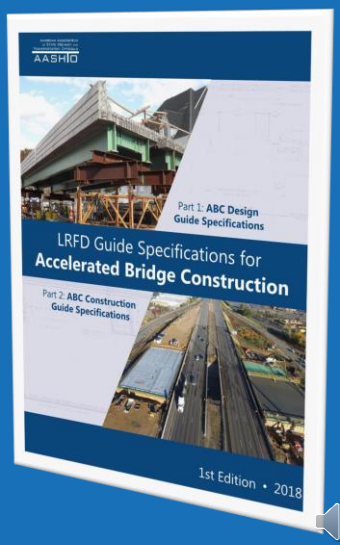
Module 1: General Provisions and Common Connections

This Module Covers:

- ABC Definitions
- Designer & Contractor Responsibilities
- Tolerances (NCHRP Project 12-98)
- Loads and Load Combinations
- Design of Prefabricated Elements
- Design of Common Connections



The first Module covers:

- ABC Definitions
- Designer & Contractor Responsibilities
- Tolerances (NCHRP Project 12-98)
- Loads and Load Combinations
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- Design of Common Connections

Section 1: Definitions

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

2

At this point I would like to walk through some of the sections of the AASHTO LRFD Guide Specifications for ABC in more detail.

Section 1 of the guide specification includes definitions for common ABC terms. It is important to use common terms in specifications and in practice. The Federal Highway Administration and the AASHTO T-4 Committee on Construction have developed consistent definitions for ABC. These definitions are used throughout the document and in this workshop.

Section 1: Definitions

Prefabricated Bridge Elements (PBE)

A single prefabricated structural component of a bridge. Prefabricated bridge elements can be made of any approved structural material.

The first major definition is for “Prefabricated Bridge Elements”. You have all probably heard the term “Prefabricated Bridge Elements and Systems” or PBES. “Prefabricated Bridge Elements” are a subset of this term.

Examples of Prefabricated Bridge Elements include individual elements such as columns, pier caps, wall panels, deck panels, etc.

Section 1: 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.

4

A "prefabricated system" is also a subset of prefabricated bridge elements and systems. These differ from prefabricated elements in that they comprise a larger portion of the overall bridge.

The most common types of Prefabricated Systems are bridge superstructures that are installed using Self-Propelled Modular Transporters or Lateral Sliding.

Section 1: 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.

5

I know, most of you are probably familiar with Self-Propelled Modular Transporters. These are highly maneuverable machines that can lift very large loads. They have high load capacity and are capable of fine movement and rotation along all three major axes.

They are commonly used to install entire bridge superstructures and as quickly as one hour.

If you are not familiar with these machine, we will go over the use of them in more detail later on in the workshop.

Section 1: 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.

6

"Lateral Slide" is a method of building a bridge adjacent to an existing bridge and quickly sliding it into its final position after removal of the existing bridge.

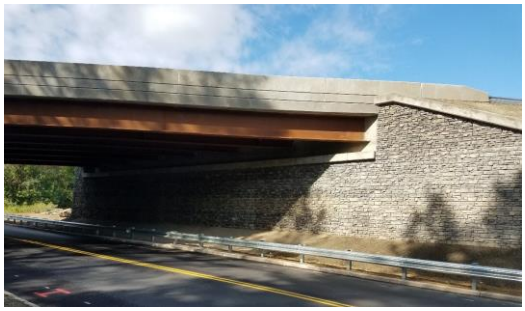
The Federal Highway Administration has also used the term "Slide In Bridge Construction" or SIBC for this method of ABC. The AASHTO Technical Committee for Construction has adopted the term "Lateral Slide", therefore that term is included in the Guide Specification.

This method of ABC has become quite popular. It is best applied to bridges over local roads and rivers. Building a bridge parallel to an existing bridge over busy feature would still impact traffic below.

Section 1: 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



7

"Geosynthetic Reinforced Soil - Integrated Bridge System" is a novel approach to building embankment abutments quickly. The key term in this long name is "integrated". This means that the superstructure is integrated into the abutment to act as one structure in a similar fashion to integral abutment bridges.

It is comprised of multiple layers of geosynthetic fabric combined with backfill soils to form a flexible abutment that can accommodate thermal movement and beam end rotations. GRS-IBS bridges do not require bearings or approach slabs, as the approach embankment is integrated into the superstructure .

A GRS-IBS bridge abutment can be constructed in a few days with minimal equipment, which is why it is considered an ABC technology. These structures are also very cost effective and aesthetically pleasing.

The Federal Highway Administration has published several documents on design of GRS IBS bridges, therefore we will not focus on this in today's workshop.

Section 1: 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.

In the ABC world, the term "closure pour" was quite common, however all joints do not have materials "poured" into them. Some joints have flexible fillers or are open. For this reason, the AASHTO Construction Technical Committee decided to use the term "closure joint" to describe all gaps between elements that are either filled or unfilled.

Section 1: 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.

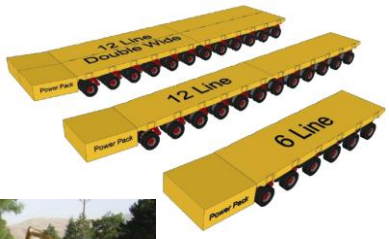
Emulation is a term that is commonly used in prefabrication design. It means that a design is detailed to “emulate” cast-in-place concrete construction.

By using emulation, the design of precast elements is simplified and the performance is well understood, which is why this method of design and detailing is popular.

Many articles in the guide specification and the research to back them up are based on emulation.

GS Section 1: Designer & Contractor Responsibilities

- Recommended responsibilities are defined for both the designer and the contractor for:
  - Prefabricated bridge elements *
  - Systems **
    - SPMTs
    - Lateral Slide



* Covered in this Part of the workshop
** Covered in Part 10 of the workshop

Section 1 of the Guide Specifications for ABC covers and defines the recommended responsibilities of the designer and the contractor on ABC projects. This section covers both prefabricated elements and prefabricated systems. The photo in the center shows a lateral slide system. The sketch to the right shows several SPMT configurations.

Early ABC projects had blurred these lines and created unnecessary rework of designs by the contractor. For example, designers would design and detail temporary falsework on contract drawings, followed by a contractor redesign in construction. Another example includes designs incorporating lifting locations and lifting methods for prefabricated elements. In many cases a fabricator would need to change these methods to accommodate the equipment available.

This is especially true for projects built with SPMTs and lateral slide technologies that are heavily depended on falsework and specialized equipment. The key to this division of work is separating out the design of the permanent portions of the bridge from contractor means and methods.

PBE: Designer Responsibilities

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 these Guide Specifications.

The Designer may 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.

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.

C1.4.1

Recommendations for the design of elements and connections are included in these Guide Specifications.

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

Article 1.4.1 of the Guide Specifications for ABC defines the responsibilities of the Designer in ABC projects. First and foremost, the Designer is responsible for designing the bridge, the elements, and the connections in accordance with the AASHTO LRFD Bridge Design Specifications, as supplemented by the Guide Specifications for ABC.

The use of emulative design means that standard design practices and specifications can be followed in most cases. The Guide Specifications for ABC include guidance on the minor changes required in design to accommodate prefabrication.

Handling: Designer Responsibilities

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 should 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 should 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 Section 8.4 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."

12

One of the most commonly misunderstood facets of ABC is the shipping and handling of elements. Designers have commonly thought that it was their role to ensure that an element could be lifted, transported, and erected without damage. The handling of prefabricated beam elements in the past has almost always fell under the realm of contractor means and methods. The exception to this is very large elements that may be unstable such as long span beams and girders.

