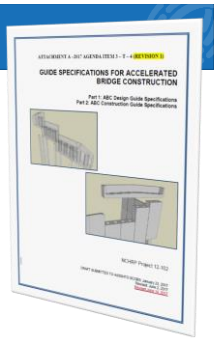
Workshop W-01

Implementation of the New AASHTO Guide Specifications for Accelerated Bridge Construction

- Module 1: NCHRP Project 12-102



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Note

This investigation is sponsored by TRB under the NCHRP Program. Data reported is work in progress. The contents of this presentation has not been reviewed by the project panel or NCHRP, nor do they constitute a standard, specification, or regulation.

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- Project Panel
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  - Tim Couples - FHWA
- Project Team
  - Michael P. Culmo – CME Associates
  - Lee Marsh – Berger ABAM
  - Stuart Bennion – Berger ABAM
  - John Stanton – UW
  - Dennis Mertz

NCHRP 12-102 Project Overview

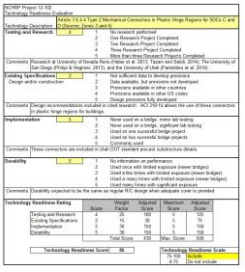
- Guide Specification Development
  - Significant synthesis project
  - No new research involved
  - Technology readiness evaluation done for each technology
- Approach:
  - Not a stand-alone document
  - Supplement to:
    - AASHTO LRFD Bridge Design Specifications
    - AASHTO Bridge Construction Specifications
  - Separate Design and Construction Parts
- Project Status
  - Final Report is in for Publication

NCHRP 12-102 Project Overview

- Sources of provisions
  - Significant technology synthesis
    - Dozens of past research projects on ABC
    - NCHRP Research
    - State DOT Research
    - International Research
    - Private research
  - Questionnaire to Each State DOT
    - What are they using?
    - What is working? What is not?
    - Lessons Learned

Technology Readiness

- Evaluation
  - Level of testing and research
  - Existing Specifications
  - Implementation
  - Durability
- Parameters and weight factors worked out with project panel



NCHRP Project 12-102
Technology Readiness Evaluation

Technology Description	Within 2.6.4.2 Type 2 Mechanical Connectors in Plastic Hinge Regions for SDCs C and D Regions, Zones 3 and 4	
Testing and Research	0	1 No research performed 2 One Research Project Completed 3 Two Research Project Completed 4 Three Research Project Completed 5 More than three Research Projects Completed
Comments	Research at University of Nevada Reno (Ameuri et al. 2016), the University of San Diego (Phillips & Ringelstetter, 2015), and the University of Utah (Tamalonis et al. 2016)	
Existing Specifications Design and/or construction	0	1 Not sufficient data to develop provisions 2 Data available, but provisions not developed 3 Provisions available in other countries 4 Provisions available in other US codes 5 Design provisions fully developed
Comments	Design recommendations included in cited research. ACI 318-14 allows the use of these connectors in plastic hinge regions for buildings	
Implementation	0	1 Never used on a bridge, never lab testing 2 Never used on a bridge, but lab testing 3 Used on two successful bridge projects 4 Used on two successful bridge projects 5 Commonly used
Comments	These connectors are included in Utah DOT standard precast substructure details	
Durability	0	1 No information on performance 2 Used once with limited exposure (newer bridges) 3 Used a few times with limited exposure (newer bridges) 4 Used a many times with limited exposure (newer bridges) 5 Used many times with significant exposure
Comments	Durability expected to be the same as regular RC design when adequate cover is provided	

Technology Readiness Rating	Score	Weight Factor	Adjusted Score	Maximum Score	Adjusted Score
Testing and Research	4	25	100	5	125
Existing Specifications	2	15	30	5	75
Implementation	5	36	180	5	180
Durability	5	36	180	5	180
		Total Score	490	Max. Score	560

Technology Readiness Score: 88

Technology Readiness Scale	
75-100	Include
0-75	Do not include

Specification Overview

- Guide Specification Contents

Part 1: ABC Design Guide Specifications

1. Introduction
2. General Design Provisions
3. Design of Prefabricated Elements
4. Detailing Requirements
5. Durability of ABC Technologies

Part 2: ABC Construction Guide Specifications

1. Introduction
2. Temporary Works
3. Fabrication and Assembly Planning
4. Layout and Tolerances
5. Concrete Structures
6. Steel Structures
7. Geosynthetic Reinforced Soil / Integrated Bridge System

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A little Grammar to start

- This document should not be considered a full specification
- Users of this document should exercise engineering judgement in conjunction with the provisions in the document.
- The document makes use of the terms "shall", "should", "may", and "recommended".
  - The term "shall" denotes a requirement for compliance with these specifications.
  - The term "should" indicates a strong preference for a given criterion.
  - The term "may" indicates a criterion that is usable, but other local and suitably documented, verified, and approved criterion may also be used in a manner consistent with the LRFD approach to bridge design.
  - The term "recommended" is used to give guidance based on past experiences.

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Reference Documents

- This is not a stand-alone design specification.
- This document is intended to supplement the AASHTO LRFD Bridge Design Specifications and the AASHTO LRFD Bridge Construction Specifications
- In many cases, provisions will refer to other AASHTO documents
  - Not by article number
  - By article Title
  - Why?

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Other References

NCHRP Project 12-98

- 2018 Guidelines on Prefab Tolerances
- 2018 Guidelines on Dynamics
  - Including SPMTs and Lateral Sliding
- Want to know more?
  - FIU ABC Webinar Archives recordings
  - They are free!

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Current Status

- The guide specification was completed late 2016
- Forwarded to the AASHTO T-4 Construction Committee for consideration
  - Home of ABC at AASHTO
- Reviewed by AASHTO Committees over the winter of 2017
- Minor revisions were made based on reviews
- Balloted for adoption in June 2017
- Passed
- Several Committees still have issues
  - T-4 is working with them to resolve some issues prior to publication
  - Publication by AASHTO in late 2017, early 2018

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Design Specification Overview

- Design Specification
  - Section 1: Introduction
  - Section 2: General Design Provisions
    - Shipping and Handling Provisions
    - Load Combinations for SPMT and Lateral Slide

CME

Design Specification Overview

- Design Specification
  - Section 3: Design of Prefabricated Elements
    - Majority of specs are based on emulation
    - Significant seismic provisions
    - Major Provisions that vary from LRFD
      - Lapped hooked and headed bars
      - UHPC connections
      - Type 2 Mechanical Connectors
      - Corrugated Metal Pipe Sockets and corrugated precast sockets
      - Link slabs

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Design Specification Overview

- Design Specification
  - Section 4: Detailing Requirements
    - Tolerances and layout of precast elements
      - Reference to NCHRP Project 12-98 guidelines
  - Section 5: Durability of ABC Technologies
    - Detailing recommendations for durability

CME

Design Specification Overview

- Construction Specification
  - Section 4: Detailing Requirements
    - Tolerances and layout of precast elements
      - Reference to NCHRP Project 12-98 guidelines
  - Section 5: Durability of ABC Technologies
    - Detailing recommendations for durability

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Definitions

- Definitions are included in Section 1 of the Design Section
- Based on FHWA and AASHTO T-4 Committee

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Key Definitions

Accelerated Bridge Construction

Bridge construction that uses innovative planning, design, materials, and construction methods in a safe and cost-effective manner to reduce the onsite construction time that occurs when building new bridges or replacing and rehabilitating existing bridges.

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Key Definitions

Conventional Bridge Construction

Bridge construction that does not significantly reduce the onsite construction time that is needed to build, replace or rehabilitate a bridge. Prefabrication is typically limited to beams and girders in this form of construction.

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Key Definitions

Prefabricated Bridge Elements and Systems (PBES)

Structural components of a bridge that are built offsite or near the site that include features that reduce the on-site construction time that occurs with conventional bridge construction.

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Key Definitions

Prefabricated Bridge Elements (PBE)

A category of PBES which comprise a single structural component of a bridge. Prefabricated Bridge Elements can be made of any approved structural material.

Key Definitions

Prefabricated System

A category of PBES that consists of an entire superstructure, an entire superstructure and substructure, or a total bridge that is procured in a modular manner such that traffic operations can be allowed to resume after placement. A Prefabricated system is rolled, launched, slid, lifted, or otherwise transported into place, having the deck and preferably the railing in place such that no separate construction phase is required after placement.

CME

Key Definitions

Self-Propelled Modular Transporters (SPMTs)

A high capacity transport device that can lift and move prefabricated elements and systems with a high degree of precision and maneuverability in all three directional axes without the aid of a tractor for propulsion.

CME

Key Definitions

Lateral Slide

A method of moving a bridge system built adjacent to the final bridge location using hydraulic jacks or cable winches while supported on sliding materials or rollers. The bridge is typically built parallel to its final alignment, facilitating the installation.

CME

Key Definitions

Geosynthetic Reinforced Soil Integrated Bridge System (GRS/IBS)

A bridge that is directly supported on a GRS abutment that blends the roadway into the superstructure to create a jointless interface between the superstructure and the approach embankment

CME

Key Definitions

Assembly Plan

A package of plans, specifications and calculations developed by the contractor that describes the process for the assembly of prefabricated elements. The assembly plan may include handling and erection plans, materials specifications, details and calculations for bridge temporary works, and construction scheduling.

Key Definitions

Closure Joint

A gap between two elements or systems that is filled with materials to form a connection. The joint may or may not include reinforcing. The width of the closure joint can vary based on the type of material used to fill the joint and the reinforcing within the joint. This feature is also referred to as a "closure pour" by some agencies.

Key Definitions

Link Slab

Links slabs are a transverse deck level connection at piers between the decks of two adjacent spans, providing a jointless bridge without continuity. The deck is made continuous across the pier, but the supporting beams or girders are not connected.

Key Definitions

Emulation

A type of design where the bridge and its elements are designed to resist forces in the same manner as a bridge built with conventional construction.

Guide Specification Format

Standard AASHTO Format

Sample Section

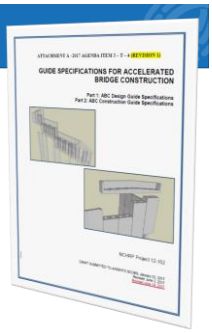
- Specifications on Left
- Commentary on Right
  - There is significant commentary
- Includes Notation definitions
- Includes references
- This is not an ABC Manual
  - Limited to Design and Construction "Specifications"
- Not a cookbook
  - You need to use engineering knowledge and judgement

Questions?

Workshop W-01

Implementation of the New AASHTO Guide Specifications for Accelerated Bridge Construction

- Module 2: General Design Provisions



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Design Responsibilities - PBEs

1.4.1 DESIGNER

The designer is responsible for the design of all elements and connections according to the AASHTO LRFD Bridge Design Specifications as supplemented in this Guide Specification.

The designer can detail a schematic specialty structure or component and require that the contractor complete the final design. This shall be clearly indicated on the plans and in the project specifications along with the specific performance and design parameters.

C1.4.1

Recommendations for the design of elements and connections are included in this Guide Specification.

Examples of this approach include but are not limited to precast culvert structures and proprietary retaining walls.

Design Responsibilities - PBEs

1.4.1.1 Designer's Role in Shipping and Handling of Elements

Project specifications shall require that the contractor be responsible for all lifting and handling of elements. The designer shall be responsible for reviewing the contractor's methods and calculations for shipping and handling for general conformance with the contract documents.

The designer shall investigate the shipping and handling requirements for very large or unusually shaped elements. Schematic erection plans and procedures shall be included in the project plans in these situations.

The investigation shall be limited to general equipment needs and stability. In some cases, the designer shall contact erection contractors to verify that an element can be erected. The designer shall also verify that the construction area is adequate to accommodate the required crane(s) and the delivery trucks.

C1.4.1.1

See Part 1, Section 1.4 and Part 2, Section 3.4 for more information on shipping and handling of elements. There are different configurations of shipping and lifting equipment for elements. The selection and design of these devices falls under the realm of "contractor means and methods".

Design Responsibilities - PBEs

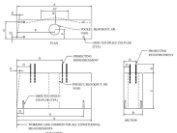
1.4.1.2 Establishment of Tolerances and Joints

The designer is responsible for the specification of element tolerances, erection tolerances, and joint width tolerances. Element tolerance details shall be shown on the plans or included in the project specifications. Erection tolerances shall be shown on a structure layout drawing, or included in the specifications. Joint width tolerances shall be included in the joint details.

C1.4.1.2

Recommended tolerances and detailing of tolerances is included in the document entitled: *Guidelines for Prefabricated Bridge Elements And Systems Tolerances* (Culmo, et al., 2017). This guideline is to be developed under NCHRP Project 12-98.

Some agencies allow contractors to fabricate elements onsite or near site. The tolerances specified in the above cited document may be followed, however, the owner may choose to specify different criteria. The cited document is recommended for a basis for establishment of owner specified tolerances.



Design Responsibilities -PBEs

1.4.1.3 Specification of Special Materials

The designer is responsible for the specification of specialized materials for use in prefabricated bridge construction. Performance or prescriptive specifications may be used.

C1.4.1.3

Performance specifications allow for more flexibility in the use of specialized materials. Examples include, but are not limited to:

- high early strength concrete
- non-shrink grout
- mechanical connectors

See Part 2 of this document for recommendations on performance specifications.

Design Responsibilities - PBEs

1.4.2 **CONTRACTOR**

In general, the contractor's role in the design of prefabricated elements is limited to means and methods of construction.

The Contractor may be required to complete the final design of the structure or components if specified on the contract plans.

C1.4.2

See Article 1.4.1 for examples of this approach.

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Design Responsibilities - PBEs

1.4.2.1 Temporary Works

All temporary works shall be designed by the contractor. The *AASHTO Guide Design Specifications for Bridge Temporary Works* shall be followed unless otherwise specified herein or by the owner.

For complex structures where the means of construction are not self-evident, the designer shall detail at least one feasible construction method on the plans.

C1.4.2.1

This approach is intended to allow for flexibility in construction and to allow the contractor to make use of equipment and materials that are on hand, thereby reducing project costs.

1.4.2.2 Element Shipping and Handling Calculations including Lifting Hardware

The contractor shall be responsible for the lifting and handling of prefabricated elements. This includes but is not limited to lifting equipment and hardware, crane sizes and delivery vehicles. The contractor shall also be required to verify that the lifting methods will not overstress the elements.

C1.4.2.2

See Part 2 of this Guide Specification for more information on element shipping and handling specifications.

CME

Designer Responsibilities - SPMTs

1.5.1 **DESIGNER**

The designer shall be responsible for the design of the entire bridge, verification that the bridge can be reasonably moved within the proposed work zone, and specifying the requirements of the contractor to complete the construction.

C1.5.1

The temporary support of the bridge in the staging area and the equipment/falsework used to move a bridge with SPMTs falls under the realm of "contractor means and methods". There are several brands of transporters in the market and falsework varies from project to project, therefore the designer should not be involved in a detailed design of these components.

1.5.1.1 Bridge Design

The designer shall make assumptions on the methods of lifting and moving the bridge using SPMTs. The designer shall take into account the assumed methods in all aspects of the design.

The designer shall clearly state all assumptions on the lifting and moving of the bridge on the design plans. The plans shall also state that variations in the lifting and moving methods which will require an evaluation of the bridge by the contractor.

C1.5.1.1

The designer shall design the bridge to resist the lifting and moving process based on the preliminary layout of the SPMTs.

CME

Designer Responsibilities - SPMTs

1.5.1.1.2 Analysis for Jacking Locations

The designer shall design the bridge system for the move. The design shall be based on the preliminary layout of the SPMTs. The system shall be evaluated for the following conditions:

- On the proposed temporary supports
- On the SPMT supports during the move
- On the SPMT supports during the move with the system displaced to the maximum specified twist

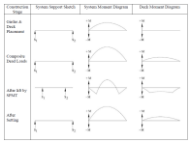
The design of the bridge shall meet or exceed all of the requirements specified in the AASHTO LRFD Bridge Design Specifications under all conditions during construction.

The designer shall account for dynamic effects during the move.

C1.5.1.1.2

The design of the bridge will most likely require changes to the beam designs and/or supplemental reinforcement in the structure. Cross frames and intermediate bearing stiffeners may be required at the temporary lift points. The designer shall identify the acceptable methods of connecting the falsework to the bridge system. The designer can detail a schematic of a basic pinned connection. The designer may also elect to allow bracing connections between the bridge system and the falsework to aid in stability of the SPMTs. If bracing connections are allowed, the plans shall clearly indicate the types of acceptable connections.

