recommended to install transverse joints in the pavement for bridges with slab over backwall.

Asphaltic plug joints are recommended for movement ranges up to 1.5 inches. For joints with this movement range at pin and hangers, a preformed joint seal is recommended.

Preformed joint seals are recommended for movement ranges of 1.5 to 3 inches.

Reinforced concrete headers with neoprene strip seals and anchored extrusions are recommended for movement ranges of 3 to 4 inches.

Finger and modular joints are recommended for movement ranges greater than 4 inches.

14.5.2.4CT—Fixed Joints

For fixed joints at abutments and piers, an asphaltic plug joint is preferable to a sawed and sealed joint. Designers should take into consideration rotational movements and evaluate the need for sawed and sealed joints.

For "slab over backwall" conditions, where roadway settlement at the abutment is expected to be minimal, such as with approach slabs supported by the backwall, asphaltic plug joints are not required for fixed joints and no transverse joint in the pavement is required.

When an approach slab is anchored on a shelf in a backwall that is integral with the bridge deck, there is no need for an asphaltic plug joint or any type of joint. The membrane shall be made continuous over the construction joint between the deck and approach slab.

C14.5.2.4CT

Although the asphaltic plug joint is more expensive than a sawed and sealed joint to install and is prone to rutting under heavy wheel loads and shoving under heavy braking forces, it handles settlement at abutments better and the exact placement of the joint is not so critical to its functioning, as with a sawed and sealed joint.

14.5.3—Design Requirements

14.5.3.2—Design Movements

The first paragraph shall be supplemented with the following:

The maximum roadway surface gap, *W*, in., for a single gap shall apply at a temperature of 20ºF.

C14.5.3.2

The first paragraph shall be supplemented with the following:

This provision considers the safety of motorcycles and bicycles, which are not typically on the road at -10ºF, a temperature of 20ºF is more practical for evaluating the maximum roadway surface gap.

The roadway surface gap is the sum of the gap at installation, the thermal movement of the joint as the bridge contracts and time-dependent movement. The deck joint gap at installation shall be set by the Designer with consideration for the maximum roadway surface gap. A joint with a thermal movement range of 3 inches will likely test the 4 inch maximum roadway surface gap limit. The application of the maximum roadway surface gap at a temperature of 20ºF will

Bridge deck joints shall be designed to the Combined Movement Range (CMR).

allow use of the single-gap joint in larger ranges of thermal movement.

Beam end rotation is most critical to check for asphaltic plug joints, which do not accommodate the sudden movements from live load as well.

14.5.6—Considerations for Specific Joint Types

14.5.6.1—Open Joints

This section shall be replaced with the following:

Open Joints, excluding finger joints, shall not be used on State projects.

C14.5.6.1

This section shall be replaced with the following:

The Department prefers not to use open joints due to the deicing chemicals and sand used during the winter.

14.5.6.1.1CT—Finger Joints

Where a proper drainage structure can be constructed behind the abutment backwall, an open finger joint can be considered. The drainage structure should be provided with an access door or manhole for cleaning. The structure shall also be connected to a storm drainage system or a standard outlet.

Where the bottom of the drainage structure is not the top of the abutment footing, a 2 foot deep sump should be detailed to catch sediment.

At piers, where the location of the joint is at the crest of a vertical curve, an open finger joint can be considered. A drainage trough shall be provided that is connected to a proper piping system, refer to Article 2.6.6.3.3CT.

C14.5.6.1.1CT

At piers with greater than 4 inches of thermal movement, modular expansion joints are the first preference for joint type.

14.5.6.4—Joint Seals

This section shall be supplemented with the following:

Joint seals shall be preformed silicone glands or foam-supported silicone joint seals.

Joint seals shall be secured between elastomeric concrete headers as specified in Article 14.5.6.11CT or between two concrete surfaces.

To install a preformed joint seal, there must be a joint gap in which to secure the sealing gland. This gap may be formed between the following components:

- Deck ends
- A backwall and a deck end
- A deck end with a double backwall
- A deck end and an approach slab
- A deck end with an approach structure
- Two parapet ends
- Two sidewalks

When the maximum roadway surface gap, W, as specified in Article 14.5.3.2, is greater than 4 inches, another type of joint shall be considered.

C14.5.6.4

This section shall be supplemented with the following:

Refer to **CTDOT** Guide Sheets, for typical details. The Designer is responsible for modifying these sheets as applicable for each project. To assist designers with selection of preformed joint seals, a spreadsheet is available on the **CTDOT** Bridge Design website. Designers shall be responsible for selecting a properly designed joint seal for the specific joint conditions and shall only be used as a guide.

The deck joint gap shall conform to the joint manufacturer's recommendations for depth of shelf, minimum gap and minimum gap at installation. The deck joint gap will vary depending on temperature. The gap will be maximum when the temperature is at its lowest. Although preformed joint seals are available for movement ranges larger than 3 inches, the roadway surface gap measured in the direction of travel is limited by Article 14.5.3.2.

Preformed silicone joint seals are divided into two groups:

- V-shaped silicone joint seals
- Foam-supported silicone joint seals

Preformed joint seals for bridges with sidewalks shall be foam-supported silicone joint seals.

For sidewalks, the shape and density of foam-supported silicone joint seals is more suitable for foot traffic and reduces the tripping hazard. Since there is no acceptable transition from a V-shaped silicone joint seal in a bridge deck joint to foam-supported silicone joint seal, the foam-supported silicone joint seal shall be continuous through the bridge deck, sidewalk and parapet.

14.5.6.4.1CT—Skew Effects

Skew affects joint seals differently, so the joint seal shall be selected and designed appropriately.

14.5.6.4.1aCT—V-Shaped Silicone Joint Seals

V-shaped silicone joint seals are designed to prevent tension from occurring in the seal or in the bonding point. The seal is adhered to both sides of the joint at the time of installation. Thermal movement causes the gap to open and close in the direction of travel for bridges on tangent alignments.

Designers shall follow the manufacturer's written guidance regarding how the skew affects the thermal movement capacity of their seal. In lieu of manufacturer guidance, the following shall be used as guidance for selection of V-shaped silicone seals with adjustments to the manufacturer's nominal capacity:

- Skews from 0 degrees to 30 degrees: No adjustment to nominal capacity is needed.

- Skews between 30 degrees and 60 degrees: Select a seal with a nominal capacity larger than:

$$\frac{TMR}{1-(\frac{\theta-30}{30})(1-0.625)}$$

where:

θ = Skew angle of bearing line

C14.5.6.4.1aCT

When the seal is installed in a joint that is skewed to the direction of travel, the seal experiences relative movements between both sides of the seal. The movement can be described as two components:

- normal to the joint
- movement parallel to the joint (racking)

Seals in non-skewed joints experience movement predominately normal to the joint. Seals in joints oriented normal to the roadway can reach their nominal capacity in thermal movement because this is how they are designed.

Seals in skewed joints experience additional movement parallel to the joint that introduces racking or shear forces into the seal. This reduces the nominal capacity of the seal. Bridges with skews larger than 30 degrees require a larger nominal size seal to accommodate racking and movement normal to the joint.

14.5.6.4.1bCT—Foam-Supported Silicone Joint Seals

Foam-supported silicone joint seals are pre-compressed foam seals with a waterproof silicone coating. They are not affected by skew to the degree that a V-shaped joint seal is because the foam is in compression at all times. The direction of movement is less important than the magnitude of movement.

Due to vector components of movement on the skewed joint, the following is recommended when selecting the size of the foam-supported silicone joint seal:

- Skews from 0 degrees to 30 degrees: Select a seal with a movement capacity 0.25 inches larger than the calculated CMR
- Skews between 30 degrees and 45 degrees: Select a seal with a movement capacity 0.5 inches larger than the calculated CMR.
- Skews above 45 degrees: Select a seal with a movement capacity 0.75 inches larger than the calculated CMR.

14.5.6.5—Poured Seals

This section shall be replaced with the following:

Poured Seals shall not be used on State projects.

14.5.6.6—Compression and Cellular Seals

This section shall be supplemented with the following:

Compression and Cellular Seals shall not be used on State projects.

14.5.6.7—Sheet and Strip Seals

This section shall be supplemented with the following:

Sheet and strip seals shall be used in conjunction with reinforced concrete headers and anchored extrusions.

Strip seals in sidewalks shall be covered with sliding steel plates, detailed to meet ADA requirements. The steel plates shall be anchored into the sidewalk on both sides of the joint.

The neoprene seal should be detailed from curb to curb and up the top of the curb portion of the parapet (approximately 11 inches above the pavement).

When the maximum roadway surface gap, W, as specified in Article 14.5.3.2, is greater than 4 inches, another type of joint shall be considered.

14.5.6.8—Plank Seals

This section shall be replaced with the following:

Plank Seals shall not be used on State projects.

14.5.6.9—Modular Bridge Joint Systems (MBJS)

14.5.6.9.1—General

This section shall be supplemented with the following:

Modular expansion joints may be used at abutments, provided that the distance between the abutment backwall and the ends of the beams and diaphragms is kept to 2 feet

C14.5.6.9.1

This section shall be supplemented with the following:

minimum in order to facilitate inspection and future maintenance.

At piers, the distance between adjacent diaphragms shall be kept to 2 feet minimum in order to facilitate inspection and future maintenance. The beam ends may be kept closer if proper maintenance can be accomplished.

At piers with a thermal movement range of greater than 4 inches, first preference for joint type should be modular expansion joints.

Joint manufacturers should be contacted for specific requirements for each joint and to ensure that each joint can fit within the bridge slab. The design and detailing of modular expansion joints is the responsibility of the manufacturer of the joint; however, the Designer should provide the proper room in the slab for the installation of the joint.

Modular joints shall be detailed from curb to curb and up to the top of the curb portion of the parapet (approximately 11 inches above the pavement).

For bridges with skews, the joint system should be run into the parapet on the skew and covered with curb plates. The curb plates shall be designed to accommodate all movements, and the free edge should overlap the parapet on the trailing edge of the parapet.

14.5.6.9.3—Testing and Calculation Requirements

This section shall be supplemented with the following:

Only joints that have successfully tested for fatigue and approved by **CTDOT** may be used.

14.5.6.10CT—Asphaltic Plug Joint

Asphaltic plug joints should generally not be specified for skews greater than 45 degrees. Use of asphaltic plug joints for skews greater than 45 degrees may be permitted with the approval of **CTDOT**.

C14.5.6.10CT

Manufacturers limit the skew at which the joint may be installed. Typically, this skew is 45 degrees.

The asphaltic plug expansion joint system shall always be placed after the final pavement has been placed on the bridge and the pavement in the area of the header has been saw cut and removed. This applies for rehabilitation and new construction.

Refer to **CTDOT** Guide Sheets, for typical details. The Designer is responsible for modifying these sheets as applicable for each project.

The asphaltic plug joint should be detailed from curb to curb.

The joint in the parapet should be sealed.

Refer to **CTDOT** Guide Sheets for details for the seal at parapets.

14.5.6.11CT—Sawed and Sealed Joints

Sawed and sealed joints shall be used on all integral abutment bridges or bridges without a traditional joint.

Saw cutting shall be used when bituminous concrete overlays are 8 inches or less in depth.

C14.5.6.11CT

Thicker overlays increase the chance of the joint location being misaligned and a crack forming next to the saw cut, leading to pavement deterioration.

For overlay thicknesses greater than 8 inches, sealing can be performed on any potential crack propagation as a part of future maintenance efforts.

The Bridge Plans shall clearly identify the total overlay depth and the location, length, and skew for all sawing and sealing operations.

14.5.6.12CT—Headers

Reinforced concrete headers are appropriate for use with strip seal, modular, finger, and plank joints. Elastomeric concrete headers are appropriate for use with preformed joint seals.

Elastomeric concrete headers shall always be placed after the final pavement has been placed on the bridge and the pavement in the area of the header has been saw cut and removed. This applies for rehabilitation and new construction.

The header material should be recessed 1/8" below the bituminous overlay to account for long-term compaction of the bituminous overlay under traffic.

C14.5.6.11CT

This applies to both elastomeric and reinforced concrete headers.

14.6—REQUIREMENTS FOR BEARINGS

14.6.1—General

The first paragraph shall be supplemented with the following:

Keeper blocks may also be used to restrain some of these loads. When anchor bolts are required at bearings, stainless steel bolts shall not be used.

The fifth paragraph of this section shall be replaced with the following:

Combinations of bearing types should not be used at the same line of bearing.

This section shall be supplemented with the following:

Bearings may also be required to allow for horizontal movement due to temperature and time dependent causes, allow rotation due to loads on the superstructure, and transmit seismic forces from the superstructure to the substructure. The selection and layout of bearings shall be consistent with the proper function of the bridge.

Steel reinforced elastomeric bearings shall be the first bearing of choice for any bridge bearing due to the low initial cost and the low future maintenance costs. These bearings should be considered for low to moderate load situations.

In general, elastomeric bearings shall be used to support prestressed deck units and prestressed concrete girders. The bearings may be either plain or steel-laminated.

On structures composed of butted deck units, the use of a single sheet of elastomer, placed continuous between the fascia units, is not permitted. Each deck unit shall rest on two individual bearings.

The design of single span bridges may be based on providing elastomeric expansion bearings at both ends of the super structure if the grade of the roadway is less than 5%. The Designer should incorporate keeper assemblies in order to maintain alignment of the superstructure. Designs of this nature will reduce the amount of expansion at the bearings and deck joints.

C14.6.1

Differing deflection and rotational characteristics may result in damage to the bearings or structure.

For simple span bridges, with a fixed and an expansion bearing, the fixed bearing should be located at the low end of the structure.

The design and layout of bearings in multi-span bridges should be based on the design of the deck expansion joints, the capacity of the bearings to accommodate the anticipated loads and movement, and the seismic design of the substructure where applicable.

Provisions for jacking of superstructures shall be provided at all locations that have bearings that will require future maintenance. These bearings include all types other than fixed bearings. Refer to Article 6.7.9CT.

Several bearing types have been recommended within this document for different situations. Other bearing devices may be used, provided that they have been approved by **CTDOT**.

14.6.3—Force Effects Resulting from Restraint of Movement at the Bearing

14.6.3.1—Horizontal Force and Movement

This section shall be supplemented with the following:

Provisions shall be made in the bearing design for both lateral and longitudinal movement based on the geometry of the deck, the layout of the deck expansion joints and keeper assemblies. For bridge with complicated deck configurations, a thermal expansion analysis of the deck should be done in order to determine the thermal movements relative to the bridge bearings. The geometry of the deck, not the structural framing, should be the basis for the expansion analysis. For narrow bridges where the effects are minimal, transverse expansion may be neglected.

14.6.5— Seismic and Other Extreme Event Provisions for Bearings

14.6.5.3— Design Criteria

This section shall be supplemented with the following:

If the bridge is designed for seismic events, the bearings may be designed to transmit seismic forces from the superstructure to the substructure. The movement due to seismic forces shall be accommodated in the design of the bearings. It is important that the bearing remain stable under the maximum anticipated bridge displacement during the seismic event.

Rocker type bearings should not be used due to the high susceptibility of overturning during seismic events.

C14.6.5.3

This section shall be supplemented with the following:

For requirements for the design of seismic isolation bearings, see Article 3.10.

14.7—SPECIAL DESIGN PROVISIONS FOR BEARINGS

14.7.4—Pot Bearings

14.7.4.5—Sealing Rings

14.7.4.5.2—Rings with Rectangular Cross-Sections

This section shall be replaced with the following:

Sealings rings with rectangular cross-sections (flat) are prohibited.

C14.7.4.5.2CT

Sealing rings with rectangular cross-sections are more prone to leakage of the elastomer.

14.7.4.5.3—Rings with Circular Cross-Sections

This section shall be supplemented with the following:

Sealing rings used to secure the elastomer disc within the pot shall be circular in cross-section.

14.7.5—Steel-Reinforced Elastomeric Bearings-Method B

This section shall be supplemented with the following:

The preferred design method for steel-laminated elastomeric bearings shall be Method A. Should various constraints limit the use of Method A, Method B may be used.

C14.7.5

This section shall be supplemented with the following:

Method A provides a more conservative design. The testing requirements for Method B can cause unnecessary increases in lead times and costs.

14.7.5.1—General

This section shall be supplemented with the following:

Steel-reinforced elastomeric bearings may be designed as either rectangular or round. Round elastomeric bearings should be considered where significant movement occurs in both the longitudinal and transverse direction.

For the design of steel bridge beams, the top of the bearing should be vulcanized under heat and pressure to a steel top plate to facilitate installation. The top plate should be bolted to a beveled sole plate. Field welding should be avoided due to the possibility of damage to the elastomer during welding.

For prestressed concrete bridge beams without steel sole plates, if the grade of the roadway is less than 5%, the bearings may be manufactured with a sloping top surface provided that the internal steel reinforcement plates are parallel and level.

14.7.5.2—Material Properties

This section shall be supplemented with the following:

Steel reinforced elastomeric bridge bearings shall only be designed with virgin neoprene, not natural rubber.

14.7.5.3—Design Requirements

14.7.5.3.2—Shear Deformations

This section shall be supplemented with the following:

If the shear force in the bearing is less than 20% of the minimum vertical load on the bearing, the interface of the bearing and the concrete bearing seat should not be attached or bonded.

For cases where the shearing force is greater, the following possibilities should be investigated:

- The bearing should be redesigned to attempt to reduce the shearing force.
- The bearing should be shop vulcanized under heat and pressure to a bottom steel plate that is anchored to the substructure.
- A PTFE slider type bearing can be considered.

14.7.6—Elastomeric Pads and Steel-Reinforced Elastomeric Bearings—Method A

14.7.6.1—General

This section shall be supplemented with the following:

The preferred design method for steel-laminated elastomeric bearings shall be Method A. Should various constraints limit the use of Method A, Method B may be used.

Cotton Duck fabric reinforced elastomeric bearings should be considered for locations with low to moderate loads combined with moderate to high movement.

The movement due to expansion is accommodated between the PTFE and the slider plate. The PTFE material should be bonded to the top surface of the bearing. The slider plate shall be welded to a top plate or the beveled sole plate.

C14.7.6.1

This section shall be supplemented with the following:

Method A provides a more conservative design. The testing requirements for Method B can cause unnecessary increases in lead times and costs.

14.7.9—Guides and Restraints

14.7.9.1—General

This section shall be supplemented with the following:

For curved superstructures, provisions shall be made in the alignment of bearing guides and keeper blocks for both lateral and longitudinal movement based on the geometry of the deck and the layout of the deck expansion joints.

Refer to Article 14.5.1.2 for direction of longitudinal movement for curved superstructures.

14.7.10—Other Bearing Systems

This section shall be supplemented with the following:

Steel bearings may be used where no movement is necessary and where the only rotation is in the transverse axis of the bridge. A 0.125-inch thick, 90 durometer random fabric pad should be used to seat the steel masonry plate on the concrete substructure bearing pad. For steel bridge beams, the anchor bolts for the bearing should not pass through the flange of the beam.

14.8—LOAD PLATES AND ANCHORAGE FOR BEARINGS

14.8.3—Anchorage and Anchor Bolts

14.8.3.1—General

This section shall be supplemented with the following:

Elastomeric bearings shall be unattached to the substructure. When anchor bolts are required, holes for anchor bolts shall not pass through the elastomeric bearing and shall be located outside the limits of the bearing.

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SECTION 15

DESIGN OF SOUND BARRIERS

Commentary is opposite the text it annotates.

15.1—SCOPE

C15.1

This section shall be supplemented with the following:

The **CTDOT** frequently uses the term "Noise Barrier." To be consistent with **BDS**, the use of "Sound Barrier" and "Noise Barrier" shall be considered interchangeable.

15.4—GENERAL FEATURES

15.4.5CT—Design and Detailing

15.4.5.1CT—Railing Requirements

Structure-mounted sound barriers may be attached to new and existing railings on bridge and wall structures.

On bridges, the railings shall be solid concrete parapets with a minimum height of 36 inches and a maximum height of 42 inches above the finished roadway surface that are connected to reinforced concrete decks. On wall structures, the railings shall be solid concrete parapets with a minimum height of 36 inches and a maximum height of 42 inches above the finished roadway surface that are connected to a wall stem constructed entirely of reinforced concrete. The attachment of a structure-mounted sound barrier to a wall stem that is wholly or partially constructed of stone masonry is not permitted. The minimum distance from the rear vertical face of the parapet to the traffic face at the top of the parapet shall be not less than 7.5 inches.

Parapets and supporting decks shall meet the requirements of Article 15.7.

Deck overhangs supporting parapets with structure-mounted sound barriers shall satisfy the applicable Strength and the Extreme Event limit states.

Parapets with structure-mounted sound barriers shall satisfy the applicable Strength and Service limit states.

New parapets used to support structure-mounted sound barriers shall be either single slope or F-shape parapets determined to be **MASH** TL-4 compliant by the **CTDOT**. Supporting structure-mounted sound barriers on new vertical shaped parapets, with or without an adjacent sidewalk, is not permitted. Deviation from this criterion is not permitted unless allowed by an approved design variance.