Article 1.4.1.1 of the Guide Specifications for ABC take a similar approach. Designers need not specify shipping and handling requirements for most elements. The Designer is responsible for reviewing the contractor's calculations for shipping and handling. The specification includes provisions for calculating handling forces for concrete elements, which should be used by the contractor or fabricator to determine the handling stresses. These provisions can also be used by the designer to review the contractor's submissions.

PBEs: Contractor Responsibilities

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.

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.

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 the Construction sections of these Guide Specifications for more information on element shipping and handling specifications.

13

The responsibilities of the contractor are outlined in Article 1.4.2. This is primarily limited to contractor means and methods such as temporary works and shipping and handling of prefabricated elements .

The AASHTO Guide Design Specifications for Bridge Temporary Works are referenced here. These provisions are consistent with current practice for conventional construction.

Handling: Designer Responsibilities

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 *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

14

One of the most common causes of problems on ABC projects is improper fit-up of elements. In many cases, this is due to lack of control on tolerances and a lack of geometry control during erection.

Article 1.4.1.2 of the Guide Specifications for ABC include recommendations for the establishment of tolerances for four elements and joints. A guideline developed under NCHRP Project 12-98 is referenced for this purpose.

NCHRP Project 12-98 focused on developing recommended tolerances and detailing requirements for tolerances for prefabricated elements. A guideline was published entitled *Guidelines For Prefabricated Bridge Elements and Systems Tolerances*. This guideline, which is written in AASHTO specification format can be used to develop details and specifications for projects with prefabricated elements. The guideline includes recommended fabrication and erection tolerances for the most common prefabricated elements in use. The guideline also includes recommended tolerances for joint width between elements. If followed, this guideline will help to ensure that prefabricated elements will fit together in the field in fall within the geometric requirements of the design.

12-98: Erection Layout Drawings

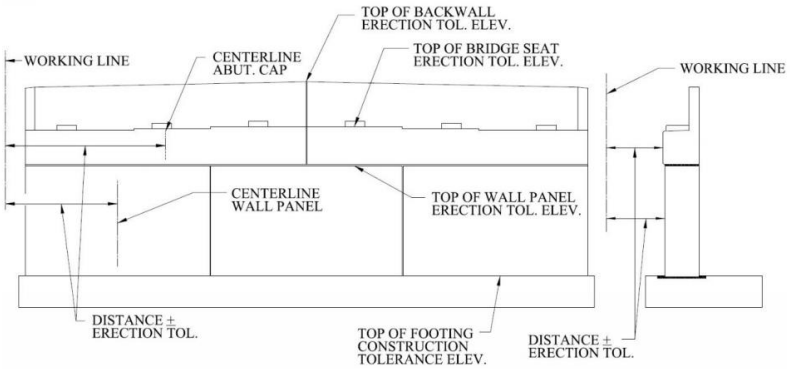
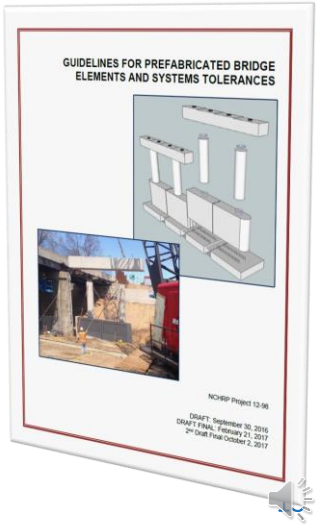


Figure 3.10-2 Example of Abutment Erection Layout
(Center of Element Method used for Longitudinal Layout
Surface of Element Method used for Transverse Layout)



Please take note that the header on the top of this slide is now purple, which denotes discussion regarding the reference document entitled *Guidelines For Prefabricated Bridge Elements and Systems Tolerances*. In this guideline, it is recommended that the designer show and specify element tolerances and erection tolerances on the contract drawings. It is recommended that all erection tolerances be measured from a common working line or working point. Designers should never specify layout based on center to center of elements , as this will lead to a buildup of tolerances and loss of the overall geometry of the bridge.

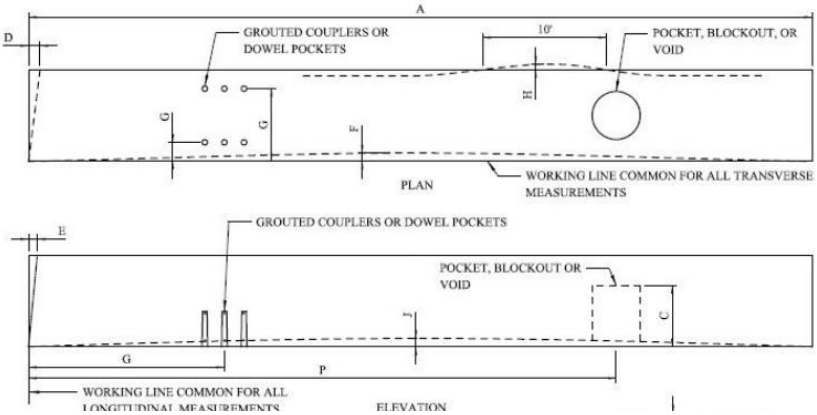
There are two methods for specifying erection tolerances. The first is the “center of element” method. In this method, erection dimensions are measured to the center of each element. This is typically used for most elements, to ensure that the length and spacing of the elements remains consistent.

The second method for specifying erection tolerances is called the “face of element” method. This method is typically used to produce a smooth face or edge along a structure. Examples would include the exposed face of walls, or the edge of a bridge deck.

It is acceptable to use both methods on an element. The details on the slide show that a wall panel can be erected lengthwise using the center of element method and transversally using the face of element method.

12-98 Element Tolerances

Pier Cap Example

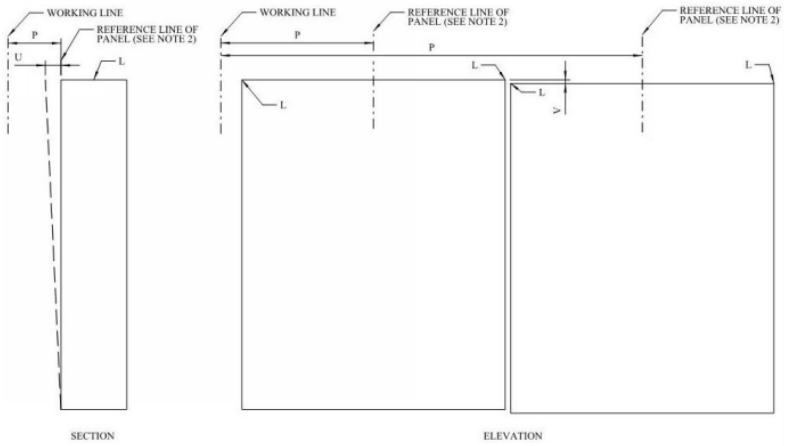


The tolerance guideline includes recommended tolerances for common prefabricated elements. This is an example of a fabrication tolerance specification detail that should be included on the plans or in the specifications. The detail shown is for a pier cap element. The tolerance guide includes recommended values for each of these tolerances. The tolerances given are based on years of industry experience.

The tolerances include overall element size, skew, and flatness. Also included are tolerances on voids inserts in projecting appurtenances.

