Implementation of the 2018 AASHTO LRFD ABC Guide Specifications

This Part Covers:
- Overview of NCHRP Project 12-102
- Reference Documents
- ABC Definitions

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

Technology Readiness

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

Specification Overview

- Guide Specification Contents
  1. Design Introduction
  2. General Design Provisions
  3. Design of Prefabricated Elements
  4. Detailing Requirements
  5. Durability of ABC Technologies
  6. Construction Introduction
  7. Temporary Works
  8. Fabrication and Assembly Planning
  9. Layout and Tolerances
  10. Concrete Structures
  11. Steel Structures
  12. Geosynthetic Reinforced Soil / Integrated Bridge System
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?

Current Status
• The guide specification was completed late 2016
• Balloted for adoption in June 2017
  • Passed
• Several Committees still had issues
  • T-4 Construction Committee worked with them to resolve some issues prior to publication
• Re-balloted in 2018
  • Adopted by AASHTO CBS in 2018
  • Published in the fall of 2018
  • Available at the AASHTO Bookstore

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

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

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
  • NCHRP Project 12-98 Website
  • They are free!

Design Specification Overview
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
Reference Documents for Today

- AASHTO LRFD Guide Specifications for ABC
- NCHRP Project 12-98 Guidelines for PBES Tolerance
- NCHRP Project 12-98 Guidelines for Dynamic Effects for Bridge Systems
- FHWA LRFD Bridge Design Examples

Blue Header Denotes AASHTO ABC Guide Specifications

Green Header Denotes PBES Tolerance Guide (NCHRP 12-98)

Purple Header Denotes System Dynamics Guide (NCHRP 12-98)

Red Header Denotes FHWA LRFD Bridge Design Examples

Section 1: Definitions

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

Prefabricated Bridge Elements (PBE)
A single prefabricated structural component of a bridge. Prefabricated Bridge Elements can be made of any approved structural material.

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.

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.
**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.

**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.

**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.

**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.

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**Part 2:**
This Part Covers:
- Designer & Contractor Responsibilities
- Tolerances (NCHRP Project 12-98)
- Design of Bridge Systems

**Section 1: Designer & Contractor Responsibilities**
- Recommended responsibilities are defined for both the designer and the contractor
- Prefabricated elements
- Systems
  - SPMTs
  - Lateral Slide
PBE: Designer Responsibilities

1.4.3—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.

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

Examples of this approach include but are not limited to cast-in-place structures and proprietary connecting tools.

12-98: Erection Layout Drawings

12-98 Element Tolerances

Pier Cap Example

Handling: Designer Responsibilities

1.4.3.1—Designer’s Role in Skidding 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 skidding and handling for general conformance with the contract documents.

The Designer should investigate the skidding and handling requirements for very large or unusually shaped structures. These procedures shall be included in the contract plans in these structures.

The investigation shall be limited to general equipment needs and suitability. In some cases, the Contractor shall contact members of the Contractor to verify that the design can be assisted. The Designer shall also investigate the Contractor’s capabilities to determine if the Contractor can accommodate the required capacity and the delivery dates.

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 resources and methods.”

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 in the contract plans.

1.4.3.2—Temporary Works

All temporary works shall be designed by the Contractor. The AASHTO LRFD-Bridge Design Specifications for Bridge Engineering Works shall be followed unless otherwise specified herein or by the Owner.

1.4.3.3—Compliance and Handling Calculations including Lifting Hardware

The Contractor shall be responsible for the lifting and handling of precast and precast elements. This includes but is not limited to lifting equipment and load-bearing Sexe cases, and delivery vehicles. The Contractor shall also be required to verify that the lifting methods will not exceed the loads.

See Article 3.1.2 for examples of this approach.

1.4.3.4—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 tolerance details shall be shown on the erection plans, or included in the project specifications. Joint width tolerances shall be shown on the plans or included in the project details.