Existing parapets used to support structure-mounted sound barriers shall be either single slope or F-shape parapets determined to be **MASH** TL-4 compliant by the **CTDOT**, single slope or F-shape **MASH** TL-4 compliant parapets meeting the requirements of ECD-2019-8 or meet the requirements of Article 15.4.5.1.1CT. Supporting structure-mounted sound barriers on existing vertical shaped parapets, with or without an adjacent sidewalk, is not permitted, except

C15.4.6.1CT

The minimum height requirement of 36 inches meets the minimum height requirement for a **MASH** TL-4 railing.

The maximum height of the parapet must consider the projecting rail elements of the structure-mounted sound barrier. The location of the projecting rail elements above the finished roadway surface is fixed and cannot be altered as it is based on crash testing of the structure-mounted sound barrier.

The article applies to existing parapets and supporting decks.

Refer to Section 13 and Article A13.4.

For **MASH** TL-4 compliant railings, refer to Section 13.

The use of new vertical shape parapets to support structure-mounted sound barriers is not permitted due to potential adverse effects of vehicle climb and rollover due to a vehicle collision on the structure-mounted sound barriers system. Since structure-mounted sound barriers are generally located along major highways and expressways, and vertical shape parapets are generally located on shorter structures on secondary routes, the condition is rarely encountered.

as allowed by Article 15.4.5.1.1CT. Deviation from these criteria is not permitted unless allowed by an approved design variance.

15.4.5.1.1CT—Parapets Supporting Existing Structure-Mounted Sound Barriers

Parapets supporting existing structure-mounted sound barriers may be used to support upgraded structure-mounted sound barriers provided the following requirements are met:

- The existing structure-mounted sound barrier is being upgraded via a project initiated specifically for that purpose
- The existing parapet is one of the following:
  1. 42 inch F-Shape Solid Concrete Parapet
  2. 32 inch F-Shape Solid Concrete Parapet with concrete curb, with or without metal traffic railing
  3. 32 inch Jersey Shape Solid Concrete Parapet with either a granite or concrete curb, with or without metal traffic railing
  4. 34 inch Brush Curb parapet, with or without metal traffic railing
  5. 42 inch vertical parapet with modified sloped curb and parapet cap
- The parapet meets the condition requirements of Article 15.7.
- The upgraded sound barrier weight per linear foot and height above the finished roadway surface is no greater than the existing sound barrier. The height of the upgraded structure-mounted sound barrier shall meet the requirements of Article 15.4.5.2CT.
- Parapets less than 36 inches shall be modified to increase the parapet height to 36 inches. Parapet modifications to increase the parapet height may be made with cast-in-place concrete cap or with a hollow structural section (HSS) anchored to the top of the parapet. The modifications shall provide continuity across all joints (paraffin-coated joints, deck joints, contraction joints from wall stems, expansion joints from wall stems), with the use of sliding splices to distribute potential CT load effects.

Parapets supporting upgraded structure-mounted sound barriers need not be investigated for CT load effects at the Extreme Event II limit state.

C15.6.4.1.1CT

The **CTDOT** has determined that upgrading structure-mounted sound barriers (replacing existing structure-mounted sound barriers on existing parapets) is acceptable provided the specified requirements are met since the structural adequacy of the upgraded structure-mounted sound barrier system to resist load effects will be no less than the existing system and the upgraded structure-mounted sound barriers include a panel retention system for added safety. The installation of a structure-mounted sound barrier on an existing parapet shall meet the requirements of Article 15.4.5.1CT.

The traffic railing portion of a combination railing shall meet the requirements of Article 15.4.5.1CT. Parapet heights less than 36 inches are required to be increased to at least 36 inches to meet **MASH** TL-4 minimum height requirements. A greater height is not required since the upgraded structure-mounted sound barrier will provide the additional height to meet OSHA requirements. Increasing the height of existing parapets above the **MASH** TL-4 minimum height requirements may subject the parapet to increased load effects due to a vehicle collision.

15.4.5.2CT—Structure-Mounted Sound Barrier Requirements

Structure-mounted sound barrier walls shall be crashworthy; include a panel retention system; meet specific material, acoustic, and performance requirements; and meet general material requirements.

The crashworthiness of structure-mounted sound barrier walls is based on the structure-mounted sound barrier wall system being successfully crash tested to meet one of the following:

C15.4.6.2CT

Structure-mounted sound barriers systems consist of sound barriers connected to solid concrete railings.

For a structure-mounted sound barrier to meet the **MASH** TL-4 crash-testing requirements, the barrier must be attached to a solid concrete railing meeting the requirements of **MASH** TL-4.

- **MASH** TL-4, Tests 4-10, 4-11 and 4-12 requirements, as documented in a signed crash test report by an accredited crash testing facility
- **MASH** TL-4, Test 4-12 requirements and written justification of meeting Test 4-10 and Test 4-11 requirements, as documented in a signed crash test report by an accredited crash testing facility

The specific material, acoustic, and performance requirements for structure-mounted sound barrier walls are addressed by the **CTDOT** construction specifications.

The minimum **MASH** TL-4 requirement meets the test levels of successfully crash-tested proprietary structure-mounted sound barriers.

The crashworthy requirement provides criteria for selecting acceptable structure mounted sound barriers. The **CTDOT** understands that if a structure mounted sound barrier is installed on a parapet with a geometric shape and structural resistance that is different than the parapet that was used in the crash tested system, the structure mounted sound barrier system may no longer be considered **MASH** TL-4 compliant. As a result, the **CTDOT** does not require the MASH compliance of structure-mounted sound barriers systems be noted on the plans.

The panel retention system prevents panels and parts of panels from dislodging and falling from the structure due to a vehicle collision.

The use of structure-mounted sound barriers with major components and elements comprised entirely of timber is not permitted.

The use of timber is not allowed due to its limited long-term durability and acoustical properties, and since it is unknown if it can meet **MASH** TL-4 crash-test requirements. Proprietary structure-mounted sound barriers are available that are comprised of wall panels constructed of durable man-made materials with adequate acoustical properties that have been successfully crash tested and meet **MASH** TL-4 requirements.

The use of structure-mounted sound barriers comprised of concrete panels placed between vertical posts is not permitted.

The use of concrete structure-mounted sound barriers is not allowed due to the weight of the concrete components, the need for supplemental support member connection to the superstructure framing, and since it is unknown if **MASH** TL-4 crash-test requirements can be met.

The maximum height of the structure-mounted sound barriers above the finished roadway surface shall be the least of the following:

The maximum height of the structure-mounted sound barrier on a structure must be limited so as not to obstruct inspection and maintenance access. Since inspection and maintenance access requirements depend on site constraints and equipment, barrier height limitations shall be determined on a project basis. Refer to Article 15.4.7CT.

- The maximum height allowed by the **CTDOT** Bridge Safety and Evaluation Unit so as not to obstruct access to the structure by inspection equipment
- The maximum height allowed by the **CTDOT** Bridge Maintenance so as not to obstruct access to the structure by maintenance equipment
- The maximum height of successfully crash-tested structure-mounted sound barriers included in the contract documents
- The maximum height based on meeting structural design requirements
- The maximum height based on noise analysis by **CTDOT** Office of Environmental Planning (OEP)

No portion or section of posts, panels or other appurtenances of the structure mounted sound barrier shall project above the maximum height limit.

The minimum height of the structure mounted sound barrier is limited by the projecting rail elements. The location of the projecting rail elements above the finished roadway surface is fixed and cannot be altered as it is based on crash testing of the structure-mounted sound barrier.

The traffic face of the structure-mounted sound barrier shall be parallel to the vertical rear (outside) face of the railing. Elements or components projecting from the traffic face of the sound barrier above the top of the railing are not permitted unless they are part of a crash-tested structure-mounted sound barrier. The offset distance from the gutter line of the parapet to the traffic face of a projecting rail element shall be no less than the distance provided for the crash-tested structure-mounted sound barrier.

At locations where the outside of the parapet projects to support other appurtenances, if the barrier panels cannot be placed to the traffic face of the appurtenance and maintain the barrier alignment due to a conflict with the appurtenance, the appurtenance must be either removed or relocated, or the barrier cannot be installed on the parapet.

Typically, crash tested structure-mounted sound barriers do not include panels that are installed discontinuously.

Posts shall only be connected to the outside face of the parapet. All posts shall be installed plumb. Anchor plates shall be used to connect the posts to the parapet. Beveled anchor plates, encompassing all the anchors at a connection, shall be used at non-vertical parapet surfaces. The use of washers at each anchor to compensate for the non-vertical parapet surface is not permitted.

Post connections to new concrete shall be made with cast-in-place anchors. Post connections to existing concrete shall be made with post installed adhesive bonded anchors. All anchors shall be installed perpendicular to the posts. The use of concrete cored holes with through-bolts resulting in a portion of the anchorage being exposed on the traffic face of the railing is not permitted. Post connections to steel superstructure or concrete superstructure framing members is not permitted. The reuse of existing anchor rods or anchorages is not permitted.

Cast-in-place anchors are specified in new concrete to simplify anchor installation and eliminate anchor testing.

Refer to Article 5.13.5CT, for post-installed adhesive bonded anchor design requirements.

Posts shall be connected to the parapet with no less than 6 anchor rods. The use of multiple anchor plates with 2 or more anchors is acceptable. The preferred distance from centerline of an anchor rod to a vertical expansion, contraction, construction, or control joint in concrete shall be a minimum of 12 inches. In no case shall an anchor rod have less than 2 vertical reinforcing bars on each side of an anchor rod before being interrupted by a vertical joint. The reduction in anchorage resistance due to free edges, vertical and horizontal joints in the concrete, and embedded appurtenances, such as conduits and junction boxes, shall be accounted for in the anchorage design.

No less than 6 anchor rods are required to ensure a redundant post connection

The top of the wall panels may be fabricated so that the top of the barrier follows the grade of the structure, or the top of the wall panel is stepped to accommodate the grade resulting in a barrier with varying heights along the structure.

Designers should be aware that some sound barriers, such as those with acrylic panels, can be fabricated so that the top of the barrier follows the grade of the structure. Other sound barriers, such as those with composite panels with an aluminum facing, the top of the barrier cannot be fabricated to follow the grade of the structure and must be stepped to accommodate the grade resulting in a barrier with varying heights along the structure.

Where multiple panel elements are stacked to form a wall panel, the panel elements shall be designed for load effects due to self-weight as well as load effects due to the wall panel elements that rest upon them.

The longitudinal joint between stacked wall panel elements, the joint between the wall panel elements and the posts, and the joint between the bottom of the wall panel and the top of the railing shall be detailed to prevent the transmission of sound from one side of the barrier to the other. Details and materials shall prevent the accumulation of water. The use of caulking for a sealant is not permitted due to its limited-service life. At the top of parapets with a full height vertical surface on the rear face, this requirement is met if the panels are extended below the top of the parapet for a distance equal to no less than 4 times the gap between the vertical parapet face and the panel's traffic face plus the depth of the chamfer at the top of the parapet.

The wall panels shall be secured to the overall sound barrier system in such a way that elements, pieces, or fragments do not fall when they are deformed or broken. The panel retention system shall be designed to withstand the self-weight of the relevant parts multiplied by a minimum safety factor of no less than 4.

The design and detailing of the sound barrier shall account for the thermal movement of the barrier components as well as the movement of the structure to which it is mounted. Details shall allow for thermal movement of the sound barrier components, thermal movement of the supporting structure, material tolerances and installation tolerances.

The leading and trailing edge of the structure-mounted sound barrier shall be designed and detailed to include a sloped crash rail to mitigate the severity of the impact of an errant vehicle. The sloped rail shall extend from the top of the parapet to the top longitudinal rail element. The slope of the rail shall be no greater than that used on the crash tested structure-mounted sound barrier.

The design and details at the ends of the structure-mounted sound barrier shall be coordinated with the Designer of the approach sound barrier

This requirement is included to ensure that the Designer of the ground-mounted sound barrier understands how the structure-mounted sound barrier will be terminated so that the termination of the approach sound barrier can be addressed appropriately in the contract documents.

15.4.6CT—Bridge Load Rating

Bridges with structure-mounted sound barriers shall meet the bridge load rating requirements of **BRSDM** [V1B] – Load Rating.

15.4.7CT—Designer's Responsibilities and Contract Documents

The need for sound barriers and their limits is determined during the design phase of a project by **CTDOT** OEP. Sound barriers on bridge or wall structures are referred to as structure-mounted sound barriers. Due to their proprietary nature, structure-mounted sound barriers are design-build contract items designed by the Contractor during the construction phase of a project.

During the design phase of a project the structural Designer is responsible for preparing and incorporating

C15.4.7CT

This article is added to provide guidance for the design of structure-mounted sound barriers.

contract documents comprised of plans, construction specifications and quantity estimates for the structure-mounted sound barriers into the project. The structural designer is responsible for ensuring the adequacy of the structure-mounted sound barriers as well as the adequacy of the structures and components that support them. Additionally, the structural designer is responsible for ensuring the constructability of the structure-mounted sound barriers to meet all design and detailing requirements as well as all site conditions and constraints. To ensure the adequacy and constructability of structure-mounted sound barriers, the structural designer shall undertake an investigation that includes, but is not limited to, the following:

- Determining the condition of the existing structure and the work required, if any, to meet the design and detailing, and existing structure condition requirements
- Determining the remaining service life of the existing structure and its components
- Determining if the existing solid concrete railings are **MASH** TL-4 compliant
- Determining which types of sound barriers that can be attached to the structure and meet all design and detailing requirements as well as all site conditions and constraints.
- Coordinate with the **CTDOT** Bridge Safety and Evaluation Unit and **CTDOT** Bridge Maintenance to determine if the maximum height of the structure-mounted sound barrier on a structure must be limited so as not to obstruct inspection and maintenance access. Since inspection and maintenance access requirements depend on site constraints and equipment, barrier height limitations shall be determined on a project basis. Designers shall coordinate with the **CTDOT** OEP on the maximum height of the sound barrier on each structure.
- Determining if the post to railing attachment can be adequately designed
- Determining the bridge load rating with the structure-mounted sound barrier

Load rating shall meet the requirements of the **BLRM** and **BRSDM** [V1B] – Load Rating.

Based on the results the investigation, provided the structure-mounted sound barrier will be adequate and constructable, and the supporting structure is adequate, the structural designer shall prepare plans of the conceptual layout and details of a structure-mounted sound barrier. The plans, construction specifications, and quantity estimates shall be incorporated into the project.

Unless otherwise shown on the plans, the construction specifications for the structure-mounted sound barrier allow the Contactor to choose the type of material to be used for the wall panels. The structural designer shall coordinate with **CTDOT** OEP to determine if the type of wall panel used in the barrier will be restricted and note the acceptable panel type on the plans. Additionally, unless otherwise shown on the plans, the construction specifications specify the color to be used for each type of wall panel. The structural designer shall coordinate with **CTDOT** OEP to determine if the color of the

The construction specification includes requirements for structure-mounted sound barriers constructed with uncoated, aluminum panels and colorless, transparent panels.

Uncoated aluminum panels are specified instead of coated panels to eliminate both field repair of the coating damaged during construction and potential future maintenance painting.

If a panel color other than those specified in the construction specification are required, the color of the ground-mounted approach sound barrier shall be taken into consideration when selecting a color.

wall panels will be other than that required by the construction specification and note the selected color on the plans.

During the construction phase of the project, working drawings and calculations for the structure-mounted sound barrier submitted by the Contractor shall be reviewed by the structural designer.

If the load effects from the structure-mounted sound barrier shown in the working drawings affects a structures live load rating, the structural designer shall revise the load rating to reflect the changed load effects before the review of the working drawings is complete.

Revised or updated load ratings shall meet the requirements of the **BLRM** and **BRSDM** [V1B] – Load Rating.

15.7—SOUND BARRIERS INSTALLED ON EXISTING BRIDGES

This section shall be supplemented with the following:

Structure-mounted sound barriers are only allowed on existing structures meeting the following condition requirements:

- The structure shall be rated to be in fair/satisfactory condition. The National Bridge Inventory (NBI) condition rating of the structure’s elements and components subject to load effects from structure-mounted sound barriers shall be no less than 6.
- The structure’s elements and components subject to load effects from structure-mounted sound barriers shall have no cracks that will compromise the structural adequacy of either component.
- The structure’s elements and components subject to load effects from structure-mounted sound barriers shall have not sustained any vehicle collision damage that will compromise the structural adequacy of either component.
- The structure’s elements and components subject to load effects from structure-mounted sound barriers shall have no evidence of deterioration, distress, or instability that will compromise the structural adequacy of either component.

A review of the existing inspection reports on the existing structure and a field inspection of the existing structure shall be performed and documented to determine whether the structure’s condition meets the preceding requirements.

The installation of a structure-mounted sound barrier on existing structures not meeting the preceding condition requirements is not permitted unless allowed by an approved design variance.

C15.7

This section shall be supplemented with the following:

The intent of these requirements is to ensure that the condition of the parapet will be adequate for the service life of the structure-mounted sound barrier.

15.8—LOADS

15.8.4—Vehicle Collision Forces

This section shall be supplemented with the following:

The structure-mounted sound barriers that have been successfully crash tested and meet the crashworthy requirements of Article 15.4.5.2CT require no further analysis due to the vehicular collision force for Extreme Event II provided post spacing and panel height do not exceed the maximum values of the crash-tested system and the materials, details and structural resistance of the sound barrier and its connections meet or exceed the materials, details and structural resistance of the similar elements of the crash tested structure-mounted sound barrier system, and the sound barrier is attached to a to new or existing parapet that meets the requirements of Article 15.4.5.1CT. If the post to parapet connection geometry (such as anchor spacing/placement) or materials (such as lower concrete strength), differ from that of the crash tested structure-mounted sound barrier system, analysis of the affected elements and components is required to ensure that the structure-mounted sound barrier meets the **BDS** requirements.

15.8.5CT—Unit Loads

Table 15.8.5-1CT includes unit weights for panels that can be used for structure-mounted sound barrier preliminary feasibility investigations and bridge load rating.

Table 15.8.5-1CT – Unit Weights for Preliminary Investigations

Wall Panel Type	Panel Weight
Aluminum	5 in. thk. x 1.64 ft. high, insulated, wt. = 6.1 psf
Transparent	For 15 mm (19/32 in.) thk. panel, wt. = 3.66 psf
	For 20 mm (25/32 in.) thk. panel, wt. = 4.86 psf
	For 20 mm (1 in.) thk. panel, wt. = 6.10 psf

The weight of the posts, stiffening members and other hardware must be added to the panel weight to obtain a permanent load of the structure-mounted sound barrier.

Only panel weights are provided since the post spacing and post heights may vary due to site constraints.

The weights provided are for panels that meet the acoustic requirements in **CTDOT** construction specifications.

For sample calculation of weight per square foot, see Appendix CT15.1.

APPENDIX CT15.1—SAMPLE CALCULATIONS

Table CT15.1-1 – Sample Calculations of Weight per Square Foot

Transparent Acrylic Panel, assumed t = 0.594 in.	3.66 psf * 14.0 ft. * 10.0 ft.	=	512	#
Aluminum Frame, assume 2.62 #/ft.	2.62 #/ft. * 14.0 ft. * 9.83 ft.	=	360	#
Upper longitudinal rail, HSS 8 x 4 x 1/4	19.02 #/ft. * 10.0 ft.	=	190	#
Lower Longitudinal rail, HSS 8 x 4 x 1/4	19.02 #/ft. * 10.0 ft.	=	190	#
Post, assumed W6x20	20 #/ft. * (13.5 + 3.0)ft.	=	330	#
Subtotal		=	1582	#
Miscellaneous: plates, fasteners, gaskets, etc.; assume % of subtotal	20%	=	316	#
Total		=	1898	#
Average weight of sound barrier per square foot of exposed panel above top of parapet	1898# / [(13.5 ft.)*(10 ft.)]	=	14	#/ft^2

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Bridge and Roadway Structures Design Manual

Release 1

Volume 3

Bridge Rehabilitation and Preservation

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DESIGN GUIDANCE FOR SELECTING A CULVERT LINER

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I. GOALS FOR LINING CULVERTS

This guidance document assumes that a decision has been made by **CTDOT** to rehabilitate a culvert using a structural liner. Nonstructural liners will be allowed as a temporary method to stabilize an existing culvert quickly to provide time to initiate a project to address the proper rehabilitation or replacement of the culvert. A nonstructural liner may also be used in combination with a structural liner to achieve a smoother interior surface. Selecting a structural liner to rehabilitate a culvert is complicated. Selection requires consideration of many stakeholders. It is important to set some goals that the culvert liner must satisfy. These typically include:

A. Meets the Purpose and Need established for the project

B. Coordinates with other projects in Planning, Design or Construction phases

C. Considers the need for culvert extension for roadway widening or to minimize the height of endwalls

D. Is structurally adequate (shall be designed by LRFD Design Methodology)

E. Is hydraulically adequate (allowable headwater depth, effects of flooding both upstream and downstream, etc.)

F. Avoids potential erosion, piping, and scour

G. Is durable

H. Is capable of providing a 50-year minimum design service life unless the Department authorizes a lesser service life

I. Considers the minimum height inside of the lined culvert for inspection access (typically 5')

J. Is maintenance free, needs little maintenance, or does not hinder access for maintenance

K. Supports habitat/passage for fish and other species where applicable

L. Considers the need for a detour or stage construction

M. Considers construction access to the inlet and outlet

N. Considers geometry of channel and available length at inlet and outlet to set liner in channel before it is pushed or pulled into the host culvert.