12-98: Erection Tolerances

Wall Panel Example



This is an example of wall panel erection tolerances. This includes tolerances on spacing, elevation, plumbness, and variation from panel to panel.

The tolerance guide includes recommended values for each of these tolerances.

12-98: Joint Tolerances

4.5.1 Specified Joint Width or Thickness

The designer should specify the joint opening defined on the plans that accounts for the minimum tolerable joint opening plus the tolerances of the two adjoining elements.

Specified vertical joint width (side-by-side elements):

$$S_{jw} = t_{min_jw} + T_{jw} \quad (4.5.1\text{-}1)$$

Specified horizontal joint thickness (stacked elements):

$$S_{jt} = t_{min_jt} + T_{jt} \quad (4.5.1\text{-}2)$$

where:

S_{jt} = specified joint thickness (in.)
S_{jw} = specified joint width (in.)
t_{min_jt} = minimum tolerable joint thickness (in.)
t_{min_jw} = minimum tolerable joint width (in.)
T_{jw} = joint width tolerance (in.)
T_{jt} = joint thickness tolerance (in.)

C4.5.1

The following articles contain recommended joint thicknesses and widths for typical element layouts. The recommended values are based on the recommended element fabrication tolerances included in Section 2 and the recommended erection tolerances included in Section 3. The following articles also contain the equations used to generate the recommended tolerances in the commentary. The designer can use these equation to calculate joint width/thickness tolerances for elements with tolerances that differ from the recommended tolerances in Section 2 and section 3.

The determination of the minimum tolerable joint width/thickness is based on the materials that are to be used within the joint. For example, a joint with non-shrink grout may have a minimum tolerable width of ½", while a joint with concrete may have a minimum tolerable width of 1" (depending on the size of the aggregate).

This slide shows a portion of the guidelines for tolerances. As you can see, the tolerance guideline is similar to AASHTO specifications with guidelines on the left and commentary on the right.

An important aspect of a successful assembly of a prefabricated bridge is the joints between the elements. Joints are used to accommodate the fabrication and erection tolerances that we specify. If you choose to specify large tolerances, you will need wide joints to accommodate them. Likewise, if you want narrow joints you need to specify tighter fabrication and erection tolerances. The goal is to strike a balance between these two approaches.

This article defines the proper way to specify tolerances on contract plans. To do this, you need to determine the specified joint width, and the joint width tolerance. The document gives guidance on how to calculate these two values for different types of elements.

The specified joint width is set equal to the minimum allowable joint width plus the joint width tolerance. The minimum joint width is based on the material that is to be placed within the joint. For example, if grout is to be used, the minimum installation width specified by the manufacturer should be used.

12-98: Joint Tolerances

Element fabrication dimensions should be based on the specified joint thickness or width. The detail call out for a joint on the plans should be the specified joint width/thickness plus or minus the joint width/thickness tolerance denoted as follows:

Horizontal joint detail call out = $S_{jt} \pm T_{jt}$
Vertical joint detail call out = $S_{jw} \pm T_{jw}$

See Article 4.5.2 to obtain specified joint width and joint thickness tolerances, T_{jw} and T_{jt} respectively.

Example for joint width detail call out:
Given:

Minimum tolerable width = ½"
Joint width tolerance = $\pm$ $^5/_8$"
Specified joint width = $1^1/_8$"
Therefore:
Detail call out = $1^1/_8$" $\pm$ $^5/_8$"

Once you have determined the specified joint width and the joint width tolerance, you can detail the plans as follows:

The callout for the specified width of the joint should be the specified joint width plus or minus the joint width tolerance. The specified joint width is simply the narrowest joint that can be accommodated by the fill material plus the joint width tolerance. If the maximum negative tolerance occurs, the joint width will equal the minimum tolerable joint.

12-98: Joint Tolerances

4.5.2 Specified Joint Opening Tolerance

The specified joint opening tolerance is set based on the following factors:

- The fabrication tolerances of the two joined elements
- The erection tolerance of the two joined elements
- The probability of occurrence of additive tolerances

C4.5.2

The specified joint width should take into account the probability that all of the maximum tolerances will occur at the same time. If maximum tolerances are used in the joint width equations, overly wide joints will result.

The specified joint opening tolerances in this article were determined through the use of probability simulations (Monte Carlo method). These simulations were used to obtain more realistic tolerance values based on the probability of occurrence of each tolerance. The simulations included both the fabrication and erection tolerances.

The basic theory of Monte Carlo simulation is the creation of many "what if" cases to determine the expected results of a study. It allows users to account for risk in quantitative analysis and decision making. The joint width/thickness values included in this Section are based on a 95% probability of achieving a success connection. The 5% probability of exceeding the joint width tolerance does not necessarily result in a significant problem. This scenario would require adjustment to be made to the elements (grinding of edges, re-setting of elements).

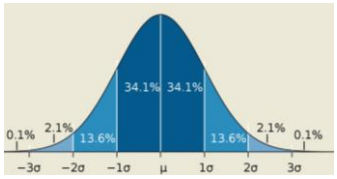
Article 4.5.2 of the tolerance guideline includes recommended tolerances for joint widths. The development of this guideline was based on significant statistical evaluation of the impact of element tolerances and erection tolerances on the width of constructed joints.

The key to specifying joint widths is understanding the probability of occurrence of additive tolerances. One way to do this is to assume that all negative tolerances occur simultaneously. If you were to do this the required width of joints would be very large. It is important to know that the probability of this occurring is near zero, therefore it is not recommended.

To account for the probability of multiple tolerances occurring at the same time, statistical Monte Carlo simulations were run in the research to establish a reasonable probability of occurrence of multiple tolerances. In the upcoming slides we will review the results of this process in more detail.

Monte Carlo simulations

$$[0.5(A_L + A_R + L_{HL} + L_{HR}) + \text{maximum of } (D_L + D_R),(H_R + H_L)]$$



$$\pm 0.34\,[0.5(A_L + A_R + L_{HL} + L_{HR}) + \text{maximum of } (D_L + D_R),(H_R + H_L)]$$

This is an example of how Monte Carlo simulations were used to develop joint with tolerance specifications. The equation at the top of the screen defines all of the tolerances that could affect the width of a joint. The terms correspond to the tolerances of the left and right elements as follows:

- The A terms correspond to the width of the elements
- The L terms correspond to the erection tolerances
- The D terms corresponds to the flatness of side of the element
- The H term corresponds to the squareness of the left and right element.

The reason for the 0.5 at the beginning of the equation is based on the assumption that the width and erection tolerance would be split on either side of the element. The reason for the maximum in the second part of the equation is the assumption that the flatness would occur along the side of the element and the squareness would affect the corner of the element, Therefore they would not occur at the same location.

Each of these terms have variability based on probability of occurrence. In a Monte Carlo simulation, we can assign the distribution of variability for each term in the equation. Research has shown that variability of tolerances are typically a statistical “normal distribution” or a Bell curve. In a Monte Carlo simulation, thousands of simulations are generated by the software to simulate a real world situation. The terms of the equation are varied based on the statistical variation defined.

The result of this analysis is a Bell curve for the <u>entire equation</u>. If we take the mean plus or minus 2 standard deviations, we will have a value that encompasses 95% of the potential outcomes. We then compare those values with the results if the maximum tolerances are inserted in the equation, and then calculate a ratio of the two. The 0.34 in the equation is that ratio.