In most cases, the designer need only check the structure for vertical dynamic effects brought on by the movement of the bridge on the SPMTs. The document entitled "Guidelines for Dynamic Effects during Bridge System Installations" is recommended for use in this analysis.



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Designer Responsibilities - SPMTs

1.5.1.1.3 Determine Allowable Deformations

The designer shall analyze the bridge to determine the allowable twist in the structure during the bridge move. The analysis shall investigate at a minimum, the stresses in the beams, girder, deck, and barriers.

The designer shall analyze the bridge to determine the allowable differential support elevations during the bridge move for systems with three or more SPMT units. The analysis shall investigate at a minimum, the stresses in the beams, girder, deck, and barriers.

The combination of twist and differential support variations need not be studied unless specified by the owner.

C1.5.1.1.3

Twist is the variation in planar supports between adjacent SPMT support points. The contractor will need this value for use in monitoring the bridge during the move. The analysis for twist shall be based on a refined analysis. Grid analysis is not recommended for calculating deck stresses.

Differential support variations are related to SPMT bridge moves that have indeterminate support conditions. For example, a bridge portion that is proposed to be moved with three independent SPMTs has the potential for one SPMT to be higher or lower than the others. This analysis is essentially the same as analysis of a bridge for differential settlement.

This recommendation is based on the assumption that the maximum twist and maximum differential support variation would not occur at the same time. The owner may elect to require a combined analysis for special structures that may be sensitive to these combined deflections.

CME

Designer Responsibilities - SPMTs

1.5.1.2 Preliminary SPMT Investigation

The designer shall prepare a preliminary SPMT layout and investigate the potential staging area and travel path for the bridge move.

C1.5.1.2

The contractor shall be responsible for the final SPMT layout, falsework design and investigation of the travel path. The designer needs to determine the feasibility of the bridge move by investigating these features of the move.

CME

Designer Responsibilities - SPMTs

1.5.1.2.1 Preliminary SPMT Layout

SPMTs can be assembled in a number of different configurations. Figure 1.5.1.2.1-1 shows several configurations. The modular nature of SPMT units is that they can be joined together longitudinally and transversely to form a larger platform. The preliminary layout of the SPMT platform should be based on the following assumed criteria:

- Width of transporter: 8 ft.
- Length of transporter: Sum of axles * axle spacing
- Length of power pack: 12 ft.
  Number of axles line per unit: 4 and 6
- Number of tires per line: 4
- Axle line spacing within unit: 4.625 ft.
- Axle line spacing between adjacent units: 4.625 ft.
- Axle line load capacity: 30 tons
- Unit height: 48 in. min., 72 in. max.

C1.5.1.2.1

The layout of the SPMTs will establish the lifting points of the bridge move. This information is necessary to complete the design requirements specified in Article 1.5.1.1.

The dimensions and capacities of SPMT units vary depending on the manufacturer. The assumed criteria are based on typical units that are used and have proven to provide an adequate estimate of the final configuration. SPMT units can be linked longitudinally or laterally to achieve the number and configuration of axle lines required by the load.

Example: 650 ton load on the SPMT unit
Number of required axle lines = 650/30 = 21.7
Use 24 axle lines (rounded up)
12 axle lines double wide
Width: 8 feet*2 = 16 ft.
Length: 4.625*12 = 55.5 ft.
Approx. capacity = 30 tons*12*2 = 720 tons

Designers should consider rounding up the number of axle lines by approximately 10 percent as shown above to account for the potential of a local failure during a move (flat tires or hydraulic problems). In this case, the Designer should specify that the Contractor use a similar approach for the final layout of the SPMTs.

CME

Designer Responsibilities - SPMTs

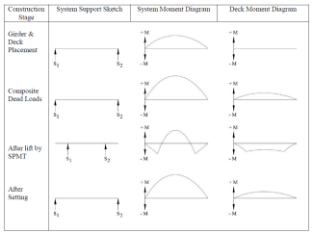
Other Designer Responsibilities

- Staging area
  - Show locations of temporary abutments
  - Recommend a foundation type for temporary abutments
  - Show support locations
- Travel Path
  - Check for clearances
  - Geometry checks (flatness)
- Bridge site geometry
  - Check for vertical issues (crowns, cross slopes, etc.)

CME

Designer Responsibilities - SPMTs

Support Type A



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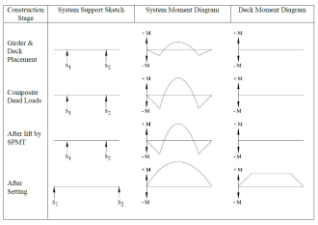
Designer Responsibilities - SPMTs

Support Type B



CME

Contractor Responsibilities - SPMTs

1.5.2 CONTRACTOR

The Contractor is responsible for the final design and detailing of the temporary support structure for the bridge in the staging area, the equipment/falsework used to move a bridge, the final layout of the SPMTs, and verification of the adequacy of the proposed travel path.

The Contractor is also responsible for maintaining the integrity of the bridge during all aspects of the bridge fabrication and move.

C1.5.2

All of the items listed fall under the realm of "contractor means and methods". There are several brands of transporters in the market and falsework varies from project to project based on available materials.

In many cases, the Contractor will contract with a specialty engineer or heavy lift engineer for falsework and SPMT designs. Therefore, any reference to the "Contractor" in this specification would include the specialty engineer or heavy lift engineer.

In the case where the designer specifies the lift points and allowable twist, the Contractor needs to ensure that these specifications are followed during all phases of the move. If these parameters are not specified, the Contractor will be responsible for all aspects of the bridge move including the calculation of lifting stresses and the allowable twist determination.

CME

Contractor Responsibilities - SPMTs

Other Contractor Responsibilities

- Staging area
  - Design of Temporary Abutments
- Travel Path
  - Final configuration of SPMTs
  - Travel Path geometry analysis
  - Monitoring system (to verify designer specified twist limits)

CME

Contractor Responsibilities - SPMTs

1.5.2.4 Bridge Analysis

A detailed analysis of the bridge structure by the contractor may not be required, provided that the final SPMT layout and falsework meets all the design assumptions that are indicated on the plans.

A full analysis of the bridge structure would be warranted under the following conditions:

- If the plans do not specify preliminary SPMT layout or pick points
- Lifting the bridge at pick points that are not consistent with the assumed locations listed on the plans
- Use of falsework that does not provide support that is consistent with the assumed support conditions
- If the project documents specify that an independent analysis is required.

If analysis of the bridge is required, the analysis shall at a minimum meet the requirements of Articles 1.5.1.1.2 and 1.5.1.1.3, or as specified by the owner.

C1.5.2.4

The basis of this provision is to reduce the level of effort for the Contractor for typical bridge moves, and to avoid duplication of effort. In many cases, the Contractor can meet the assumed preliminary SPMT layout assumptions.

The Contractor needs to demonstrate to the Designer that the falsework can provide the required support conditions.

This approach may be warranted for complex bridges that are sensitive to minor variations in the layout of the SPMTs and falsework.

CME

Designer Responsibilities Lateral Slides

1.6.1.2 Preliminary Jacking Forces and Jacking Locations

The designer shall identify the acceptable jacking/support locations on the bridge. This is applicable to the horizontal jacking points, vertical jacking points, and the support locations for the sliding devices.

The assumed jacking/support locations and associated forces shall be shown on the plans.

CME

Designer Responsibilities Lateral Slides

1.6.1.4 Resolution of Sliding Forces

The approach to the preliminary layout of the slide system shall account for the resolution of friction forces. Two approaches can be used for friction force management and the interaction of the temporary falsework and the bridge substructure/foundation.

- Falsework attached to the bridge substructure: Figure 1.6.1.4-1 is a schematic representation of this approach. The connection between the falsework and the substructure shall be designed to resist the total sliding friction force in the system. If this assumption is used for the design of the substructure, the designer shall indicate the need for this connection in the contract documents.
- Falsework not attached to the bridge substructure: Figure 1.6.1.4-2 is a schematic representation of this approach. The falsework and the substructure shall be designed to resist the total sliding friction force in the system.

C1.6.1.4

In this scenario, the forces in the lateral slide system form a continuous force loop resulting near zero force in the bridge substructure and the falsework structure. This will result in no significant changes to the design of the bridge substructures. The design implications are limited to the connection of the falsework and a load path for the friction forces to flow through the bridge seat area.

This approach will affect the design of the substructures foundation and the design of the temporary falsework. The friction forces will generate sliding and overturning forces that will need to be addressed in the design of both the substructures and falsework.

CME

Designer Responsibilities Lateral Slides

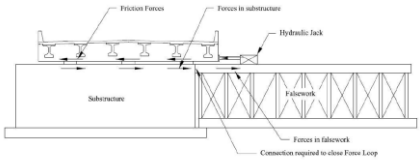


Figure 1.6.1.4-1 Falsework Connected to Substructure (Resolution of Friction Forces)

CME

Designer Responsibilities Lateral Slides

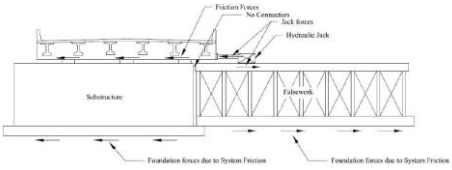


Figure 1.6.1.4-2 Falsework not Connected to Substructure (Resulting in Additional Foundation Forces)

CME

Designer Responsibilities Lateral Slides

Other Designer Responsibilities

- Geotechnical Design
  - Recommend foundation type for temporary supports
- Jacking forces
  - Schematic layout of slide system
  - Design for estimated temporary forces acting on substructure and superstructure
- Specifications
  - Require jacking plan submission
  - Allowance for alternate jacking systems and supports

CME

Contractor Responsibilities Lateral Slides

- Design of falsework
- Design of sliding system
- Determine final jacking forces
- Detailing of slide system

Loads and Load Combinations

2.4 LOADS AND LOAD COMBINATIONS

Prefabricated bridge elements and systems shall be designed to meet all of the load and load combination requirements of the AASHTO LRFD Bridge Design Specifications based on the completed structure.

C2.4

The requirements of this article are based on the emulative design process, where the elements are designed and connected to behave as a structure built using conventional construction methods.

Loads and Load Combinations

2.4.1.1 Shipping and Handling

All prefabricated elements shall be analyzed for shipping and handling forces during fabrication, shipping and erection.

The following load combinations shall be used for shipping and handling calculations:

Service I = γ_DDL (2.4.1.1-1)
Strength I = γ_p γ_D DL (2.4.1.1-2)

where:

γ_D = dynamic dead load factor for handling of prefabricated elements.
γ_p = dead load factor as specified in the *AASHTO LRFD Bridge Design Specifications*

The following values for γ_D are recommended:

C2.4.1.1

The Contractor is typically responsible for shipping and handling of elements, since there are many ways to lift and secure an element during fabrication and erection (see Article 1.4.2). For example, a deck panel can be lifted with a 4 point pick with sloped slings, or an 8 point pick with a spreader beam. Both methods can be acceptable, however the stresses imparted in the element would be different. Therefore, the handling of prefabricated elements should fall under the realm of "contractor means and methods".

All of these provisions are not applicable to girders (form finish), however values for yard handling and shipping may be used. Special care needs to be exercised with girder shipping and handling methods, especially with regard to long slender elements that may tip or buckle during shipping and handling (PCI, 2016).

Loads and Load Combinations

1. Dead load factors for removal of precast elements from forms:

Form finish	γ_D
Flat, with removable side forms no false joints or reveals	1.3
Flat, with false joints and/or reveals	1.4
Fluted with proper draft (tees, channels, fluted panels)	1.6
Sculptured	1.7

The values contained herein are based on the *Precast/Prestressed Concrete Institute Design Handbook* MNL-120 (2010).

2. Dead load factors for yard handling and erection: $\gamma_D = 1.2$

3. Dead load factor for shipping: $\gamma_D = 1.5$

These values for handling, erection and shipping account for dynamic effects during construction. They would be applicable to prefabricated elements fabricated with any material (concrete, steel, etc.).

Loads and Load Combinations

SPMT Installations

2.4.2.1 Service Load Combination for the Bridge

The Service I Load Combination as specified in the *AASHTO LRFD Bridge Design Specifications* shall be checked to control cracking in the deck during the bridge move, cracking in prestressed concrete beams subjected to negative bending during the move, and the bridge falsework (if designed using working stress methods). The following load combination shall be used:

Service I = 1.0DL + 0.4(VDL) (2.4.2.1-1)

where:

DL = Dead loads applied to the bridge, falsework, and SPMTs
VDL = Vertical dynamic loads applied to the bridge, falsework, and SPMTs

Other applicable loads and load factors specified in the *AASHTO LRFD Bridge Design Specifications* for the Service I Load Combination shall also be applied.

C2.4.2.1

This limit state can also be used to check the SPMTs for working limit loads.

Horizontal dynamic forces will not affect the design of the bridge since it is considered a rigid body. Therefore they are excluded from this check.

The basis of this provision is the document entitled *Guidelines for Dynamic Effects during Bridge System Installations* (Culmo, et al. (2017), which is recommended for use in calculating dynamic effects during SPMT bridge moves.

Research conducted for the development of this document involved establishing the upper bound for vertical dynamic loads with a loaded SPMT traversing uneven ground at a travel speed of approximately 4 miles per hour. The vertical dynamic effects are a function of the square of the travel speed of the SPMT. The load factor of 0.40 accounts for a travel speed value of approximately 2.5 miles per hour ($2.5^2/4^2$), which is a more realistic safe maximum travel speed. Designers and owners may elect to specify lower travel speeds along with corresponding load factors. Project specifications shall limit the travel speed to this level in order to provide adequate service limit state performance.

Loads and Load Combinations

SPMT Installations

2.4.2.2 Strength Load Combination for the Bridge

The Strength I Load Combination as specified in the *AASHTO LRFD Bridge Design Specifications* shall be used to check the flexural capacity of the beams and the bridge. The following combination shall be used:

Strength Load = γ_pDL + 1.0(VDL) (2.4.2.2-1)

where:

γ_p = dead load factor as specified in the AASHTO LRFD Bridge Design Specifications
DL = Dead loads applied to the bridge, falsework, and SPMTs
VDL = Vertical dynamic loads applied to the bridge, falsework, and SPMTs

Other applicable loads and load factors specified in the *AASHTO LRFD Bridge Design Specifications* for the Strength I Load Combination shall also be applied.

C2.4.2.2

The basis of this provision is the document entitled *Guidelines for Dynamic Effects during Bridge System Installations* (Culmo, et al. (2017), which is recommended for use in calculating dynamic loads during SPMT bridge moves.

Horizontal dynamic forces will not affect the design of the bridge since it is considered a rigid body. Therefore they are excluded from the check.

The 0.4 factor for travel speed in Equation 2.4.2.1-1 is replaced with a load factor of 1.0 in this equation, since the loads calculated in Article 2.4.2.1 represent an upper bound for vertical forces.

Loads and Load Combinations

SPMT Installations

2.4.2.3 Load Combinations for Falsework and SPMTs

The falsework and SPMTs shall be designed according to the *AASHTO Guide Design Specifications for Temporary Works* (2017). The following load combination shall be used:

$$ASD_{other} = 1.0DL + 0.75(HDL) + 0.40(VDL) \quad (2.4.2.3\text{-}1)$$

where:

DL = *Dead loads applied to the bridge, falsework, and SPMTs*

HDL = *Horizontal dynamic loads applied to the bridge, falsework, and SPMTs*

VDL = *Vertical dynamic loads applied to the bridge, falsework, and SPMTs*

Other applicable loads and load factors as specified in Load Combination ASD_{12} shall also be applied to the falsework and SPMTs.

C2.4.2.3

The basis of this provision is the document entitled *Guidelines for Dynamic Effects during Bridge System Installations* (Culmo, et al. (2017), which is recommended for use in calculating dynamic loads during SPMT bridge moves.

The primary cause of horizontal dynamic load (HDL) is the acceleration and deceleration of the SPMT. An emergency stop would represent the largest horizontal dynamic force, which is the basis for the loads calculated in the cited guideline. This is similar to a seismic load; however the probability of occurrence is higher. Load Combination ASD_{12} of the *AASHTO Guide Design Specifications for Bridge Temporary Works* (2017) accounts for seismic forces. A load factor of 0.53 is specified for seismic loads. The load factor of 0.75 in this provision is an approximate value based on the anticipated probability of occurrence.

See C2.4.2.1 for discussion regarding the 0.40 factor for vertical dynamic load.

CME

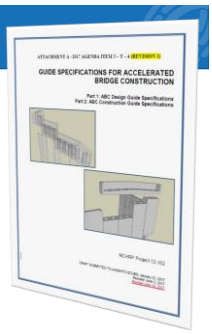
Questions?