Recommended tolerances and drawing of tolerances is included in the document Guidelines for Prefabricated Bridge Elements and Segmental Concrete Bridge Infrastructures (AASHTO, 2011). This guideline is to be developed by the Owner to cover different conditions.
12-98: Erection Tolerances

Wall Panel Example

12-98: Joint Tolerances

Element fabrication dimensions should be based on the specified joint thicknesses or welds. The detailed cut outs for a joint or an edge should be based on the joint thickness or weld dimension, not the tolerances shown. The tolerances are dimensions that should be added to the joint thickness or weld dimension for the final tolerance. For example:

- Vertical joint detail cut out: \( d_L = \frac{1}{2} L + T_e \)
- Horizontal joint detail cut out: \( d_L = \frac{1}{2} L, \ T_e \)
- Vertical joint detail cut out: \( d_L = \frac{1}{2} L, \ T_e \)
- Horizontal joint detail cut out: \( d_L = \frac{1}{2} L + T_e \)

See Article 6.5.2 to obtain specified joint thicknesses and joint tolerance columns, \( T_e \) and \( L \) respectively.

12-98: Joint Tolerances

**Table 3.3.2.3: Vertical Joint Tolerances for Weld or Tolerance, \( T_e \)**

<table>
<thead>
<tr>
<th>Component</th>
<th>( L )</th>
<th>( T_e )</th>
<th>( d_L )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Cap</td>
<td>( \frac{1}{2} L )</td>
<td>( T_e )</td>
<td>( \frac{1}{2} L + T_e )</td>
</tr>
<tr>
<td>Wall Cap</td>
<td>( \frac{1}{2} L )</td>
<td>No Tolerance</td>
<td>( \frac{1}{2} L )</td>
</tr>
</tbody>
</table>

**Example for joint width detail cut out:**

- Minimum tolerable width: \( \frac{1}{2} L \)
- Joint with tolerance: \( \frac{1}{2} L + T_e \)

Therefore:

Detail cut out: \( \frac{1}{2} L + T_e \)

12-98: Joint Tolerances

**Table 3.3.2.3: Horizontal Joint Tolerances**

<table>
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**Example for joint width detail cut out:**

- Minimum tolerable width: \( \frac{1}{2} L \)
- Joint with tolerance: \( \frac{1}{2} L + T_e \)

Therefore:

Detail cut out: \( \frac{1}{2} L + T_e \)
Section 1: SPMT Systems

- Designer responsibilities
  - Design the bridge
  - Determine preliminary SPMT layout
  - Specify pick points – Analyze structure for move
  - Verify that travel path is reasonable
  - Determine and specify allowable deformations
  - Check the bridge for the lift and move
    - Vertical Dynamic effects should be included
    - More to come on this subject....

Section 1: Preliminary SPMT Layout

- Recommended Assumptions
  - Width of transporter: 8 ft
  - Length of transporter: Sum of axles + axle spacing
  - Length of power pack: 12 ft
  - Number of axles per unit: 4 and 6
  - Number of lines per unit: 2
  - Axle line spacing within unit: 4.625 ft
  - Axle line spacing between adjacent units: 4.625 ft
  - Axle line load capacity: 30 kips
  - Unit height: 49 in, 72 in, max.

Section 1: Preliminary SPMT Layout

- Example
  - 650 Ton Bridge Weight
  - Number of axles required = 650/30 = 21.7
  - Use 24 axle lines (rounded up)
  - Use two 12 line SPMTs
    - Width = 8 feet
    - Length = 4.625*12 = 55.5 feet
  - Locate the SPMTs in a desired location under the bridge
    - Verify that the bridge can be picked at that location
    - Check travel path to make sure the layout works along the path

Section 1: Bridge Analysis

- Two methods for support in staging area:
  - Method A: End supported during deck casting
  - Method B: Supported at pick points during deck casting
  - Designer needs to specify this

Method A: End supported during deck casting

Include vertical dynamic effects

Method B: Supported at pick points during deck casting

Include vertical dynamic effects
Section 1: SPMT Systems

- Contractor responsibilities
  - Design of all falsework
  - Staging area
  - Above the SPMTs
  - Include horizontal dynamic effects
  - Final SPMT Layout and Travel Path
  - Determine preliminary SPMT layout
  - Develop monitoring system (twist, deformations)
  - Bridge Analysis if different pick points are used

Section 1: Lateral Slide Systems

- Designer Responsibilities
  - Design the bridge
  - Identify Preliminary jacking locations
  - Analyze structure for these jacking locations
  - Determine Preliminary friction forces
  - Verify that the bridge can resist these forces
  - Develop schematic falsework layout
  - Specify method to resolve sliding forces