12-98: Joint Tolerances

Table C4.5.2.1 – Vertical Joint Tolerance for Side-by-Side Elements, T_{jw}

The equations in this table were used to develop the vertical joint width tolerances noted in this article. The 0.21 to 0.34 factors are the adjustment to the maximum tolerances to account for the probability of all maximum tolerances occurring simultaneously.

Element Description		Vertical Joint Width Tolerance
Left	Right	T_{jw}
Approach Slab	Approach Slab	$\pm 0.33\ [0.5(B_L + B_R + L_{HL} + L_{HR}) + \text{maximum of } (D_L + D_R),(H_L + H_R)]$
Approach Slab	Sleeper Slab	$\pm 0.34\ [0.5(A_L + B_R + L_{HL} + L_{HR}) + \text{maximum of } (D_L + D_R),(H_L + H_R)]$
Sleeper Slab	Sleeper Slab	$\pm 0.34\ [0.5(A_L + A_R + L_{HL} + L_{HR}) + \text{maximum of } (D_L + D_R),(H_R + H_L)]$
Pier Cap	Pier Cap	
Panel, Footing, Wall Cap	Panel, Footing, Wall Cap	
Full-Depth Deck Panel	Full-Depth Deck Panel	$\pm 0.21\ [0.5(A_L + A_R + L_{HL} + L_{HR}) + \text{maximum of } (D_L + D_R),(H_R + H_L + F_L + F_R)]$
Modular Deck Beam	Modular Deck Beam	$\pm 0.28\ [0.5(A_L + A_R + L_{HL} + L_{HR}) + \text{maximum of } (D_L + D_R),(H_R + H_L + F_L + F_R)]$

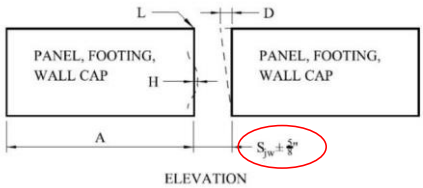


Figure 4.5.2.1-5 Panel to Panel, Footing to Footing, Wall Cap to Wall Cap

where:

A_L = maximum fabrication length tolerance of left element (in.)
A_R = maximum fabrication length right element (in.)
B_L = maximum fabrication width tolerance of left element (in.)
B_R = maximum fabrication width tolerance of right element (in.)
D_L = maximum fabrication end skew/squareness tolerance of left element (in.)
D_R = maximum fabrication end skew/squareness tolerance of right element (in.)
F_L = maximum fabrication sweep tolerance of left element (in.)
F_R = maximum fabrication sweep tolerance of right element (in.)
H_L = maximum fabrication smoothness tolerance of left element (in.)
H_R = maximum fabrication smoothness tolerance of right element (in.)
L_{HL} = maximum horizontal erection tolerance of left element (in.)
L_{HR} = maximum horizontal erection tolerance right element (in.)

This is an example of the results of the Monte Carlo simulations and how it can easily be used in the detailing of a prefabricated bridge. It is possible to write a logical equation that defines the width of the joint required based on the maximum tolerances for fabrication and erection. The equations shown in the table to the left were developed for various element-to-element joints. Certain assumptions were made.

We do not intend to have designers perform Monte Carlo simulations for each design. If designers use the recommended tolerances in the guideline, the answer is given as a simple value. That is shown in the right figure on the slide. The equations are given for cases where a designer or owner might choose to use different tolerances than those recommended in the guideline. In this case, the Designer would simply enter the tolerances into the equation given in the table and calculate the joint width tolerance.

12-98: Joint Tolerances

Example of Vertical Joints between Approach Slabs

B_L	=	$^1/_4$ (in.)	(Article 2.7.6)
B_R	=	$^1/_4$ (in.)	(Article 2.7.6)
L_{HL}	=	$^1/_2$ (in.)	(Article 3.9.1 assuming an approach slab is similar to a footing)
L_{HR}	=	$^1/_2$ (in.)	(Article 3.9.1 assuming an approach slab is similar to a footing)
D_L	=	$^1/_2$ (in.)	(Article 2.7.6)
D_R	=	$^1/_2$ (in.)	(Article 2.7.6)
H_L	=	$^1/_8$ (in.)	(Article 2.7.6)
H_R	=	$^1/_8$ (in.)	(Article 2.7.6)

Vertical Joint Width Tolerance

$= \pm\, 0.34\, [0.5(B_L + B_R + L_{HL} + L_{HR}) + \text{maximum of } (D_L + D_R),(H_L + H_R)]$

$= \pm\, 0.34\, [0.5(^1/_4 + {}^1/_4 + {}^1/_2 + {}^1/_2) + \text{maximum of } (^1/_2 + {}^1/_2),\ (^1/_8 + {}^1/_8)]$

$= \pm\, 0.34\, [0.5(1\tfrac{1}{2}) + 1'']$

$= \pm\, 0.34\, [1.75]$

$= \pm\, 0.6''$ *(say 5/8")*

Example for joint width detail call out:

Given:

Minimum tolerable width = ½"

Joint width tolerance = $\pm\, ^5/_8$"

Specified joint width = $1\,^1/_8$"

Therefore:

Detail call out = $1\,^1/_8$" $\pm\, ^5/_8$"

This slide shows an example of how a designer could calculate a joint width tolerance based on revised element and/or erection tolerances.

The upper portion of the slide shows the tolerances that designer might specify. The designer simply needs to enter these maximum tolerances into the equation to calculate the required joint width tolerance.

In this example, the joint width tolerance was calculated at 0.6 inches, or 5/8 inches. For this example, the Designer has determined that the minimum tolerable joint width is 1/2 inch. Therefore, the specified joint width would be the total of the minimum joint width plus the joint width tolerance, which equals 1 1/8 inches. The detail call out for this joint would then be 1 1/8 inches plus or minus 5/8 inches.

If the maximum negative tolerance occurs, the joint width will equal the minimum tolerable joint. For this example, this would be 1 1/8 inches minus 5/8 inches, which equals ½ inch.

This can be a little confusing at first, but it does make sense once you look at the potential scenarios.

GS Section 2: 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.

Section 2 of the Guide Specifications for ABC also includes recommended provisions for loads and load combinations.

As is discussed before, emulation is commonly used for the design of prefabricated ABC bridges. Based on this, the design would logically follow the provisions of the AASHTO LRFD bridge design specifications.

GS Section 2: 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_hDL (2.4.1.1-1)
Strength I = γ_p γ_h DL (2.4.1.1-2)

where:

γ_h = 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_h 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).

25

As we previously discussed, shipping and handling calculations are typically done by the contractor, fabricator, or erector. Article 2.4.1.1 includes provisions for calculating shipping and handling forces and stresses for use by these entities. Designers can use these for reviewing calculations submitted by these entities.

The forces acting an element during shipping and handling are dynamic in nature. This is handled by applying a dynamic dead load factor to the elements at various stages of their construction. This is a similar approach to the use of dynamic load allowance for live load in the AASHTO LRFD Bridge Design Specifications, except the factors are applied to the dead load of the element.

GS Section 2: Loads and Load Combinations

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

Form finish	γ_h
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:
   $\gamma_h = 1.2$

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

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.).

The Guide Specifications for ABC include recommended dynamic dead load factors for fabrication, shipping and handling of precast concrete elements. The factors are shown in the slide.

As you can see, different factors are applicable to different shipping and handling situations. It is also important to note that the strength of concrete elements may vary at these different situations. For example, the form finished factors should be applied to a precast element with the concrete strength specified by the fabricator at time of removal from the forms. The same is true for yard handling of the element.