CME

Workshop W-01

Implementation of the New AASHTO Guide Specifications for Accelerated Bridge Construction

- Module 3: Design of Prefabricated Elements



Michael Culmo, PE
CME Associates, Inc.

Note

This investigation is sponsored by TRB under the NCHRP Program. Data reported is work in progress. The contents of this presentation has not been reviewed by the project panel or NCHRP, nor do they constitute a standard, specification, or regulation.

Section 3: Design of PBEs

Largest Section in the Guide Specification

- 85 pages

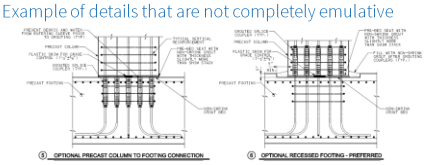
Seismic combined with non-seismic provisions

- The team originally considered a separate seismic section
- Panel decided that a combined section was preferred
  - Many details and connections are applicable to both (avoid duplication of provisions)
  - Consistent with the AASHTO LRFD Bridge Design Specifications
- This presentation will focus on non-seismic details
  - Stuart Bennion will cover seismic in the next module

Section 3: Design of PBEs

Most common design approach

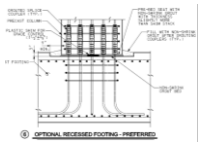
- Emulative design
  - The same as non-prefabricated element
- Example of details that are not completely emulative



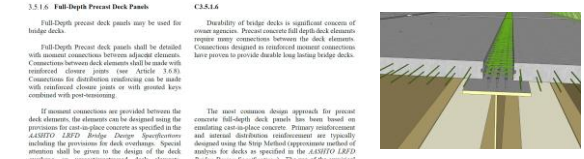
Couplers and Design

Couplers have a minor impact on the design

- Concrete Cover on non-coupler designs
  - Design section based on required cover
  - Detail bars accordingly
- Coupler Designs
  - The diameter of the coupler is larger than the bar
  - There are multiple coupler manufacturers and each has different dimensions (diameter)
  - In order to maintain cover over the coupler, the designer needs to:
    - Move the rebar cage for design
    - The distance has to be bracket all manufacturers (more on this in Module 5)



PCFD Deck Panel Design



3.5.1.6 **Full-Depth Precast Deck Panels**

Full-Depth precast deck panels may be used for bridge decks.

Full-Depth Precast deck panels shall be detailed with moment connections between adjacent elements. Connections between deck elements shall be made with reinforced closure joints (see Article 3.6.8). Connections for distribution reinforcing can be made with reinforced closure joints or with grouted keys combined with post-tensioning.

If moment connections are provided between the deck elements, the elements can be designed using the provisions for cast-in-place concrete as specified in the *AASHTO LRFD Bridge Design Specifications* including the provisions for deck overhangs. Special attention shall be given to the design of the deck overhang on precast/prestressed deck elements. Designers shall check the development length of the prestressing strand and its impact on the design sections for barrier impact. Designers shall also check serviceability requirements and requirements for deck overhangs.

C3.5.1.6

Durability of bridge decks is significant concern of owner agencies. Precast concrete full depth deck elements require many connections between the deck elements. Connections designed as reinforced moment connections have proven to provide durable long lasting bridge decks.

The most common design approach for precast concrete full-depth deck panels has been based on emulating cast-in-place concrete. Primary reinforcement and lateral distribution reinforcement are typically designed using the Strip Method (approximate method of analysis for decks as specified in the *AASHTO LRFD Bridge Design Specifications*). The use of the empirical design method is not allowed for precast full depth deck elements. The design of the deck overhang is typically controlled by the forces due to a vehicle impact on the barrier, which will generate large forces (bending combined with transverse tension) in the deck at the base of the barrier. Mild reinforcement may be required to supplement the prestressing strand if the strand is not developed at this section.

PCFD Deck Panel Design

Precast concrete full depth deck panels can be made composite with the supporting beams or girders with shear connector through the use of pocket connections (see Article 3.6.8.3) or within a closure joint.

Pockets can be formed in the precast deck elements at regular intervals. Shear connectors that project from the supporting beam project into the pockets. The pockets are then filled with grouts or other cementitious materials to form the connection. The gap between the underside of the deck element and the top of the beam (often referred to as the "haunch") is also filled with grout or cementitious material. Some agencies have detailed a closure joint over the supporting beam which provides a location for placement of shear connectors.

Research has shown that partial depth "hidden" pockets can provide full composite action (Badie and Tadros, 2008). These pockets are open on the underside of the deck panel, but closed on the top surface. Shear connectors are installed in pre-determined locations on the girders prior to placement of the deck panels. Once the panels are set, the pockets are filled with grout through ports in each pocket.

The PCI Report (2011) on full depth deck panels is a good resource for more information on design of full depth deck elements. This document also contains numerous references to the many studies that have been completed on precast concrete full depth decks.

CME

Adjacent Precast Deck Beam Design

3.5.1.7 Adjacent Deck Beams

Adjacent deck beams consist of precast beam elements where the top of the beam forms the riding surface of the bridge. The beams are placed adjacent to each other to form the bridge superstructure. The adjacent deck beams shall be designed according to provisions of the *AASHTO LRFD Bridge Design Specifications*.

The connections between the adjacent deck beams can be designed as reinforced moment connections or as a shear connection. The type of connection will affect the live load distribution factor used in the design of the beam. Moment connections are recommended for bridges with a high volume of truck traffic.

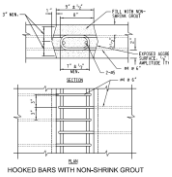
The design of the top flange reinforcing in adjacent deck beams with projecting flanges can be based on emulation of a cast-in-place concrete deck as specified in the *AASHTO LRFD Bridge Design Specifications*. The provisions for maximum skew of primary reinforcement are applicable to modular deck beams.

C3.5.1.7

Adjacent deck beams have proven to be cost effective structures. They include precast box beams, voided slabs, solid slabs, double tees, deck bulb tees, and inverted tees.

The *AASHTO LRFD Bridge Design Specifications* contain provisions for live load distribution that are based on the type of connection between the deck beams. If an adequate moment connection is detailed between the deck beams, they can be assumed to be "sufficiently connected to act as a unit". Deck beams designed with a shear connection have proven to be durable for bridges with low volumes of truck traffic; however these bridges have not performed as well on high volume roadways. Therefore, a more robust moment connection is recommended for increased durability on high volume roadways.

The behavior of an integral deck on a flanged deck beam is akin to a cast-in-place deck on an independent beam. Primary reinforcement and distribution reinforcement are typically designed using the Strip Method. Designers shall also design the deck for serviceability and for deck overhang reinforcement.



CME

Modular Deck Beam Design

3.5.2.2 Modular Decked Beams

Modular steel decked beams consist of two or more steel beams that are connected to a concrete deck which is cast in a fabrication facility (off-site or near site). This forms a modular unit that is similar to a precast double tee beam.

The design of modular decked beams shall follow the provisions in the *AASHTO LRFD Bridge Design Specifications* for beams with a cast-in-place concrete deck. The deck level connections between the adjacent deck beams shall be designed as reinforced moment connections. The provisions for maximum skew of primary reinforcement are applicable to modular deck beams.

C3.5.2.2

Modular decked beams (MDB) are also referred to as Prefabricated Bridge Units (PBUs) in some states. The most common modular decked beam has a concrete deck that is cast in a fabrication facility. To date, all MDBs have been made with steel beams. It is possible to make a MDB with concrete beams. The provisions contained in this article would still be applicable to a MDB made with concrete beams. It is also possible to use an orthotropic steel deck connected to the top of a steel beam. This steel deck option is a specialized system that is not well documented, therefore this article focuses on the concrete deck option.

The behavior of an integral deck on steel beams is akin to a cast-in-place deck on independent beams. Primary reinforcement and distribution reinforcement are typically designed using the Strip Method as specified under the "Alternate Methods of Analysis" article in Chapter 4 of the *AASHTO LRFD Bridge Design Specifications*. Designers shall also design the deck for serviceability and deck overhang reinforcement.

CME

Precast Railing Design

3.5.1.9 Precast Concrete Railings

Precast concrete railing (also referred to as a barrier or parapet) shall be designed to emulate cast-in-place concrete according to the requirements of the *AASHTO LRFD Bridge Design Specifications*. The connection of the precast railing to the bridge deck is critical to the physical performance of the bridge.

C3.5.1.9

The *AASHTO LRFD Bridge Design Specifications* requires that traffic barriers and railings be crashworthy. Precast Barriers would be included in this requirement. Appendix A13 of this specification can be used for the preliminary design of the barrier or railing. Barriers or railings shall be physically crash tested prior to use.

In most cases, the design of the reinforcement within the barrier can be similar to cast-in-place concrete. The spacing of deck connections in precast barriers may be somewhat larger than typical cast-in-place barriers, therefore adjustments to the design internal reinforcements may be required.

The durability of the barrier or railing is an important part of the design. It is important to prevent gutter flow water from migrating under the barrier where it can deteriorate the anchor bolts and deck. Special detailing of joints at the base of the railing is required. Consideration

CME

Connection Design

3.6.1 GENERAL PROVISIONS

Provisions in Section 3.6 are not intended to preclude the use of other preferred details as approved by the Owner and so long the designer can demonstrate adequate load transfer as well as consideration for compatibility of deformations and constitutive relations of materials.

The designer shall design and detail all connections between prefabricated elements. The connections shall be structurally adequate and meet the durability requirements of the Owner.

The connections between elements shall be designed to resist all the forces that are transmitted from one element to the adjoining element.

Designers shall consider the constructability of all connections based on the anticipated construction methods.

C3.6.1

Specific connection provisions in Section 3.6 correspond to details for which adequate structural performance has been demonstrated through experimental and analytical research. While such details are recommended in the proposed guide specifications, they are not exclusive of other details or a combination thereof provided that the designer can demonstrate, using fundamental principles of mechanics, the intended structural performance of the system.

The importance of durability cannot be overstated. It is imperative that the connection of elements achieve durability that is equal to or better than the connected elements.

Connections in emulative design and detailing of precast concrete structures replace the typical construction joints in a bridge. The construction joints are either replaced with a mechanical connection or a cast-in-place closure joint.

Constructability should be checked for all connections to ensure that the elements can be placed and connected without conflicts.

CME

Connection Design

3.6.2 CAST-IN-PLACE CONCRETE CLOSURE JOINTS USING LAPPED BAR REINFORCEMENT

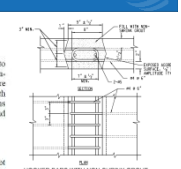
Connections made with cast-in-place concrete closure joints shall be designed to resist the forces acting on them. The provisions contained herein are based on achieving a connection that can develop the nominal resistance for the Strength Load Combination in a fashion that emulates cast-in-place concrete. The designer shall identify the load path through the element to provide proper detailing for force transfer to adjacent elements.

The design of the joints shall satisfy the strength and serviceability requirements specified in the *AASHTO LRFD Bridge Design Specifications*.

C3.6.2

Connections included in this section are intended to emulate reinforced concrete connections. Other non-emulative connections such as post-tensioned joints are included in this specification. Closure joints made with cast-in-place concrete can be used in a variety of locations on a bridge including superstructure connections and substructure connections.

An exception for this could be a connection that is not covered in the *AASHTO LRFD Bridge Design Specifications*, but thoroughly tested in a simulated laboratory experiment. Designers should obtain the Owner's approval for use of any design connection that is based on laboratory testing.



CME

Connection Design

Connections may be designed using an initial compressive strength of concrete in order to open the bridge to vehicular loading prior to achieving final strength. The contract documents shall clearly indicate the interim concrete strength required and the allowable loading on the connection at the interim strength.

Connections shall be designed and detailed based upon the fabrication and erection tolerances shown in the contract documents. This shall include the dimensions of the joint and the spacing and length of lap splice reinforcing.

Tolerances can also affect lap splices of reinforcing. The detailing of the reinforcing in the joint shall accommodate tolerances via spacing limitations and length of laps.

In certain ABC projects with very short construction time constraints, it is reasonable to design the connections using an interim concrete design compressive strength (less than the final specified design compressive strength). This is based on the fact that most cement based products require multiple days to gain final strength, which may not be consistent with the proposed ABC schedule. For example, a connection can be specified to have a final concrete compressive strength of 6 ksi, but the connection can be designed based on an interim concrete compressive strength of 3 ksi concrete, thereby allowing construction to proceed at a faster pace.

The acknowledgement of tolerances leads to wider closure joints and longer projection lengths of reinforcing bars. Specifying large tolerances will lead to larger closure joints; however specifying very small tolerances will lead to higher fabrication costs. Designers should strive to strike a balance between these two extremes. The publication entitled *"Guidelines for Prefabricated Elements and Systems Tolerances"* developed under NCHRP Project 12-98 provides further guidance on this subject.

CME

Connection Design

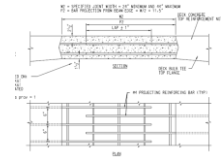
3.6.2.1 Reinforcing Bars in Closure Joints

If straight reinforcing bars projecting from one element are lapped with other straight reinforcing bars projecting from another element within a closure joint, the design of the lap splice within the closure joint shall be as specified in the *AASHTO LRFD Bridge Design Specifications*.

The spacing and length of lap splices with projecting bars shall account for the specified fabrication and erection tolerances so that the minimum non-contact splice dimension is not exceeded.

C3.6.2.1

The design of lap splices for closure joint areas is the same as for construction joints in cast-in-place concrete construction.



② STRAIGHT BARS WITH 4 KSI CONCRETE

CME

Connection Design

3.6.2.2 Hooked Reinforcing Bars in Closure Joints

Hooked bars may be lapped within the closure joint. The minimum length of the lap between adjacent hooked reinforcing bars shall be equal to the hook development length ℓ_{dh} specified in *AASHTO LRFD Bridge Design Specifications*. The detailing of the hooks shall include at least one transverse bar of equal size set within the inside radius and in contact with each hook, which would require a minimum of two transverse bars per lapped hook connection.

In a non-contact hooked bar splice, the center-to-center spacing of the spliced bars shall not exceed 4 inches

C3.6.2.2

The *AASHTO LRFD Bridge Design Specifications* is silent with regard to provisions for lapped hooked bars; however the provisions of this article have been successfully used by designers for many years in conventional construction. The typical adjustments for lap splices outlined in the *AASHTO LRFD Bridge Design Specifications* are based on straight lapped reinforcing bars. The failure mechanism of hooked bars is different than straight bars, therefore the provisions for lap spliced bar need not be applied. Testing by French et al. (2011) Brush (2004) and Sheng et al. (2013) has shown that the standard development length of a hooked bar is adequate. These tests included the use of a bearing bar within the hook.

This requirement is consistent with testing noted above. Uniform projecting bar spacing is recommended with a maximum spacing of 8 inches on center in each element (see Figure 3.6.2.2-1). By using this approach, the minimum spacing requirement should not be exceeded even after accounting for tolerances. If an element is placed in an offset (but within tolerance) location, the maximum non-contact lap spacing of one adjacent bar might be exceeded on one side, but would end up closer to the next adjacent bar on the other side, thereby meeting the requirements of this provision. Figure 3.6.2.2-1 shows the maximum recommended spacing of lapped hooked bars.

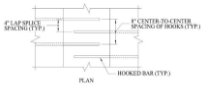


Figure 3.6.2.2-1 Hooked Lap Splice

CME

Connection Design

3.6.2.3 Headed and Mechanically Anchored Deformed Reinforcing Bars in Closure Joints

Headed and mechanically anchored deformed reinforcing bars meeting the requirements of ASTM A970 shall be considered effective in developing and splicing bars in tension.

3.6.2.3.1 Development of Headed and Mechanically Anchored Deformed Reinforcing Bars in Tension

The basic development length for headed and mechanically anchored deformed reinforcing bars in tension, ℓ_{dt}, measured from the underside of the head to the point of development shall satisfy:

$$\ell_{dt} \geq \frac{0.31\, f_y d_b}{\sqrt{f'_c}} \qquad (3.6.2.3.1\text{-}1)$$

but not less than:

- 6 in.

where all of the following criteria are met:

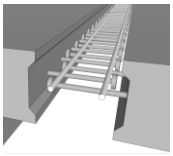
C3.6.2.3

The behavior of headed and mechanically anchored deformed reinforcing bars is similar to that of hooked bars. The force in the bars is transferred to the concrete through a combination of a bearing force at the head and bond forces along the bar.