Section 1: Lateral Slide Systems

- Resolution of Sliding Forces – With connection to substructures

![Figure 1.1.4-1 - Falsework Connected to Substructure (Resolution of Friction Forces)](image)

Section 1: Lateral Slide Systems

- Resolution of Sliding Forces – Without connection to substructures

![Figure 1.1.4-2 - Falsework not Connected to Substructure (Resulting in Additional Foundation Forces)](image)

Section 1: Lateral Slide Systems

- Contractor Responsibilities
  - Design of Falsework and sliding system
  - Final design of jacking forces and jacking locations
  - Detail the sliding system
  - Develop methods to monitoring movement during the slide

Section 2: Loads and Load Combinations

- Loads and Load Combinations
  - The requirements of the loads are based on the
  - Foundation Systems
  - Structures for the loads are designed to resist these loads based on the
  - Compliant Structures

![Figure 2.4](image)
Section 2: Loads and Load Combinations

### 2.4.3 Shipping and Handling:

All prefabricated elements shall be marked for shipping and handling issues during fabrication, shipping, and storage.

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

- Live Load: 1.5
- Dead Load: 1.0

**Example:**

- Live Load: 1.5
- Dead Load: 1.0

**Equation:**

\[
F_{ci} = \frac{F_{ck}}{K_{ci}}
\]

Where:

- \(F_{ci}\): Compressive strength of concrete
- \(F_{ck}\): Compressive strength of concrete
- \(K_{ci}\): Coefficient for compressive strength

**Notes:**

- Dimensional tolerances are not considered in this calculation.
- All calculations shall be performed in accordance with the American Concrete Institute (ACI) Building Code.

### 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
- After stripping: use \(f'_{ci}\)
- Shipping: use \(f'_{c}\)

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Section 2: Loads and Load Combinations

### 2.4.2 Section Load Combinations for the Bridge

The section load combinations are specified in the ASCE/SEI 7-16 Bridge Design Specifications for Steel Bridges. The following combinations shall be used:

- **Dead Load:** 2.0
- **Live Load:** 1.5

**Notes:**

- The values in parentheses are based on the American Society of Civil Engineering (ASCE) Bridge Design Handbook (2016).
- These values for handling, erection, and shipping combine the dead load with the live load.

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Section 2: Loads and Load Combinations

### 2.4.2 Section Load Combinations for the Bridge

The section load combinations are specified in the AASHTO Bridge Design Specifications. The following combinations shall be used:

- **Dead Load:** 2.0
- **Live Load:** 1.5

**Notes:**

- The values in parentheses are based on the American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications (2016).
- These values for handling, erection, and shipping combine the dead load with the live load.
Part 3:
This Part Covers:
• Design Example
• SPMT System Dynamics

Horizontal SPMT Dynamics: Example
• NCHRP Project 12-98
• Completed physical tests of SPMTs
• Measured accelerations under various maneuvers
• Varied the load to measure effect

SPMT Testing
Sensor Placement
• Measured acceleration along all three axes
  • Longitudinal
  • Transverse
  • Vertical

Loads
• Three Cases (50T, 25T, 15T)
• Investigate this if this has an effect on accelerations
• Three data points define a curve
• Extrapolate out to full load (2017)

Travel Path
• Goal is to simulate all potential SPMT motions
  • Starting and Stopping
  • Raising and Lowering
  • Traversing rough terrain
  • Turning
  • Spraying
• All at highest speeds
• Upper bound
• Rough terrain chosen
• Upper bound
Figure 3.5.1: Response Spectra Mean and Mean Plus One Standard Deviation for Vertical Accelerations, 15T

\[ S_{\text{am}} = 3 \times PPA \]

\[ T_n \leq T_1 \]

\[ T_n > T_1 \]

Figure 3.5.2: Response Spectra Mean and Mean plus One Standard Deviation for Horizontal Accelerations, 50T
Design Spectra - Horizontal

Idealized Spectra Curve

\[ \text{SAmax} = \begin{cases} 0 & T_0 \leq T_n \leq \frac{T_1}{2} \\ \frac{2}{4} \times \text{PPA} & T_n > \frac{T_1}{2} \end{cases} \]