The shipping and erection factors should be applied to the element based on the specified concrete strength at each time interval. For example, the final concrete strength would most likely be used for the erection situation.

In many cases, the controlling load case is during the removal of the precast element from the forms.

Handling Calculations - PCI

- Determine the dead load factors
- Determine the pick locations (rigging)
- Analyze the element to obtain handling stresses
- Check the element for cracking
  - No discernable cracking criteria = modulus of rupture/1.5
- At removal from form: use f'ci
- Shipping: use f'c

The Guide Specifications for ABC reference the PCI Design Handbook for recommended methods for shipping and handling calculations. This document has been used by the precast industry for years, it has provided satisfactory results.

The designer should specify the acceptable tension in the concrete during shipping and handling. The PCI Design Handbook recommend a value equal to the Modulus of Rupture divided by 1.5 as an acceptable way to achieve "no discernible cracking". It should be noted that the modulus of rupture calculated in the AASHTO LRFD Bridge Design Specifications can be quite variable, therefore this prediction is not guaranteed. Experience has shown that this will be sufficient in most cases. We will go through an example of these types of calculations later on in this workshop.

Review the steps for these calculations as noted above

Section 3: Design of PBEs

Largest Section in the Guide Specification

- 85 pages

Includes Seismic provisions

- Not covered today workshop
- This workshop
  - Non-seismic details
  - Many provisions applicable to seismic regions as well

Section 3 of the Guide Specification covers the design of prefabricated bridge elements and connections between elements. It is the largest section in the guide specification at 85 pages.

This section includes seismic provisions, however they will not be covered in today's workshop. A separate seismic workshop module will cover those provisions.

The workshop was set up this way since many states are not required to design for seismic events.

Section 3: Design of PBEs

3.5—PREFABRICATED ELEMENT DESIGN

3.5.1—CONCRETE ELEMENTS

Concrete elements shall be designed and detailed to meet all the requirements in the *AASHTO LRFD Bridge Design Specifications*.

Most common design approach:
Emulative design

C3.5.1

The use of concrete PBEs is very common. Concrete PBEs have been used for virtually every element in a bridge.

This article contains descriptions of various concrete PBEs that are in use. These elements are manufactured using precast concrete. In some cases, the elements are prestressed (pretensioned and/or post-tensioned).

Emulative design, where the elements are designed to emulate cast-in-place concrete behavior, is the most common approach for precast concrete elements. Emulative design of precast concrete elements is consistent with the design of cast-in-place concrete; therefore the elements shall be designed to meet all of the requirements of the *AASHTO LRFD Bridge Design Specifications*, including strength and serviceability. Likewise, prestressed elements are designed using the appropriate provisions of the *AASHTO LRFD Bridge Design Specifications.* Prestressed elements have their unique design, construction and behavior; thus, they will not typically be emulative of CIP concrete, especially pretensioned elements.

Article 3.5.1 of the Guide Specifications for ABC notes that the most common method for design of prefabricated elements is the emulative design method. This method is not new to the precast industry, as the American Concrete Institute had published a document on emulative design many years ago.

Article 3.5.1 of the Guide Specifications for ABC recommends that the design of concrete elements and connections be in accordance with the AASHTO LRFD Bridge Design Specifications, which is the basis for the emulative method of design.

Section 3: Connection Design

Most common connections covered:

- CIP closure joints using lapped bar reinforcement
  - Straight bars
  - Headed bars
  - Hooked bars
  - UHPC with straight bars
- Mechanical Connectors
- Bars inserted in grouted pt ducts
- Pocket connections:
  - Receive reinforcing from other element
  - CMP Pipe Voids
- Socket connections:
  - Portion of one element inserted into a void in another

A significant portion of Section 3 in the Guide Specification covers connections between elements. This is an important aspect of ABC design that is not well covered in the AASHTO LRFD Bridge Design Specifications, therefore significant specification language was justified. The provisions included in the section are based on past research that was gathered during project 12-102.

The connections included were grouped as follows:

- Reinforced cast in place concrete closure joints are very common. These joints are typically reinforced using different reinforcement details. These consists of straight lapped bars, headed lapped bars, or hooked lapped bars. These joints are filled with a variety of different cementitious materials such as grout, normal concrete, high early strength concrete, and ultra high performance concrete.
- The second type of joint covered includes mechanical reinforcing bar connectors. These are allowed in the AASHTO LRFD Bridge Design Specifications. There a few proposed subtle changes in the Guide Specifications for ABC.
- The third type of joint covered includes reinforcing bars grouted into post tensioning ducts embedded in adjacent elements. The ducts can be filled with grout or ultra high performance concrete.
- The last two joint types are referred to as pocket and socket connections. A pocket connection involves a void in one element where projecting reinforcement from an adjacent element is inserted and filled with grout or concrete. A socket connection is defined as a connection where a portion of one element is inserted into a socket in grouted or concreted in place.

Section 3: 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.

Article 3.6.2 of the Guide Specification for ABC covers cast-in-place concrete closure joints using lapped bar reinforcement. This covers a number of the joints described in the previous slide.

By using the emulative design approach, the connections need to be designed for the forces acting on them, and satisfy the strength and serviceability requirements specified in the AASHTO LRFD Bridge Design Specifications.

The term "concrete" in this title in intended to cover all cementitious materials such as grout, concrete, and ultra-high performance concrete.

Section 3: CIP Closures with Reinforcing

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_{hb}, 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* are 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 maximum 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

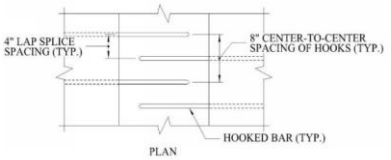
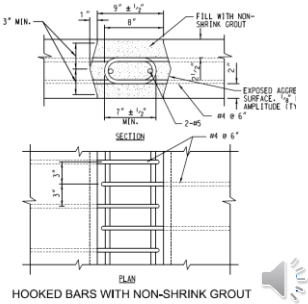


Figure 3.6.2.2-1—Hooked Lap Splice



Article 3.6.2.2 covers the design of closure joints reinforced with lapped hooked bars. This is a common and effective detail used by many states. The hooked bars can be lapped over a short distance resulting in a narrow closure joint.

The current AASHTO LRFD Bridge Design Specifications do not include provisions for lapped hooked bars. The research noted in the commentary has shown that it is reasonable to use a standard hook development length as the lap length provided that the non-contact distance is less than 4 inches.

The research also included at least one bar within the hook, therefore this requirement was added to the provision. The bar within the hook should be of equal size to the lapped bar.

Section 3: CIP Closures with Reinforcing

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_{dth}, measured from the underside of the head to the point of development shall satisfy:

$$l_{dth} \geq \frac{0.51 f_y d_b}{\sqrt{f'c}} \qquad (3.6.2.3.1\text{-}1)$$

but not less than 6 in., where all of the following criteria are met:

- 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 $2d_b$;
- 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 $4A_b$;

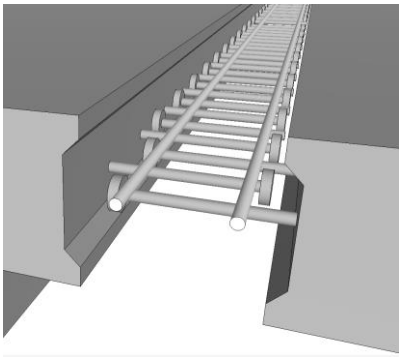
C3.6.2.3

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 Specification* 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.*



Article 3.6.2.3 covers the use of headed and mechanically anchored deformed reinforcing bars in closure joints. Sub article 3.6.2.3.1 covers the development length of headed bars in concrete.