C3.6.2.3.1

The basis of this provision including limitations on concrete strength, steel yield strength, bar size, normal weight concrete, bar cover and clear spacing is the *Building Code Requirements for Structural Concrete (ACI-318-14)* adjusted for *AASHTO LRFD Bridge Design Specifications* units. Testing results by French et al. (2011) and Oesterle et al. (2009) completed on the use of these bars is consistent with these provisions.

The clear cover and clear spacing requirements are based on dimensions measured to the bar, not the head; however the head is considered to be part of the bar for the purposes of satisfying the specified cover requirements in the *AASHTO LRFD Bridge Design Specifications*.



CME

Connection Design

where all of the following criteria are met:

- f'_c used in the calculation shall not exceed 6 ksi;
- Concrete shall be normal weight;
- Clear cover for the bar shall not be less than $3d_b$;
- Clear spacing between bars being developed shall not be less than $4d_b$;
- f_y shall not exceed 60 ksi;
- Bar size shall not exceed No. 11;
- A_{brg} shall not be less than $4A_b$;

where:

f'_c = nominal compressive stress of concrete (ksi)
f_y = specified minimum yield stress of reinforcing bars (ksi)
A_{brg} = net bearing area under the head of the bar
A_b = area of individual bar (in^2)
d_b = nominal diameter of a reinforcing bar (in.)

The basic development length, ℓ_{dt}, shall be multiplied by the following factors:

- Epoxy coated reinforcement 1.2
- Anchorage or development for the full yield stress of reinforcement is not required, or where reinforcement in flexural members is in excess of that required by analysis ..... $\frac{(A_s\ required)}{(A_s\ provided)}$

CME

Connection Design

3.6.2.3.2 Development of Headed and Mechanically Anchored Deformed Reinforcing Bars in Compression

The heads on the bars shall not be considered effective in developing bars in compression.

C3.6.2.3.2

Data is not available that demonstrates that the use of headed bars adds significantly to the anchorage strength in compression. The deformed portion of the bar can be considered in the development of the bar in compression.

CME

Connection Design

3.6.2.3.3 Splices of Headed and Mechanically Anchored Deformed Reinforcing Bars in Tension

The length of non-contact tension lap splice of headed and mechanically anchored deformed reinforcing, l_{dh}, measured between the underside of the opposing heads of the spliced bars shall satisfy:

$$l_{dh} \geq l_{dh} + 0.708S_b \quad (3.6.2.3.3\text{-}1)$$

where the following criterion is met:

$$S_b \leq 2c \quad (3.6.2.3.3\text{-}2)$$

where:

l_{dh} = basic development length for headed and mechanically anchored deformed reinforcing bars in tension as specified in Article 3.6.2.3.1 (in.)

c = the dimension measured from the center of the bar to the nearest surface of the element (in.)

S_b = center-to-center spacing of spliced headed and mechanically anchored deformed reinforcing bars (in.)

C3.6.2.3.3

Research on the use of spliced headed bars by Thompson et al. (2003) included the study of non-contact lap splices via strut and tie modeling. A strut angle of 55° was recommended. Figure C3.6.2.3.3-1 shows the assumed strut geometry. The adjustment to development length of the headed bars is simply derived from this geometry.

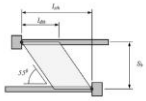


Figure C3.6.2.3.3-1 Strut and Tie Model for Non-contact Lapped Headed Reinforcing Bars

UHPC Connection Design

3.6.2.4.1 Development of Deformed Reinforcing bars in Tension in UHPC

The basic development length for straight or hooked deformed bars cast into UHPC, l_{db}, shall satisfy:

For reinforcing bars with:

$$f_y \leq 75 \text{ ksi}$$

$$l_{db} \geq 8d_b \quad (3.6.2.4.1\text{-}1)$$

where:

d_b = nominal diameter of bar (in.)

f_y = specified yield stress of reinforcing bars (ksi)

For reinforcing bars with:

$$75 \text{ ksi} < f_y \leq 100 \text{ ksi}$$

$$l_{db} \geq 10d_b \quad (3.6.2.4.1\text{-}2)$$

where all of the following criteria are met:

- Field cast UHPC shall contain a minimum of 2 percent (by volume) steel fiber reinforcement
- The minimum compressive strength of the UHPC shall be greater or equal to 14 ksi.
- Bar size shall not exceed No. 8.
- Clear cover for the bar shall not be less than $3d_b$.

The basic development length, l_{db}, shall be increased by $2d_b$ if:

$$2d_b \geq \text{minimum clear cover} + 3d_b$$

C3.6.2.4.1

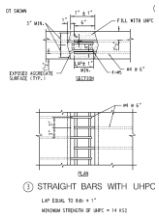
These provisions are applicable for all types of steel reinforcement and for epoxy coated bars. Adjustments noted in the *AASHTO LRFD Bridge Design Specifications* are not applicable to these provisions including increases in development length for epoxy coated bars.

The values for straight bars are conservatively used for hooked bars based on limited testing on hooked bars in UHPC.

The compressive stress of 14 ksi is set to facilitate the use of UHPC in Accelerated Construction when early application of construction loads to newly completed connections is advantageous. The specified minimum compressive strength at 28 days should still be specified.

The limitation on bar size is based on limited testing data on larger bars.

A decrease in the cover results in reduced confinement of the bar and then an increase in the required development length.



UHPC Connection Design

3.6.2.4.2 Splices of Lapped Reinforcing Bars in Tension in UHPC

The splice length of straight and hooked deformed reinforcing bars l_{ds} in tension that are cast into UHPC shall satisfy:

$$l_{ds} \geq 0.75l_{db} \quad (3.6.2.4.2\text{-}1)$$

where all of the following criteria are met:

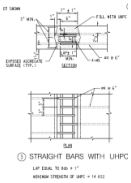
- The maximum space between adjacent spliced bars in a non-contact lap splice shall not exceed l_{ds}.
- Precast component interfaces onto which the field-cast UHPC will bond shall be detailed to include shear keys that project into the prefabricated element with exposed aggregate or roughened finish.

C3.6.2.4.2

Research by Graybeal et al. (2010) has demonstrated that passive reinforcement embedded into a connection and spliced with adjoining bars to have a lap of $0.75l_{db}$, sustain static and cyclic loads that cause rupture of the reinforcing bar outside of the connection.

The values for straight bars are conservatively used for hooked bars based on limited testing on hooked bars in UHPC.

The shear key facilitates compression strut transfer of applied loads without relying on dowel action of reinforcing bars.



UHPC Connection Design

3.6.2.4.3 Minimum Spacing of Reinforcing Bars in UHPC

The clear spacing between adjacent reinforcing bars around which UHPC must flow during casting shall be at least 1.5 times the length of the longest fiber reinforcement in the UHPC.

C3.6.2.4.3

Tight spacing of discrete reinforcements can restrict the flow of UHPC during casting and can impede the uniform distribution of fiber reinforcement throughout the component. The most common steel fiber deployed in UHPC applications is a 0.008-inch diameter by 0.5-inch long straight fiber, therefore a minimum spacing of 0.75-inch would be required. If fiber length is not specified, a conservative value of $2d_b$ may be used.

This provision is not applicable to the space between contact lap splices and bundled bars. The contact lap splicing of two reinforcing elements in combination with the allowance for appropriate adjacent clear spacing is an acceptable practice.

UHPC Connection Design

3.6.5.5 Development of Deformed Steel Bars in Corrugated Steel Ducts using UHPC

Ducted substructure connections shall be detailed to fully develop the ultimate strength of the embedded deformed steel reinforcing bar. Failure modes to consider include bar pullout from field-cast UHPC, duct pullout from conventional concrete, conventional concrete conical failure around duct, and bar rupture.

The development length for straight deformed bars cast into UHPC within a corrugated steel post-tensioning duct, l_{dsd}, shall satisfy:

For reinforcing bars with:

$$f_y \leq 75 \text{ ksi}$$

$$l_{dsd} \geq 8d_b \quad (3.6.5.5\text{-}1)$$

C3.6.5.5

The development of deformed reinforcing bars commonly used as longitudinal reinforcement in substructure components can be completed in shorter distances when embedded in field cast UHPC contained in ducted connections. Static and seismic testing has demonstrated the emulative nature of these connections (Tazarv and Saiidi (2013)). Tests were completed on field-cast UHPC with 2-percent (by volume) steel fiber reinforced, compressive strengths above 19 ksi and bar sizes ranging from No. 8 to No. 11. Appropriate detailing of the duct and the surrounding precast concrete are critical to the overall performance of the connection.

Mechanical Connectors

3.6.4.2 Type 1 Mechanical Connectors

Type 1 Mechanical Connectors are used to connect two in-line reinforcing bars. They shall be capable of developing 125% of the specified yield strength of the spliced bars.

Type 1 mechanical connectors can be used for most applications in which transfer of axial forces from one reinforcing bar into another reinforcing bar is required. The specification requirements for Type 1 mechanical connectors shall be according to the provisions for "Mechanical Connections" in the *AASHTO LRFD Bridge Design Specifications*, including the limitation on their use in columns.

C3.6.4.2

The *AASHTO LRFD Bridge Design Specifications* refer to this as a full "mechanical connection" and contain provisions for allowable slip within the connector as a function of bar size.

Mechanical Connectors

3.6.4.3 **Type 2 Mechanical Connectors**

Type 2 Mechanical Connectors are used to connect two in-line reinforcing bars. They shall be capable of developing 100% of the specified static tensile strength of the spliced bars.

Type 2 mechanical connectors shall be permitted in the following locations and conditions:

- Outside of the plastic hinge regions.
- Inside plastic hinge regions for Seismic Zone 1/ SDC A or inside of plastic hinge regions for Seismic Zone 2/ SDC B with the Owner's approval.
- Inside plastic hinge regions for Seismic Zones 3 and 4 or SDCs C and D when the requirements of Article 5.6.4.4 are met.

Plastic hinge region shall as defined in the article "Reinforced Concrete Column Plastic Hinge Region" of *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

C3.6.4.3

The current *AASHTO LRFD Bridge Design Specifications* do not include specifications for Type 2 mechanical connectors. The *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, on the other hand, prohibit the use of splices in plastic hinge regions of columns for SDCs C and D. An exception is made for piles or shafts in SDC D where liquefaction is anticipated because the locations of potential plastic hinging in the liquefied and non-liquefied condition may differ significantly. In that case, Type 2 mechanical connector may be specified. Caltrans Seismic Design Criteria, on the other hand, prohibit the use of splices all together at any location where nonlinear actions can be expected as a result of seismic actions.

The *ACI Building Code Requirements for Structural Concrete (ACI-318-14)* contain specifications allowing Type 2 connectors to be used in plastic hinge zones of special moment frames that are cast in place. For special moment frames with "ductile connections" constructed using precast concrete, mechanical splices for beam reinforcement are not permitted within a distance of 0.5 times the beam depth from the face of the column. This more restrictive requirement is to ensure ductile behavior of monolithic special moment frames by avoiding strain concentrations over a short length of reinforcement adjacent to a splice device. However, for special moment frames with "strong connections" *ACI 318-14* allows connectors anywhere so long it is ensured - -by using capacity-design techniques- that the connection remains elastic following the formation of plastic hinges.

- This remains a controversial subject with seismic states
- This is subject to change
- More information on the use of Type 2 Connectors in the next Module

Integral Abutment Connections

3.6.6.6 **Abutment-to-Pile Pocket Connections**

Pocket connections are an alternative to connect a pile (integral abutment stem) or to connect two elements by passing reinforcement into the pocket. When the pile itself is embedded into the void, the connection shall be characterized as socket connection and meet the provisions from Article 3.6.7.

The contribution of the corrugated steel pipe to the tension capacity of the pile-to-abutment connection shall be neglected.

C3.6.6.6

Provisions in this article are based on research conducted on a precast integral abutment bridge (Wipf, T.J., et al. (2009)). Figure C3.6.6.6-1 shows a typical integral abutment stem with pockets formed with corrugated steel pipes.

Integral Abutment Connections

The detailing of the pile top for integral abutment designs shall be the same as conventional cast-in-place concrete construction.

The nominal shear transfer resistance at the pocket-to-precast abutment interface, V_n, shall be calculated as follows:

$$V_n = 0.13\sqrt{f'_{cp}}\,A_{cv} \qquad (3.6.6.6\text{-}1)$$

where:

f'_{cp} = nominal compressive stress of the pocket concrete (ksi)

A_{cv} = Cylindrical shaft area of the pocket (in^2)

The cylindrical shaft area of the pocket A_{cv}, shall be calculated as:

$$A_{cv} = \pi\, d_c\, h_c \qquad (3.6.6.6\text{-}2)$$

where:

d_c = inside diameter of the CMP pocket (in.)

h_c = effective height of the CMP pocket (in.)

The effective height of the pocket for integral pile connections shall be based on the detailing of the embedded pile, accounting for the pile embedment length and the distance from the pile top to the top of the abutment stem.

Figure C3.6.6.6-1 Corrugated Steel Pipe Pocket Used in Integral Abutment Stem

Testing only included galvanized steel pipes. Other corrugated materials such as plastic pipe are therefore not recommended. Aluminum pipes are not recommended because the material can react with cement and lead to corrosion.

Testing showed that the presence of a corrugated metal pipe filled with concrete had no effect on the performance of the abutment stem.

This connection has proven to emulate the behavior of a CIP connection, therefore no special treatment is required for pile detailing. Designers have specified steel pile tops with welded stud shear connectors. While this is not detrimental to the design or performance of the system, the use of welded studs is typically not used in integral abutment detailing, therefore it is not necessary with CMP pocket connections.

Link Slab Connections

3.6.9 LINK SLABS

Links slabs shall be designed to resist dead load and live load (deck) forces as well as forces induced by beam end rotation. Links slabs can be used for steel beam superstructures and concrete beam superstructures.

In general, link slabs can be designed and detailed with normal deck concrete.

The detailing of link slabs involves the debonding of a portion of the bridge deck at the ends of the spans. Designers need to verify the design of the beam in this zone based on non-composite action.

The skew of the bridge may be neglected in link slab design.

C3.6.9

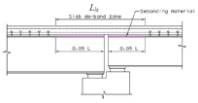
Links slabs are a means of providing a jointless bridge without continuity. The deck is continuous across the substructure element, but the supporting beams or girders are not. The basic design approach for link slabs is to design them to accommodate anticipated beam end rotations (Caner, A. and Zia P. 1998).

Links slabs are applicable to Accelerated Bridge Construction because they allow for the use of span-by-span construction as opposed to span continuity. This provides the potential to build multi-span jointless bridges quickly without the need for girder splicing or casting of integral end diaphragms. Modular Deck Beam bridges are particularly well suited for this technology.

Special concrete mixes with favorable properties have been used by some agencies. Designers should consider the impacts of special mixes on the construction process and the cost implications of special mixes (Li et al. 2005).

In most cases, a non-composite section at the ends of a bridge can be accommodated in the design, since the bending moments are small at the non-continuous beam ends.

Studies have shown that in general, the bending moments in link slabs decreases with increases in bridge skew (Akten et al. 2011). In general, reductions were found to be limited for skews up to 20 degrees and more pronounced for higher skewed bridges. Based on this study, it is conservative to assume zero skew in the design of link slabs.



Link Slab Connections

3.6.9.1 **Primary Reinforcement**

The primary reinforcement shall be designed and detailed to resist dead load and live load forces as specified in the *AASHTO LRFD Bridge Design Specifications* including the provisions for deck overhangs.

C3.6.9.1

In a typical bridge with longitudinal beams or girders, the primary reinforcement runs transversely across the bridge deck. The design of this transverse reinforcement is not different than the design of normal deck reinforcement. The design of the transverse reinforcement must account for the overhang forces including barrier impacts.

Link Slab Connections

3.6.9.2 **Link Slab Reinforcement**

Longitudinal link slab reinforcement shall be designed to resist the induced rotations in the adjacent beams.

Links slabs shall not be assumed to provide any continuity of composite dead loads or live loads.

The link slab shall be detailed as non-composite over its entire length through the use of a physical bond breaker between the link slab and the supporting beam. The minimum recommended length of the link slab shall be equal to the total of 5 percent of each span length that is connected to the link slab, measured from the end of the supporting beams.

C3.6.9.2

Link slab reinforcement is not designed for typical live load forces. Instead, it is designed to accommodate the beam end rotations. The rotation of the beams leads to forces that are result of the stiffness of the link slab.