Figure 3.6.2-3: Proposed Design Spectrum for Horizontal Acceleration

Design for Horizontal Dynamics

- Design of falsework would be akin to design of bridge piers for seismic
- Use ultimate strength design method
- The design horizontal accelerations would represent an "emergency stop" situation
- Design for acceptable damage, no collapse
- Use R Factors to account for different falsework systems
  - R = 2.0 for falsework with limited ductility
  - R = 2.5 for falsework with medium ductility
- Similar to AASHTO Guide Specifications for Bridge Temporary Works

Design Method – SPMT Dynamics

We need to account for stiffness of falsework and the bridge
- Use method based on the Uniform Load method in AASHTO LRFD
- Single Mass-Spring Oscillator
- Apply a uniform load to superstructure at C.G. (vertical and horizontal)
- Calculate maximum deflection
- Calculate period
- Obtain Dynamic Response Coefficient
- Modify previous load by multiplying it by the Dynamic Response Coefficient
- Analyze the bridge and falsework for this new load

Simplified Method

- Assume the period is short
- Design for plateau of response spectrum curve

\[ p_{pk} = \left( 9.72 \times 10^{-4} \right) \cdot W \cdot (1/E) \]

where:
- \( p_{pk} \): peak value of the SPMT load
- \( W \): total weight of the bridge and falsework
- \( E \): length of the bridge load
- \( K \): falsework response modification factor
  - 2 for falsework with limited ductility
  - 2.5 for falsework with medium ductility

Calculation of Vertical Dynamic Forces

- Use a method similar to the uniform load method for seismic

1. Calculate the static vertical displacement \( w_t \) due to an assumed uniform load \( p_{vk} \), as shown in figure C7.4.2-4.
2. Calculate the vertical stiffness \( K_v \) of the bridge and falsework from the following expression:

\[ K_v = p_{vk} \cdot A_{vk} \]

where:
- \( A_{vk} \): area of the bridge deck (ft²)
- \( p_{vk} \): maximum value of \( w_t \) (kip/ft)
- \( A_{vk} \): uniform vertical load applied in the area of the bridge deck (kip per square foot)

Calculation of Vertical Dynamic Forces

1. Calculate the total weight, \( W \), of the bridge and falsework.
2. The weight should take into account the dead weight of the bridge, falsework, and all other loads that might be on the SPMT during the bridge move. The weight of the SPMT should not be included in this weight.

3. Calculate the vertical period of the bridge and falsework, \( T_v \), using the expressions:

\[ T_v = 2\pi \sqrt{\frac{K_v}{W}} \]

where:
- \( g \): acceleration of gravity (ft/sec²)

4. Calculate the vertical period of the bridge and falsework, \( T_v \), using the expressions:

\[ T_v = 2\pi \sqrt{\frac{K_v}{W}} \]

where:
- \( g \): acceleration of gravity (ft/sec²)
Calculation of Vertical Dynamic Forces

1. Calculate the static moment of vertical dynamic load, \( p_s \), from the expression:

\[
p_s = C_m p_{ay} (2.42.5)
\]

where:
- \( C_m \) = the dynamic coefficients of vertical dynamic load
- \( p_{ay} \) = the maximum dynamic load

2. The PDA load represents the load factor at the SPMT. PDA should be used to determine the vertical dynamic force. The PDA is calculated as follows:

\[
PDA = \frac{2.42.5 \times \text{Speed in mph}^2}{C_m}
\]

3. The PDA load represents the load factor at the SPMT. PDA should be used to determine the vertical dynamic force. The PDA is calculated as follows:

\[
PDA = \frac{2.42.5 \times \text{Speed in mph}^2}{C_m}
\]

The PDA load represents the load factor at the SPMT. PDA should be used to determine the vertical dynamic force. The PDA is calculated as follows:

\[
PDA = \frac{2.42.5 \times \text{Speed in mph}^2}{C_m}
\]

Calculation of Horizontal Dynamic Forces

1. Calculate the static moment of horizontal dynamic load, \( p_s \), from the expression:

\[
\text{horizontal dynamic load} = \frac{2.42.3 \times \text{Speed in mph}^2}{C_m}
\]

where:
- \( C_m \) = the dynamic coefficients of horizontal dynamic load
- \( p_{ay} \) = the maximum dynamic load