Headed reinforcing bars are used in the vertical construction industry , however they are not currently included in the AASHTO LRFD Bridge Design Specifications. The ACI 318 building code does have provisions for development length of headed bars. Those provisions were added to the Guide Specification for ABC, with minor modifications to account for different units used in AASHTO.

The limitations noted in this provision are taken from the ACI code.

The use of headed bars in compression is prohibited, based on the availability of limited testing data, which is noted in the ACI code.

There are two important provisions regarding the development of headed bars in concrete.

The first is that the bar size is limited to #11 and smaller. This is based on the fact that past research has not investigated bars larger than #11.

The second is that the area of the head in bearing must be greater than or equal to four times the area of the bar. This is an important specification, since all headed bars do not meet this specification. There are multiple manufacturers of headed bars that can meet the specification, therefore this specification would not result in a proprietary supply problem.

Section 3: CIP Closures with Reinforcing

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_{sth}, measured between the underside of the opposing heads of the spliced bars shall satisfy:

$$l_{sth} \geq l_{dth} + 0.70S_b \qquad (3.6.2.3.3\text{-}1)$$

where the following criterion is met:

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

where:

l_{dth} = 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.

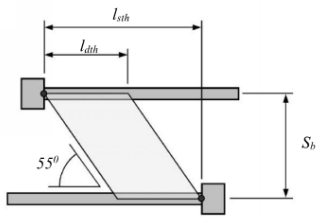


Figure C3.6.2.3.3-1—Strut and Tie Model for Non-Contact Lapped Headed Reinforcing Bars

The splicing of headed bars is currently not covered in either AASHTO or ACI. The failure mechanism of a headed bar is primarily a cone shaped failure plane emanating from the head of the bar. In order to transfer force from one bar to the next, the force need to transfer through the concrete from one head to the next via a compression strut.

The research noted in the commentary studied this force transfer. The logical result is that as the spacing increases the length of the lap splice must increase as shown in the diagram in the commentary. A strut slope angle of 55 degrees was recommended in the research. This is similar to approaches that are used in Europe for this situation.

Precast Full Depth Deck Panel (FDDP): Design

Closure joint with Ultra-High Performance Concrete (UHPC): Lap length

3.6.2.4—Reinforced UHPC Connections in Closure Joints

Ultra-High Performance Concrete (UHPC) may be used to reduce the development length and splice length of deformed reinforcing bars in closure joint connections.

C3.6.2.4

UHPC is a very high strength cementitious concrete mix that is reinforced with steel fibers to enhance the tensile and bond properties of the material.

The provisions in this section are based on work by Graybeal et al. (2014).

Refer to Section 10.4.4 of this document for information on the material of UHPC. In general, the specified concrete compressive strength of concrete at 28 days is equal to or greater than 21 ksi.

Ultra-High Performance Concrete is an important tool in the tool chest for ABC. UHPC is a highly specialized cementitious material that gains tremendous strength in compression. It also includes steel fibers that greatly improve the tensile resistance of the material. This can result in very short development and lap splice lengths, which allows designers to detail narrow closure joints between elements without sacrificing strength and ductility.

There has been significant research and development of specifications by the Federal Highway Administration in several states. FHWA has issued several Tech Briefs on the subject. These documents form the basis of the provisions in the Guide Specifications for ABC.

In general, in order for a concrete to be classified as UHPC, it needs to have a 28 day compressive strength of 21ksi or greater and 2% steel fibers by volume.

Designers have asked if nonmetallic fibers can be used. At this time, only steel fibers can be used. Other types of fibers are in the market, however it is believed at the performance will not be equal to UHPC with steel fibers.

UHPC Joint

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_{dtu}, shall satisfy:

For reinforcing bars with:

$f_y \leq 75$ *ksi*

$$l_{dtu} \geq 8d_b \qquad (3.6.2.4.1\text{-}1)$$

where:

d_b = nominal diameter of bar (in.)
f_y = specified yield stress of reinforcing bars (ksi)

For reinforcing bars with:

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

$$l_{dtu} \geq 10d_b \qquad (3.6.2.4.1\text{-}2)$$

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_{dtu}, shall be increased by $2d_b$ if:

$2d_b \leq$ *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 thus an increase in the required development length.

Article 3.6.2.4.1 covers the development of deformed reinforcing bars in tension in UHPC. The minimum development length is specified has eight bar diameters. The basic development length shall be increased by two bar diameters If the cover is less than three bar diameters.

For use of this provision, the yield strength of the bar must be less than 75ksi. The minimum compressive strength of the UHPC must exceed 14ksi in order to open the bridge to traffic. The specified 28 day compressive strength still needs to be 21ksi in order to meet this specification.

Bar sizes are limited to #8 and smaller. This is based on limited testing at the time of publication. Designers should keep abreast of the latest FHWA publications on UHPC, as testing continues to this day. FHWA is also developing a UHPC design specification. Once published, that specification will most likely supersede the Guide Specification for ABC.

UHPC Joint

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_{su} in tension that are cast into UHPC shall satisfy:

$$l_{su} \geq 0.75 l_{dtu} \qquad (3.6.2.4.2\text{-}1)$$

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_{su}.
- 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 $0.75 l_{dtu}$ 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.

Article 3.6.2.4.2 Covers lapped reinforcing bars in tension in UHPC. The minimum lap length is equal to 75% of the tension development length. The maximum non-contact spacing between bars shall not exceed the lap length. These provisions can also be used for hooked bars in UHPC .

This provision also requires that the interface with the connected element include a shear key and have an exposed aggregate or roughened finish. We will go into more detail on this subject later today.

Mechanical Connectors

3.6.4—MECHANICAL REINFORCING BAR CONNECTORS FOR ED AND CP APPLICATIONS

Mechanical reinforcing bar connectors are mechanical connections that splice reinforcing bars. Two types of mechanical splices can be used for bridge structures. The Designer shall specify which type of connector is to be used in each connection.

3.6.4.1—General

Mechanical reinforcing bar connectors are devices that allow direct axial load transfer between reinforcing bars.

Connections of columns to pier caps or footings in moderate and high seismic design require special detailing in design and construction to ensure emulative behavior of CIP reinforced concrete. Such connections are generally in or near ED regions of the substructure. The Designer shall identify the load path through the element to provide proper detailing for force transfer to adjacent elements, and shall identify the connection as ED or CP.

Two types of mechanical connectors, Types 1 and 2, may be used for bridge structures according to the limitations listed in Articles 3.6.4.2 and 3.6.4.3. The Designer shall specify which type of connector will be used in each connection. Type 1 may only be used for CP connections.

C3.6.4

The basis of this article is the *ACI Building Code Requirements for Structural Concrete* (ACI-318-14). Chapter 18–Earthquake Resistant Structures, of this code includes provisions for "Special Moment Frames and Special Structural Walls". These structures are essentially the same a bridge substructure.

C3.6.4.1

Mechanical connectors in current use are mostly proprietary and include threaded sleeve, grouted sleeve, and sleeve-swaged. The most common connector in use is the grouted sleeve, since it is relatively easy to insert a projecting bar from one element into a sleeve cast into the connected element. Contract documents should include testing and approval procedures whenever a proprietary connector is used.