Links slabs do not provide measurable resistance to negative bending moments, and their stiffness is

significantly less than the adjacent composite beams, therefore the joint can be assumed to be a theoretical pinned connection. This approach is similar to the assumption made with the design of an integral abutment bridge.

Figure C3.6.9.2-1 shows a typical link slab debond zone.

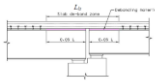


Figure C3.6.9.2-1 Links slab debond zone

Link Slab Connections

The design bending moment in the link slab per beam M_b shall be calculated as follows:

$$M_b = 2E_{ls}I_{ls}\Theta/L_b \qquad (3.6.9.2\text{-}1)$$

where:

E_{ls} = *modulus of elasticity of the link slab (ksi)*

I_{ls} = *moment of inertia of the link slab using gross gross section properties (in⁴)*

The design bending moment can be derived from the theory that the slab is fixed at each end of the link slab. A rotation is applied to the slab from either end resulting in a maximum bending moment at the fixed end. Figure C3.6.9.2-2 shows the free body diagram for this situation.

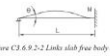


Figure C3.6.9.2-2 Links slab free body diagram

The moment of inertia of the link slab is calculated in the same plane as the adjacent girders. For an interior beam the calculation would be as follows:

$$I_{ls} = S\,d^3/12 \qquad (C3.6.9.2\text{-}1)$$

where:
S = beam spacing (in.)
d = gross section depth of the link slab (in.)

Similar calculations would be used for fascia beams accounting for the width of the overhang. The use of gross section properties is conservative since it results in a larger section property than a cracked section.

Designers should not increase the thickness of the link slab in order to provide a more robust connection. Increased thickness will lead to increased stiffness, which will develop more bending moment in the link slab, potentially leading to more cracking. Maintaining the same thickness as the adjacent span slab is recommended. Likewise, the use of higher strength concrete is not recommended since it will have a higher modulus of elasticity, which will lead to higher link slab moments.

Link Slab Connections

Θ = *maximum girder end rotation applied to the link slab (rad.)*

The maximum girder end rotation is the largest end rotation that would be applied to the link slab zone after the casting and curing of the link slab region. Provisions for continuous spans may be used for the application of the vehicular live load.

L_{ls} = *total length of link slab (in.)*

The total length of the link slab shall be the debond zone as shown in Figure C3.4.9.2-1.

The links slab shall be designed to resist the induced bending moment. The section shall also be checked for serviceability using the "Control of Cracking by Distribution of Reinforcement" provisions in the *AASHTO LRFD Bridge Design Specifications*.

The loads factors used to calculate rotations need to be consistent with the design provisions being used (strength vs. service). These provisions will ensure that the crack width in the link slabs will provide a durable structure. The provisions for Class 2 exposure are recommended.

Steel Beam Connections

3.6.10.3.2 Simple Span for Dead Load, Continuous for Live Load

This design and detailing method consists of span-by-span beams that are connected after casting of a composite concrete deck. The beams are supported on bearings at each end, and are designed as simple spans for most dead loads, and as continuous spans for live loads and any dead loads applied to the bridge after completing the connection.

The connection at interior supports consists of two components:

- Compression transfer at the bottom of the composite beam: This connection can consist of a bolted flange splice, a welded flange splice via a common sole plate, or end bearing between two end plates connected to the beam flanges.
- Tension transfer at the top of the composite beam: This connection can be made with a bolted flange splice, lapped mild reinforcement, supplemental steel connections, or post-tensioning (or a combination of these).

If bearings are used at the ends of each beam, a shear connection need not be included in the design of the continuity connection.

C3.6.10.3.2

Several connection methods have been tested in laboratories. The connections listed herein were shown to provide adequate resistance for ultimate loads. One test included a compression connection that consisted of a transfer of force from the compression flange to the surrounding concrete. This connection was found to be inadequate to resist ultimate loads (Azizinamini et al. 2005).

Source: SHRP2 ABC Toolkit

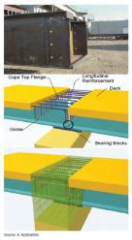


Figure A.5. Pier continuity details.

Superstructure to Substructure

3.6.11 INTEGRAL SUBSTRUCTURE-TO-SUPERSTRUCTURE CONNECTIONS

The recommended approach to design and detailing of full integral superstructure-to-substructure connections is to incorporate a reinforced closure joint in the design. The designer shall identify the load path through the element to provide proper detailing for force transfer to adjacent elements. This type of connection, or system of connections, is always a CP connection.

Designers shall consider non-integral or semi-integral designs for ABC bridges with very short construction durations.

Semi-integral abutment connections may be used to facilitate the construction of bridge system installations using SPMT and Lateral Sliding methods.

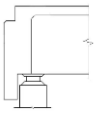
C3.6.11

The connection of typical integral superstructure to substructure connection involves the connection of a multiple prefabricated elements. The fabrication and erection tolerances of each individual element will complicate the fit-up of each element. Cast-in-place reinforced concrete closure joints are an effective means of connecting the various elements to create a positive moment and shear connection at the joint.

Integral pier designs are preferred in high seismic zones. It is possible to design and detail a durable non-integral bridge in a high seismic zone with acceptable performance. The result will mostly like include larger foundations. Designers should investigate the trade-offs between foundation design and construction speed.

There are several different types of semi-integral abutments that are used. In general, semi-integral abutments have a portion of the abutment stem inside integral with the superstructure, however a hinge joint is detailed between the superstructure and the substructure. Figure C3.6.11-1 depicts a semi-integral abutment connection that has been used.

Semi-integral abutment designs are common for SPMT and Lateral Slide installations. The superstructure can be moved into position and set down without the need for a closure joint.



Other Deck Beam Connections

3.6.13.1 Welded Tab Deck Connection

Deck connections may be made using a welded connection combined with a grouted shear key. Steel plates are embedded within the edge of the deck at regular intervals. The shear keys typically run the entire length of the deck.

There is no specific design procedure for these connections since they are based on experimental studies and/or field experience. Agencies that make use of these connections have developed typical details that can be used for detailing.

3.6.13.2 [Transverse Post-Tensioning

Transverse post-tensioning may be used to connect deck beam elements. The post-tensioning system is typically combined with a continuous grouted shear key to form the connection. The provisions for "Precast Deck Bridges" included in the *AASHTO LRFD Bridge Design Specifications* shall be used for the design of the deck connection.

C3.6.13.1

Welded tab connections have been found to produce durable connections on bridges with low to moderate volume of truck traffic. Bridges with higher truck traffic volumes should be detailed with more substantial moment connections (see Article 3.6.2).

Designers should coordinate with the Owner agency regarding the use of welded tab connections.

GRS/IBS Design

Provisions are based on FHWA Implementation Guide

- There is some concern within the AASHTO Bridge Sub-committee
- Under debate at this time

Questions?

CME

NCHRP Project 12-102 Seismic

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CME Associates, Inc.

Lee Marsh, Ph.D., P.E.
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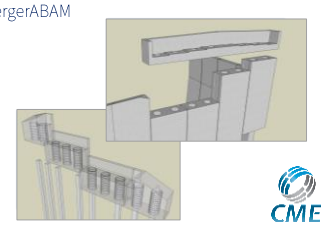


Review of New Guide Specifications for ABC

Design Guide Specifications for ABC

- Section 1: Introduction
- Section 2: General Design Provisions
- Section 3: Design of Prefabricated Elements
- Section 4: Detailing Requirements
- Section 5: Durability of ABC Technologies

Construction Guide Specifications for ABC

DESIGN OF PREFABRICATED ELEMENTS

3.4 Seismic Design for Accelerated Bridge Construction

3.5 Prefabricated Element Design

3.6 Connection Design and Detailing

3.7 GRS IBS

3.8 Accelerated backfill

3.4 SEISMIC DESIGN FOR ABC

3.4.1 Seismic Analysis & Design Methods

3.4.2 Load Path

3.4.3 Seismic Systems, Elements, & Sub-Systems

3.4.4 Energy Dissipation

3.4.5 Capacity Protection

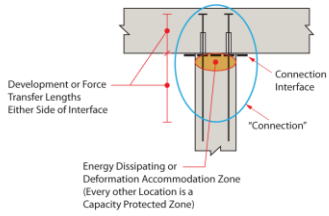
3.4 SEISMIC DESIGN FOR ABC CONT.

Seismic analysis and design of prefabricated elements shall be performed using either:

- Force-Based Design AASHTO LRFD Bridge (2014)
- Displacement-Based Design AASHTO Seismic GS (2011)

… set up to accommodate continuing research and advancements.

3.4 SEISMIC DESIGN FOR ABC CONT.



In high seismic areas inelastic ductility is required; thus clearly all members must have sufficient strength and ductility to form the intended structural plastic mechanism.

… ideal locations for ABC connections

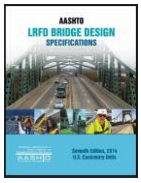
3.4.1.2 AASHTO LRFD (2014)

The force-based seismic design methodology, as defined in AASHTO LRFD Bridge Design Specifications, is permitted when R-factors and prescriptive detailing for energy dissipating PBEs and their connections have been shown through laboratory testing to provide acceptable performance. This document limits the application of force-based design procedures for some prefabricated connections in Zones 2 through 4 when detailed understanding of material behaviors is necessary.

3.4.1.3 AASHTO SEISMIC GS (2011)

Displacement-based design, as defined in AASHTO Guide Specifications for LRFD Seismic Bridge Design, shall be used to ensure proper performance of prefabricated elements and their connections in Seismic Design Categories (SDCs) B through D.

3.4 SEISMIC DESIGN FOR ABC CONT.



…Select a methodology and use it.

Do not mix and match!

3.4 SEISMIC DESIGN FOR ABC CONT.

1. Continuity of load path under load reversals
2. Development of cyclic inelastic deformations
3. Maximum forces (moments) occur where we would like to connect prefabricated elements
4. Certain element/material behaviors may cause rapid loss of cyclic resistance
   - Limitations of Rotation Capacity
   - Local Buckling
   - Strain Concentrations

… Detailing is important!

3.4.2 LOAD PATH

A continuous load path shall exist between the source of the seismic loads and the foundation. The load path shall enable the transfer of forces between the elements and connections through which it passes.

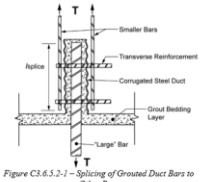


Figure C3.6.5.2-1 – Splicing of Grouted Duct Bars to Other Bars

Example Load Path Detail

3.4.3 SEISMIC SYSTEMS, ELEMENTS, & SUB-SYSTEMS

3.4.3.1 Earthquake-Resisting Systems (ERS)

3.4.3.2 Earthquake-Resisting Elements (ERE)

3.4.3.3 Earthquake-Resisting Sub-Systems (ERSS)

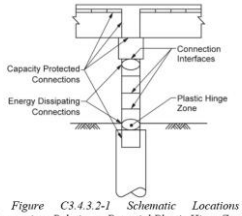
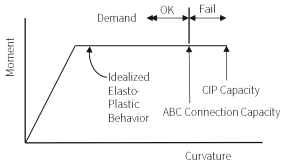


Figure C3.4.3.2-1 Schematic Locations of Connections Relative to Potential Plastic Hinge Zones.

3.4.3.2 DEFINING EREs W/ ABC

Determine what is different from CIP and make a plan for addressing it in the design.



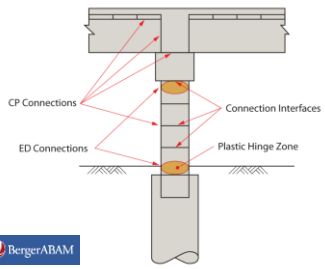
3.6 CONNECTION DESIGN & DETAILING

3.6 CONNECTION DESIGN & DETAILING

3.6.1 GENERAL CONNECTION DESIGN

Provisions in Section 3.6 are not intended to preclude the use of other preferred details as approved by the Owner and so long the designer can demonstrate adequate load transfer as well as consideration for compatibility of deformations and constitutive relations of materials.

3.6.1 GENERAL CONNECTION DESIGN

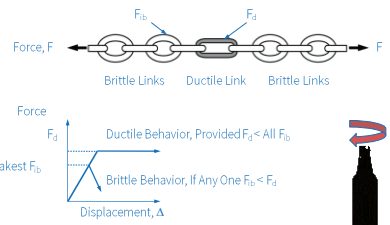


Note ~ In this Example: Moment Continuity is at Top and Bottom of Column

ED – Energy Dissipating (Ductile)

CP – Capacity Protected (Non-ductile)

3.6.1 GENERAL CONNECTION DESIGN



3.6.4 MECHANICAL BAR CONNECTORS

- Viable option for connecting concrete elements
- Defined Type 1 and Type 2 Connectors

  Defined in ACI-318-14 (Building Code) requirements for structural concrete (Special Moment Frames & Special Structural Walls)

  Implied in AASHTO SGS (2011) under "Splicing of Longitudinal Reinforcement in SDCs C & D

Type 1 mechanical connectors shall not be used to splice longitudinal column reinforcement within a distance equal to twice the maximum column dimension from the top of the footing or from the bottom of the pier cap, or at any other location where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements.

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3.6.4 MECHANICAL BAR CONNECTORS

- Only Type 2 mechanical permitted in plastic hinge regions

Type 2 mechanical connectors shall be permitted in the following locations and conditions:

- Outside of the plastic hinge regions.
- Inside plastic hinge regions for Seismic Zone 1/ SDC A or inside of plastic hinge regions for Seismic Zone 2/ SDC B with the Owner's approval.
- Inside plastic hinge regions for Seismic Zones 3 and 4 or SDCs C and D when the requirements of Article 3.6.4.4 are met.

- Length of mechanical coupler (L_{MC}) is limited to $L_{MC} \leq 15\ d_{barl}$

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3.6.4.4.1 FORCED-BASE DESIGN

- Reduced modification factor R_r

$$R_r = \gamma R$$

- 0.8 for $L_{MC} \leq 4\ d_{barl}$
- 0.5 for $L_{MC} > 4\ d_{barl}$

On-going research, but current research is limited to #8 bars and smaller. Revisions are expected until additional research is complete.

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3.6.4.4.2 DISPLACEMENT-BASED DESIGN

The displacement capacity of each bridge bent shall be calculated using the "Nonlinear Static Procedure (NSP)" outlined in AASHTO Guide Specification for LRFD Seismic Design. Plastic moment capacity shall be based on the moment-curvature analysis of the column section with no mechanical connector and the analytical plastic hinge length shall be taken as the reduced analytical plastic hinge length given by:

$$L_p^{sp} = L_p - \left(1 - \frac{H_{sp}}{L_p}\right)\beta L_{sp} \leq L_p \qquad (3.6.4.4.2\text{-}1)$$

On-going research, but included as a starting point and expected to be updated as research is developed further.

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3.6.4.4.2 DISPLACEMENT-BASED DESIGN

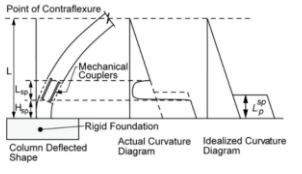

Figure C3.6.4.4.2-1 – Column Deflected Shape and Curvature Diagram

$$L_p^{sp} = L_p - \left(1 - \frac{H_{sp}}{L_p}\right)\beta L_{sp} \leq L_p$$

Table 3.6.4.6-1-- Rigid length factor for permitted mechanical connectors

Mechanical Connector Type	Rigid Length Factor, β
Grouted Sleeve Coupler (GC)	0.65
Headed Reinforcement Coupler (HC)	0.75

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3.6.4.5 DEBONDING OF COLUMN REINF.

Applicable in footing or pier cap when couplers are used. The minimum debonding length, L_{deb}, and anchorage length, L_a, in inches, shall satisfy:

$$L_{deb} \geq \max\begin{cases} 0.4D_c \\ 5000\varepsilon_{ye}d_{bl} \end{cases} \qquad (3.6.4.5\text{-}1)$$

where:

D_c = diameter of the column (in.)
ε_{ye} = nominal yield strain of the column longitudinal reinforcement
d_{bl} = diameter of column longitudinal bar (in.)

$$l_a \geq L_{deb} + l_d \qquad (3.6.4.5\text{-}2)$$

where l_d is the tension development length

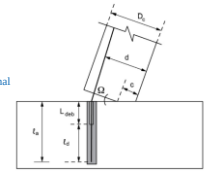

Fig. C3.6.4.5-1 – Approach to Determine the Debonded Length in Column-to-Footing Connection

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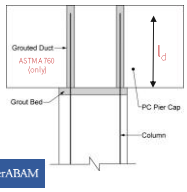
3.6.5 GROUTED DUCTS

- Connect reinforcing bars that projects from one element into a corrugated ducts embedded in receiving member



Source: Brenes (2006)

3.6.5 GROUTED DUCTS

3.6.5.1 Minimum Development Length

The anchorage length for column grade 60 bars developed into corrugated steel ducts shall satisfy:

$$l_d \geq \frac{0.67 d_{bl} f_{ye}}{\sqrt{f'_g}} \qquad (3.6.5.1\text{-}1)$$

where:

l_d = anchored length of longitudinal reinforcing bars into a pier cap or footing (in.)
d_{bl} = diameter of column longitudinal bar (in.)
f_{ye} = expected yield stress of the longitudinal reinforcement (ksi)
f'_g = nominal compressive strength of grout (ksi)

In lieu of specific data it shall be permitted to use $f_{ye} = 68\ ksi$ for grade 60 reinforcement conforming to ASTM A706 or ASTM A615.