2. The PDA load represents the load factor at the SPMT. PDA should be used to determine the horizontal dynamic force. The PDA is calculated as follows:

\[
PDA = \frac{2.42.5 \times \text{Speed in mph}^2}{C_m}
\]

3. The PDA load represents the load factor at the SPMT. PDA should be used to determine the horizontal dynamic force. The PDA is calculated as follows:

\[
PDA = \frac{2.42.5 \times \text{Speed in mph}^2}{C_m}
\]

Calculation of Horizontal Dynamic Forces

1. Calculate the horizontal period of the bridge and foundation, \( T_{h} \), using the expression:

\[
T_{h} = 2\pi \sqrt{\frac{m_y}{m_z}}
\]

where:
- \( m_y \) = the mass of the bridge
- \( m_z \) = the mass of the foundation

2. The PDA load represents the load factor at the SPMT. PDA should be used to determine the horizontal dynamic force. The PDA is calculated as follows:

\[
PDA = \frac{2.42.5 \times \text{Speed in mph}^2}{C_m}
\]

The PDA load represents the load factor at the SPMT. PDA should be used to determine the horizontal dynamic force. The PDA is calculated as follows:

\[
PDA = \frac{2.42.5 \times \text{Speed in mph}^2}{C_m}
\]
Calculation of Horizontal Dynamic Forces

6. Calculate the member forces for use in the design of the bridge and framework by applying pV to the unknown and comparing it with the maximum member forces obtained in the first step above. The value of pV is determined by the formula:

\[ pV = \left( \frac{a}{A} \right) \left( \frac{b}{B} \right) \]

where:
- \( a \) is the length of the bridge parallel to the direction of the dynamic load.
- \( b \) is the length of the framework parallel to the direction of the dynamic load.
- \( A \) is the area of the bridge deck.
- \( B \) is the area of the framework.

If the bridge is considered to be a rigid body, an equivalent linear dynamic load equals to \( pV \) can be used for the analysis of the structure. The dynamic load \( pV \) is not a linear proportion of the static load. The application of the dynamic load should be treated as a non-linear load condition. See Article 2.4.3 for discussions regarding the application of this load for the design of the bridge and framework.

Simplified method - Vertical

• Assume period of the structure and falsework is at the plateaud

2.4.3.1 Simplified Vertical Dynamic Analysis

In lieu of the methods described above, the equivalent static dynamic load \( pV \) may be taken as:

\[ pV = \left( \frac{a}{A} \right) \left( \frac{b}{B} \right) \]

where:
- \( a \) is the length of the bridge parallel to the direction of the dynamic load.
- \( b \) is the length of the framework parallel to the direction of the dynamic load.
- \( A \) is the area of the bridge deck.
- \( B \) is the area of the framework.

This simplified method is based on the assumption that the period of the structure and framework is less than 0.5 seconds, which is conservative. This method still accounts for the percentage of load on the SPMT.

Simplified method - Horizontal

• Assume period of the structure and falsework is at the plateau

2.4.2.1 Simplified Horizontal Dynamic Analysis

This simplified method is based on the assumption that the period of the framework and structure is less than 3 seconds, which is conservative. This method still accounts for the percentage of load on the SPMT.

Horizontal Dynamic Calculations

Example Calculations: Horizontal Dynamic Forces

Note: This example is for longitudinal horizontal dynamic forces. A similar calculation would be required for transverse horizontal dynamic forces.

Given:
1. Bridge Length: 60 ft
2. Bridge Width: 40 ft
3. Weight of Bridge and Falsework: 1400 kips
4. SPMT Load: Two – 8 ton units
5. Wheel load (rated capacity of SPMT): 40 kips

Horizontal Dynamic Calculations

Example Calculations: Horizontal Dynamic Forces

Note: This example is for longitudinal horizontal dynamic forces. A similar calculation would be required for transverse horizontal dynamic forces.