Although research has indicated that emulative design procedures are suitable for elastic analysis (Haber et al. 2013), these may not be appropriate for all splice types in the inelastic range of response. Placement of mechanical splices within the plastic hinge zone can alter the strain and plastic curvature distribution thus leading to reduced displacement ductility capacity. See Article 3.4.3.2

Mechanical connectors for reinforcing bars are an important aspect of many ABC connections. The current AASHTO LRFD Bridge Design Specifications have provisions for mechanical bar connectors. In general, these connectors must be capable of developing 125% of the specified yield strength of the connected bar.

The ACI 318 Building Code has similar provisions, but also has provisions for higher strength mechanical bar connections. Higher strength bar connections specified in the ACI code are required to develop 100% of the specified tensile strength of the connected bar.

These higher strength connectors have proven to be useful in seismic applications. Significant research has been completed on this subject. For this reason, both types of mechanical connectors are included in the Guide Specifications for ABC.

Type 1 Connectors

3.6.4.2—Type 1 Mechanical Connectors

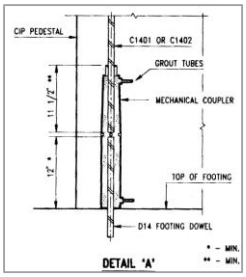
Type 1 Mechanical Connectors are used to connect two in-line reinforcing bars. They shall be capable of developing 125 percent of the specified yield strength of the spliced bars.

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.

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.

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.



- Example of a mechanical connector (Grouted Bar Coupler)
- There are other similar devices in the market

The first type of connector is referred to as a Type 1 Mechanical Connector. This connector is essentially what is specified in the current AASHTO LRFD Bridge Design Specifications. It can develop 125% of the specified yield strength of the spliced bar.

There are limitations on use specified in the AASHTO LRFD Bridge Design Specifications. They should not be used in columns at points of potential yielding or hinge zones, such as above the footing and below the pier cap. There are also limitations on placement within columns, such as not allowing all bars to be located at one cross section in column. This is all applicable to elements that make use of Type 1 Connectors. Reference is made to the AASHTO LRFD Bridge Design Specifications for guidance on the use of Type 1 Connectors.

Type 2 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 percent of the specified minimum tensile strength of the spliced bars. Type 2 Mechanical Connectors shall be limited to bars with a specified yield strength of 60 ksi or less.

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.

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

C3.6.4.3

These provisions are applicable to Grade 60 reinforcing bars (both ASTM A615 and ASTM A709) based on testing completed to date and availability of connectors that can achieve the required specified tensile strengths. There is not sufficient data to justify the use of connectors for higher grades of steel.

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.

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

The second and higher strength mechanical connector for reinforcing bars is referred to as a Type 2 Mechanical Connector. As previously stated, these connectors can develop 100% of the specified minimum tensile strength of these splice bars.

Let's compare the design strength both types of connectors using ASTM A615 Grade 60 bars. A Type 1 connector can develop 125% Fy, which equates to 75ksi. A Type 2 connector can develop 100% Fu, which equates to 90ksi.

The ACI Code has allowed the use of these connectors in plastic hinge zones for seismic applications in vertical construction for many years. Recent research on bridges have shown that they are acceptable for use in bridges with certain limitations and requirements.

The details for Type 2 connectors are similar to Type 1 connectors, except that the thickness and dimensions of the connector may vary slightly in order to achieve the required strength.

Grouted Duct Connections

3.6.5—GROUTED DUCT CONNECTIONS

Grouted duct connections are used to connect individual reinforcing bars in tension by grouting the reinforcing bar that projects from one member into a corrugated steel duct embedded in a receiving member. Tensile force transfer occurs from the bar into the grout, from the grout into the duct, and then from the duct into the surrounding concrete and finally into adjacent reinforcement..

The connection between a precast column segment and the pier cap or footing is typically considered an ED connection. Connections using grouted ducts to join adjacent segments of a column are considered CP connections if the connection is outside the detailing limits of the plastic hinge zone.

C3.6.5

Grouted ducts can be used for the connection of column-to-footing, pipe-to-pile cap, precast pile-to-cap, column-to-column, or column-to-pier cap as shown in Figure C3.6.5-1. While column and piles can be precast or CIP, footings are usually CIP because their weight is too large to permit transportation. Grouted ducts differ from sleeve-type bar connectors in that the latter are in-line tension splices, whereas force transfer in the former occurs by lap-splicing. In addition, sleeve-type bars connectors are significantly stiffer than grouted ducts. While column and piles can either be precast or CIP, footings are usually CIP because their weight is too large and concentrated to permit transportation.

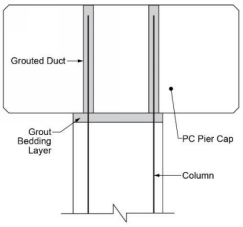


Figure C3.6.5-1—Column-Pier Cap Grouted Duct Connection

Article 3.6.5 of the Guide Specifications for ABC covers Grouted Duct connections. This innovative connection involves inserting a reinforcing bar into a corrugated steel post tensioning duct cast into the connected element. The duct is then filled with grout to complete the connection. By using these provisions, the bar can be fully developed.

A word of warning with the use of these connections. Designers should carefully check the layout of reinforcing in the element that has the ducts. In most cases these connections are used in pier caps, which normally have congested reinforcing. The designer should layout the pattern of ducts and reinforcing bars to ensure that the geometry is satisfactory. Many designers choose to use fewer ducts with larger bars for the connection for this reason.

Grouted Duct Connections

3.6.5.1—Minimum Development Length of Reinforcing Steel in Grouted-Duct Connections

Column longitudinal reinforcement shall be extended into footings and pier caps as close as possible to the opposite face of the footing or pier cap to produce effective force transfer in the joint region. When grouted ducts are used force transfer through the entire joint region must be accommodated. Accordingly, a combination of force transfer between conventional reinforcing bars and the duct may be necessary.

C3.6.5.1

Extending the column longitudinal bars will help provide an adequate load path through the joint (joint shear design). Results from monotonic pullout tests (Eberhard et al. 2009) showed that large diameter reinforcing bars can develop their tensile capacity in shorter lengths when these are embedded in corrugated steel ducts filled with grout as compared to bars in direct contact with concrete. Where concrete splitting cracks are prevented, the bars can develop their ultimate strength if grouted at least 16 bar diameters into the ducts. This length

42

Article 3.6.5.1 of the Guide Specification includes provisions for the development length of reinforcing steel within a grounded duct connection. These provisions are based on the research noted in the commentary.

The development length for a bar in a grouted ducts is more than for a typical mechanical connector, but it is less than a typical bar development length. This makes this connection a less expensive effective connection.

Grouted Duct Connections

The minimum 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)

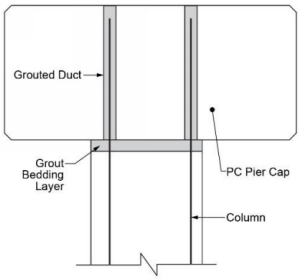


Figure C3.6.5-1—Column-Pier Cap Grouted Duct Connection

Note: There are additional requirements for seismic applications

This is the equation for the development length of the bar within the grouted duct. It should be noted that the design should be based on the expected yield stress of the longitudinal bar as opposed to the specified yield stress. This provision this primarily for seismic applications where it is required that the plastic hinge forms in the column and not the cap.

There are additional requirements in the Guide Specification for this connection when used for seismic applications. For this presentation, we will not go into specifics for this connection in seismic applications, as it is covered in a separate seismic workshop.