O. Minimizes environmental impact from construction activities

P. Is constructible

Q. Considers worker safety for culvert entry and construction techniques

R. Considers water handling needs

II. INFORMATION COLLECTION

The Preliminary Design phase begins with a Rehabilitation Study Report (RSR) to investigate culvert lining options and studying viable options to present alternates. Before viable options can be investigated, information about the culvert to be lined and the site where the culvert is located must be collected. Begin by collecting original construction plans, as-built plans, and plans for subsequent projects for rehabilitation or extension of the culvert. If the culvert conveys a watercourse, the previous hydraulic analysis and reports should be obtained as well. Bridge maintenance memos and responses shall also be obtained. The Bridge file contains several Bridge Safety Inspection Reports, load rating reports, and numerous other documents that are valuable sources of information. Other documents that may provide useful information are the Proposed Project Information (PPI) developed for the culvert and OEP Early Resource Screening information. These documents contain preliminary purpose and need as well as environmental concerns at the project location. This can provide an early notice of fisheries preliminary findings and recommendations.

The following information is typically available in the Bridge Safety Inspection Report, but if not, should be collected:

A. Age of culvert (year built)
B. Geographic location
C. Feature carried (if a roadway, include width of roadway, configuration of lanes and shoulders and position of roadway between inlet and outlet of culvert)
D. ADT and ADTT and classification of roadway carried by culvert
E. Feature conveyed through culvert (roadway, path, stream, etc.)
F. Structure type (Pipe, pipe arch, vertical or horizontal ellipse, box culvert, arch, etc.)
G. Shape and size (span, rise, diameter, etc.)
H. Material and thickness or gage (asphalt-coated corrugated steel, galvanized steel, masonry, etc.)
I. Maximum and minimum height of fill over culvert
J. Embankment slopes
K. Channel characteristics, including width, shape, depth, natural or paved bottom, etc.
L. Condition rating of the culvert, wingwalls, endwalls, etc.
M. Documented damage to the structure, including loss of coating systems, overall corrosion, loss of thickness at isolated locations and along invert, and perforations
N. Deflections and obstructions within the culvert
O. Voids below invert and behind culvert walls
P. Presence of erosion and scour at inlet and outlet or presence of a cutoff wall or protective apron
Q. Piping of water below invert
R. Flow depth measurements

III. ENVIRONMENTAL TESTING

When a culvert rehabilitation/replacement project is initiated, the bridge inspection report should be reviewed to determine if there is excessive corrosion in the culvert at and below the high-water mark inside the culvert. Reports for culverts along the same waterway upstream and downstream can also be reviewed to determine if they have experienced excessive corrosion. Excessive corrosion can be related to corrosion plus abrasion caused by the streambed material, but if the rate of deterioration has accelerated, consideration shall be given to perform environmental testing at the culvert site to determine the potential presence of corrosive elements. Such testing will typically be requested at project initiation and shall be performed before preparing RSR alternates for culvert lining to avoid selecting liner materials that could be vulnerable to corrosion at the site. A request for environmental testing is warranted when the watershed area upstream of the culvert contains any of the following:

- Past or present industries such as mines, tanneries, steel mills, pulp mills, textile plants, metal and plating industries, production of fertilizers, soap, paper, dyes, fungicides and insecticides
- Water discharge including sewage treatment, industrial wastewater, household wastewater, runoff from a heavily salted roadway, a hazardous waste site or naturally decaying material
- Farmlands that are likely being fertilized.
- Tidal or brackish water

The environmental testing shall be performed by a qualified environmental engineer using properly cleaned equipment and acceptable sampling methods and should include the following tests:

1. pH
2. Resistivity
3. Chloride concentration (testing not necessary in marine environments, but required in potentially brackish water)
4. Sulfate concentration (testing not necessary in marine environments, but required in potentially brackish water)

Sampling shall include:

- Water in the stream
- Water outfall from pipes that outlet upstream of the culvert
- Water from below the channel bottom

Since culvert liners are grouted within the host culvert, they are not as vulnerable to soil-side corrosion as the host culvert. Environmental testing should focus on water in and below the channel and water drainage/runoff that flows into the channel at or near the culvert. During low flow periods, water in the channel mixes with and is affected by subsurface water that interacts directly with the soil. This interaction may alter the pH and resistivity of the water and increase chloride and sulfate concentrations. These changes can produce an environment that is corrosive to the culvert liner.

The Designer shall consider the effect of pH and Resistivity on calculation of service life using the Culvert Liner Selection Worksheet. The effect of chlorides and sulfates is more difficult to quantify but understanding the role these factors play in corrosion and deterioration of the culvert liner is helpful in making a recommendation for a culvert liner in the RSR.

When environmental testing is not warranted or test results are not available during preparation of the RSR, the following assumptions can be made for input into the liner selection worksheet and to assist in determining service life of the liner material being considered:

1. pH:

    - Freshwater rivers: 6.5 (6.0 within pine forests)

- Saltwater (marine environments): 8.0 (7.5 to 8.5)
- Brackish water: 7.0 (6.0 to 8.5)
- Stormwater runoff channels: 5.5 (5.0 to 6.0)
  Rainwater in Connecticut has a pH of about 5.0. These runoff channels may have very low flows or be dry a great deal of the year. Therefore, it is not reasonable to use the lowest pH to estimate service life.

2. Resistivity:

   - Freshwater rivers: R = 4,000 ohm-cm
   - Stormwater runoff channels: R = 5,000 ohm-cm
   - Rivers with direct runoff upstream from heavily salted roads: R = 2,500 ohm-cm
   - Brackish water: R = 200 to 2,000 ohm-cm
   - Saltwater (marine environment): R = 25 ohm-cm

3. Chlorides:

   - Freshwater rivers: 100 ppm or higher chloride concentration in water with a pH of 7.3 or lower can cause corrosion of metal culvert liners. pH may not matter at Abrasion Level 3 and higher.
   - Brackish water: Assume chlorides are at a sufficient concentration to promote corrosion if the tidal influence is anticipated at the culvert site.
   - Saltwater (Marine environment): Assume chlorides are at a sufficient concentration to promote corrosion.

   Chlorides are present at culverts in a marine environment or brackish water in areas of tidal influence. Water in marine environments typically has a higher pH than in rivers and streams containing fresh water. When the pH is above 7.3, resistivity has a much greater influence than pH on the corrosion rate of metal culvert liners. Resistivity in saltwater can be very low compared to freshwater. This can significantly shorten the service life of metal culvert liners. A steel liner with a protective polymeric or Type II aluminum coating are protected by their barrier coatings from corrosion and can provide an acceptable service life if abrasion does not destroy the coating first. Galvanized steel and aluminum liners are more vulnerable in saltwater environments and should generally not be specified.

   Chlorides may also be present at culverts that receive surface water runoff from roads treated heavily with de-icing salts in the winter. Surface water can seep into roadway embankments and migrate into streams. Drainage outlets can deposit water from catch basins close to the inlet of a culvert. Freshwater streams usually have a pH lower than 7.3. When pH is low and chlorides reduce resistivity, the protective oxidative layer that forms on metal can be dissolved, and corrosion can accelerate. When pH is 7.3 or lower, the equation for service life is a function of both pH and resistivity. At lower pH and resistivity levels, the effect of both variables dramatically shortens the service life of metal culvert liners. Understanding the environment for which the culvert liner is designed is very important.

The presence of de-icing salts is not a year-round phenomenon. Heavy spring rains and melting snow often wash away salts or dilute the concentration in the water that passes through the culvert. Because of the limited seasonal use of de-icing salts, and the fact that corrosion reactions decrease in the colder temperature, corrosion of culvert liners due to chlorides from road salt does not significantly shorten the service life of a culvert. Testing of chloride concentration and resistivity during low flow periods will give the best indication of how chlorides in the water will affect corrosion of a metal culvert liner. Designers shall use judgment when considering how the seasonal fluctuation of chlorides impacts the determination of service life.

4. Sulfates:

   100 ppm or higher Sulfate concentration in water can cause corrosion of metal culvert liners. When pH is below 7.0 and sulfate concentration is 1,500 ppm or more, Designers shall specify low-permeability concrete for invert liners. Type II cement is recommended to resist sulfates.

   Sulfates are mineral salts containing sulfur. Sulfate ions are detrimental to the passive oxide film that protects steel from corrosion. The sulfates do not chemically react to form the corrosion product but are an activator of corrosion. Once the corrosion reaction begins, even if conditions return to those favoring passivation of the metal, corrosion still progresses. Sulfates can act synergistically with chlorides to further accelerate corrosion on metal surfaces. Hydrogen sulfide when present in water makes the water acidic and causes rapid corrosion – even in the absence of oxygen.

   The decay of plants and animals and some industrial processes produce these salts. Mines, tanneries, steel mills, pulp mills, and textile plants also release sulfates in the environment. Industrial wastewater, household wastewater, runoff from a hazardous waste site or naturally decaying material can put sulfates into waterways, rivers, lakes and streams. Water that flows through rock or soil containing gypsum could contain significant sulfate concentration. Water that contains sulfates seeps through soil and contaminates groundwater. Fertilizer applied to fields on farmlands supplies sulfates that can be carried by surface runoff to drainage channels, streams and rivers.

   Sulfates can lower the pH of water, causing corrosion to metals. If sufficient acidic water is available, it can neutralize the pH of concrete, leading to corrosion of reinforcing steel. Sulfates can combine with the lime in cement to form calcium sulfate (gypsum), which creates structural weakness in concrete culverts and liners and shortens the life of the concrete.

IV. SITE VISIT AND DESIGN INSPECTION

A site visit is a required step in the Preliminary Design phase and is critical for preparing contract documents. A Design Inspection collects information that is relevant to structural evaluation of the culvert, to the presentation of alternates, and to the preparation of contract documents for a construction project.

Determine if it is safe to enter the culvert. Contact Bridge Safety for confined space requirements. Culverts that are determined to be unsafe to enter may still be able to be surveyed using 3-D laser scanning technology by setting the instrument up at the inlet and outlet ends.

For culverts that are safe to enter, perform a design inspection. The inspection shall include confirmation of all information collected as well as the physical condition of the culvert. It shall include observations of all conditions that affect the proper functioning of the culvert and the rehabilitation of the culvert. Bridge Safety can provide guidance on safety of entering culverts.

Document or verify the following during the design inspection:

A. Roadway width and approach geometry.

B. Ease of access to inlet and outlet.

C. Presence of utilities, roadway drainage, roadway barrier, noise barrier wall and any other potential obstacle to access or constructability.

D. Proximity of residences, businesses and private property and their access to the roadway near the culvert.

E. Depressions, sinkholes and cracks in the roadway or ground above the culvert, erosion, scour, undercutting of the stream banks and other embankment instability concerns.

F. The presence of regulated areas and potentially sensitive habitats.

G. Identify the presence of sediment, rocks and other debris on the bottom of the culvert. Document the size and type of material present and the depth of material in the culvert.

H. Section loss due to corrosion, abrasion, or other environmental factors. Include general section loss and isolated section loss. Identify the presence and locations of perforations. Section loss will be used to determine the capacity of the culvert to support earth and other loads. If debris is present in the bottom of the culvert, then it may need to be removed to observe and measure damage to the invert. If it is not possible or practical to remove debris during the design phase, the designer shall document the decision to not remove debris. In such cases the designer shall make conservative assumptions regarding the condition of the invert and review those assumptions with the Department for approval. Removal of debris by the construction contractor and inspection of the invert during construction may be an alternative to confirm assumptions made during design. During design, the designer shall develop an alternate plan for rehabilitation should the proposed method of rehabilitation be determined during construction to not be feasible. The Department will decide whether to proceed with the proposed alternate and if the alternate plan is reasonable to implement by change order. It may be necessary to re-advertise the project if the condition is determined during construction to be considerably worse.

I. Isolated deformations, deflection and distortion. Settlement of the soil beside the culvert or excessive live load can cause deflection or deformation of the culvert. Noncircular culverts are particularly sensitive to bending distortion. A large rock in the backfill can apply a concentrated load on a culvert wall, deforming it unsymmetrically in a localized area. Voids in the backfill or unbalanced compaction during installation can cause unsymmetrical loading on a culvert cross section. Some deformations are caused by the failure of bolts at a seam, which leads to cusping of the culvert plates.

J. Cross sectional measurements of the culvert along its length. Before measuring cross sections, it is recommended to lay out a survey baseline through the culvert to which cross sections may be referenced. Cross sections should be developed at regular intervals along the baseline and at additional, critical locations identified by the designer. Most culverts deflect under load and do not have the same size or shape as when they were first installed. Deflections will vary along the length due to differences in loading. Identify and measure bolt heads, which reduce the available inside dimensions for lining. Removal of debris in the culvert may be required to obtain measurements.

Document if the cross section is uniform or changes along the culvert length. Note also if there is a horizontal curvature, sharp bends or angle points in the alignment that would limit the length of liner that can be installed. If any portion of the culvert must be removed to install a liner, note where the removal will begin and end along the length and/or cross section and how much material must be removed.

K. Profile of the culvert invert. High embankments in the middle of the culvert length cause greater settlement in the culvert than at the ends. This can create a sag in the profile of the culvert invert. Elevations along the invert along the entire length of culvert can help a designer determine if there is a sag.

L. Joints (type and condition), if present, between culvert sections. Indicate if the joint is leaking and take note if soil or sediment is washing through the joint.

M. Presence of coatings remaining (usually missing at the bottom) and where along the perimeter the coatings have been worn away. Identify limits of corrosion and abrasion around the perimeter for use in designing a liner for abrasion.

N. Voids in the bedding or backfill. Often, voids in the bedding can be seen through perforations in the invert, but if not visible, voids can be detected by hammer-tapping the culvert to detect a hollow sound). Pay particular attention to voids at the bottom corners of pipe arches. Loss of bearing at the corners can quickly result in failure of the culvert.

O. Average flow condition/depth and high water marks.

P. Observation of whether water flows below the invert instead of/in addition to flowing through the culvert. Flow through the bedding is called "piping," and may create voids below the invert.

Q. Baffles, roughness rings or other instream fish passage or energy dissipation features inside the culvert.

R. Configuration of inlet and outlet. Document wingwall and headwall configuration and condition, mitered or square culvert ends, presence of tapered inlet, flow diversion walls/deflectors, endwalls, slope protection, cutoff walls, aprons, nosing between multiple culvert barrels, debris collection, rock weirs, composition of the channel bed material, sedimentation or erosion of the channel, scour holes, vegetation, etc. Document the inlet configuration for potential hydraulic improvement.

S. Determine if water is salty/brackish (riverine or tidal condition).

T. Test water for pH and resistivity as this can affect the longevity of culvert liners.

U. Width of channel upstream and downstream (Bankfull width). Is the water channelized or does it overflow the banks and cover a floodplain? Does water rise quickly in a flood event or is there a short timeframe in which water can be handled before it overtops a cofferdam?

V. DETERMINATION OF ABRASION LEVEL

Classify the abrasion level at the culvert on a scale of 1 to 6 as defined in Appendix B, with “1” being the least severe and “6” the most severe. Abrasion Level will be needed during the selection of culvert liner material. Abrasion Level will be used to determine the appropriateness of material and in the calculation of target design service life. Abrasion level is one of the most important determinations in culvert liner selection. Abrasion level directly affects the rate at which liner material wears and helps determine if a liner material is suitable at all. Wear rates typically outpace corrosion rates at Abrasion Level 4 and above. If careful evaluation of a site can lead to a determination of Level 3 abrasion or lower, abrasion typically does not control the selection of a liner material. Although particle size is extremely important, availability of a moderate volume of abrasive aggregate of the specified size and smaller in the channel bed can be the difference between classification of abrasion as Level 3 or 4.

Before the abrasion level at a specific culvert can be classified, the type and volume of bedload available to cause abrasion shall be investigated. This can be performed by visual examination during a site visit. It is recommended that a geotechnical engineer, accompanied by a hydraulic engineer and the Designer inspect the channel and culvert bottom to identify the type (including angularity) and size of aggregates present. Before visiting the site, the hydraulic engineer shall perform or obtain a preliminary hydraulic analysis to determine the velocity and depth of flow through the culvert during a two-year event. Determine the size aggregate that will likely be moved by the two-year flow (a table is available in Appendix B labelled "Bed Materials Moved by Various Flow Depths and Velocities"). Particle size is the most useful property in determining if bed material will move during specific flow conditions. In the field, observe the available volume of that particle size and smaller that can be moved by a two-year flow and classify the volume as:

- No bed load
- Minor
- Moderate
- Heavy

Presence of material by itself should not be the sole determinant of bed load volume. In addition to observing the volume of aggregate at or below a specified size that is available in the stream bed to move during a 2-year event, the following observations will help in determining whether the available bed material will be likely to move and cause abrasion:

- The slope of the channel profile
- Roughness of the channel
- The cross sectional shape of the channel
- Angularity and size distribution of bedding material
- Compaction level of bed material
- Bed armoring
- Evidence of historical scour/erosion/deposition/bank cutting
- Obstacles to flow (or lack thereof), such as rocks, rock weirs, scour holes, debris in the channel, paved inverts, etc.

It is important to observe the channel just upstream of the culvert as well as a significant reach of channel upstream, since that material can be transported downstream to the culvert being rehabilitated. Hydraulic Engineering Circular, HEC-20 – Stream Stability at Highway Structures provides guidance in qualitatively determining sediment transport characteristics of the watercourse at the culvert being lined or replaced. It is not anticipated that a sieve analysis will be needed for the purpose of determining abrasion level, however, a bar probe may be used to examine the channel bottom and determine the layer depth of channel bottom material contributing to the determination of volume. Penetration of the bar into the channel bed (or lack of penetration) can provide insight into the ability of the material to armor the channel bottom, potentially reducing erosion and sediment transport. However, armor materials often appear in a shallow layer over sand or finer material. Transport of this material can expose the fine bedding material below it which also must be considered in evaluating the bed load volume since its particle size is less than the size of particles in the armor layer.

Observe and document abrasion within the existing culvert (and nearby culverts, if possible). If scour holes are present, look for deposition of abrasive materials in the scour hole as well, since the velocity may have slowed enough over the scour hole to deposit the material that has caused abrasion.

VI. STRUCTURAL CULVERT LINER MATERIAL OPTIONS

Structural liner materials can first be evaluated for abrasion and corrosion at the given culvert site to determine liners that may be eliminated from consideration. With a list of viable structural liner options, designers can begin to eliminate options based on other goals discussed in Section I above. Liner materials that are appropriate for environmental conditions for the desired service life and for other goals can be preliminarily sized for structural capacity to support the height of fill and all live loads that the culvert is expected to carry. A structural culvert liner shall be designed as if in direct burial without composite action with the host culvert. Although the grout in the annular space between the host culvert and the liner adds stiffness and strength to the liner, the maximum stiffness of the grout used in design calculations shall not exceed that of well-compacted, well-graded angular sand and gravel backfill. The culvert liner shall be designed for the known soil envelope around the host culvert. If the soil modulus of the backfill envelope is not known, it may be selected from Table 1 in Appendix B assuming Type III soil. The capacity of the liner shall be determined in accordance with **BDS** [12] – Buried Structures and Tunnel Liners.

Be sure to check if the liner can support all Design, Legal and Permit Vehicles. Structural liners spanning 6’ or more shall be load rated in accordance with the **CTDOT BLRM** and a load rating report shall be submitted during the design phase by the 90% plan submission.

For most culverts when fill heights exceed 8’ to 10’, live loads will not be distributed to the culvert but will bridge the culvert through soil arching and be applied to the compacted soil columns on both sides of the culvert. For these situations, a Capacity/Demand Ratio shall be computed in accordance with the **BLRM** for the culvert to confirm that the liner has sufficient capacity to support the dead load of the soil prism above. For each liner span and height of fill, a preliminary design of the liner shall be performed to confirm that the liner can support the loads.

If a liner cannot perform structurally under live load, consider constructing a distribution slab beneath the roadway pavement to span the culvert and distribute load to the soil beside the culvert instead of to the crown of the culvert. Removing live load from the structure won’t increase the service life of a culvert liner, but it can increase the number of options that are structurally acceptable. Designers should note that a distribution slab may interfere with buried utilities and roadway drainage. Preliminary thickness of distribution slabs may be estimated as 1” thickness for every foot of span. Precast concrete slabs may be specified to expedite installation. Such slabs should be designed with a longitudinal key, dowels or other mechanical connection between slabs to transfer live load to adjacent slabs transversely. A 6” thick minimum compressible layer of lightweight fill or foam should be placed just below the center half of the distribution slab so load is distributed to the ends of the slab instead of bearing directly above the crown of the culvert. If a concrete distribution slab is considered to remove live load effect from the liner be sure to include the extra cost of installing a distribution slab in the RSR alternative.