3.6.5 GROUTED DUCTS

3.6.5.2 Splicing w/ Ductility Demands (ED) for SDCs C and D in Grouted Duct Connections

Where tensile force is transferred between reinforcing anchored into steel ducts using high-strength grout and bars adjacent to the duct, the splice length, l_{splice}, shall be the longer of the anchorage length of the bar inside the duct or the splice length of the bars on the outside of the duct.

Transverse steel shall enclose both the duct and the bars outside the duct.

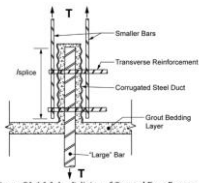


Figure C3.6.5.2-1 – Splicing of Grouted Duct Bars to Other Bars

3.6.5 GROUTED DUCTS

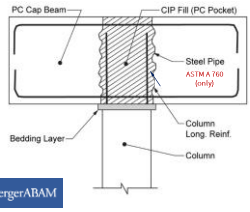
3.6.5.3 Debonding of Column Longitudinal Reinforcement in Grouted Duct Connections

3.6.5.4 Bedding Layer in Grouted Duct Connections

3.6.5.5 Development of Deformed Steel Bars in Corrugated Steel Ducts using UHPC

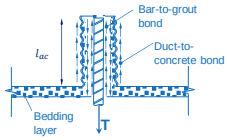
3.6.6 POCKET CONNECTIONS

- Column with projecting reinforcement and a receiving precast footing or pier cap with a corrugated steel-pipe-formed pocket



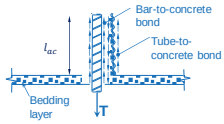
3.6.5 & 6 DEVELOPMENT LENGTH

3.6.5.1 Grouted Duct Development Length



$$l_{ac} \geq \frac{0.67 d_{bl} f_{ye}}{\sqrt{f'_{cg}}}$$

3.6.6.3 Pocket Connection Development Length



$$l_{ac} \geq \frac{d_{bl} f_{ye}}{\sqrt{f'_{cg}}}$$

3.6.6.4 CORRUGATED PIPE THICKNESS

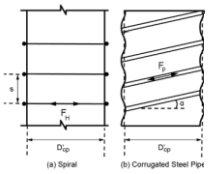


Figure C3.6.6.4-1 – Confinement Provided by Spiral a Corrugated Steel Pipe

A minimum thickness of corrugated steel pipe:

- For SDC B (Seismic Zone 2):

$$t_{pipe} \geq max \begin{cases} \dfrac{0.11\sqrt{f'_{cp}}\, D'_p}{4 f_{ypipe} \cos(\alpha)} \\ 0.060\ in. \end{cases} \quad (3.6.6.4\text{-}1)$$

- For SDCs C and D (Seismic Zones 3 and 4):

$$t_{pipe} \geq max \begin{cases} \dfrac{\rho_s D'_{cp} f_{yh}}{4 f_{ypipe} \cos(\alpha)} \\ 0.060\ in. \end{cases} \quad (3.6.6.4\text{-}2)$$

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3.6.6.4 CORRUGATED PIPE THICKNESS

Additional prescriptive transverse reinforcement is also required, as described below.

ρ_s = volumetric ratio of transverse reinforcement

ρ_s shall be calculated as the minimum joint shear reinforcement ratio for SDCs C and D in *AASHTO Guide Specifications for LRFD Seismic Bridge Design.*

When principal tension stress in the joint is not calculated, ρ_s in Equation (3.6.6.4-2) shall be taken as:

$$\rho_s = \max \begin{cases} \dfrac{0.11\sqrt{f'_{cp}}}{f_{yh}} \\ 0.40\dfrac{A_{st}}{(l_{ac} l_d)^2} \end{cases} \quad (3.6.6.4\text{-}3)$$

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3.6.6 OTHER SECTIONS

- 3.6.6.5 Bedding Layer
- 3.6.6.6 Abutment to Pile Pocket Connections

Nominal shear transfer resistance, V_n at the pocket to precast abutment interface and area of the pocket A_{cv}, shall be:

$$V_n = 0.13\sqrt{f'_{cp}}\, Acv$$

$$A_{cv} = \pi\, d_v\, h_v$$

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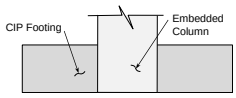
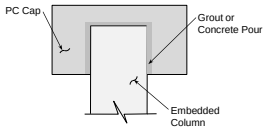
3.6.7 SOCKET CONNECTIONS

- Embedment of precast column or pile into receiving element
- Socket can be:



a) Wet (cast-in-place)



b) Formed (precast)

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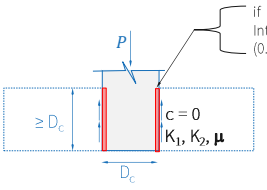
3.6.7.2 SOCKET IN CIP FOOTING



if $L_c \leq 1.5D_c$
Intentionally roughening (0.25") is required

(concrete placed against a clean concrete surface), Except μ is limited to 0.5 when $L_c < 1.1\ D_c$

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3.6.7.2 SOCKET IN CIP FOOTING

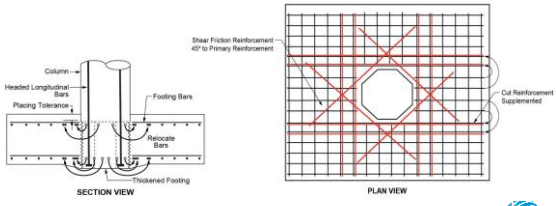

Figure C3.6.7.2-1 Placement of Footing Bars with a Socket Connection

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3.6.7.3 SOCKET IN SHAFT

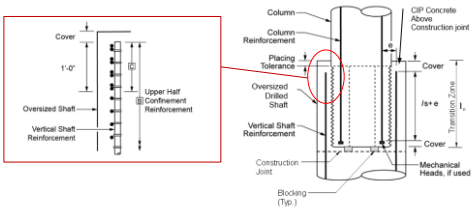

Figure 3.6.7.3.1-1 Column-to-Shaft Longitudinal Bar Arrangement

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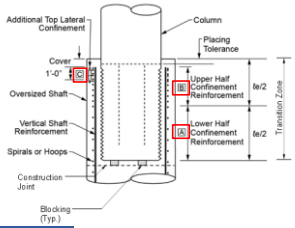
3.6.7.3 SOCKET IN SHAFT



Spiral/Hoops Requirement

$$\frac{A_{sh}}{s_{max}} \geq \frac{k f_{ul} A_l}{2\pi f_{ytr} l_s}$$

Location

- A (k = 0.5)
- B (k = 1.0)
- C (k = 2.0)

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3.6.7.4 SOCKET IN PC ELEMENT

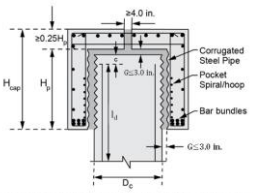

Figure C3.6.7.4.2-1 Details of Pier Cap with Socket

$$H_p \geq max \begin{cases} D_c + G \\ l_d + c + G \end{cases}$$

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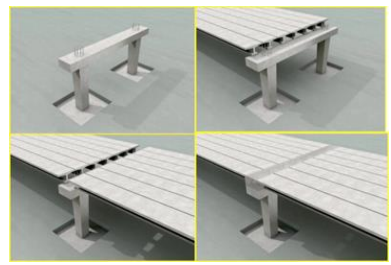
3.6.11 INTEGRAL CONNECTIONS



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Questions?

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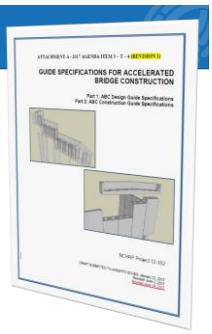
Workshop W-01

Implementation of the New AASHTO Guide Specifications for Accelerated Bridge Construction

- Module 5: Other Topics



Michael Culmo, PE
CME Associates, Inc.

Note

This investigation is sponsored by TRB under the NCHRP Program. Data reported is work in progress. The contents of this presentation has not been reviewed by the project panel or NCHRP, nor do they constitute a standard, specification, or regulation.

Section 4: Detailing Requirements

Detailing of bridges for ABC and Prefabrication is not significantly different

- Designers still need to depict the construction and layout
- There are a few items that are applicable to ABC that need to be shown on the contract plans

Section 4: Detailing Requirements

4.4 PREFABRICATED ELEMENTS

The following are the typical details that shall be included in project plans:

1. Plan view(s) of the structure showing the horizontal geometry and layout of the elements and features.
2. Elevation view showing the vertical geometry.
3. The location of all joints and closure pours shall be shown on the applicable details.
4. Details of elements including typical sections showing dimensions and the required reinforcement (concrete elements).
5. Concrete reinforcing bar bend diagrams and tables for elements (if required by an agency for conventional construction).

C4.4

There is no specific way to detail a prefabricated bridge. The recommendations herein are based on past successful projects. Detailing practices vary from agency to agency, therefore minor adjustment to these recommendations would be in order. For example, some agencies do not detail reinforcing bar bend diagrams on contract drawings. This work is left to the fabricator. In this case, the bar bend diagrams need not be included in the plans.

Prefabrication does not relieve the designer of the responsibility to design all of the prefabricated elements in the bridge. Portions of the vertical construction industry schematically detail structures and leave the final design up to the fabricator. This is not the case in the prefabricated bridge construction industry. See Part 1, Section 3 for more information on the design of prefabricated elements.

Section 4: Detailing Requirements

4.4.1 LAYOUT AND WORKING LINES

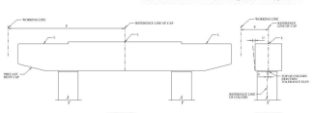
All layout details shall be based on working lines, working points or grid coordinates.

Working lines may be shown on the plans or specified to be determined by the contractor and shown on the erection plans for the elements.

C4.4.1

This approach is common practice for most agencies, therefore normal detailing protocol should suffice.

Working points or working lines need not be based on a specific element in the bridge. They are often offset from the structure. Historically, this approach has been used in the building industry through the use of batter boards and string lines. Modern construction methods employ sophisticated survey methods to accurately define locations of working lines and points.



Section 4: Detailing Requirements

4.4.2 ELEMENT AND ERECTION TOLERANCES

The designer shall develop schematic erection plans for large elements and very complex bridge sites. The plans shall include preliminary layout of cranes and delivery vehicles. The erection plans shall be consistent with the specified traffic management approach.

The designer shall specify both element tolerances and erection tolerances on the plans or in the specification. Element tolerances shall be shown graphically using schematic element drawings. Erection tolerances shall be shown on plan or elevation details.

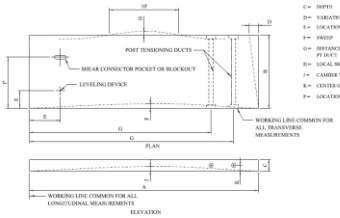
C4.4.2

Erection plans are not required for all projects. Projects with smaller elements and significant work areas need not include erection plans. Schematic erection plans are recommended for sites where work areas are small and element weights are large. The designer should investigate the approximate size of cranes required to erect the prefabricated elements to ensure that the bridge can be built within the available right of way.

The designer should be responsible for establishing and specifying tolerances. The values for element tolerances and erection tolerances have an impact on the required width of joints in the structure. Large tolerances will require large joints in order to ensure that the elements will fit together. The document entitled *Guidelines for Prefabricated Bridge Elements and Systems Tolerances (2017)* contains recommended element tolerance details and values, and recommended element erection details and values. This document also contains more information on determining and specifying tolerances.

Section 4: Detailing Requirements

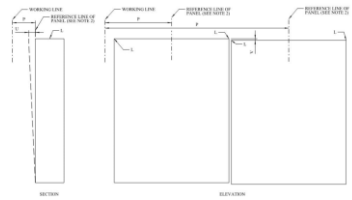
Element tolerance example



A=	LENGTH	± 1/4"
B=	WIDTH	± 1/4"
C=	DEPTH	± 1/8"
D=	VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	± 1/4"
E=	LOCATION OF LEVELING DEVICE	± 1"
F=	SKEWP	± 1/4"
G=	DISTANCE FROM COMMON WORKING POINT TO CENTER OF ANY PT DUCT	± 1/8"
H=	LOCAL SMOOTHNESS OF ANY SURFACE	± 1/4" IN 10 FEET
J=	CAMBER VARIATION FROM DESIGN CAMBER	± 1/4"
K=	CENTER OF PT DUCT AT EDGE OF SLAB	± 1/8"
P=	LOCATION OF SHEAR CONNECTOR POCKET OR BLOCKOUT	± 1/2"

Section 4: Detailing Requirements

Erection tolerance example



Section 4: Detailing Requirements

4.4.3 JOINT WIDTHS

The specified width of joints between elements shall be based on the specified element tolerances, erection tolerances, and the specified materials that are to be used within the joint.

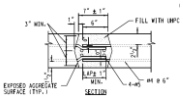
Reinforced closure joints shall be sized to accommodate the required lap lengths of the reinforcement as well as element and erection tolerances.

The materials used to fill a joint shall be considered for narrow joint openings. The recommended minimum joint width for concrete filled joints shall be set to 1.5 times the size of the maximum aggregate size in the concrete. The recommended minimum joint width for UHPC filled joints shall be set to 1.5 times the length of the longest steel fiber used in the mix.

C4.4.3

The document entitled *Guidelines for Prefabricated Bridge Elements and Systems Tolerances (2017)* contains recommended approaches to determining and specifying joint tolerances. The minimum width of joints should account for the materials that are specified to fill the joints. For example, a joint that is to be filled with concrete should have adequate width to accommodate the size of the specified aggregate.

The most common steel fiber deployed in UHPC applications is a 0.008-inch by 0.5-inch long straight fiber. Using joints that are narrower than the specified value can result in parallel orientation of the steel fibers, which can affect the performance of the UHPC.



Section 4: Detailing Requirements

4.4.4 MATERIALS

The designer shall specify all materials that are required for the bridge including joint materials. The designer shall also specify coatings of materials.

Concrete materials for prefabricated elements can be specified in two ways:

1. Prescriptive specifications using standard agency mix designs.
2. Performance specifications with limitations on acceptable admixtures.

The contractor shall be responsible for determining the materials for all temporary works. Embedded devices shall be detailed to provide equal or better corrosion protection when compared to the element.

C4.4.4

Materials for structural steel and reinforcing steel are standardized in most agencies.

The use of prescriptive concrete mixes can be problematic with precast concrete fabrication. Many precast concrete fabrication facilities have internal batching equipment. Fabricators will also use concretes with strengths that are higher than specified strengths in order to expedite form stripping. The use of prescriptive concrete mixes may lead to longer fabrication times and higher costs. Performance specifications are commonly used in prestressed girder specifications for this reason.

The selection of materials that are not part of the completed structure fall under the realm of "contractor means and methods". Materials for temporary works include, but are not limited to:

- Falsework framing
- Embedded lifting devices
- Lifting hardware including slings, spreader beams, shackles, etc.
- Temporary support of excavation

SPMT Systems

4.5 SPMT SYSTEMS

The designer shall design and detail the bridge structure including all connections. The detailing of the interface between the bridge system being moved and the supporting structure at the bridge site shall account for the movement and setting of the bridge with the SPMTs.

The designer shall detail a schematic preliminary representation of the SPMT installation methods. The plans shall include, but not be limited to:

- Weight and center of gravity of the portion of the bridge to be moved
- Preliminary SPMT sizes and layout
- Allowable twist in the structure during the move
- Allowable differential deflection between support locations
- Estimated dynamic effects used for the design of the bridge
- Locations of support for the bridge in the staging area
- Locations of support for the bridge on the SPMTs
- Assumed static reactions at each support point during the bridge move and variation in the reactions with under the application of the maximum allowable twist

C4.5

Special care should be taken when detailing the connection at the interface of the bridge system and the supporting elements. Features such as sloping bridge seats, pedestals and cheekwalls can make the final setting difficult. This may require the movement of the bridge at a higher elevation and jacking of the bridge during the final setting. See Article 1.5 for more information on SPMT installations.