Given:
1. Bridge Length: 60 ft
2. Bridge Width: 40 ft
3. Weight of Bridge and Falsework: 1400 kips
4. SPMT Load: Two – 8 ton units
5. Wheel load (rated capacity of SPMT): 40 kips
Horizontal Dynamic Calculations

Calculate the peak platform horizontal acceleration:

\[ \text{PGA}_{\text{peak}} = 0.3 \frac{1}{T^2} \times 10^{-0.16} = 0.3 \left( \frac{1}{T^2} \right) \times 10^{-0.16} = 0.159 \]

This equation was derived from the empirical curve shown in Figure 3-3.

Figure 3-7: Peak platform acceleration in horizontal direction

Check Bridge and Fairwork for Dynamic Loads:
The Fairwork and SMIT shall be designed using the load combinations specified in the ASHSTI Guide for Dynamic Specifications for Overhead Cranes (2013). Load combinations AD0, is recommended for use as the basis for the design of Fairwork and the SMIT. Based on this, the following is the recommended load combinations:

\[ \text{ADA} = 1.2 \left( \frac{1}{T^2} \right) \times 10^{-0.16} \times 1.5 \]

The horizontal load applied to the Fairwork shall be calculated for the present service load by using a load combination factor (1.1.2). A value of 1.5 is assumed here as the assumption that the Fairwork is based with flexible cabling.

**Horizontal load on Fairwork:**

- \[ \text{NHL} = 1.2 \left( \frac{1}{T^2} \right) \times 10^{-0.16} \times 1.5 \times 1.5 \]
- \[ \text{VOL} = 0.85 \times \text{NHL} \]

These horizontal loads would then be factored as noted in Eq. 2.4.3.1-1, and applied to the Fairwork and SMITs.

Discussion on Results:
The results for this example indicate a horizontal dynamic load factor of 7.8% (1.75% of the load). This is consistent with typical assumptions used in the heavy I.H. industry. Values between 5% and 15% have been used in calculations, however these values were based on engineering judgment. This new method is a systematic approach that accounts for the weight of the load on the SMIT and the flexibility of the Fairwork.
Load Factors for Systems

Design of the Bridge
- Follow AASHTO LRFD Bridge Design Specifications

Dynamics
- NCHRP Project 12-06 developed "upper bound" dynamic forces
- Considered to be "ultimate load" forces
- Load factors are based on this

Load Factors for SPMT Systems

2.4.2—Bridge Systems—SPMT Installation

Several load combinations shall be checked for the design of the bridge, crawler, and SPMTs to ensure horizontal and vertical dynamic loads during movement.

The designer is responsible for the selection of the beams and deck. The Contractor is responsible for the fabrication on the work and SPMTs.

Load Factors for SPMT Systems

2.4.3—Bridge Systems—Lateral Slide Installation

The maximum lateral forces shall be checked for the bridge, crawler, and SPMTs to ensure dynamic loads during movement.

The designer is responsible for the selection of the beams and deck. The Contractor is responsible for the fabrication on the work and SPMTs.

Load Factors for Lateral Slide Systems
2.4.3.1—Shipment 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:

\[
\begin{align*}
\text{Service Load (A)} & = \text{Load from AAR (2.4.3.1.1)} \\
\text{Erection Load (B)} & = \text{Load from AAR (2.4.3.1.2)} \\
\end{align*}
\]

where:

- \( S \) = dynamic load factor for handling of prefabricated elements
- \( p \) = dead load factor as specified in the AASHTO LRFD Bridge Design Specifications

The following values for \( p \) are recommended:

1. Dead load factors for removal of prefabricated elements from forms:
   - Form (\( S_0 \)) = 0.15
   - Ship, unload, storage, or mobile crane (\( S_{\text{mob}} \)) = 1.3
   - Ship, with hoist or crane (\( S_{\text{cr}} \)) = 1.4
   - Ship, with hoist or crane, and scaffolding (\( S_{\text{cr}} \)) = 1.8
   - Ship, with crane (\( S_{\text{c}} \)) = 1.7

2. Dead load factors for yard handling and erection:
   - \( S_{\text{eh}} = 1.0 \)

3. Dead load factor for shipping:
   - \( S_{\text{sh}} = 1.5 \)

**Part 4:**

This Part Covers:
- Design of Prefabricated Elements
- Design of Connections

**Section 3: Design of PBEs**

Largest Section in the Guide Specification
- 85 pages
Includes Seismic provisions
- Not covered in this workshop

This workshop:
- Precast deck panels with UHPC Joints
- Link Slab
- Precast Integral Abutment
- Precast Pier Bent

Most common design approach: Emulative 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

Connections required in this section are intended to achieve sufficient strength connections. Other non-structural connections are to be determined by the designer. Connections are to be designed in accordance with the requirements of the AASHTO LRFD Bridge Design Specifications, including strength and compatibility.