Structural culvert liner options are categorized as follows:

A. Thermoplastic/Thermosetting Resin

1. High Density Polyethylene (HDPE)
    a. Solid Wall
    b. Corrugated
        i. Smooth Interior
        ii. Corrugated Interior
    c. Profile Wall - Smooth Interior
2. Polyvinyl Chloride (PVC) – Solid Wall
3. Fiberglass – Solid Wall

B. Metal

1. Steel-Reinforced Polyethylene (SRPE)
2. Aluminum
    a. Spiral Rib Smooth Interior
    b. Corrugated
    c. Structural Plate
    d. Tunnel Liner
3. Steel - Uncoated or coated (Coatings include galvanized, aluminized, and polymeric)
    a. Solid Wall
    b. Corrugated (spiral or annular ribs)
        i. Smooth Interior
        ii. Corrugated Interior
    c. Structural Plate (galvanized)
    d. Tunnel Liner

C. Concrete

A. THERMOPLASTIC/THERMOSETTING RESIN

Availability

Thermoplastic, fiberglass and PVC culvert liners are available as shown in Table VI.1 below:

Table VI.1 - THERMOPLASTIC / THERMOSETTING RESIN LINER AVAILABILITY

WALL TYPE	INTERIOR	MAX. SPAN	WALL THICKNESS	AVAILABLE STIFFNESS*
PVC				
Solid Wall	Smooth	15” O.D.	0.44”	NA
Profile Wall	Smooth	48” I.D.	0.19” Inner Wall	NA
Fiberglass				
	Smooth	Any reasonable span	Custom	NA
HDPE				
Solid Wall	Smooth	63” O.D.	O.D./Dimension Ratio	SDR 41 SDR32.5 SDR 26
Corrugated	Corrugated or Smooth	60” Nom. I.D.	Varies by diameter See AASHTO M294	Varies by diameter See AASHTO M294
Profile Wall	Smooth	120” I.D.	Varies by Dia. and Ring Stiffness Coefficient (RSC)	RSC 160 RSC 250 RSC 400

*Liner stiffness:

SDR = Standard Dimension Ratio = O.D./Wall Thickness

RSC = Ring Stiffness Constant = load applied between parallel plates needed to deflect the pipe 3%, measured in pounds per foot per percent deflection (**CTDOT** requires RSC 160 min. stiffness)

Pipe stiffness of corrugated liner is measured at 5% deflection in psi, and varies by diameter

PVC liner pipe is only available in standard diameters up to approximately five feet with a smooth inner wall type. Wall thickness is also standard and not customizable.

Fiberglass liner is available in any shape and span and can be ordered to custom lengths. Fiberglass liner can be customized to fit any culvert within tight tolerances if desired. In addition, the thickness of fiberglass liner can be custom designed to resist abrasion and support all required loads.

HDPE liner, like PVC, is available in standard sizes only. Solid wall and corrugated HDPE are available in limited maximum diameters, but profile wall HDPE is available in a wide range of standard diameters. Solid wall HDPE is available in the same limited span range as corrugated HDPE but offers various thicknesses for the same diameters. Corrugated HDPE offers both smooth wall and corrugated wall types, which may make it more desirable in certain applications. Corrugated HDPE may offer cost advantages for lining small diameter culverts that are not subjected to live load or moderate to high abrasion. Most HDPE selected for culvert lining will be profile wall because of the wider range of sizes and choices for stiffness (stiffer pipe offers a thicker inner wall). Profile wall HDPE pipe is available in open and closed profiles. It is recommended to specify closed profile. Closed profile pipe includes an inner and an outer wall. This offers extra protection against abrasion perforating the liner and additional structural capacity if the inner liner becomes perforated.

Selecting the size of the liner is often critical for meeting hydraulic requirements. Designers shall account for known deformations in the host culvert and anticipate continued deformation until the liner is installed and select a liner that will easily clear these obstacles. The outside diameter of bell and spigot type joints shall also be accounted for so they will also clear obstructions in the host culvert. Host culverts that are metal with bolted seams have additional obstructions from nuts or bolt heads inside the structure. When possible, it is recommended to create a template of the liner and pull it through the culvert to demonstrate that the specified size liner will be able to be installed.

Liners made of different materials are not all supplied in the same standard dimensions or increments. Solid wall liners are typically specified by outside diameter whereas corrugated and profile wall liners are specified by inside diameter. To make matters even more complicated, corrugated liner is specified by a nominal inside diameter while profile wall inside diameter is actual. Designers shall check the availability of the desired liner dimension before assuming that material may be specified. Availability varies between suppliers, so check with more than one supplier before assuming that a product is not available in the desired material and size.

Designers looking for the advantages that HDPE, PVC and fiberglass have to offer may also consider steel-reinforced HDPE liner. This material is addressed under metal liners, but comes with a thinner wall, which may allow a larger diameter to fit in the host culvert.

Abrasion and Corrosion Resistance

Abrasion Levels 1 to 3:

HDPE, PVC and Fiberglass are all suitable materials. Abrasion is negligible.

Abrasion Levels 4 to 6:

- PVC liner abrades two times faster than steel
- HDPE liner abrades at a rate 2 to 5 times faster than steel
- Fiberglass liner abrades 12 to 30 times faster than steel.

Smooth liners facilitate higher flow velocities and allow abrasive bedload to move faster through the culvert. Abrasion of smooth liners is more uniform along the invert. Corrugated liner wears faster along the upstream face of the corrugations due to more direct impact from abrasive bed load. Corrugations tend to cause stones to tumble and bounce through the culvert, impacting the liner with more energy.

HDPE, PVC and fiberglass liners do not corrode. They are generally unaffected by pH, Resistivity, chlorides and sulfates. Before a Designer selects one of these liner materials, the longterm section loss due to abrasion must be considered. After calculating the loss of thickness, the Designer shall evaluate the liner’s capacity to resist loads with the reduced section properties. Corrugated and profile wall liners have relatively thin walls to begin with. Although they have sufficient section properties to resist short-term loads, the reduction in long-term material properties, coupled with losses due to abrasion makes them vulnerable to failure due to buckling. Grout around the liner helps resist global buckling, but local buckling may still be a failure mechanism. Liners with closed profiles can resist local buckling better than liners with open profiles.

Hydraulics

HDPE, PVC and fiberglass liners are all available with smooth inner walls. Installing a smooth liner inside a corrugated metal host culvert, even though the inside diameter is smaller, may provide a hydraulically acceptable solution. In some cases where the flow through the culvert is inlet-controlled, the constriction created by the liner can adversely affect the hydraulic capacity of the lined culvert. In these instances, consider designing an improved inlet. Improved inlets may be created by beveling the grout in the annular space at the upstream end of the liner. Alternatively, structures can be installed at the upstream end to funnel water into the liner. Fiberglass can be used to create special structures to create improved entry conditions. Such structures can be used at the leading end of the liner as an integral extension of the liner.

In some cases it is desirable to slow the flow of water down before it exits the culvert. HDPE in diameters up to 60" offers a corrugated inner wall. Corrugations can be used to slow the flow velocity to reduce erosion of the stream bed at the outlet. Corrugated interior pipe shall not be used in Abrasion Levels 4 to 6 without considering the service life of the liner.

Baffles or sills can also be installed in the invert to slow the flow of water and to encourage sedimentation of natural stream bed materials in the culvert. The diameter and length of sections of liner play a role in accessibility to install baffles. Longer sections of liner require a minimum diameter of approximately 48 inches to provide sufficient access to attach baffles or sills in the middle of the length. Due to maximum diameter limitations, long lengths of PVC, solid wall HDPE and corrugated HDPE liner should not be used if baffles would be installed in the middle of the length. Long lengths of profile wall HDPE in smaller diameters should also not be used if baffles must be installed in the middle of the length. Baffles in HDPE, PVC and fiberglass pipe can be damaged by larger stones and debris moving at high velocities. Before selecting a liner material, check with the supplier of the liner to ensure that baffles can resist the forces and impact loads that are expected and provide the desired service life.

Installation Methods and Constructibility

HDPE, PVC and fiberglass liners are installed by slip lining or segmental construction methods. When slip lining is the method of installation, check with the supplier of the liner to determine the maximum length that can be installed by this method. Slip lining requires joining the segments outside of the host culvert as the liner is pushed or pulled through the culvert. Liners that are pulled through require joining methods that will not allow the sections of liner to be separated as the liner is pulled. HDPE, PVC and GRP liners all offer high quality joints. If pushing the liner into the host culvert, care shall be taken not to crack or buckle the liner by pushing too hard. Corrugated and profile wall liner are vulnerable to buckling, and solid wall liner (especially PVC) is vulnerable to cracking if pushed too hard. Bells of bell-and-spigot type joints – particularly corrugated HDPE liner - can snag on corrugations, nuts or deformations in the host culvert. The Designer shall select a diameter that will facilitate installation without snagging but should also review the Contractor's proposed method of installation carefully to ensure that steps are taken to avoid snagging. Corrugated HDPE does not perform well being pulled into the culvert because of the fit of the joints. They can also pull apart in long runs due to thermal contraction.

Grouting HDPE, PVC and fiberglass liners is a critical operation. It starts by specifying lightweight cellular grout where possible. Reducing grouting pressure and buoyant forces on the liner during installation can prevent damage to the liner and shifting of the liner within the host culvert. Thin walls such as provided by corrugated and profile wall liner are particularly vulnerable to grouting pressure. Grouting is typically a contractor's responsibility, but contract documents shall require that grout ports be installed in the liner to verify that grout has achieved specific depths and that the grout pour should stop. Controlling the height of the grout lift can avoid sudden failures. The Contractor shall insist that the supplier install grout ports as indicated on the working drawings.

Segmental lining is often selected due to physical constraints related to the culvert site. Shorter segments are needed when there is a limited lay-down area at the inlet or outlet end to assemble a long length of slip liner. Shorter lay down areas can be caused by natural bends in the channel, steep profiles, and placement of water handling cofferdams close to the inlet of the culvert. Deformities or obstructions within the host culvert can also discourage installation by slip lining methods in lieu of shorter segments that can be pulled or pushed in lengths that can clear obstructions.

Weight or size may also drive the need to install the liner using the segmental method. This may affect transportation, access to deliver the liner to the end of the culvert, or even the ability to move the liner into position within the host culvert. Heavier liner such as fiberglass can be split into upper and lower halves to reduce the weight and size. Also, shorter lengths can be fabricated to reduce weight.

The tunnel lining method is typically used for steel and aluminum liners, but fiberglass has been used on rare occasions by some contractors and can be customized for this purpose.

Environmental

HDPE, PVC and fiberglass liners do not typically provide an ideal environment for fish and aquatic life. When it is desirable to establish a more natural stream bottom, one of these materials may be considered as a liner if placement of baffles and sills in the invert can be designed to encourage natural sedimentation. If high velocities during less frequent storm events could move large rocks through the lined culvert, baffles (and the liner itself) could be damaged. Designers shall consider this possibility before specifying baffles with HDPE and PVC. Fiberglass liner is solid and can accommodate stronger connections for baffles.

It may be necessary to import natural material into the culvert to establish a natural bottom. This should only be specified when the span and rise of the culvert will accommodate delivery and placement of the natural stream bed material within the lined culvert. Other features may need to be constructed to facilitate the natural sedimentation. Placement of boulders upstream of the culvert, or a series of rock weirs may need to be constructed to slow velocities enough to encourage deposition of material within the culvert.

Disruption of the environment to install the liner is another consideration. Access roads to transport large liners requires removal of trees and vegetation and upon completion, restoration of the environment. HDPE liners are particularly lightweight and can easily be delivered to the end of culverts with limited access.

The need for significant cofferdams for water handling can cause disruption over a large footprint of the stream bed and can limit the available room for setting a liner before it is moved into the host culvert. Solid wall products such as PVC and fiberglass can more easily be installed in a wet environment because due to their weight, they are less vulnerable to being moved by shallow depths of flowing water. This can allow for a smaller cofferdam or none at all.

Structural

In general, the most structurally vulnerable culvert liner is HDPE. The wall of the liner is not stiff enough to support the loads by itself – it must rely on the stiffness of a well-compacted soil envelope around the host culvert to resist the loads. To transfer loads to the structural liner, grout is placed between the host culvert and the liner. Although the liner may develop additional capacity through composite action with the grout and host culvert, the liner shall be designed as if in direct burial. For design of HDPE, PVC and fiberglass liners with height of cover less than 8’, assume that the existing soil envelope is silty sand compacted to 85% unless specific information is available to use for design. HDPE is generally not suitable for use under shallow fills (< 3') with heavy, concentrated live loads above them. Although the **BDS** states that the minimum height of cover is 12", Table 30.6-1 of the Construction Specification requires at least 36" of cover for culverts with a span of 3.5' or more to distribute loads from construction vehicles. In Connecticut, these construction vehicles are Legal Vehicles and can move freely along State roads. Before specifying a HDPE culvert liner under a roadway, check the height of cover to see if it meets the minimum.

When designing thermoplastic liners with Cover Height of 8' or greater, Table 1 of Appendix B may be used to select a constrained soil modulus. Unless actual soil backfill information is available, assume Class III soil and 95% compaction. 95% compaction assumes that the soil over and beside the culvert is well-compacted after many years of service. If it is suspected that there may be voids and/or active settlement in the backfill, then assume 85% compaction.

HDPE liners of the profile wall type offer a higher quality material than corrugated HDPE and have a more robust profile to support loads. Wall thicknesses of corrugated HDPE liner are thin and may buckle when loaded. Calculations show that corrugated HDPE liners generally do not perform well under the vehicular live loads they are expected to support in Connecticut. Heavy, concentrated loads over shallow fill shall be avoided. Corrugated HDPE may be suitable in installations that do not carry live load, but care shall be exercised during installation to ensure that construction loads do not fail the liner.

Under sustained loading, material properties change for thermoplastic and thermosetting resins. The stiffness of the plastic can drop to 20% to 25% of its initial value and yield strength can drop to less than half. With such reduced longterm properties, loading from high earth fills will cause continued longterm deflection of the structure. **BDS** limits deflection to 5% of the span. Limiting deflection can reduce stress cracking. Designers shall check deflection under longterm properties. Although thermoplastic liner is quite flexible, large deflection of the liner wall is discouraged by the Design Specification. With reduced longterm stiffness, the thin wall is also prone to local or global buckling failure. Section loss due to abrasion can also weaken the wall of HDPE liner over time. Even though the abrasion rate is low and there is no corrosion, wall buckling due to a thinner wall over time caused by abrasion is a possible failure mechanism. The wall is substantially braced by the material in the annular space between host culvert and liner. However, the wall can still buckle inward where it is not supported by grout.

Thermoplastic and fiberglass liners rely heavily on the soil envelope around the host culvert to provide adequate stiffness for resisting applied loads, since the plastic wall itself typically lacks adequate stiffness -especially for corrugated HDPE liner. Soil stiffness depends on the type of soil and the level of compaction. Soil envelopes can be easily disturbed by voids, settlement or adjacent excavation for other drainage structures or utility installations. Disturbance due to excavation may be caused by third parties without any oversight by **CTDOT**. Restoration of the soil envelope by the third party may not be at the desired level of compaction. For this reason, and because in-situ soils are often used to backfill drainage structures, **CTDOT** recommends that the soil envelope be evaluated as silty sand with 85% compaction. In addition, the water table should be assumed to be 3' above the top of the culvert for design purposes. These design assumptions greatly reduce the capacity of thermoplastic and thermosetting resin culvert liners compared to designs prepared for ideal soil and compaction conditions but must be adhered to when designing thermoplastic and thermosetting resin materials for **CTDOT** culverts.

Of the liners in the thermoplastic and thermosetting resin category, fiberglass is the most versatile. This solid-wall liner can be fabricated to any shape, size, and thickness. The capacity can be increased by specifying a thicker wall, and a greater interior liner thickness can be specified to resist anticipated abrasion over the design service life. Thermosetting resin properties change over time under sustained loads, so designers are reminded to design the thickness using longterm properties. Large spans/rises can be fabricated as upper and lower halves and in shorter segments that can be transported and handled more easily. The segments are assembled in the field to create the full shape of the liner. Fiberglass liner can be custom made to tight tolerances to fit any culvert. When special sizes, shapes or custom fit aren't as critical, HDPE or PVC standard sizes and shapes should be considered as structural options to reduce weight and cost.

Steel-Reinforced Thermoplastic liner is technically categorized as a metal culvert liner. It offers greater stiffness than unreinforced thermoplastic liners, while providing a smooth, corrosion-resistant interior. Its light weight and long lengths allow it to be installed quickly. Due to a very thin plastic wall, steel-reinforced thermoplastic liner is recommended for Abrasion Levels 1 through 3 only. High performance bell-and-spigot or welded joints are available if desired to provide silt-tight or water-resistant joints.

B. METAL

i. STEEL-REINFORCED POLYETHYLENE (SRPE)

Availability

Table VI.2 - SRPE LINER AVAILABILITY

WALL TYPE	INTERIOR	MAX. SPAN	WALL THICKNESS	AVAILABLE STIFFNESS*
Ribbed	Smooth inner wall	120” Nominal I.D.	Varies by diameter See AASHTO M335	NA

*SRPE is classified as a metal pipe by AASHTO because the steel ribs provide the primary structural capacity of the pipe while the HDPE inner wall is the conduit for water. Consult the manufacturer for physical properties of the pipe for design.

SRPE is available in standard diameters. Wall thickness cannot be customized. The smooth inner wall is reinforced by steel ribs that are encapsulated in HDPE and are integral with the inner liner. The profile is an open profile (there is no exterior wall). SRPE has a shallower wall profile than profile wall HDPE because its stiffness comes from the steel reinforcing ribs.

Abrasion and Corrosion Resistance

Abrasion Levels 1 to 3:

SRPE shall be specified for these levels only.

Abrasion Levels 4 to 6:

SRPE shall not be specified. The thin inner wall will abrade quickly at higher abrasion levels.

SRPE, like HDPE liner, does not corrode. Steel ribs are encased in HDPE so unless the encasement is damaged, the ribs are protected from corrosion. Encasement could be damaged by abrasion. HDPE can be repaired in the field, so all is not lost if localized damage occurs.

Hydraulics

SRPE has a very smooth interior and a low profile. Because of the shallow wall thickness and available sizes, SRPE has an advantage over HDPE to maximize the hydraulic capacity of the lined culvert.

Installation Methods and Constructibility

SRPE is lightweight, which is desirable for shipping and handling the liner. Pushing and pulling the liner in place takes less effort than heavier liners. SRPE can be damaged if forced into the host culvert. Special devices should be attached between the ribs at the bottom to prevent damage to the liner and support it as it is pulled along the skids into the host culvert. Snagging is one of the primary causes of damage during installation. Another source of damage is the grouting method. Because the lightweight pipe has such thin components, they are subject to buckling when loaded improperly. The pressure from grout can be too large for the thin walls if the density of the grout is high or if the grout lift itself is too high. Designers shall specify lightweight cellular grout where possible to minimize pressure. Grout ports shall be installed in the liner at appropriate heights to properly grout the annular space in lifts. Requests to not install grout ports, if proposed by the supplier during construction, should not be accepted.

Grouting of larger diameter SRPE shall take into account that a few of these sizes do not offer bell and spigot type joints. Instead, internal bands are used at the joints once the liner is in position within the host culvert.

Environmental

SRPE does not provide an ideal environment for fish and aquatic life. If a more natural stream bottom is desired, baffles may be installed in SRPE liner with inside diameter 48” or greater, which provides sufficient access to enter the pipe and install the baffles. SRPE liner can be ordered in lengths as short as 14’, so if baffles can be spaced at 14’ intervals, then they may be installed in smaller diameters as well. If high velocities during less frequent storm events could move large rocks through the lined culvert, baffles (and the liner itself) could be damaged. Designers shall consider this possibility before designing baffles with SRPE. Baffles (and the inner wall) can be repaired in the field.

Structural

SRPE behaves like a metal pipe due to the stiffness of the steel ribs encased in HDPE. Longterm properties need not be checked, since steel, not HDPE, is being subjected to the sustained load. Even though SRPE is stiffer than its unreinforced HDPE counterpart, the thin walls make the structure vulnerable to buckling failure. Designers are reminded to check this mode of failure to ensure that SRPE can adequately support all loads. SRPE relies less on the soil envelope for its capacity than unreinforced HDPE.

ii. ALUMINUM

Availability

Table VI.3 - ALUMINUM LINER AVAILABILITY

WALL TYPE	INTERIOR	MAX. SPAN	WALL THICKNESS	AVAILABLE STIFFNESS
Corrugated	Smooth inner wall available	120” I.D. 144” Span for Pipe Arch	10 gauge	NA
Structural Plate	Annular ribs	316” I.D.	0.25”	NA

Tunnel Liner 2-Flange	Corrugated	As-designed	0.239”	NA
Tunnel Liner 4-Flange	Corrugated	As-designed	0.375”	NA

The most typical aluminum liners are spiral rib smooth interior, corrugated or profile wall. The strongest shape fabricated from aluminum is tunnel liner plate, which has corrugations. Aluminum is available in standard sizes for round pipes and pipe arches. Wall thickness can be selected by the Designer. A smooth inner wall is available by ordering Type IR aluminum pipe. Aluminum tunnel liner has a shallow wall profile that maximizes the hydraulic opening however, tunnel liner has a corrugated shape that will tend to slow the flow compared to a smooth liner.