SPMT Systems

- Schematic details of the allowable methods of attachment of the bridge to the falsework
- Assumed travel path for the SPMTs
- Assumed staging area including schematic details of the temporary supports in the staging area
- Geotechnical information in the staging area and along the travel path including results of subsurface investigations
- Allowable temporary foundation types for the temporary supports in the staging areas including allowable bearing pressures or recommended pile sizes and allowable pile loads

The plans shall also list all other design assumptions that can affect the SPMT layout. The plans or specifications shall note the changes to the SPMT system that would require a structural evaluation of the bridge system by the contractor (see Article 1.5.2.4).

Lateral Slide Systems

4.6 LATERAL SLIDE SYSTEMS

The designer shall design and detail the bridge structure including all connections. The detailing of the interface between the bridge system being moved and the supporting structure at the bridge site shall account for the non-moved and setting of the bridge.

The designer shall detail a schematic preliminary representation of the lateral slide installation methods. The plans shall include, but not be limited to:

- Weight of the portion of the bridge to be moved
- Schematic details of the allowable sliding bearing type(s)
- Assumed location of jacking/pulling force including assumed friction values
- Assumed vertical jacking and support locations before, during and after the slide installation
- Schematic details of the temporary falsework (including connections and connection details of connection) between the temporary falsework and the permanent bridge substructures
- Geotechnical information beneath the temporary falsework including results of subsurface investigations
- Allowable temporary foundation types for the temporary supports in the staging area including allowable bearing pressures or recommended pile sizes and allowable pile loads

The plans shall also list all other design assumptions that can affect the lateral slide.

The plans or specifications shall note the changes to the lateral slide system that would require a structural evaluation of the bridge system by the contractor.

C4.4

The interface with the bridge system and the substructure is typically detailed as a flat joint. Sliding a structure along a gusset is not recommended.

The most common detail for bearing lines is a reinforced or post-tensioned integral diaphragm with discrete support points called slide shoes that are not necessarily connected with the beam. Figure 4.6-1 shows a schematic representation of a slide shoe detail. This detail is applicable to several different types of mechanical sliding systems. The slide bearings or rollers can be placed under the slide shoe or between the slide shoes. The permanent bearings are typically placed under the slide shoes, but the bearings can be placed alongside the slide shoes. This approach allows significant flexibility in the design of the slide system. It also reduces the total number of bearings.

It is also feasible to detail a non-integral diaphragm, thereby placing a sliding bearing directly under each beam. This will require more slide bearings and permanent bearings than the integral diaphragm option. In this case, the connection between the beam and the diaphragm must be designed and detailed to resist vertical jacking forces.

The connection between the falsework and the substructure can affect the detailing of the bridge. The connection eliminates the friction forces in the foundations by resolving the force loop between the sliding bearing friction and the jacking/pulling equipment. The connection between the falsework and permanent bridge foundations and the falsework will need to be designed and detailed to resist lateral friction forces. This connection also prevents differential vertical deflection between the falsework and the substructure, which can lead to binding of the slide system (see Article 1.4 for more information on Lateral Slide Systems).

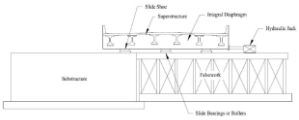


Figure 4.6-1 Integral Diaphragm with Slide Shoes

Durability of ABC Technologies

No need to sacrifice durability with ABC and prefabrication

- Many good details have been developed that have proven to be very durable
- Utah DOT completes routine inspections on specific ABC details
  - Many of the provisions in this section are based on owner experiences

Durability of ABC Technologies

5.4 SIMPLIFIED DETAILING FOR DURABILITY

The designers should strive to design prefabricated bridges with simplified element details that are easy and fast to construct, while providing a durable structure. The detailing should be consistent with the proposed ABC technology, and provide for adequate tolerances during assembly.

C5.4

Complex details can lead to fit-up problems during assembly. These problems can result in delays, which is contrary to the goals of ABC. Poor fit-up at expansion joints can also lead to potential leaks and poor performance.

There are many ways to detail an ABC bridge since there are different type of ABC technologies. Installation of prefabricated elements can be detailed with emulative joint details (see Part 1, Section 3), while bridge system installations might require a different type of connection in order to facilitate the installation.

Durability of ABC Technologies

5.4.1 INTEGRAL ABUTMENTS AND PIERS

Integral abutments and piers should be considered for ABC projects where feasible.

Reinforced closure joints are recommended for the connection of the superstructure to the substructure. The time to cast and cure closure joints shall be considered in the project scheduling.

C5.4.1

There are several benefits to integral bridges. Integral design can reduce foundation costs, eliminate deck expansion joints, and eliminate the need for bearings. Leaking expansion joints have proven to be a source of long-term deterioration, especially in northern regions that use de-icing treatments. Integral bridges result in bridge without deck expansion joints, which will lead to better long term performance.

The connection of a superstructure to a substructure is normally a complex connection involving multiple elements that need to fit together. Detailing of this connection with a tight tolerance connection is not recommended due to the high potential for fit-up problems.

Durability of ABC Technologies

5.4.2 SEMI-INTEGRAL ABUTMENTS

Semi-integral abutments can be used to accelerate construction of the superstructure to substructure connection. Semi-integral abutment connections are recommended for bridge system installations using SPMT and Lateral Slide methods.

C5.4.2

Figure C5.4.2-1 depicts a schematic representation of a semi-integral abutment detail that has proven to be easy to build and durable. The backwall for the abutment is attached to the superstructure. The backwall is the lapped over and behind the abutment stem. The overlap prevents groundwater from leaking onto the bridge seat. The expansion joint is also behind the backwall, therefore no water can leak onto the bridge seat or beam ends. This detail was adapted from conventional construction details in several states.

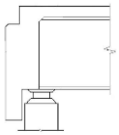


Figure C5.4.2-1 Semi-Integral Abutment

Good solution for many ABC Technologies

- Fast
- Economical
- Durable

Durability of ABC Technologies

5.5.2 ELEMENT CONNECTIONS

The connections between prefabricated elements shall be detailed to be structurally adequate and durable.

Connections in critical locations shall be detailed as reinforced moment connections. Joint filler materials shall provide equal or better durability when compared to the connected elements.

C5.5.2

Connections in prefabricated bridge element construction have been the source of concern regarding durability by some designers. The concern arises from the presence of joints in a structure that may lead to future deterioration. Joints in structures are not limited to prefabricated bridges. All bridges have construction joints throughout. The goal of structure emulation detailing is to replace construction joints with durable connection joints. If properly detailed, these connection joints can provide equal durability when compared to typical construction joints.

Part 1, Section 3 contains more information on element connections. All of the connections included in Part 1, Section 3 have proven to be durable. Common durable connections include:

- Cast-in-place concrete closure joints using lapped bar reinforcement
- Mild reinforced joints with mechanical reinforcing bar connectors combined with a grouted joint.
- Grouted post-tensioned joints combined with grouted keys
- Deck link slabs

Durability of ABC Technologies

5.5.2.1 Concrete Cover with Mechanical Connectors

The detailing of reinforcement in elements with mechanical connectors shall account for the size of the coupler. Reinforcing bars shall be detailed to provide adequate concrete cover over the connector.

The designer shall specify the distance to the center of the spliced bars. The specified dimension shall be based on the maximum diameter of the available mechanical connectors plus the required cover.

C5.5.2.1

The mechanical connector will be larger than the connected bar. There are several manufacturers of mechanical connectors, however there is no standard connector size. This issue requires a different approach to the detailing of concrete cover.

Table C5.5.2.1 shows the recommended detailing dimensions for commonly available grouted connectors. These connectors are well suited for prefabrication and have been used on many successful projects. These dimensions encompass the dimensions of these common connectors. These dimensions are preliminary and not guaranteed. Designers should contact approved companies to verify these dimensions.

Bar Size	Outside Diameter (in.)	Length of Connector (in.)
4	2.63	14.13
5	3.00	14.13
6	3.00	14.13
7	3.00	18.75
8	3.50	18.75
9	3.50	18.75
10	3.50	23.50
11	4.00	23.50
14	4.00	28.38
18	4.50	39.63

Table C5.5.2.1-1 Dimensions of Common Grouted Reinforcing Bar Connectors

Other mechanical connectors can be used, provided that they can meet the strength requirements of Article 3.6.4 of this guide specification. Designers should contact manufacturers to determine the required dimensions for detailing.

CME

Durability of ABC Technologies

5.5.2.2 Concrete Cover with Headed Reinforcement

The detailing of reinforcement in elements with headed anchorage ends shall account for the size of bar head. Reinforcing bars shall be detailed to provide adequate concrete cover over the bar head.

C5.5.2.2

There are different sizes of headed bars in the market. The specification for headed reinforcing bars is ASTM A970 (2016). The provisions in Article 3.6.2.3 of this guide specification are based on the use of Class HA headed bars. This class of bar is included in Annex A1 of ASTM A970 (2016). This specification requires that the head bearing area be equal to or greater than 4 times the area of the bar. There are several manufacturers of headed bars that meet this specification, however there is no standard head size, and sizes vary from one manufacturer to another.

This issue requires a different approach to detailing of concrete cover. In lieu of specifying cover over the bars, the designer should specify the distance to the center of the spliced bars. The specified dimension should be based on the maximum diameter of the available mechanical connectors plus the required cover.

Table C5.5.2.2-1 lists the diameter of round heads for various bar sizes based on ASTM A970 (2016) and material data from the most common headed bar manufacturers. These dimensions encompass the dimensions of these companies. These dimensions are preliminary and not guaranteed. Designers should contact approved companies to verify these dimensions.

Bar Size	Head Diameter (in.)
3	1.1
4	1.6
5	2.0
6	2.4
7	2.8
8	3.2
9	3.6
10	4.0
11	4.5
14	4.5
18	5.1

Table C5.5.2.2-1 Dimensions of Common Headed Reinforcing Bars

CME

Durability of ABC Technologies

5.5.2.3 Location of Joints and Joint Protection

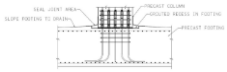
Joints between prefabricated elements should either be detailed for durability, protected from moisture ingress, or located in areas that are not exposed to deleterious elements.

Where protection from moisture ingress cannot be ensured, non-corrosive materials such as stainless steel reinforcing or Ultra High Performance Concrete can be considered.

C5.5.2.3

Properly detailed joints can provide excellent durability regardless of their location on the structure. Cast-in-place reinforced closure joints and grouted reinforced closure joints have performed well, even in harsh environments as documented in studies by the Utah DOT entitled *Performance of Accelerated Bridge Construction Projects in Utah* (2009, 2010, 2011, 2013).

Joints subjected to harsh environments can be protected in several ways. Deck joints can be protected through the use of waterproofing membranes and overlays or thin polymer overlays. Some joints can be protected via secondary grouting or encasement in concrete. Figure C5.5.2.3-1 depicts a recessed grouted joint between a column and a footing, a seal around the joint, and a sloped footing top. All or some of these measures can provide additional corrosion protection to the joint. The primary protection is the reinforced grouted joint. It should be noted that the footing recess can lead to other issues. In this case, the footing thickness needs to be increased in order to recess the top reinforcing layer deeper into the footing, resulting in a heavier footing.



CME

Durability of ABC Technologies

5.5.2.5 Materials for Closure Joints

Materials used for closure joints shall have equal or better durability than the connected elements. The following durability characteristics should be chosen for closure joint materials:

- Low permeability
- Low shrinkage
- High bonding
- Freeze thaw resistance (northern environments)

The materials should also have strength gain characteristics that are consistent with the intended construction schedule, and flow characteristics that allow for proper placement and consolidation.

C5.5.2.5

Materials for closure joints can range from normal concrete to complex grouts made with various materials. Pre-mixed bagged grouts are common for small joint closures. Part 2 of this guide specification contains more information on specifying closure joint materials.

CME

Durability of ABC Technologies

5.5.3.1 DECK CONNECTIONS

Connections between prefabricated deck elements shall be detailed as moment connections using lapped bar reinforcement, post-tensioning, or mechanical devices.

The surface treatment of precast deck panels should be fabricated with an exposed aggregate finish in order to improve the bond of closure joint materials. The exposed aggregate surface need not be detailed with a specific minimum amplitude surface.

C5.5.3.1

Closure joints differ from expansion joints. Properly detailed closure joints have proven to be as durable as conventional concrete decks as documented in studies by the Utah DOT (2009, 2010, 2011, 2013).

This recommendation is based on the anecdotal experience of several state agencies where deck leakage was found at the panel closure joint interface. Exposed aggregate surfaces have shown to provide better aggregate interlock and contain less potential for surface contaminants such as carbonation. The amplitude of the exposed aggregate surface is not critical to provide adequate performance.

The studies noted above included a number of bridges built with post-tensioned connections built without exposed aggregate surface on the panel edges. These bridges have performed very well over a number of years.

CME

Durability of ABC Technologies

5.5.4 CORROSION PROTECTION OF POST-TENSIONING TENDONS AND ANCHORAGES

Post-tensioning systems shall be properly protected from corrosion. The level of protection shall be commensurate with the exposure of the tendons and anchorages to aggressive environments.

The corrosion protection of post-tensioning systems shall be based on standardized protection levels. As a minimum, the following three levels of corrosion protection should be used for prefabricated bridge systems:

- Concrete cover for ducts and anchorages equal to or greater than that used for element reinforcing steel.
- Corrosion protected ducts with quality duct splice devices
- High quality duct grout

Anchorages for deck post-tensioning systems should not be detailed directly at a deck expansion joint. The anchorage assembly should be encased in a short closure pour at the end of the deck. Adequate concrete cover should be specified around the anchorage assembly.

C5.5.4

The FHWA manual entitled *Post-tensioning Tendon Installation and Grouting Manual (2013)* contains information on corrosion protection. There are six level of potential corrosion protection for post-tensioning systems:

Level 1: Exterior surface of elements
Level 2: Concrete cover
Level 3: Corrosion resistant ducts
Level 4: Grout within ducts
Level 5: Sheathing or coatings on strand
Level 6: Corrosion resistance strand

The basis for these recommendations is from the Post-Tensioning Institute (PTI) Committee M-50/ASBI Joint Task Group. This group has identified standardized Tendon Protection Levels (PL) for Post-Tensioning Systems and can be found in the PTI document entitled *Guide Specifications for Grouted Post-Tensioning* (2010).

Small closure pours are common in precast deck systems. The pours address several issues including the accommodation of tolerances and joint systems. The closure pour can also provide corrosion protection for the end of the anchorage. Concrete cover above and below the anchorage can be difficult to achieve in a thin precast concrete deck. The cover should be equal to or greater than the cover for the reinforcing steel in the deck. Several manufacturers produce anchorages that are designed for thin decks. Designers should verify that these anchorages can fit within the deck system with the required cover.

CME

Part 2: Construction Guide Specifications

Similar to AASHTO LRFD Bridge Construction Specifications

- Specification on left, Commentary on right
- Not as detailed as state project specifications
  - There is no standard format for state specs
  - State specs reference other state specs
  - Focus is on guidance

Includes measurement and payment specifications where applicable

Section 2: Temporary Works

2.3 ROLES AND RESPONSIBILITIES

The general roles and responsibilities for bridge temporary works for the designer and the Contractor can be different on certain types of ABC projects when compared to conventional construction. The provisions listed below define the recommended approach to various types of ABC technologies.

2.3.1 DESIGNER

The designer should be responsible for the review of Contractor submissions regarding temporary works.

C2.3.1

The design and construction of bridge temporary works falls under the realm of "contractor means and methods."

Section 2: Temporary Works

2.3.1.1 Bridge Systems

The designer shall include schematic details of bridge temporary works on the plans indicating the falsework required to support the bridge system off site during construction and the falsework required for the movement of the bridge system.

If falsework is to be connected to or supported by the permanent structure, the designer shall estimate the connection forces and indicate them on the plans. The nature of the connection shall also be indicated.

The plans and specifications shall include allowances for modifications to the schematic plans provided that the modifications are verified and certified by the Contractor.

C2.3.1.1

Bridge system installations require significant falsework systems. Complete falsework details are not required since the actual bridge temporary works will be designed and detailed by the Contractor. The details should be schematic, showing the support and connection locations. The plans should not show specific members or elements. The plans should also contain notes that indicate that the design and layout of the falsework is by the Contractor.