Connections included in this section are intended to achieve structural strength connections. Other non-structural connections are to be determined by the designer. Connections are to be designed in accordance with the requirements of the AASHTO LRFD Bridge Design Specifications, including strength and compatibility.
Section 3: CIP Closures with Reinforcing

3.2.2.3 - Bonded Reinforcing Bars in Closure Joints

Bonded reinforcement in a closure joint shall be designed to meet the force requirements of the joint. The provisions contained herein are based on the assumption that the concrete is in a state of normal stress and that the reinforcement is adequately anchored at the ends of the joint. The anchorages shall be designed to accommodate the forces resulting from the bending moment and shear forces developed in the closure joint.

Requirements for Reinforcing in Closure Joints

- The anchorages shall be designed to accommodate the forces resulting from the bending moment and shear forces developed in the closure joint.
- The anchorages shall be designed to accommodate the forces resulting from the bending moment and shear forces developed in the closure joint.
- The anchorages shall be designed to accommodate the forces resulting from the bending moment and shear forces developed in the closure joint.

Section 3: CIP Closures with Reinforcing

3.2.3.3 - System of Redundant Reinforcement

The system of redundant reinforcement shall be designed to meet the load requirements of the joint. The provisions contained herein are based on the assumption that the concrete is in a state of normal stress and that the reinforcement is adequately anchored at the ends of the joint. The anchorages shall be designed to accommodate the forces resulting from the bending moment and shear forces developed in the closure joint.

Requirements for Reinforcing in Closure Joints

- The anchorages shall be designed to accommodate the forces resulting from the bending moment and shear forces developed in the closure joint.
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- The anchorages shall be designed to accommodate the forces resulting from the bending moment and shear forces developed in the closure joint.

FDDP: Design

3.6.1.4 - Reinforced UHPC Connections in Closure Joints

Connections in reinforced UHPC connections in closure joint shall be designed to meet the force requirements of the joint. The provisions contained herein are based on the assumption that the concrete is in a state of normal stress and that the reinforcement is adequately anchored at the ends of the joint. The anchorages shall be designed to accommodate the forces resulting from the bending moment and shear forces developed in the closure joint.

Requirements for Reinforcing in Closure Joints

- The anchorages shall be designed to accommodate the forces resulting from the bending moment and shear forces developed in the closure joint.
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- The anchorages shall be designed to accommodate the forces resulting from the bending moment and shear forces developed in the closure joint.

Closure Joint with UHPC: Lap length

The length of reinforcement in a closure joint shall be determined by the following equations:

\[ L_{reinforcement} = \frac{P_{capacity}}{P_{resistance}} \]

where

- \( L_{reinforcement} \) = length of reinforcement in closure joint
- \( P_{capacity} \) = capacity of the joint
- \( P_{resistance} \) = resistance of the reinforcement

The length of reinforcement in a closure joint shall be determined by the following equations:

\[ L_{reinforcement} = \frac{P_{capacity}}{P_{resistance}} \]

where

- \( L_{reinforcement} \) = length of reinforcement in closure joint
- \( P_{capacity} \) = capacity of the joint
- \( P_{resistance} \) = resistance of the reinforcement

Figure 3.6.1.4 - Reinforced UHPC Connections in Closure Joints

The figure shows the location of the reinforcement in the closure joint. The reinforcement is shown as a continuous line, with arrows indicating the direction of the force.

 UDPC is a very high strength concrete with a compressive strength of 160 MPa. The concrete is cast in the form of precast panels and the panels are connected using a special connection system. The connection system is designed to transfer the forces from the panels to the structure and to provide a tight seal between the panels.
11/13/2019

**UHPC Joint**

**Mechanical Connectors**

Mechanical reinforcing bar connectors are mechanical connections that either thread in or thread out. Two types of mechanical options can be used for reinforcing bars. The designer shall specify which type of connector to be used in each connection.