Abrasion and Corrosion Resistance

Abrasion Levels 1 to 3:

> Aluminum is a suitable material. Abrasion is negligible.

Abrasion Levels 4 to 6:

> Aluminum can be considered in thicknesses up to ¼” for structural plate but will likely not give a suitable target design service life.

Moderate abrasion can remove the oxide coating that forms on the surface of aluminum liners. Exposing aluminum below the oxide layer will allow additional oxidation of the aluminum. Moderate and highly abrasive environments can also remove base metal, resulting in longterm reduction in section properties and reduced capacity of the liner.

Specifying extra thickness of aluminum can compensate for some loss of material, but for aggressive environments, aluminum may not be the best choice for a liner. Aluminum abrades quickly at Abrasion Level 4 and above. Designers may specify aluminum above Level 3 but must increase the thickness to provide the desired service life. When aluminum liner is used within a range of pH from 5.5 to 8.5 and Resistivity is above 1,500 ohm-cm aluminum forms an oxide coating that is resistant to corrosion. Outside of this pH range the protective oxide layer dissolves and stops protecting the aluminum. Chlorides above 100 ppm can provide a favorable environment for corrosion. For this reason, it is recommended to refrain from specifying aluminum liner in brackish water or marine environment.

Aluminum bolts could be specified for structural plate aluminum, but the strength of a galvanized bolt is usually preferred. The resistivities of these materials are close enough to not cause an adverse electrochemical reaction.

Hydraulics

Aluminum liner is offered with a smooth interior and a low profile. This type of liner can be one of the most hydraulically efficient liners when a strong liner is needed to maximize flow in existing culverts that were sized close to the desired hydraulic capacity. A smooth surface and maximized hydraulic area make this product desirable when deeper profile liners cannot deliver hydraulic results. Corrugated aluminum liner can be used to slow the water velocity before it exits the culvert.

Installation Methods and Constructibility

Aluminum is lightweight, which is desirable for shipping and handling the liner. Pushing and pulling the liner in place takes less effort than heavier liners. Aluminum can be damaged if care is not taken to restrict the height of grout lifts. The pressure from grout can be too large for the thin walls if the density of the grout is high or if the grout lift itself is too high. Bracing may be required inside the liner until grout has been placed. Designers shall specify lightweight cellular grout where possible to minimize pressure. Grout ports shall be installed in the liner at appropriate heights to properly grout the annular space in lifts.

Grout can react with the aluminum liner when placed in contact with the aluminum. This reaction is mild and lasts only through the hydration process until the cement cures. The miniscule section loss during this reaction is not of any structural consequence. Attempts to coat the outside wall of the aluminum liner with zinc chromate paint or bituminous coating have not worked well. The coatings were severely damaged during handling and installation and could not be touched up when the liner was in its final position. Touch up is time consuming and costly. Coating aluminum liner is not recommended.

Environmental

Aluminum does not provide an ideal environment for fish and aquatic life. If a more natural stream bottom is desired, baffles may be installed in aluminum liner to slow the flow and cause natural deposition of streambed material. Baffles would not typically be specified within a smooth-wall aluminum liner because the smooth wall is specified to increase flow while baffles are intended to slow the flow. Do not specify baffles in a liner with inside diameter less than 48” because access to enter the pipe and install the baffles would be difficult.

Structural

Aluminum liner is much stiffer than HDPE liners and its longterm material properties remain the same as its initial properties under sustained loads. While not as strong as steel, aluminum can support most heights of fill and loadings to which a culvert will be subjected. For very shallow fill, aluminum liner may not be able to support large, concentrated wheel loads. For height of cover greater than approximately 30 feet, aluminum liner may not be capable of supporting the soil prism. When aluminum culverts are installed in direct burial, external ribs can be used to increase the capacity of the culvert. However, when aluminum culvert liner cannot support the loads, it is generally impractical to attach external ribs without reducing the internal diameter excessively so the ribs can fit into the culvert. A stronger material such as steel may need to be considered.

iii. STEEL

Availability

Table VI.4 - STEEL LINER AVAILABILITY

WALL TYPE	INTERIOR	MAX. SPAN	WALL THICKNESS	AVAILABLE STIFFNESS
Solid Wall	Smooth	As specified	1.25” Max. (Up to 2” Possible)	NA
Corrugated Galvanized	Smooth inner wall available	144” Round or Pipe Arch	8 gauge	NA

Corrugated Aluminized	Smooth inner wall available	144” Round or Pipe Arch	10 gauge	NA
Corrugated Polymer Coated	Smooth inner wall not available	144” Round or Pipe Arch	10 gauge	NA
Structural Plate	Annular ribs	316” I.D.	0.25”	NA
Tunnel Liner 2-Flange	Corrugated	As specified	0.239”	NA
Tunnel Liner 4-Flange	Corrugated	As specified	0.375”	NA

The most typical steel liners are spiral rib smooth interior, corrugated or profile wall. Although these are fabricated in standard sizes, special orders are possible with steel structural plate liner. Wall thickness is specified by the Designer. Solid wall steel liners can be custom designed and fabricated to fit any size or shape culvert and the Designer chooses the thickness. One of the strongest shapes fabricated from steel is tunnel liner plate, which has corrugations. Steel tunnel liner has a shallow wall profile that maximizes the hydraulic opening however, tunnel liner has a corrugated shape that will tend to slow the flow compared to a smooth liner. Steel tunnel liner corrugations can be filled with mortar to create a smooth interior surface. The mortar is anchored to the tunnel liner with studs or mesh secured by the bolts. The mortar is usually built up to cover the inside crests of the tunnel liner corrugations.

Abrasion and Corrosion Resistance

Abrasion Levels 1 to 3:

> Steel is a suitable material. Abrasion is negligible.

Abrasion Levels 4 to 6:

> Steel (coated or uncoated) can be considered. Additional thickness shall be considered to extend the design target service life.

Moderate abrasion can remove the oxide coating that forms on the surface of steel liners. Exposing steel below the oxide layer will allow additional oxidation of the steel. In moderate and highly abrasive environments the oxide coating can be removed and re-formed repeatedly. This is known as the abrasion-corrosion cycle and can lead to accelerated section loss. Abrasive environments also remove base metal, resulting in longterm reduction in section properties and reduced capacity of the liner.

Solid wall steel liners are typically designed with extra thickness to compensate for corrosion and abrasion and provide the desired service life. Some solid wall steel liners use a steel alloy that corrodes at about half the rate of conventional steel. Specifying extra thickness of steel can compensate for some loss of material, but coatings can also extend the service life of a steel liner. Galvanized steel has historically been used to extend the life of culverts. Aluminized steel has also been used and has been shown to provide a longer service life than galvanized steel when compared to one another in a less aggressive environment. Polymer coated steel combines a tough coating with the protection of galvanized steel below. Polymer-coated steel has been very effective in moderate to low abrasion environments, but the coating will abrade quickly in more aggressive environments. Polymer coating is available on steel up to 10-gauge thickness. Polymer coating can be damaged during shipping, handling and installation and must be repaired before installation of the liner.

Designers shall assume that polymeric coatings add twenty years of service life to a steel liner for Abrasion Levels 1 through 3, ten years for Level 4, five years for Level 5 and 0 years for Level 6. In some case histories, polymer coating has provided more than thirty years of corrosion protection to the steel liner at lower abrasion levels.

Asphalt coatings are also available but are considered to add only up to eight years to the service life of the steel. Asphalt coatings are unfriendly to the environment and are flammable. Thick concrete invert liners can offer the most protection against abrasion, but the concrete thickness raises the invert significantly and reduces the hydraulic area of the culvert. Thinner mortar coatings may be used for protection if anchored well to the liner. Mortar coating that fills the corrugations and creates a smooth interior also reduces the Mannings coefficient and can improve flow through the lined culvert. Greater velocity of flow can increase abrasion, however.

Steel liners perform better when pH is > 6.0 with resistivity above 2,000 ohm-cm.

Chlorides above 100 ppm can provide a favorable environment for corrosion. For this reason, it is recommended to refrain from specifying steel liner in brackish water or a marine environment. Sulfates at 100 ppm or more may cause a drop in pH that can accelerate corrosion in steel liners. Sulfates can adversely interact with chlorides to cause corrosion in metal. Sulfates lower the threshold at which chlorides cause corrosion. Aluminized liners may be considered for use in Abrasion Levels 1 and 2 when sulfate levels exceed 100, but only when pH is within allowable levels. When resistivity is above 1,500 ohm-cm, the recommended range of pH is between 5.0 and 7.2. When resistivity drops to 1,000 ohm-cm, the recommended range of pH is from 7.2 to 9.0. Polymer coated steel liners have performed well in Abrasion Levels 1, 2 and 3 in salty environments.

Hydraulics

Steel liner is offered with a smooth interior and a low profile. This type of liner can be one of the most hydraulically efficient liners when a strong liner is needed to maximize flow in existing culverts that were sized close to the desired hydraulic capacity. A smooth surface and maximized hydraulic area make this product desirable when deeper profile liners cannot deliver hydraulic results. Corrugated steel liner can be used to slow the water velocity before it exits the culvert. Baffles can also be installed in steel liners to slow the velocity.

Installation Methods and Constructibility

Steel is heavier than aluminum liners but offers much greater stiffness, which is desirable for shipping and handling the liner. Pushing and pulling the liner in place is less likely to cause damage to steel liner. Despite greater stiffness, steel can be damaged if care is not taken to restrict the height of grout lifts. The pressure from grout can be too large for the thin walls if the density of the grout is high or if the grout lift itself is too high. Bracing may be required inside the liner until grout has been placed. Designers shall specify lightweight cellular grout where possible to minimize pressure. Grout ports shall be installed in the liner at appropriate heights to properly grout the annular space in lifts. The liner must also be secured against buoyancy, so it is not displaced during grouting.

Environmental

Steel does not provide an ideal environment for fish and aquatic life. If a more natural stream bottom is desired, baffles may be installed in the steel liner to slow the flow and cause natural deposition of streambed material. Baffles would not typically be specified within a smooth-wall steel liner because the smooth wall is specified to increase flow while baffles are intended to slow the flow. Do not specify baffles in a liner with inside diameter less than 48" because access to enter the pipe and install the baffles would be difficult.

Structural

Steel is the strongest of liners and its long-term material properties remain the same as its initial properties under sustained loads. Steel liners can support most heights of fill and loadings to which a culvert will be subjected. For very shallow fill, steel liners subjected to large, concentrated wheel loads could be damaged. Designers shall consider this type of failure when checking the design.

Long-term strength can be achieved by designing the thickness of steel liners for losses due to abrasion and corrosion and still meet the desired service life. Uncoated, solid wall alloy steel liner plate will provide a strong liner that minimizes the liner thickness and maximizes the hydraulic area available. Some steel alloys have double the resistance to corrosion as conventional structural steel. These solid wall steel liners can be customized to a particular culvert size and shape. However, they are heavy and require field welding.

C. CONCRETE

Availability

Concrete is not typically used to line the entire circumference of a culvert. It is sometimes used to restore a culvert bottom that has become deteriorated, perforated or is completely missing. Concrete inverts that make the culvert hydraulically inadequate or have a negative impact on stream channel habitat may still be able to be specified if a supplemental culvert is installed that does provide these characteristics.

A dense, low-permeability concrete is desirable. Strength of the concrete is determined by the Designer based on structural capacity needed to rehabilitate the host culvert. A 4,400 psi concrete is the preferred minimum strength with a maximum aggregate size of 3/8" to facilitate pumping.

Abrasion and Corrosion Resistance

Abrasion Levels 1 to 3:

> Concrete is a suitable material but is likely not the first choice for a liner. Abrasion is negligible.

Abrasion Levels 4 to 6:

> Concrete has the fastest rate of abrasion of all materials but can provide a sacrificial invert to extend the life of the host culvert or protect the invert of a new liner.

Concrete is adversely affected by a combination of pH level below 7.0 and sulfate concentration above 1,500 ppm. When resistivity of the water is 1,000 ohm-cm or greater, sulfate is not a concern. The Designer shall specify a low-permeability concrete with a water/cementitious material ratio of 0.40 or lower. If sulfate concentration is above 2,000 ppm, Type II or V Portland cement shall be specified.

Hydraulics

A concrete invert is smooth, but the thickness typically reduces the capacity of the culvert. An improved inlet can increase the flow of water into the culvert when the culvert is inlet controlled. A hydraulic analysis is needed to determine if the flow in the lined culvert is adequate, a supplemental culvert is needed, or the culvert needs replacement. Often, the water velocity will increase due to the smooth concrete liner and a reduced hydraulic area. Sills or baffles may be designed to slow water velocity to address scour concerns at the outlet.

Installation Methods and Constructibility

If a concrete invert is selected, water shall be handled in a manner that allows concrete to be placed in the dry with sufficient time to gain the specified strength with proper curing in place. This typically requires placement of a temporary water handling cofferdam and pumping or diverting flow while the invert concrete is placed. Depending on the length of the host culvert, concrete may need to be pumped from one or both ends due to pumping distance limitations.

Voids behind the host culvert and at the invert must be filled to allow for the proper transfer of loads to the rehabilitated culvert.

Concrete can be cast in place, troweled on, or applied by low-pressure spray to restore a culvert invert or provide a smooth surface inside the culvert. Concrete can be mixed near the ends of the culvert or from the roadway above.

Environmental

Concrete inverts can be designed with low-flow channels and baffles or sills to encourage natural streambed material to be deposited to accommodate fish habitat and passage. Because of the thickness of the liner, creating a natural bottom can create an undesirable invert elevation and reduce hydraulic capacity. Close coordination between Hydraulic Designers and Environmental Planning is needed before proposing a concrete invert.

Structural

For concrete to restore the structural capacity of the culvert and protect the culvert against further deterioration, the concrete slab must be securely anchored to the culvert wall. If the as-built culvert did not have adequate capacity originally, the Designer may want to consider another option to rehabilitate the culvert. The slab shall be of sufficient thickness and reinforced to resist bending and axial forces imposed on it by the culvert walls as load is transferred from the walls to the concrete slab. Sufficient overlap with and anchorage to the host culvert wall will reduce prying forces at the edges of the concrete slab, allowing the concrete slab to act as a continuous, integral part of the culvert. Designers shall specify sufficient concrete cover above the reinforcing to allow for loss of material due to abrasion and erosion. Reinforcing should be galvanized, stainless or GFRP to avoid corrosion, minimize spalling and extend the life of the invert.

VII. NON-STRUCTURAL CULVERT LINERS

Most culvert liners are being proposed because a culvert condition has been identified as deficient or structurally incapable of supporting all required loads. However, not all culvert liners must be selected to provide full structural capacity. A culvert liner can be proposed to proactively protect the host culvert against corrosion, abrasion, and environmental attack to extend the service life of the culvert. It can also be used in conjunction with a proposed structural liner to provide a lower Mannings coefficient. In such an application, the non-structural liner must be selected to provide the desired service life. The three non-structural liners noted below will likely provide the desired service life when abrasion is classified as low. However, these liners may not provide the desired service life if installed improperly or under moderately abrasive conditions. A nonstructural liner can also be specified as a temporary measure until a structural solution is designed and implemented. For example, perforations in a culvert invert can allow bedding or backfill to be drawn into the culvert, creating voids, which can lead to structural failure of the culvert. Grouting the voids and installing a liner can prevent voids from developing or progressing. The non-structural liner must still be able to provide the service life anticipated for the temporary installation. Tools to assist in selecting non-structural liners are available in Appendices A and B.

Some examples of non-structural liners are:

A. Spiral wound HDPE or PVC

B. Centrifugally cast mortar or geopolymer

C. Cured in place resin.

The reason these are listed as "non-structural" is **BDS** does not offer a method of calculating the capacity of these liners. To be considered a structural culvert liner, a culvert that qualifies to have a bridge number must also be load rated by the Load and Resistance Factor Rating (LRFR) method. Structural liners for culverts that are too small to be assigned a bridge number shall still be designed in accordance with AASHTO. Allowable Stress or Load Factor Design methodologies are not acceptable for structural liners on the National Bridge Inventory whose load rating will reported to FHWA.

A. SPIRAL WOUND

A continuous strip of HDPE or PVC extruded with a ribbed profile and wound mechanically with a machine that joins the edges to form a cross-sectionally closed shape. The liner can be made to conform to the shape of the host culvert. They can be expanded against the host culvert or fabricated as a fixed size and grouted between the liner and host culvert. Some profiles offer steel reinforcement embedded within the plastic.

Spiral wound liner can be installed while the host culvert carries water - up to 25 – 30% flow.

B. CENTRIFUGALLY CAST MORTAR OR GEOPOLYMER

Cementitious mortar or geopolymer liner applied by an electric or air powered rotating head. The host culvert must be clean and dry. All voids must be filled, and invert repaired to allow the machine to ride on skids through the culvert. Cementitious or geopolymer material is sprayed onto the culvert interior wall. Material is typically high strength and reinforced with fibers for tensile strength and to control cracking. When spraying over corrugations, hand finishing is often required if a smooth finish is desired. A number of states have experienced problems with cracking, leaking and debonding of the liner. This liner is difficult to install on culverts with discontinuities or sharp deformities.

C. CURED IN PLACE RESIN

Cured In Place Pipe (CIPP) is a term coined for a flexible tube material such as felt reinforced with fiberglass and impregnated with polyester resin. The tube is pulled through the culvert and expanded to tightly fit the interior of the host culvert. The resin is cured to a solid using heated water, steam, or ultraviolet light.

VIII. SELECTION OF CULVERT LINER WITH CONSIDERATION FOR METHOD OF INSTALLATION

There are three typical methods of structurally lining culverts: slip lining, segmental lining and tunnel lining. Each of these methods has limitations that will be discussed in the following paragraphs. All methods of structurally rehabilitating an existing culvert will not necessarily result in a hydraulically acceptable liner. Sometimes the strategy must be to address the structural needs of the host culvert and install a supplemental pipe or culvert to accommodate hydraulic needs.

A. SLIP LINING

Slip lining is typically the fastest and most economical method of culvert rehabilitation. Slip lining involves the insertion of a prefabricated structural liner that is smaller in cross section than the host culvert into the inlet or outlet end of the culvert. The liner is either prefabricated or assembled outside of the host culvert and pushed or pulled into the culvert. Longer culverts may require that a portion of liner be assembled, then moved partially into the host culvert while additional lengths are attached. This process is repeated until the liner is inserted completely into the host culvert. This technique is also used when there is insufficient length of channel at the inlet or outlet in which to stage the entire assembled liner before moving it into the culvert. After insertion of the liner, grout is pumped into the annular space between the host culvert and the liner.

Designers should look for the following conditions to be met for selecting a slip liner:

1. The host culvert is structurally safe to enter
2. Minimal obstructions exist within the host culvert to allow the free movement of a liner through the culvert
3. The dimensions and elevations of the host culvert are measured and documented
4. The Designer has verified that based on the documented measurements of the host culvert, the liner cross section will physically fit as it is pushed or pulled through the host culvert.
5. The length of culvert liner being pushed into the host culvert does not exceed the manufacturer’s recommended push length for that type of liner.

B. SEGMENTAL LINING

Segmental lining involves the insertion of culvert liner segments into a host culvert to form a continuous liner. To form a continuous liner, the segments must be connected by gaskets, bands, bell and spigot joints, welding, fusion, or other acceptable joints. Grout bulkheads can also join liner segments of different sizes or shapes. Segmental lining is often specified where:

1. Sharp bends or misalignments occur in the host culvert that would prevent a slip liner from being inserted
2. Significant damage or isolated deformations of the host culvert exist, preventing a continuous slip liner to be inserted.
3. The channel alignment at the inlet or outlet has insufficient length to allow a slip liner to be set in the channel outside of the culvert.
4. Insufficient access is available to the culvert ends to deliver a long slip liner to the channel bed.

5. The size and/or weight of the liner is not conducive to transporting, handling or inserting a slip liner.
6. The push length for slip lining is less than the length of culvert to be lined.

To avoid using tunneling methods to remove obstructions, a cast-in-place concrete or grout transition segment can be formed and cast between two liner segments. This cast-in-place transition can facilitate the smooth flow of water from one segment size and shape to another. If necessary, different size segments can be inserted from opposite ends of the culvert to avoid the obstruction. However, if multiple obstructions exist, removal of obstructions may be necessary to line the full length of culvert. When the extent of removal of obstructions is determined to be too great to use the segmental lining technique, consider using a tunnel lining technique. Tunnel lining may also be partially used to eliminate obstructions and open the culvert for slip lining or segmental lining methods.