The forces can include dead load, live loads and dynamic effects. The estimation of dynamic forces for bridge system installation can be found in the document entitled "Guidelines for Dynamic Effects during Bridge System Installations".

Section 2: Temporary Works

2.3.1.2 Geotechnical Investigations

The designer shall conduct preliminary geotechnical investigations for areas that are to support major bridge falsework. If geotechnical investigations are not undertaken during design development, the specifications shall require that the Contractor undertake the geotechnical investigations.

C2.3.1.2

The determination of major and minor falsework is subjective and should be left to best commercial practice and engineering judgement. Major bridge falsework would typically include structures that are supporting the entire bridge superstructures (bridge systems), or significant portions of the bridge structure. The recommended investigations are not mandatory. The intent is to reduce contractor risk, which can lead to lower project costs.

Minor falsework can be designed using presumptive soil bearing values included in the *AASHTO Guide Design Specifications for Bridge Temporary Works* (2017). Major falsework and falsework in locations with poor soils may justify a more detailed geotechnical investigation.

Section 2: Temporary Works

2.3.2 CONTRACTOR

In general, the Contractor is responsible for the design of temporary works.

2.3.2.1 Bridge Systems

The Contractor shall be responsible for the design of all temporary work required to install a bridge system. This includes construction of the bridge system off site and the falsework required for the movement of the bridge system.

All details and supporting calculations for temporary works shall be submitted as a working drawing to the designer or owner for review prior to the start of construction of the temporary works. This information shall be included in the Assembly Plan submission.

If falsework is to be connected to or supported by the permanent structure, the Contractor shall calculate the actual connection forces and indicate them on the Assembly Plans.

C2.3.2

The Contractor is typically given latitude in the construction of temporary works in order to take advantage of materials and equipment on hand.

C2.3.2.1

This requirement is consistent with current *AASHTO LRFD Bridge Construction Specifications*.

See Part 2, Section 3 for requirements of Contractor submissions for Assembly Plans.

Section 3: Fabrication and Assembly Planning

Fabrication specification expanded beyond what is in the AASHTO LRFD BCS

- Some agencies are allowing contractor precasting and near site precasting

What is assembly planning?

- Bicycle analogy
- Step-by-step plan for assembling the bridge
- An enhanced erection plan
- Schedule is an important part

Fabrication

Shop Drawings
- Requirements for shop drawings

Fabrication Checklists
- Pre-pour Inspection
- Casting Inspection including curing
- Post-pour Inspection
- Final Inspection Prior to Shipping

Dry-fit of assemblies
- Can be used for complex structures
- Sub-assemblies can be used

Repairs of non-conformances
- Use PCI Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products

CME

Assembly Planning

3.5 ASSEMBLY OF PREFABRICATED BRIDGE ELEMENTS

The Contractor shall be responsible for the development of an Assembly Plan document. The Assembly Plan shall be considered a Working Drawing and shall be certified by a registered Professional Engineer with specific knowledge of the Contractor's equipment and "means and methods".

Fabrication of bridge elements, or construction of the bridge shall not commence until the Assembly Plan is reviewed by the owner.

The Contractor shall coordinate the development of the Assembly Plans with the development of the Shop Drawings to ensure consistent detailing.

The Assembly Plan shall include, but not necessarily be limited to the requirements noted below.

C3.5

An Assembly Plan is a step by step process for the assembly of the prefabricated elements. It is closely linked to the schedule of operations and the interim material strengths necessary for the work to progress.

For example, if additional lifting hooks, grout ports, leveling devices, etc. are required, they shall be added to the shop drawings prior to approval.

CME

Assembly Plan - PBES

- Assembly Sequence and Construction Methods
- Lifting and Handling Procedures and Details
- Geometry Control Plan
- Interim Material Strength
- Post-tensioning (if applicable)
- Assembly sequence for post-tensioned full-depth deck panels (if applicable)
- Assembly time schedule

CME

Assembly Plan - Systems

- Assembly Sequence and Construction Methods
  - Temporary Falsework Plans
  - Lateral Slide Plan Requirements
  - SPMT Installation Plan Requirements

CME

Section 4: Layout and Tolerances

- Fabrication and Erection Tolerances
  - Should be included on the plans or in the specifications
  - If not specified---
    - Use "Guidelines for Prefabricated Bridge Elements and Systems Tolerances" (NCHRP 12-98 document)

CME

Section 4: Layout and Tolerances

- Structure Layout
  - Use of common horizontal datum
  - Use of common vertical datum
  - Use of working lines and working points
  - Geometry Control during construction

CME

Section 5: Concrete Structures

- Performance based concrete specifications
- Minimum requirements
  - Concrete Strength
  - Allowable Cement Types
  - Air Entrainment (if required)
  - Permeability (if required)

CME

Non-shrink Grouts

5.4.3 **NON-SHRINK GROUTS**

Materials for non-shrink grout shall be specifically formulated to meet the requirements of the intended purpose. Typical requirements include:

- High early strength gain (commensurate with the project schedule)
- Minimal shrinkage or slight expansion
- Good bond to adjacent concrete
- Durability equal to or better than the connected elements
- Easily placed (flowable for confined joints)
- Ability to extend grout to fill large volumes

C5.4.3

Non-shrink grouts are highly engineered materials. Pre-mixed package grouts are preferred over site mixed grouts due to the high demand placed on these materials.

CME

Non-shrink Grouts

The Contractor shall use pre-bagged non-shrink grout that is approved by the owner. The following testing shall be used for qualification of non-shrink grouts. In lieu of contract specific acceptance limits, the acceptance levels listed shall be met.

1. Compressive strength in accordance with AASHTO T 22 (ASTM C39) (Cylinders) or AASHTO T 106 (ASTM C109) (Cubes). The minimum compressive strength shall be 5 ksi measured at 24 hours after initial set.
2. Restrained shrinkage in accordance with AASHTO T 334 (ASTM C1581) (Ring Test). The specimen shall exhibit no cracking in 4 days.
3. Bond strength according to ASTM C1583. The minimum acceptable bond stress shall be 200 psi.
4. Durability Factor in accordance with AASHTO T 161 (ASTM C666). The specimen shall have a minimum durability factor of 80%.
5. Permeability in accordance with AASHTO T 277 (ASTM C1202) (Rapid Chloride Permeability).

The maximum permeability value shall be 1500 coulombs measured at 56 days.

In most cases, it is not feasible (cost and time) to perform pre-qualification testing for an individual project. It is recommended that owners require pre-qualification testing of non-shrink grouts for the establishment of a pre-qualified list of approved non-shrink grouts to be used in ABC connections.

There are a number of potential tests for the prequalification process. The document entitled "Material Characterization of Field-Cast Connection Grouts" (Swanty, Graybeal, 2011) includes a listing of potential tests and the results of testing on a number of commercially available grouts. There is no standard for the levels of performance for some of these tests. The owner should review these parameters and adjust the testing regime accordingly.

CME

High Early Strength Concrete

5.4.4 **HIGH EARLY STRENGTH CONCRETE**

In lieu of owner specified concrete mixes, the Contractor shall design and be responsible for the performance of a concrete mix used for closure joints and other rapid construction processes according to Section 8 of the *AASHTO LRFD Bridge Construction Specifications*, as amended herein.

The concrete shall be designed to meet the following goals:

- Meet the required compressive strength (both interim and final) in an accelerated manner
- Reduce the cure time for the concrete
- Provide durable (low permeability) concrete
- Provide low shrinkage properties to reduce cracking in the field

C5.4.4

This specification is based on performance specifications that have been successfully used in several states. This type of concrete is often used in closure joints. This specification has been used to produce concretes with strength of 4 ksi attained within 12 hours after initial set.

This specification does not have a specific strength or strength gain timeframe, since both will vary from project to project. The timeframe for strength development will greatly affect the concrete mix design and cost. Designers should consult with concrete producers for mixes that are required to develop strength in less than 12 hours in order to determine the feasibility of developing this type of mix using local materials.

CME

High Early Strength Concrete

The high early strength concrete shall conform to the requirements of Section 8 of the *AASHTO LRFD Bridge Construction Specifications* amended as follows:

1. Portland cement shall be Type II, IIA or III conforming to AASHTO M85 or M240, as appropriate.
2. Portland cement shall conform to AASHTO M85 with compatible admixtures and air entraining agent.
3. Water-cementitious material ratio shall not exceed 0.4 by weight, including water in the admixture solution and based on saturated surface dry condition of aggregates.
4. The amount of entrained air shall be 6.0 +/- 1.5% when tested according to AASHTO T 152.
5. The early strength characteristics of the concrete shall be commensurate with the intended construction procedure that is developed by the Contractor in the Assembly Plan.

The most significant concern with this use of concrete is restrained shrinkage leading to cracking. Therefore, special admixtures and testing is required to develop a mix that will cure with minimal cracking.

Some of the provisions contained herein are applicable to concretes used in northern environments where the concrete is exposed to freeze thaw cycles and de-icing salts. The designer may waive the applicable portions of these specifications for bridges in more temperate climates.

CME

High Early Strength Concrete

6. A shrinkage reducing admixture shall be tested by an approved testing lab and meet the requirements of ASTM C494 Type S, except that in Table 1 length change shall be measured as: Length Change (percent of control) shall be a minimum of 25% less than that of the control. Table 1 Length Change (increase over control) shall not apply. Shrinkage reducing admixtures shall not contain expansive metallic materials.
7. It is recommended that the mix be tested for restrained shrinkage according to AASHTO T334. The test is considered successful if there is no cracking in the sample at 14 days maturity.

The cited restrained shrinkage test is a test that can be used to compare one mix against another during mix development. The cited recommended age to cracking is approximate based on typical test results of successful High Early Strength concretes. Cracking of a sample before 14 days does not necessarily mean that the mix is inadequate. Likewise, cracking of a sample after 14 days does not guarantee that the material will not crack in the field. The field test specified in Article 5.4.4.2 is a more accurate test of intended field performance. The AASHTO T334 test can be used to compare the High Early Strength mix to normal deck concrete. If the two mixes perform in a similar manner, the Owner and Contractor may elect to move forward with the field test noted in Article 5.4.4.2.

8. The maximum allowable total chloride content in concrete shall not exceed 0.1% by weight of cement.
9. The concrete mix design shall have a maximum rapid chloride ion permeability of 1500 Coulombs at not more than 56 days when tested according to AASHTO T 277. Multiple samples shall be tested using the installed curing methods in order to establish the required cure times for the mix.

CME

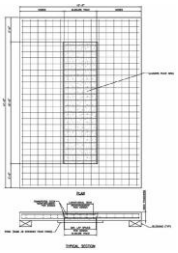
High Early Strength Concrete

- Additional Requirements
  - Mix design development (by the contractor)
  - Use of maturity meters (allowed)
  - Confined shrinkage test (AASHTO Ring Test)
  - Field Trial Placement
    - Large scale confined shrinkage test
    - Includes recommended pass fail requirements

CME

High Early Strength Concrete

Field trial placement



CME

Ultra-High Performance Concrete

Performance Specification

- Developed by FHWA
- General mix requirements

5.4.5 **ULTRA HIGH PERFORMANCE CONCRETE FOR CLOSURE JOINTS**

The Contractor shall supply a pre-mixed manufactured Ultra High Performance Concrete (UHPC) that meets the performance requirements specified below.

The general mix requirements shall conform to the following:

- Water-to-cementitious materials ratio < 0.25
- Portland cement content ≥ 1,000 lb/yd^3
- Maximum aggregate size ≤ 0.25 times the fiber length.
- Maximum aggregate size ≤ 0.125 inches (3.2 mm)
- Contains fiber reinforcement ≥ 2 percent (by volume)

UHPC shall be capable of resisting a sustained post-cracking tensile stress of greater than 725 psi.

C5.4.5

The provisions contained in this Article are based on the FHWA Publication entitled "Design and Construction of Field-Cast UHPC Connections" (Graybeal, 2014), which is a synthesis of research completed at FHWA and other organizations.

The availability of UHPC products that meet this specification may be limited. If only one company can produce UHPC to these specifications, the owner should treat the item as a proprietary product and obtain the required approvals for use of such items.

Materials for fiber reinforcement can vary. The most common UHPC in use contains steel fiber reinforcement that is approximately 0.008 inches in diameter and 0.5 inches in length. The fibers are straight fibers with a minimum tensile strength of 290ksi. Other fiber materials can be used if the required material performance requirements of Article 5.4.4.1 are met.

A specific AASHTO or ASTM specification is not available for tensile testing of UHPC. The Manufacturer should provide testing results that demonstrate the behavior of the material in sustained tension after cracking.

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Performance Testing

Table 5.4.5.1-1 – Recommended Performance Testing for UHPC Mixes

Test Method	ASTM	Pre-Const. Testing	Constr. Testing	Testing Frequency	Acceptance Requirements
Flow	C1437	Yes	Yes	Once per mix	• Flow range from 7 to 10 inches
Compressive strength	C39 C109	Yes	Yes	At least once per 25 yd^3 (19 m^3) or once per 12-h shift	• > 14 ksi after 4 days • > 21 ksi after 28 days • > 14 ksi before application of construction or live loads
Chloride ion penetrability	C1202	Yes	Not Required	N/A	• ≤ 500 coulombs by 28 days
Freeze-thaw resistance	C666A	Yes	Not Required	N/A	• RDM ≥ 95 percent after 300 cycles
Shrinkage	C157	Yes	Not Required	N/A	• ≤ 800 microstrain at 28 days • Consider curing scenarios

Note: RDM = Relative dynamic modulus of elasticity

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Other requirements

- Surface preparation
  - Exposed Aggregate Finish
  - SSD Conditions
- Formwork for UHPC
  - Properly sealed
  - Use of top forms for sloped pours
  - Slight overfilling to provide slight pressure head
- Mixing UHPC
  - Temperature monitoring
  - Mixing equipment

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Placing UHPC

- Follow manufacturer's recommendations
- No Vibration

Finishing UHPC

- Cast slightly higher than required
  - There can be settlement during set
- Surface grinding
  - Hand grind after set
  - Machine grind after 14 ksi

Curing UHPC

- Sealed forms
- No freezing until strength reaches 10 ksi
- Live load may be applied when strength reaches 14 ksi

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Other Concrete Construction Specs

Surface Preparation for Concrete Connections

- Connections subjected to water
  - Exposed aggregate finish or cleaned of laitance
  - Saturated Surface Dry Condition prior to casting

The interface of the concrete elements shall be prepared to a Saturated Surface Dry condition (SSD) prior to placement of cementitious joint materials. The surface shall be moist, but free of standing water.

Saturated Surface Dry is defined as the condition of a surface where the surface is "dry" (*i.e.*, surface adsorption would no longer take place), but the inter-particle voids are saturated with water. The process of preparing a surface to a SSD condition normally requires pre-wetting for at least several hours prior to placement of joint materials.

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Other Concrete Construction Specs

Grouted Joints and Keys

- Forming
- Limited use of backer rods
- Mock-ups for complex grouting

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Other Concrete Construction Specs

Grouted Coupler Connections

- Materials
  - Prequalified Lists
  - Certified Test Reports
  - Grout supplied by coupler manufacturer
- Installation Methods
  - According to manufacturer
  - Includes recommended installation sequence
- Field testing
  - Grout cube tests

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Other Concrete Construction Specs

Post-tensioned connections

Closure Joints

- Headed Reinforcing Bars
- Precast Full Depth Deck Panel Shear connector blockouts on PS Girders
- Link Slabs

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Section 6: Steel Structures

Not a lot required for this specification

- Covered by AASHTO LRFD Bridge Construction Specifications

Items covered in minor detail

- Simple span bridges made continuous for live load
  - Pre-assembly in fab shop for complex details
- Modular decked beams
  - Need for coordination between steel fabricator and precast fabricator
- Links slab beams
  - Need for surface smoothness in link slab regions

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Section 7: GRS/IBS

Based on FHWA Implementation Guide

- This guide spec references the implementation guide for more details

Facing Blocks

- Higher level of quality for freezing environments
  - CMU Block (dry cast): Freeze thaw resistance, higher strength, lower moisture absorption
  - Precast block (wet cast) facing allowed

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Section 7: GRS/IBS

Other material specifications
- Backfill and geosynthetic materials

Construction requirements
- Reinforced soil foundation
- Placement of backfill and compaction
- Placement of geosynthetic reinforcements
- Wall facing
- Leveling course
- Beam seat and superstructure placement
- Approach integration
- Site drainage
- Quality control

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Questions?

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