Pockets and Sockets

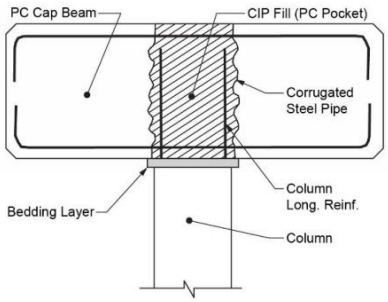


Figure C3.6.6.1-1—Pocket Connection

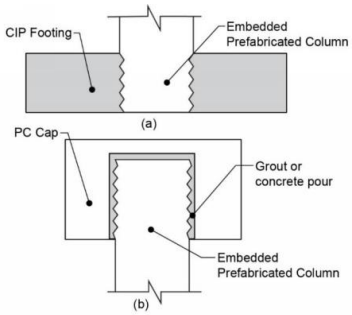


Figure C3.6.7-1—Examples of Socket Connections

Two terms are used within the Guide Specification for two similar connection categories. They are "Pocket Connections" and "Socket Connections". There has been some confusion in the industry regarding the definition of these two terms. It is important to get these two terms clarified for use in the Guide Specifications for ABC.

Pocket connections are made by inserting reinforcing bars from one element into a pocket in the connected element.

Socket connections are made by inserting a portion of one element into a socket in the connected element. It also covers inserting a portion of one element within a concrete pour in the connected element. This can best be analogous to screwing a lightbulb into a light "Socket".

Pocket Connections

3.6.6—POCKET CONNECTIONS

Pocket connections may be used to connect elements with a formed recess in a precast concrete element that is used to receive reinforcement or other mechanical devices that are projecting from another element or a previously cast element.

The Designer shall identify the load path through the element to provide proper detailing for force transfer to adjacent elements. Typically, such connections are adjacent to plastic hinging locations and the anchorage of reinforcement from the column into the pocket is considered part of the ED element.

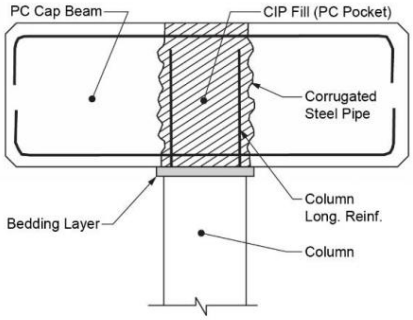


Figure C3.6.6.1-1—Pocket Connection

Note: The AASHTO LRFD Guide Specs for ABC have extensive design requirements for these connections

Article 3.6.6 of the Guide Specification covers pocket connections. There are extensive provisions for this connection type.

The research that led to these provisions was based on seismic performance, however they can also be used for non-seismic applications. The pockets are typically formed with corrugated metal pipe, which serves several functions.

- The corrugations transfer vertical shear from the column to the cap.
- The Research showed that the pipe also provides some confinement for the projecting column reinforcing. Additional confinement steel is also required around the pipe.

Pile Cap Pocket Connection

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.

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:

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

where:

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

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

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.

Figure C3.6.6.6-1—Corrugated Steel Pipe Pocket Void used in Integral Abutment Stem

One of my favorite and most versatile connections is the use of corrugated metal pipe void for the connection of piles to a precast integral abutment.

The photo on the right shows an abutment being constructed in Massachusetts. The construction is very simple. The piles are driven, the precast cap is set and leveled, and the void is filled with normal concrete. Steel H piles are shown, however, this connection can be made with concrete piles or drilled shafts (by projecting the shaft reinforcing cage into the pocket).

It is important to note that piledriving tolerances need to be carefully maintained for this connection. More on that later in the workshop.

The vertical shear resistance of this connection is significant. The equation shown is based on conventional interface shear provisions in the AASHTO LRFD Bridge Design Specifications. We will explore a design example later today.

Pile Cap Socket Connection

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

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

where:

d_v = inside diameter of the CMP pocket (in.)
h_v = 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.

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 seismic system, the use of welded studs is typically not used in integral abutment detailing, therefore it is not necessary with CMP pocket connections.

It should be noted that the effective height is contingent on the detailing of the pile top. For steel piles, it is most likely measured from the top of the pile cut off. Drilled shafts would be measured from the top of the concrete of the completed shaft.

Pile Cap Socket Connections

The research conducted on integral abutment pile connections (Wipf, T.J., et al., 2009) did not include an ultimate load test. The recommendation included in this article is based on engineering judgement. The assumption is that the concrete within the pocket will shear in a plane that is defined by the concrete between the corrugation crests. Designers may wish to add a strength reduction factor to this calculation to account for the fact that ultimate load testing has not been completed for this connection. This should not be problematic in most cases since this resistance would be significant. For example, the shaft resistance of a 24-inch diameter CMP pocket filled with 4ksi concrete would be 226 kips per foot of corrugated pipe. In most cases, the length required to resist typical pile loads would be reasonable. A strength reduction factor of 0.75 should be considered.

In the commentary for this provision, research at Iowa State University is cited. In this research, the ultimate strength of this connection could not be determined due to the limitations of the testing equipment.

A strength reduction factor of 0.75 can be considered based on the fact that this provision is based on a certain amount of engineering judgment, which includes the use of interface shear provisions. This factor should not affect the detailing of connections as the capacity is typically significantly higher than the capacity of the pile. For example, a 24 inch diameter corrugated metal pipe can develop over 226 kips per linear foot of pipe using these equations.

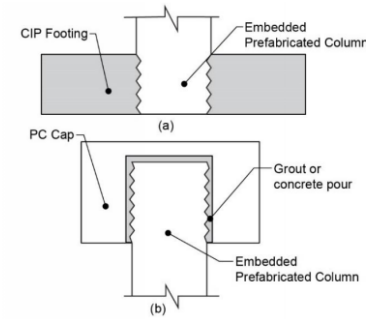
More Socket Connections

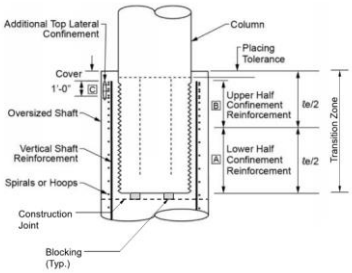
3.6.7—SOCKET CONNECTIONS

Socket connections can be used to connect elements by inserting one element into an opening in another element or into a cast-in-place concrete pour. .

C3.6.7

If the receiving element is precast, the connection is completed by filling the interstitial space in the opening with grout or concrete. Figure C3.6.7-1 depicts two types of socket connections with a cast-in-place and a precast receiving element.





Note: The AASHTO LRFD Guide Specs for ABC have extensive design requirements for these connections

A variety of socket connections are covered in Article 3.6.7 of the Guide Specifications for ABC, which contains detailed requirements for these connections. Since these types of socket connections are often used for seismic applications, we will not cover them in detail today. They are covered in the separate seismic design module.

These connections can also be used in non-seismic applications. Later today when we go through a design example, we will demonstrate one type of socket connection.

End of Module 1

Available Modules

Non-seismic

1. General Provisions and Common Connections
2. Substructure Design
3. Superstructure Design
4. Bridge Systems
5. Durability of ABC Designs
6. ABC Construction Specifications

Seismic

7. Introduction to ABC Seismic
8. Seismic Connection Design and Detailing
9. Seismic Pocket and Socket Connections
10. Seismic Integral Connections