C. TUNNEL LINING

Tunnel liner plate is corrugated structural plate with right angle flanges for butt-type bolted seams on two sides or four edges of each rectangular plate. Four-flange plates create circumferential and longitudinal seams between plates. Two-flange plate uses bolted lap splices along the non-flanged edges to assemble rings. The flanged edges of the rings are butted together and bolted to join the rings to each another to extend the liner. Butted flange splices are weaker than lapped splices, so seam strength must be considered in the selection of liner plate. Tunnel lining ring segments are typically short in length – approximately 18" wide rings. The short segment length allows some flexibility to rotate the rings to navigate the liner around obstructions within the culvert to avoid removal of small obstructions in the host culvert. Removal may also be avoided by assembling a smaller ring at the obstruction if the reduction in hydraulic cross section can be tolerated. It is recommended to grout the smaller ring with a higher strength grout since the grout will be exposed to the flow of water. In the small separation between liner rings, grout from the annular space can be formed to create a smooth transition from the larger to the smaller liner ring if needed to improve the hydraulic flow. The downstream end of the smaller ring should not require a grout transition.

Tunnel lining is to be considered by the Designer when culvert failure is imminent, or other lining methods may cause failure of the culvert. Here are some of the conditions to look for:

- When any portion of the culvert cross section has changed shape excessively due to section loss and/or bending or buckling of the wall.
- When a bolted seam fails and cusping of the plates causes a sharp discontinuity in the culvert wall. Sometimes cracks form in the culvert plate from the bolt hole to the edge of the plate signaling impending failure before cusping of adjacent plates occurs.
- When large perforations or heavy section loss can be seen in the bottom accompanied by curling of the adjacent plates as the culvert walls are thrust downward into the bedding.
- When voids form in the bedding and/or backfill. Voids themselves are not warrants for tunnel lining, but when removal of portions of the host culvert are required, tunnel liner may offer a safe solution. Voids can be seen through perforations or detected by tapping the culvert wall and listening for a hollow sound. Voids indicate loss of support of the culvert that can lead to bending or buckling of the culvert wall.

Rings of tunnel liner plate are assembled progressively from the inlet or outlet end. Each ring of liner is bolted to the previous ring and adds to the support of the host culvert. Bulkheads may be constructed at desired intervals to grout the annular space between the tunnel liner and the host culvert for greater safety.

Workers can install additional tunnel liner rings from inside the tunnel liner that has just been installed. Tunnel liner plate is considerably stronger than other types of liner and even when not yet grouted provides a safe place from which to construct the next ring. Grouting the tunnel liner in short segments can afford much greater safety to workers inside the liner.

Tunnel liner rings can be assembled while temporary water handling pipes conduct water through the culvert. Removal of material from the host pipe and excavation of backfill material should proceed in short segments within the culvert to allow tunnel liner rings to be assembled within the removal limits to minimize exposure time of the excavated areas. Depending on the extent of removal of the host culvert wall and soil before a segment of liner is installed, union workers with tunneling experience may be needed. The culvert span and rise, condition of culvert, backfill soil type and presence of voids, and height of water table are pieces of information that should be documented before attempting to determine if the work will be classified as “tunneling.” Contract documents should identify tunneling needs when applicable. Designers should coordinate with Construction to determine the need for union tunnel workers. Typically, tunnel lining segments are short enough to not require special tunneling methods or workers.

Tunnel liner is available in galvanized steel or aluminum. Aluminum is more durable than galvanized steel – especially in low to moderate abrasion level environments. Neither material is durable under heavy abrasion from fast moving water carrying angular stone. Tunnel liner can be protected by applying a mortar lining to the invert along the portion of the cross section where abrasive action is anticipated. Mortar lining shall be anchored securely to the tunnel liner plate. When mortar lining is used, water velocity may increase, and erosion measures may need to be incorporated to protect the channel at the outlet of the culvert. Mortar lining will reduce the hydraulic opening, so a hydraulic analysis shall be performed to confirm that a mortar lining within a tunnel liner is an acceptable option.

IX. CONSTRUCTIBILITY

Constructibility plays an important role in selecting a culvert liner and lining method.

Access to the inlet and outlet ends of the culvert is needed:

- For trucks and equipment access
- For setting up cranes
- To deliver material
- For storing material
- For setting up water handling
- For removal of debris within the culvert
- For placement of the liner
- For grouting of the liner

The size and weight of liner segments can influence the selection of an alternate if site access makes it difficult to deliver and install the liner. Some liners can be fabricated in manageable segments, while others cannot. Delivering individual tunnel liner plates into the culvert for example could overcome access difficulties for delivery of larger liners but assembling plates inside of the culvert will take much longer to install the liner.

Inlet and outlet channel geometry can also affect the selection of a liner and method of lining. For example, a slip liner might be the most economical liner method, but if the channel turns a sharp bend just upstream or downstream of the culvert, there may be insufficient room to place a slip liner in the channel before moving it into the culvert. In another example, the profile grade of the channel may change sharply near the inlet or outlet, preventing a slip liner from being placed on a suitable alignment with the culvert invert. If site conditions require placing a water handling cofferdam near the inlet of the culvert, there may be insufficient room to place a liner before slipping it into the culvert. Any of these constraints could steer a designer toward selecting a culvert liner that can be installed by the segmental or tunnel lining methods instead of slip lining. This means more joints in the liner, more cost and more time to install the liner.

Handling water through the culvert during installation of the liner may be an important site requirement if other options are not available. The Designer shall indicate in the RSR how water is anticipated to be handled with each alternate presented. Consideration shall be given to the location and height of cofferdam and storm frequency at which the cofferdam would overtop. Channels with a tendency to flash flood may require special consideration in selection of liner and installation method to minimize the risk of injury or damage from a flash flood.

Another constructibility concern is insufficient survey of the host culvert. When the condition of host culvert does not allow entry for survey, a greater risk exists that the liner will not fit. In such cases tunnel lining methods must be selected or a replacement structure considered. Tunnel lining methods are slower and will affect the length of the construction schedule. There are few liners that can be specified with tunnel lining and the installation of these are slower than slip lining and segmental lining. Slower advancement of the liner comes with greater risk of storm events causing overtopping of the cofferdam and delays to the work. Temporary diversion of the water (if required) will cost more, take longer and may cause undesirable impacts to regulated areas and natural habitats.

X. SERVICE LIFE

A report entitled, “Design Guide for Bridges for Service Life” (the Guide) was published in 2014 by the Transportation Research Board (TRB) at the Transportation Research Board website (http://www.trb.org/Design/Blurbs/168760.asp0x). This report provides information and guidance and defines procedures to systematically approach service life and durability for both new and existing bridges. The framework for this report is the SHRP2 Project R19A, “Bridges for Service Life Beyond 100 Years: Innovative Systems, Subsystems, and Components.”

The Guide defines three terms that are important to understand:

- Service Life
- Design Life
- Target Design Service Life

Service Life is defined as “The time duration during which the bridge element, component, subsystem, or system provides the desired level of performance or functionality, with any required level of repair or maintenance.”

Design Life is defined as “The period of time on which the statistical derivation of transient loads is based: 75 years for the current version of **BDS**.”

Target Design Service Life is "The time duration during which the bridge element, component, subsystem, and system is expected to provide the desired function with a specified level of maintenance established at the design or retrofit stage.

The approach to designing a new structure or a rehabilitation starts with public safety through structural adequacy. An important step in selecting a culvert liner is to eliminate options that are structurally inadequate to support the fill height and transient loads. As noted in the definition of "Design Life" above, the statistical derivation of transient loads is based on a design life of 75 years. While rehabilitating a culvert to last an additional 75 years is possible, it may not be practical or cost-effective. Steps taken to ensure a longer life may adversely affect other goals such as hydraulic capacity. There may be no available solutions that will provide such a long service life under the given conditions.

The Guide notes that "The service life of a given bridge element, component, subsystem, or system could be more than the target design service life of the bridge system." A culvert's target design service life is often defined as the time it takes for the culvert invert or wall to have its first perforation due to corrosion and abrasion. Despite the perforation, the soil envelope may still be able to distribute the loads to the culvert safely and the culvert wall may still have sufficient capacity to support the distributed loads. Why, then, should the target design service life be defined as the time it takes until the culvert first becomes perforated?

New culverts in direct burial that become perforated due to corrosion and/or abrasion expose bedding and backfill material that could be drawn into the culvert by flowing water or wash into the culvert when ground water infiltrates the culvert during low flows. Eventually, voids and sink holes will develop around and above the culvert and the structural capacity will eventually be affected by the loss of support. Unacceptable capacity will not likely occur at the time of first perforation. The time between first perforation and unacceptable structural capacity can vary widely among culverts. There is no reliable way to estimate the timeframe from first perforation to unacceptable capacity. Some culverts in Connecticut have lasted fifteen years or more after first perforation without showing signs of failing. Establishing the target design service life of a culvert in direct burial as "the number of years until the first perforation forms in the culvert invert or wall" is a reliable way to design the culvert "to provide the desired function with a specified level of maintenance established at the design or retrofit stage." If a perforation forms, an assessment shall be made on a case-by-case basis how quickly the culvert should be repaired or replaced.

Rehabilitation does not have to be reactive; it can be proactive by lining the culvert at a pre-determined age before a perforation occurs. Such actions are classified as "preservation," and can maintain culverts in a state of good repair. Preventative maintenance allows work to be performed on a schedule, which allows the cost of such work to be budgeted. The cost of preventative maintenance may be slightly higher than maintenance based on condition and needs, but it offers greater reliability and allows the owner to maintain the inventory in a state of good repair.

First perforation in a culvert liner does not expose the bedding and backfill the same way as with an unlined culvert in direct burial because the liner is surrounded by a grouted annular space. Perforation reduces section properties of the liner wall but grout in the annular space will provide some additional capacity of the liner. The contribution will depend on the strength and composition of the grout. In many cases, the host culvert is still actively contributing to the capacity through composite action with the grout. Therefore, the "time until first perforation" for a culvert liner should not necessarily define the target design service life of the culvert liner. In the design of culvert liners, Designers are directed to discount the composite action when designing the liner as a conservative design approach. There are too many site-specific variables that would make it difficult to define a generic approach to determining how much additional capacity from the host culvert and grout the Designer may assume in calculating the required capacity of the liner. Load rating engineers, however, may consider all known variables that contribute to composite action and distribution of load in assessing the capacity of a perforated culvert liner. With biennial or special inspections of the lined culvert, additional service life can be expected beyond first perforation.

The Guide emphasizes that "The end of service life for a bridge element, component, or subsystem does not necessarily signify the end of bridge system service life as long as the bridge element, component, or subsystem could be replaced or resume its function with a retrofit." The culvert itself is a subsystem that contributes to the bridge system, which includes the culvert and the compacted backfill envelope. If the backfill integrity can be maintained, the culvert is a subsystem that can be retrofitted with a liner to allow the culvert to "resume its function." Therefore, when designing a new culvert, it would be wise to anticipate the future need for a liner and design the culvert opening to accommodate a liner.

The Guide recognizes that there may be multiple solutions to enhance or increase the service life of a culvert and that not all solutions must have a 75 or 100-year service life. The Guide states that Life Cycle Cost Analysis (LCCA) can help identify an optimum solution. See "Life Cycle Cost Analysis Guidance for CTDOT Bridge Projects" under "Guides" on the "Engineering and Construction Information Resources" web page. When all options have a similar service life with no maintenance, a LCCA may not be needed to compare options. Using LCCA, solutions that require periodic rehabilitation or preservation activities can be compared to solutions that require no rehabilitation or maintenance. The documentation of assumptions in developing the service life and LCC for each solution shall include rehabilitation or preservation activities that would extend the life of the liner. The Guide indicates that actions to extend the service life may be "based on assessment of condition and need, or they can be based on a program of preventive maintenance actions planned for similar elements on a group of bridges." A culvert that has served for 40 or 50 years and can be relined may be able to last another 40 or 50 years with no maintenance, thus extending the service life of the original culvert to 80 or 100 years. Relining comes at an expense and, in some cases, may not provide the desired hydraulic capacity. Should a supplemental pipe be proposed to provide the additional capacity required, the LCCA shall include the cost of the supplemental pipe in the estimate for a more appropriate evaluation of alternates.

The benefit of performing a LCCA is that it accounts for the different service lives of the various alternates. Consider the following example of three potential solutions to address a deteriorated culvert:

- Alternate 1: Culvert Replacement
- Alternate 2: Culvert Reline with a 16-gauge, aluminum, spiral rib, smooth liner that is hydraulically acceptable
- Alternate 3: Culvert reline with a 10-gauge, corrugated aluminum liner of a smaller diameter than Alternate 2 plus a supplemental 36" diameter jacked concrete pipe.

In the alternates above, Alternate 2 meets structural and hydraulic requirements, but abrasion and corrosion will greatly reduce the service life at this site. Alternate 2 is determined to have a relatively short service life but is considerably less costly than Alternates 1 and 3. Alternate 3 also meets structural and hydraulic requirements but the service life is greatly increased because a much thicker gauge liner is selected. The corrugated liner must have a smaller internal diameter than the smooth liner to fit within the host culvert. The corrugations also make the liner rougher than the smooth liner in Alternate 2, requiring that a supplemental pipe be jacked beside the culvert to meet hydraulic capacity needs.

Alternates 2 and 3 are both valid rehabilitation solutions. LCCA provides a tool that helps compare Alternates 2 and 3 to each other and to Alternate 1 to find an optimal solution. In addition to cost, other considerations like constructability, site-related concerns and perhaps availability of funds will influence the selection of a solution. Target design service life for a rehabilitation alternate need not meet the goal of

50 or 75 years, provided the evaluation shows the solution to be optimum for that site when compared to alternates that offer a longer service life.

XI. CULVERT LINER SELECTION CHECKLIST

A. Identify all liner options that could perform structurally

B. Eliminate liner options that are undesirable based on:

1. Durability and Longevity
2. Hydraulics
3. Environmental and Regulatory Considerations
4. Method of Installation (Slip, Segmental, or Tunnel Liner)

C. Identify liner options for constructability considering the following:

1. Access
2. Site Constraints
3. Water Handling
4. Construction Schedule

D. Prepare Alternates, and Determine Service Life and Life Cycle Cost of Alternates

E. Select A Recommended Alternative

See Appendix A for lists of Advantages and Disadvantages for structural and non-structural liner material to assist in evaluating liner options. These lists should be included in the RSR for alternates that are presented. Standardizing the list of advantages and disadvantages will create better consistency between reports prepared by different designers.

Appendix B presents Culvert Liner Selection Aids.

Appendix C presents worked examples of culvert liner calculations for corrosion and abrasion.

APPENDIX A – ADVANTAGES AND DISADVANTAGES OF LINER OPTIONS

1. FIBERGLASS

Advantages:

- Cost-effective for specialized applications
- Low maintenance
- Durable
- Does not corrode
- Highly impact and abrasion-resistant (thickness of interior layer can be increased for abrasion)
- Chemical-resistant
- Lightweight – Short lengths or segments of a cross section are easily transported, handled and installed; Large sections available in split segments with tongue and groove joints to make them transportable and more easily handled
- Liner thickness can be customized to provide greater stiffness and strength and longer life
- Available in custom sizes and shapes, including short-radius bends and flared sections; lateral connections also available
- Fabricated to tight tolerances
- Manning's roughness coefficient as low as 0.009
- Self-cleaning; maintains smooth surfaces
- Capable of installing baffles or sills to slow water velocity and cause sedimentation
- Leak-free joints available

Disadvantages:

- Higher cost than high density polyethylene
- Heavier than high density polyethylene for comparable lengths
- Less abrasion-resistant material than high density polyethylene and pvc
- Low modulus of elasticity (stiffness)
- Low strength - additional capacity is gained at the expense of added weight and cost of material
- Material properties change significantly with time under sustained loads; longterm material properties are 50 - 65% of initial structural values for strength and stiffness. Data is good to 50 years but claims of 100-year service life have not been substantiated.
- Not available in profile wall and is therefore limited to the capacity of a solid wall liner

2. HIGH DENSITY POLYETHYLENE (HDPE) AND POLYVINYL CHLORIDE (PVC)

Advantages:

- Cost-effective
- Low maintenance
- HDPE and PVC are not subject to chemical attack
- Material does not corrode
- Highly impact and abrasion-resistant
- Lightweight - Easily transported, handled and installed
- Available with smooth interior or corrugated interior
- Self-cleaning; maintains smooth surfaces
- Capable of installing baffles or sills to slow water velocity and cause sedimentation
- Long lengths are available, minimizing number of joints
- Leak-free joints available

Disadvantages:

- PVC available in diameters up to 48"
- PVC displays less resistance to abrasion than HDPE when pH < 4.0
- Low strength and stiffness (PVC is stronger and stiffer than HDPE)
- Relies on soil stiffness for capacity calculations
- Stiffness and strength drop drastically over time under sustained loads, making the wall susceptible to buckling and allowing excessive deflections
- Subject to stress cracking that may reduce longevity of HDPE. The existing test for antioxidants required for long-term service life of HDPE in AASHTO M 294 is inadequate, bringing into questions claims of service life of 75 to 100 years. Changes to M 294 have weakened cell classification requirements, potentially shortening the service life of HDPE pipe fabricated to this specification.
- Wall failure or buoyancy during grouting of annular space is a weakness with thin-walled HDPE liner. High groundwater table also increases buoyancy effects.
- Liner is flammable and subject to damage by brush fire and vandalism
- High thermal expansion affects long culverts with few joints or culverts with fused joints
- Poor weathering resistance

3. ALUMINUM

Advantages:

- Relatively strong – can support most fill heights and vehicle live loads
- Relies more on its own stiffness than on the soil envelope
- Material properties are constant over time
- Cost-effective
- Low maintenance
- Durable material in many environments
- Does not need protective coatings
- Impact and abrasion-resistant in low to moderately abrasive environments
- Gauge can be increased for longevity in moderately abrasive environments
- Lightweight - Easily transported, handled and installed
- Available in smooth or corrugated wall profiles
- Capable of installing baffles or sills to slow water velocity and cause sedimentation
- Long lengths are available, minimizing number of joints

Disadvantages:

- Low modulus of elasticity (stiffness) compared with steel
- May not be able to support large, concentrated loads under shallow fill
- Not suitable for highly abrasive stream flow conditions
- Not recommended for marine environments or where chloride concentration is high
- Should not be in direct contact with steel to avoid galvanic corrosion

4. STEEL

Advantages:

- Strongest of structural liners
- Very stiff without the soil envelope
- Resists grouting pressure with less bracing
- Material properties are constant over time
- Able to be coated for corrosion protection
- Polymer-coated steel and mortar lined steel can resist moderate abrasion
- Available in smooth or corrugated interiors
- Available in many gauges to provide extra thickness to compensate for abrasion and corrosion effects to meet service life goals
- Liner profile is thinner than other materials for same capacity
- Long lengths of liner are available, minimizing the number of joints
- Solid wall liner can be customized to any shape, size and thickness
- Solid wall liner has alloy to improve corrosion resistance. Thickness can be increased to account for longterm section loss due to corrosion.

Disadvantages:

- Heavier than other liners
- Coatings are vulnerable to more highly abrasive stream flow conditions
- Requires corrosion protection for longevity
- Galvanized coating is not suitable for a 75-year life without increasing the liner thickness
- Coated and uncoated steel may exhibit rust staining
- Few liner options are available with a smooth wall
- Solid wall requires welded joints and dry installation conditions
- Steel bands at joints are typically lower gauge and vulnerable to abrasion, requiring replacement before the target design service life

5. STEEL-REINFORCED THERMOPLASTIC

Advantages:

- Stronger than unreinforced thermoplastic liner
- Relies less on the soil envelope for stiffness
- Longterm solution for non-abrasive environments
- Resists grouting pressure with less bracing than for HDPE
- Material properties are constant over time
- Long lengths of liner are available, minimizing the number of joints
- High performance joints are available

Disadvantages:

- Susceptible to local buckling, which affects the capacity for the strength limit state
- Vulnerable to abrasive stream flow conditions due to the thin plastic wall
- Cannot include sills or baffles to slow velocity of flow and encourage sedimentation of natural stream bed material.

6. NON-STRUCTURAL LINERS

a. SPIRAL WOUND PVC OR HDPE

Advantages:

- Has the benefits of a slip liner but does not need long length of inlet or outlet channel from which to push or pull the liner.
- The liner can be installed while water is flowing in the host culvert.
- Minimal disruption to the public
- Minimal staging area is needed
- Smooth interior
- Essentially jointless
- Environmentally friendly (plastic is mechanically wound – avoiding contamination of water with styrene or thermal pollution
- The liner has structural capacity
- No shoring, excavating or backfilling
- The liner profile is relatively shallow, maximizing hydraulic area of the lined culvert

Disadvantages:

- Cannot be designed by LRFD or load rated by LRFR

- Non-round shapes are particularly susceptible to bending failure
- Liner thickness cannot be increased to provide a longer service life for abrasive environments
- Liner is flexible and vulnerable to deflection greater than 5% maximum specified by AASHTO. Gasketed seams may leak under excessive deflection.
- Thermoplastic long-term material properties drop dramatically compared to initial properties.
- Flammable – vulnerable to brush fires and vandalism.
- No owned spec exists to control product selection or prequalification

b. CENTRIFUGALLY CAST CONCRETE

Advantages:

- Fast installation
- Smooth interior surface is possible
- Staging does not typically disrupt the public

Disadvantages:

- Design methodology assumes (incorrectly) that the liner is designed for uniform radial external pressure. This is true for manholes, but not for horizontal culverts. Liner is not designed to resist eccentric loads.
- Fiber reinforcement may not adequately resist bending stresses.
- Thin liner is not suitable for direct burial due to potential for buckling failure.
- Uneven thickness of mortar – especially in corrugated structures and especially in non-round structures. Thickness is controlled by computer equipment that surveys the non-uniform cross section and controls the spray pressure and speed of the machine.
- Verification of mortar thickness is difficult
- Post installation troweling required for smoother liner
- Requires water handling for installation
- Liner mixes are proprietary. This makes it difficult to pre-design liner for design-bid-build. Inability to provide a load rating during design is not acceptable and shifts the responsibility to the Contractor. Requiring the Contractor to provide load rating calculations can lead to delays and claims.
- Poor record of success in multiple states (cracking, leakage, rust-staining, spalling)
- Difficult to assess the liner condition with so many signs of distress
- Impossible to assess the length of service life
- Abrasion resistance is different for each proprietary mix design
- Bond to host culvert is questionable
- Surface prep of host culvert is required.

- Should not be applied to failed culverts (cusped, deformed, voids, etc.)
- Invert recommended to be paved to allow the machine's skids to travel
- Requires significant work by hand methods in addition to the machine
- Length of liner dependent on material placement limitations
- Should not be applied with high ground water table percolating through culvert wall
- No owned spec exists to control product selection or prequalification

c. CURED IN PLACE RESIN

Advantages:

- Quick installation
- Forms a continuous, jointless liner
- Minimal disruption to the public
- Resists corrosion well
- Abrasion-resistant
- Smooth interior
- Conforms to any shape
- UV curing is fast, cost-effective and friendly to the environment
- Cured liner is suitable for potable water
- Liner does not shrink when cured
- Thickness can be increased

Disadvantages:

- Requires a dry pipe for installation
- Requires a machine using air or water pressure to install the liner within the host culvert
- Heated water or steam curing is not friendly to the environment and can be difficult to mitigate
- Cannot be designed by LRFD or load rated by LRFR
- Non-round shapes are particularly susceptible to bending failure
- Thermoplastic long-term material properties drop dramatically compared to initial properties.
- Liner is flexible and vulnerable to deflection greater than 5% maximum specified by AASHTO.
- Does not handle irregularities in host culvert well
- Contractor must consider the availability and location of a wet-out facility to apply the resin
- QA/QC plan is critical to success of the liner installation and performance
- No owned spec exists to control product selection or prequalification

APPENDIX B – CULVERT LINER SELECTION AIDS

In the absence of as-built information, the Table 1 may be used to select a constrained soil modulus for use in designing thermoplastic liners with Cover Height of 8’ or greater. Unless actual soil backfill information is available, assume Class III soil and 95% compaction. 95% compaction assumes that the soil over and beside the culvert is well-compacted after many years of service. If it is suspected that there may be voids and/or active settlement in the backfill, then assume 85% compaction.

Table 0.1 - Constrained Soil Modulus

Secant Constrained Soil Modulus, M_s

	Constrained Soil Modulus at Various Depths, Compaction								
	Class I		Class II			Class III			
	Crushed Stone		GW, GP, SW, SP			GM, SM, ML(1), GC and SC with <20% passing the 200 sieve			
Cover Height	Compacted	Uncompacted	95%	90%	85%	95%	90%	85%
Feet	psi	psi	psi	psi	psi	psi	psi	psi
1	2350	1280	2000	1280	470	1420	670	360
5	3180	1440	2450	1440	510	1610	720	380
10	3900	1580	2840	1580	550	1730	750	400
15	4460	1660	3090	1660	590	1790	760	410
20	4980	1730	3270	1730	620	1840	770	420
25	5500	1800	3450	1800	650	1880	790	430
30	5900	1860	3610	1860	690	1920	810	450
35	6300	1920	3770	1920	720	1960	830	460
40	6700	1980	3930	1980	780	2010	860	480
45	7100	2040	4090	2040	790	2050	880	490
50	7500	2100	4250	2100	830	2090	900	510
55	7860	2180	4400	2180	860			
60	8220	2260	4550	2260	895			
65	8580	2340	4700	2340	930			
70	8940	2420	4850	2420	965			
75	9300	2500	5000	2500	1000			

Notes:
1) M_s values presented in the table assume that the native material is at least as strong as the backfill material. If the native material is not adequate, it may be necessary to increase the trench width. Refer to ASTM D2321 for additional information on over-excavation.
2) M_s may be interpolated for intermediate cover heights.

Table 0.2 - Recommended Abrasion Levels

Liner Material/Wall Type	Wall Profile	Available Sizes and Shapes	Material Spec	Max. Abrasion Level[1] Recommended
Fiberglass	Solid	All (Can be custom made)	ASTM D3262	Level 6
HDPE	Solid; Corrugated; Profile Wall	Round Round Round	AASHTO M326/ASTM F714 AASHTO M294 ASTM F894	Level 6
PVC	Solid	Round		Level 4
Aluminum	Spiral Rib Smooth Interior; Corrugated; Structural Plate; Tunnel Liner	Round, Pipe-Arch Round, Pipe-Arch Round, Pipe-Arch, Horiz. & Vertical Ellipses Round, Pipe-Arch and Underpass	AASHTO M 197 AASHTO M 196 AASHTO M 219 Alloy 5052-H141	Level 3 (A higher level may be considered if thickness is increased)
Steel (Galvanized)	Spiral Rib Smooth Interior; Corrugated; Structural Plate; Tunnel Liner	Round, Pipe-Arch Round, Pipe-Arch Round, Pipe-Arch, Horiz. & Vertical Ellipses Round, Pipe-Arch and Underpass	AASHTO M 218 AASHTO M 218 AASHTO M 167 ASTM A1011	Level 5 (A higher level may be considered if thickness is increased)
Steel (Aluminized, Type 2)	Spiral Rib Smooth Interior; Corrugated Tunnel Liner	Round, Pipe-Arch Round, Pipe-Arch Round, Pipe-Arch and Underpass	AASHTO M 274 AASHTO M 274 AASHTO M 274 ASTM A929	Level 5 (if thickness is increased)
Steel (Polymer-Coated)	Spiral Rib Smooth Interior; Corrugated	Round, Pipe-Arch Round, Pipe-Arch	AASHTO M 246 AASHTO M 246	Level 5 (if thickness is increased)
Steel (Uncoated)	Solid	All (Can be custom made)	ASTM A36, Grade B	Level 5 (if thickness is increased)
Steel-Reinforced Thermoplastic	Corrugated Smooth Interior	Round	AASHTO M 335 and MP-40; ASTM D3350, Resin Class 345464C	Level 3

Footnotes:

1. Abrasion levels are rated from “1” (no abrasion) to “6” (Heavy Abrasion)

The following information is referenced from the California Department of Transportation Highway Design Manual, Chapter 850 (Physical Standards).

https://dot.ca.gov/-/media/dot-media/programs/design/documents/chp0850-a11y.pdf

Table 0.3 - Classification of Abrasion Levels

Abrasion Level	General Site Characteristics	Possible Culvert Lining Options
1	Virtually no abrasive bed load Water velocity unlimited	All liners possible
2	Moderate bed load of sand or gravel 1 ft/s ≤ Velocity ≤ 5 ft/s	All liners possible Check nearby culverts for abrasion to be sure that abrasion is not a concern.
3	Moderate bed load of sand, gravel and small cobbles 5 ft/s < Velocity ≤ 8 ft/s	All liners possible. Steel should consider polymer coating, aluminized coating or galvanized coating and both should consider additional gauge thickness for target design service life. Aluminum should consider additional gage thickness for target design service life.
4	Moderate bed load of angular sands, gravels, and/or small cobbles/rocks 8 ft/s < Velocity ≤ 12 ft/s	Corrugated HDPE (Type S only), SDR HDPE, PVC, fiberglass, Steel should consider polymer coating, aluminized coating or galvanized coating and should consider additional gauge thickness. Aluminum not recommended.
5	Moderate bed load of angular sands, gravels, and rocks 12 ft/s < Velocity ≤ 15 ft/s	SDR HDPE, fiberglass, Steel should consider polymer coating, aluminized coating or galvanized coating and should consider additional gauge thickness.
6	Moderate bed load of angular sands, gravels, and rocks 15 ft/s < Velocity ≤ 20 ft/s (Note: If minor bed load, use Abrasion Level 3)	SDR HDPE (2.5” thick min.), fiberglass, Steel should be specified with additional gage thickness to yield the target design service life. Concrete not recommended for velocity >15 ft/s unless a larger, harder aggregate than the bed load is embedded in concrete liner. Consider culvert replacement with an armored channel bed or construct settling basins upstream of inlet to collect cobbles and rocks. This requires regular maintenance to remove collected materials.

Table 0.4 - Bed Materials Moved by Various Flow Conditions

Bed Material	Grain Dimensions (inches)	Approximate Nonscour Velocities (feet per second)			
		Mean Depth (feet)			
		1.3	3.3	6.6	9.8
Boulders	more than 10	15.1	16.7	19.0	20.3
Large cobbles	10 – 5	11.8	13.4	15.4	16.4
Small cobbles	5 – 2.5	7.5	8.9	10.2	11.2
Very coarse gravel	2.5 – 1.25	5.2	6.2	7.2	8.2
Coarse gravel	1.25 – 0.63	4.1	4.7	5.4	6.1
Medium gravel	0.63 – 0.31	3.3	3.7	4.1	4.6
Fine gravel	0.31 – 0.16	2.6	3.0	3.3	3.8
Very fine gravel	0.16 – 0.079	2.2	2.5	2.8	3.1
Very coarse sand	0.079 – 0.039	1.8	2.1	2.4	2.7
Coarse sand	0.039 – 0.020	1.5	1.8	2.1	2.3
Medium sand	0.020 – 0.010	1.2	1.5	1.8	2.0
Fine sand	0.010 – 0.005	0.98	1.3	1.6	1.8
Compact cohesive soils					
Heavy sandy loam		3.3	3.9	4.6	4.9
Light		3.1	3.9	4.6	4.9
Loess soils in the conditions of finished settlement		2.6	3.3	3.9	4.3

Notes:

(1) Bed materials may move if velocities are higher than the nonscour velocities.

(2) Mean depth is calculated by dividing the cross-sectional area of the waterway by the top width of the water surface. If the waterway can be subdivided into a main channel and an overbank area, the mean depths of the channel and the overbank should be calculated separately. For example, if the size of moving material in the main channel is desired, the mean depth of the main channel is calculated by dividing the cross-sectional area of the main channel by the top width of the main channel.

The following information is from a Final Report (FHWA/CA/TL – CA01-0173, EA 680442) entitled, "Evaluation of Abrasion Resistance of Pipe and Pipe Lining Materials," published in September 2007.

https://rosap.ntl.bts.gov/view/dot/27517

Table 0.5 - Relative Abrasion Resistance Properties

Material	Relative Wear (dimensionless)
Steel	1
Aluminum	1.5 – 3
PVC	2
Polyester Resin (CIPP)	2.5 – 4
HDPE	4 – 5
Concrete (RCP 4000 – 7000 psi)	75 – 100
Calcium Aluminate (Mortar)	30-40
Calcium Aluminate (Concrete)	20 – 25
Basalt Tile	1
Polyethylene (CSSRP)	1 – 2

* Evaluation of Abrasion Resistance of Pipe and Pipe Lining Materials Final Report FHWA/CA/TL-CA01-0173 (2007).

The table above offers a comparison of abrasion resistance of a number of materials. All materials are compared to steel, where steel is the most resistant to abrasion. The rate of wear is very high in this study due to the extremely aggressive bed load of angular material and the high velocities.

The following tables present wear rates for various liner materials and are in no particular order.

Table 0.6 - Abrasion Wear Rates - Concrete

ABRASION OF CONCRETE LINER						
Abrasion Level	Velocity ft/s		Concrete Thickness (For 50-Year Life) Inches		Wear Rate Inch/Year	
	Low	High	Low	High	Low	High
4	8	12	2	4	0.04	0.08
5	12	15	4	13	0.08	0.26
6	15	20	13	15	0.18	0.30

Table 0.7 - Abrasion Wear Rates - Steel

ABRASION OF STEEL LINER						
Abrasion Level	Velocity ft/s		Steel Thickness (For 50-Year Life) Inches		Wear Rate Inch/Year	
	Low	High	Low	High	Low	High
4	8	12	0.052	0.052	0.00104	0.00104
5	12	15	0.052	0.18	0.00104	0.0036
6	15	20	0.18	0.5	0.00218	0.01

Table 0.8 - Abrasion Wear Rates - Aluminum

ABRASION OF ALUMINUM LINER						
Abrasion Level	Velocity ft/s		Aluminum Thickness (For 50-Year Life) Inches		Wear Rate Inch/Year	
	Low	High	Low	High	Low	High
4	8	12	0.075	0.164	0.0015	0.00328
5	12	15	Provide invert protection		NA	NA
6	15	20	Provide invert protection		NA	NA

Table 0.9 - Abrasion Wear Rates - PVC

ABRASION OF PVC LINER						
Abrasion Level	Velocity ft/s		PVC Thickness (For 50-Year Life) Inches		Wear Rate Inch/Year	
	Low	High	Low	High	Low	High
4	8	12	0.1	0.1	0.002	0.002
5	12	15	0.1	0.35	0.002	0.007
6	15	20	0.35	1	0.005	0.02

Table 0.10 - Abrasion Wear Rates - HDPE

ABRASION OF HDPE LINER						
Abrasion Level	Velocity ft/s		HDPE Thickness (For 50-Year Life) Inches		Wear Rate Inch/Year	
	Low	High	Low	High	Low	High
4	8	12	0.125	0.25	0.0025	0.005
5	12	15	0.25	0.875	0.005	0.0175
6	15	20	0.875	2.5	0.0125	0.05

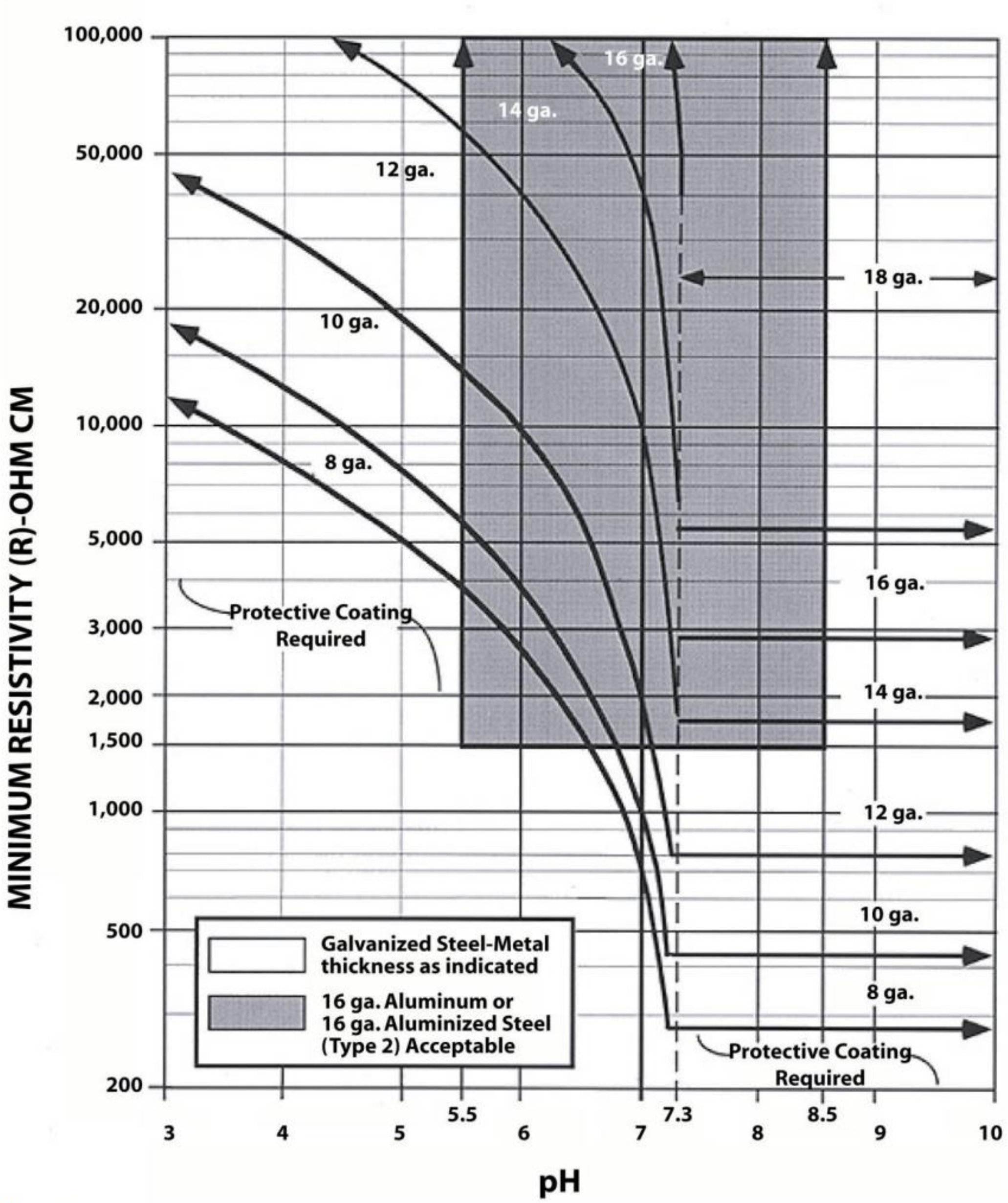
Table 0.11 - Abrasion Wear Rates - CIPP

ABRASION OF CIPP LINER						
Abrasion Level	Velocity ft/s		CIPP Thickness (For 50-Year Life) Inches		Wear Rate Inch/Year	
	Low	High	Low	High	Low	High
4	8	12	0.1	0.3	0.002	0.006
5	12	15	0.3	0.7	0.006	0.014
6	15	20	0.7	2	0.01	0.04

Polyester Resin Cured-In-Place Pipe (CIPP)

Figure 0-1 - Minimum Thickness of Metal for 50-year Service

Minimum Thickness of Metal Pipe for 50-Year Maintenance-Free Service Life (2)



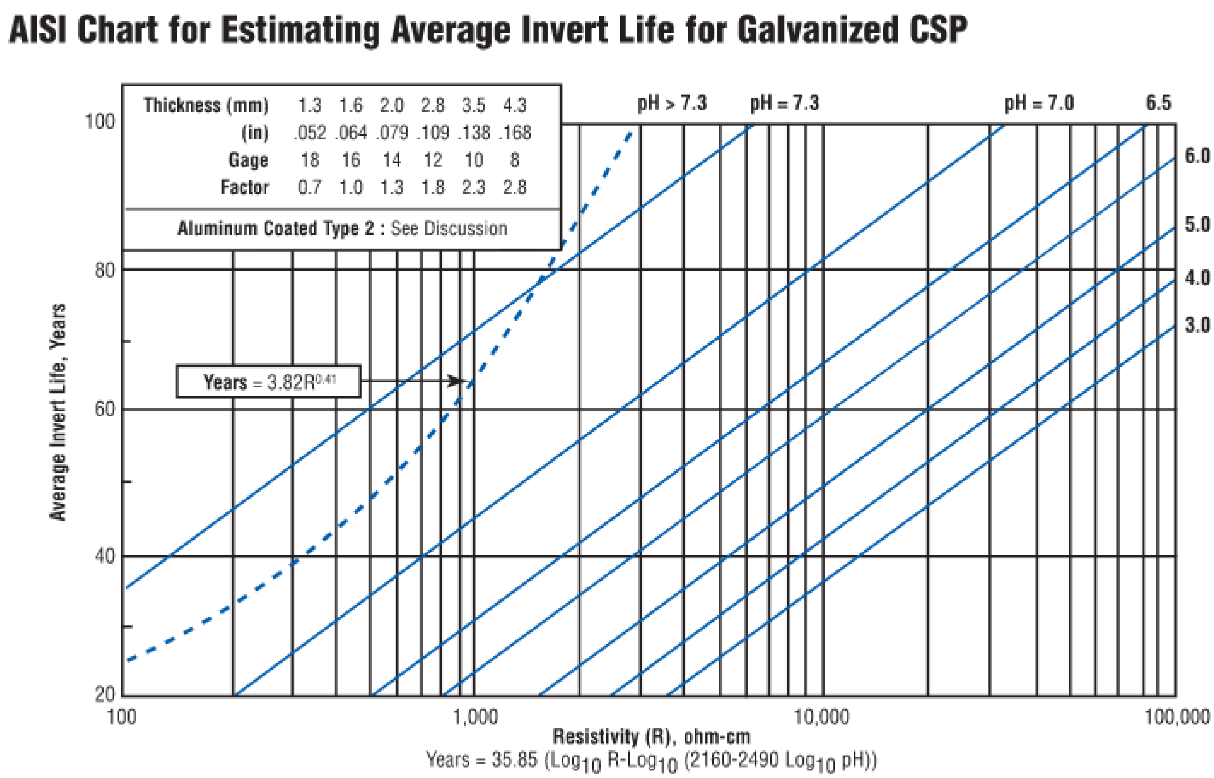
Notes:

(1) For pH and aluminum resistivity levels not shown refer to Fig. 855.3B steel pipes. (California Test 643)
(2) Service life estimate are for various corrosive conditions only.
(3) Refer to Index 852.3(2) and 852.4(2) for appropriate selection of metal thickness and protection coating to achieve service life requirements.

The following chart is referenced from the CSP Durability Guide, published by the National Corrugated Steel Pipe Association (NCSPA), May 2000. It directly applies to galvanized corrugated steel liner. To apply the AISI Chart to aluminized steel (Type 2), multiply the Average Life x 1.3. The chart is for corrosion only and does not consider the effects of abrasion.

http://pcpipe.com/wp-content/uploads/2019/05/NCSPA-CSP_Durability-Guide.pdf

Figure 0-2 - Estimating Service Life for Galvanized CSP



Steps in Using the AISI Chart

The durability design chart can be used to predict the service life of galvanized CSP and to select the minimum thickness for any desired service life. Add-on service life values are provided in the table on page 5 for additional coatings.

1) Locate on the horizontal axis the soil resistivity (R) representative of the site.
2) Move vertically to the intersection of the sloping line for the soil pH. If pH exceeds 7.3 use the dashed line instead.
3) Move horizontally to the vertical axis and read the service life years for a pipe with 1.6 mm (0.064 in.) wall thickness.
4) Repeat the procedure using the resistivity and pH of the water; then use whichever service life is lower.
5) To determine the service life for a greater wall thickness, multiply the service life by the factor given in the inset on the chart.

Table 0.12 - Corrosion Rates of Corrugated Steel

Corrosion Rates of Corrugated Steel Pipe (CSP)						
			Galvanized		Aluminized	
Gauge	Thickness, T (Inch)	Factor	Years, Y_G[1] to First Perforation	Corrosion Rate inch/year	Years, Y_A to First Perforation	Corrosion Rate inch/year
18	0.052	0.7	0.7 * Y_{G_16}	T_{18}/Y_{G_18}	1.3 * Y_{G_18}	T_{18}/Y_{A_18}
16	0.064	1	Y_{G_16}	T_{16}/Y_{G_16}	1.3 * Y_{G_16}	T_{16}/Y_{A_16}
14	0.079	1.3	1.3 * Y_{G_16}	T_{14}/Y_{G_14}	1.3 * Y_{G_14}	T_{14}/Y_{A_14}
12	0.109	1.8	1.8 * Y_{G_16}	T_{12}/Y_{G_12}	1.3 * Y_{G_12}	T_{12}/Y_{A_12}
10	0.138	2.3	2.3 * Y_{G_16}	T_{10}/Y_{G_10}	1.3 * Y_{G_10}	T_{10}/Y_{A_10}
8	0.168	2.8	2.8 * Y_{G_16}	T_8/Y_{G_8}	1.3 * Y_{G_8}	T_8/Y_{A_8}

Note: This table is for corrosion only and does not consider loss due to abrasion.

Footnote 1: Enter Resistivity, R, and pH into the following formula to calculate Years to first perforation of 16-gauge galvanized liner, Y_G: (Calculates value from Chart on previous page)

$$Y_{G\_16} = 35.85\ (\text{Log10}\ R - \text{Log10}\ (2160 - 2490\ \text{Log10}\ pH))$$

Use the table below to select Y_{G_16} for a known pH and Resistivity. Enter Y_{G_16} in the table above to calculate the corrosion rates for galvanized and aluminized liners of various gauges.

Table 0.13 - Years to First Perforation for 16-gauge Galvanized Steel

Years to First Perforation for 16-Gauge Galvanized Steel Culvert Liner, Y_{G_16}												
	Resistivity											
pH	500	1000	1500	2000	3000	4000	5000	6000	7000	8000	9000	10000
4		6	13	17	24	28	32	34	37	39	41	42
5	3	14	20	24	31	35	39	41	44	46	48	49
6	13	23	30	34	41	45	48	51	54	56	58	59
6.5	20	31	37	42	48	53	56	59	61	63	65	67
7	34	45	51	56	62	67	70	73	75	77	79	81
7.3	60	71	78	82	88	93	96	99	101	104	105	107

APPENDIX C – WORKED EXAMPLES

1. GALVANIZED AND ALUMINIZED STEEL CULVERT LINER

The following discussion provides design guidance on how to use the AISI Chart in Figure B of Appendix B to graphically determine the service life of a selected culvert liner. Tables 7 and 8 in Appendix B provide a numerical calculation of years to first perforation. These calculations can be confirmed by the Liner Selection Worksheet that is available on State Bridge Design's web page.

For this example, it is determined that 16-gauge galvanized steel is needed for structural capacity. The liner will be grouted inside the host culvert, which is conservatively assumed to support no load. The AISI chart calculates time to first perforation. For corrugated galvanized steel experiencing no abrasion, first perforations can occur anywhere along the corrugations. Although capacity is reduced by a perforation, the grout, together with the host culvert will provide additional composite action and distribute loads to the bedding. Assume that the full design capacity of the liner is available at first perforation.

Little/no abrasion (Abrasion Levels 1 and 2),

Go to the AISI Chart for Estimating Average Invert Life for Galvanized CSP (or see Table 8above for 16-gauge galvanized steel culvert liner). For this example, assume that environmental testing was performed that indicates that the water has a pH of 6.0 and a resistivity of 2,000 ohm-cm. From the table, the Average Invert Life for a 16-gauge liner to first perforation is 34 years. If a life of 75 years is desired, consider a thicker gauge. There are two ways to calculate the thickness needed for a different gauge liner:

- Calculate the corrosion rate of 16-gauge liner:

  The corrosion rate is calculated by dividing the thickness of liner by the years of estimated service life: T_{16}/Y_{G_16} = 0.064 inch thick/34 years = 0.00188 inch/year

  Using the corrosion rate calculated above for the 16-gauge liner, calculate the desired thickness of liner to resist corrosion for 75 years: 75 years x 0.00188 inch/year = 0.141 inch. A 10-gauge liner is 0.138 inch thick < 0.141 inch (an 8-gauge liner would provide considerably more life).

- Use the inset of the AISI Chart in Figure B to directly calculate the life:

  The corrosion life of a 10-gauge galvanized steel liner is (34 years for a 16-gauge liner) x (a factor of 2.3 from the inset of the AISI Chart) = 78.2 years > 75 years.

  Note: the corrosion rate of 10-gauge galvanized plate is 0.138" thick/78.2 years = 0.00176 inch per year <0.00188 calculated for 16-gauge. 75 years x 0.00176 inch/year = 0.132" required < 0.138" for 10-gauge.

Moderate abrasion within the culvert (Abrasion Levels 3, 4 and 5),

Assume for this example that the type of streambed material is angular sands and gravels and the water velocity for a 2-year storm event = 12 fps. Classify this culvert as Abrasion Level 4. Select the thickness of the steel to account for material lost due to abrasion as well as corrosion. Since a 10-gauge was selected for corrosion alone, consider an 8-gauge liner.

8-gauge galvanized liner has a corrosion life of 2.8 x life of 16-gauge = 2.8 * 34 years = 94.2 years. The rate of corrosion is 0.168 inch / 94.2 = 0.00176 inch/year. Combining corrosion and abrasion rates for 8-gauge galvanized steel: 0.00104 inch/year + 0.00176 inch/year = 0.00280 inch/year. For a 75-year life a thickness of material must be 75 years *0.00280 inch/year = 0.21 inch > 0.168” 8-gauge galvanized steel liner.

Since 8-gauge is the thickest galvanized plate available, consider adding a concrete liner to help protect the 8-gauge galvanized steel liner. The 8-gauge galvanized steel liner is expected to last 0.0168 inch thick /0.0028 inch/year = 60 years. The concrete must last for 75 years – 60 years = 15 years.

From Table 6A - Abrasion of Concrete Liner in Appendix B, select the rate of abrasion of concrete for velocity = 12 fps to be 0.08 inch/year. 15 years x 0.08 inch/year = 1.2 inches of concrete is required to protect the 8-gauge galvanized steel liner.

Consider aluminized steel without a concrete liner. Aluminized steel is available in thicknesses up to 10 gauge. Select a 10-gauge aluminized steel liner. As seen previously, a 10-gauge galvanized liner without abrasion will last 78.2 years to first perforation. Aluminized 10-gauge will last 1.3 * 78.2 years = 101.7 years.

The corrosion rate of this liner is 0.138 inch / 101.7 years = 0.00136 inch/year.

Add the abrasion rate of 0.00104 inch/year for a total material loss of 0.00136 + 0.00104 = 0.00240 inch/year.

The calculated years to first perforation is: 0.138 inch / 0.00240 inch/year = 57.5 years. If a 75-year service life is desired, a concrete liner could be added. The required thickness of concrete liner is: (75 – 57.5) * 0.08 inch/year = 1.4”.

2. POLYMER-COATED STEEL CULVERT LINER

At Abrasion Level 4, polymer-coated steel is presumed to contribute an additional 10 years of service life to the culvert liner. The steel below the polymer is galvanized. Calculate the gauge required to provide a 75-year design service life. First, subtract the additional 10-year life that the polymer offers:

75 years – 10 year (polymer) = 65 years of life expected from the galvanized steel below the polymer coating. Note: Polymer-coated steel is available in thicknesses up to 10-gauge.

Next, determine the rate of section loss due to corrosion and abrasion:

> Corrosion: From the example above, a 10-gauge galvanized liner will corrode at the rate of 0.00176 inch/year
>
> Abrasion: The abrasion rate of steel is 0.0010 inch/year
>
> The combined wear rate = 0.00176 + 0.0010 = 0.00276 inch/year

Calculate the required thickness to last 65 years: 65 years x 0.00276 = 0.179 inch > 0.138 (10-gauge)

Since 10-gauge does not provide the required thickness, the thickest available polymer-coated liner will not be sufficient for a 75-year life. If the Department would accept a design service life of 50 years, or it can be shown that there will be adequate structural capacity at 75 years (remember that only a 16-gauge steel liner was required), the 10-gauge polymer-coated steel liner would be a possible alternate.

3. ALUMINUM CULVERT LINER

Using the design situation in the example for the design of galvanized and aluminized steel liner, consider if aluminum liner can be proposed. pH is 6.0, Resistivity is 2,000 ohm-cm, Abrasion Level 4 with water velocity = 12 fps and a moderate bed load of angular sand and gravel.

Although aluminum liner is typically recommended for Abrasion Level 3 and lower, consider how it will perform in this situation. The service life will be shortened for Abrasion Level 4 and higher. Increasing the thickness is an option to increase service life if there is a compelling reason to consider aluminum.

From the Abrasion of Aluminum Liner table in Table 6C of Appendix B, the rate of abrasion is 0.00328 inch/year.

From Appendix B, Figure A, the corrosion rate is 0.064 inch/50 years = 0.0013 inch/year.

Calculate the years until first perforation of 0.25 inch-thick aluminum liner: 0.25 inch / (0.00328 + 0.0013inch per year) = 54.5 years. An aluminum liner of the maximum available thickness would perform for approximately 55 years until first perforation. If the life cycle cost for this liner for this service life is the lowest cost of the alternatives or is otherwise acceptable to the Department, aluminum could be considered an alternate in the RSR. The 0.25-inch-thick aluminum liner weighs less than the 10-gauge polymer-coated steel liner and offers five additional years of service life. If it were difficult to handle the steel in the field due to its weight or touching up the polymer coating was thought to add excessive time and cost, aluminum may provide a more constructable alternative. Before selecting the aluminum alternative, check the structural capacity at the end of service life to confirm it is adequate.

4. HDPE CULVERT LINER

Since HDPE culvert liner will not corrode, consider abrasion only. From Table 6E – Abrasion of HDPE Liner the abrasion rate at a velocity of 12 fps (Abrasion Level 4) is 0.0050 inch/year. The required sacrificial thickness for a 75-year life is 75 x 0.0050 inch per year = 0.375 inch. This will require a smooth wall Standard Dimension Ratio (SDR) HDPE liner or a profile wall liner with an interior lining of at least 0.375". SDR liner may be too heavy, too expensive, or may not be available at all. HDPE profile wall liner is lightweight, available and less expensive. Specify a minimum Ring Stiffness Coefficient (RSC) of 160. If the thickness is not adequate, select RSC 250 or stiffer.

5. FIBERGLASS CULVERT LINER

Fiberglass resin abrades at a rate of six times that of HDPE. The interior layer of the wall thickness can be designed to be sacrificed for the service life of the culvert. At an abrasion rate of 0.030 inch/year, the interior layer shall be 75 years x 0.030 inch/year = 2.25" thick. The design of the fiberglass total wall thickness shall account for this sacrificial loss to ensure that the liner can support the desired loads at the end of its service life. Contact a supplier for preliminary design guidance.

EXISTING BEARING REPLACEMENT AND REHABILITATION

The replacement or rehabilitation of existing bearings on a bridge being rehabilitated shall consider the bearing's type, age (remaining service life), material, condition, and location relative to a deck joint, condition of the existing member ends (to remain in place), and the scope of the structural work on the bridge.

For bridge rehabilitation alternatives proposing a superstructure replacement, all the existing bearings, regardless of bearing type, age, material, or condition, shall be replaced.

For bridge rehabilitation alternatives proposing a deck replacement, existing all-metal expansion bearings, all-metal fixed bearings, and high-load multi-rotational (HLMR) bearings, regardless of age or condition, shall be replaced. Existing elastomeric bearings, including isolation bearings, may remain provided:

1. the elastomer is uncracked, free of tears, has uniform and equal deformations and bulges (both vertically and laterally) compared to all bearings in the line
2. the exposed steel components do not have any heavy, laminated, or impacted rust and
3. the bearings do not have any conditions that will impact the function of the bearings for all loading conditions and load effects for the remaining service life of the structure without replacement

Commentary: All-metal expansion bearings include high-profile steel rocker bearings and bearings with steel and other metals, such as low-profile self-lubricating bronze plate steel bearings. All-metal fixed bearings include high-profile steel fixed pin bearings and low-profile steel fixed pin bearings with a radiused sole plate and a masonry plate. HLMR bearings include pot, spherical and disc bearings.

Per AASHTO Guide Specification for the Service Life of Highway Bridges, since the service life of all metal bearings and HLMR bearings is approximately 75 years or less, these types of bearings shall be replaced when the deck is being replaced. Elastomeric bearings are estimated to have a service life of 75-100 years.

Conditions that may impact the function of elastomeric bearings include the bearings' location relative to the centerline of bearing (indicating the bearing is "walking"), or bearings with less than 100% contact with the member it supports or the supporting surface of the concrete bearing pad.

The term "load effects" refers to not only forces, but also translational and rotational movements.

If an evaluation of the bearings for load conditions and load effects result in excessive deformation of the bearing, the bearings would not meet the existing elastomeric bearing criteria and should be replaced.

For bridge rehabilitation alternatives proposing a bridge widening, the bearings shall be addressed as follows:

1. existing all-metal expansion bearings, regardless of age or condition, shall be replaced.
2. existing all-metal fixed bearings may remain provided:
   a. the bearing components do not have any heavy, laminated, or impacted rust and

   b. the bearing components do not have any conditions, such as section loss, loss of bearing area, or loss of restraint, that will impact the function of the bearings for all loading conditions and load effects for the remaining service life of the structure without replacement

3. existing elastomeric bearings may remain provided:

   a. the elastomer is uncracked, free of tears, has uniform and equal deformations and bulges (both vertically and laterally) compared to all bearings in the line

   b. the exposed steel components do not have any heavy, laminated, or impacted rust and

   c. the bearings do not have any conditions that will impact the function of the bearings for all loading conditions and load effects for the remaining service life of the structure without replacement

For all other bridge rehabilitation alternatives, bearings shall be considered for replacement if:

1. the elastomer has cracks or tears, and does not uniformly and equally deform both vertically and laterally compared to all bearings in the line
2. the exposed steel components of the bearings have section loss or heavy, laminated, or impacted rust
3. the bearings do not function appropriately for all proposed loading conditions and load effects for the remaining service life of the structure without replacement.
4. the bearings have been modified from their original construction, such as self-lubricating bronze plate bearings that had the anchor bolts through the flange and bearing cut short and replaced with keeper plates

Commentary: Since the "other bridge rehabilitation alternatives" is not a defined scope of work, the direction "bearings shall be considered for replacement" has been given. The designer shall determine whether the bearings should be replaced or rehabilitated based on the criteria provided and structural scope of work.

On existing bridges undergoing bearing replacement, the condition of the member ends must be adequate to support and connect the replacement bearings. If the member ends have existing deficiencies, such as section loss, that cannot be corrected, the replacement of the bearings may not be feasible.

Commentary: Steel section loss at member ends may result in a tapered bottom flange edge that no longer has sufficient edge thickness (i.e.. "knife edge") for a weld to connect a replacement sole plate to the flange.

The replacement bearings and sole plates shall be sized to avoid conflicts with full length cover plates unless the cover plates will be modified.

Existing concrete bearing pads shall only be reused if they are in good condition. Existing concrete bearing pads with cracks and spalls shall be replaced.

Commentary: At locations undergoing bearing replacement, since the structure will be raised, concrete bearing pads with cracks and spalls shall be replaced to better ensure a longer service life of the component.

Where existing concrete bearing pads will be reused, the reuse of existing anchor bolts is not permitted. The existing anchor bolts shall be cut off and removed to below the top surface of the pad, the exposed anchor bolt shall be coated with 2 coats of brush applied cold galvanizing compound (The use of aerosol spray is not permitted), the void filled with cementitious patching material and the entire pad shall be coated with "Penetrating Sealer Protective Compound." Designers shall meet bearing anchorage requirements. Designers shall ensure that the installation of replacement anchor bolts is feasible.

Commentary: Where bearings are being replaced, the structural resistance and remaining service life of existing anchor bolts shall be assumed to be inadequate so that the anchor bolts cannot be reused. The anchorage that they provide must be addressed at the time of the bearing replacement.

Designers should understand that installation of anchor bolts for bearings below an existing structure can be difficult because existing structure components may limit worker, equipment, and material access. Designers shall ensure the constructability of the replacement anchor bolts.

At bearing replacement locations, existing concrete bearing pads shall be replaced if the pad height along with the height of the replacement bearing will adversely impact the portion of the structure to remain in place or if the pad bearing area provides less than 100 percent of the bearing area required by the replacement bearing.

Commentary: The edges of concrete bearing pads are beveled reducing the area available to support bearings.

On existing bridges undergoing elastomeric bearing replacement that have abutments with a slab over backwall condition, the bearing shall be designed and detailed to prevent the slab from resting on the top of the backwall after all bearing deflection has taken place. The joint between the underside of the slab and the top of the backwall shall be filled with a combination of 2 sealants to prevent the backfill from migrating into the joint and the intrusion of water onto the bridge seat. Once the bridge is resting on the replacement bearings, from the underside of the bridge, after cleaning the joint with compressed air, install expanding spray applied open cell foam beginning at the rear face of the back wall (end of deck) to within 1 inch of the front face of the backwall. Complete the sealant installation, by installing a non-sagging elastomeric joint sealant in the remaining 1 inch depth of the joint.

Commentary: All work described should be performed from the underside of the bridge. Access will be limited and constrained. The installation of the expanding spray applied open cell foam may require use of rigid or flexible extension tube to access the joint void at the rear face of the backwall. The extension tube should be withdrawn as the void is filled. No excavation of the roadway approach to the bridge is required.

Rehabilitation of existing bearings is limited to cleaning and painting the exposed steel surfaces of the bearing. All cleaning and painting of the exposed steel portions of existing bearings shall conform to the **CTDOT** special provisions for paint removal and field painting. The cleaning and painting of rocker bearings or fixed bearings with a radiused sole plate that have heavy, laminated, or impacted rust between the bearing radius and the masonry plate is not permitted. The rehabilitation of bearings by removing, disassembling, cleaning, re-lubricating and reinstalling the bearing is <u>not permitted</u>.

Commentary: Abrasively blast cleaning rocker bearings or fixed bearings with heavy, laminated or impacted rust between the radiused bearing surface and the surface of the masonry plate typically reveals section loss on each surface leaving nonuniform "gaps" between the surfaces and a "flattening" of the bearing radius. Since the resulting condition impacts the functionality of the bearing and will adversely impact the condition rating of the bridge element, bearing replacement should be considered provided bearing replacement is included in the structural scope of work for the project. Designers should be aware of these concerns on projects where the structural scope of work is limited to the abrasive blast cleaning and field painting of an existing structure.

The rehabilitation of self-lubricating bronze bearings by removing, disassembling, cleaning, re-lubricating and reinstalling the bearing was once practiced by ***CTDOT****. This method of rehabilitation was determined to be ineffective and is no longer permitted.*