1.6.3.3—General

Mechanical reinforcing bar connectors are devices that allow direct axial load transfer between reinforcing bars. Connections of columns to pile caps or footings in random and high seismic design require special detailing to design and construct to ensure elimination of all UHPC movement. Joints of mechanical reinforcing bar connectors shall be so detailed that the ultimate load capacity of the reinforcement is not less than the nominal load capacity of the reinforcement. The designer shall detail the base connections. Details of mechanical reinforcing bar connectors shall be selected based on the use conditions and the limits set for ultimate load capacity of the reinforcement. The designer shall detail the base connections.

1.6.3.4

The basis of this article in the ACI Building Code features a seminar (ACI 335-14, Chapter 1—Earthquake Resistant Structures, of this code includes procedures for "Special Moment Frames and Special Structural Walls." These structures are essentially a type of reinforced concrete structure. Mechanical connectors in special moment frames must be tested and approved for use in those structures. The designer shall detail the base connections. Details of mechanical reinforcing bar connectors shall be selected based on the use conditions and the limits set for ultimate load capacity of the reinforcement. The designer shall detail the base connections.

**Type 1 Connectors**

Type 1 connectors are used in structure and the reinforcing bars. They shall be capable of developing 100 percent of the specified yield stress when the reinforcing bars are connected. Type 2 connectors shall be used in special moment frames that are part of the structure and reinforced concrete structure. The designer shall detail the base connections. Details of mechanical reinforcing bar connectors shall be selected based on the use conditions and the limits set for ultimate load capacity of the reinforcement. The designer shall detail the base connections.

1.6.3.5

The basis of this article in the ACI Building Code features a seminar (ACI 335-14, Chapter 1—Earthquake Resistant Structures, of this code includes procedures for "Special Moment Frames and Special Structural Walls." These structures are essentially a type of reinforced concrete structure. Mechanical connectors in special moment frames must be tested and approved for use in those structures. The designer shall detail the base connections. Details of mechanical reinforcing bar connectors shall be selected based on the use conditions and the limits set for ultimate load capacity of the reinforcement. The designer shall detail the base connections.

**Type 2 Connectors**

Type 2 mechanical connectors are used in structure and the reinforcing bars. They shall be capable of developing 100 percent of the specified yield stress when the reinforcing bars are connected. Type 2 connectors shall be used in special moment frames that are part of the structure and reinforced concrete structure. The designer shall detail the base connections. Details of mechanical reinforcing bar connectors shall be selected based on the use conditions and the limits set for ultimate load capacity of the reinforcement. The designer shall detail the base connections.

1.6.3.6

The basis of this article in the ACI Building Code features a seminar (ACI 335-14, Chapter 1—Earthquake Resistant Structures, of this code includes procedures for "Special Moment Frames and Special Structural Walls." These structures are essentially a type of reinforced concrete structure. Mechanical connectors in special moment frames must be tested and approved for use in those structures. The designer shall detail the base connections. Details of mechanical reinforcing bar connectors shall be selected based on the use conditions and the limits set for ultimate load capacity of the reinforcement. The designer shall detail the base connections.

**GROUTED DUCT CONNECTIONS**

GROUTED DUCT CONNECTIONS are used in structure and the reinforcing bars. They shall be capable of developing 100 percent of the specified yield stress when the reinforcing bars are connected. Type 2 connectors shall be used in special moment frames that are part of the structure and reinforced concrete structure. The designer shall detail the base connections. Details of mechanical reinforcing bar connectors shall be selected based on the use conditions and the limits set for ultimate load capacity of the reinforcement. The designer shall detail the base connections.

1.6.3.8

The basis of this article in the ACI Building Code features a seminar (ACI 335-14, Chapter 1—Earthquake Resistant Structures, of this code includes procedures for "Special Moment Frames and Special Structural Walls." These structures are essentially a type of reinforced concrete structure. Mechanical connectors in special moment frames must be tested and approved for use in those structures. The designer shall detail the base connections. Details of mechanical reinforcing bar connectors shall be selected based on the use conditions and the limits set for ultimate load capacity of the reinforcement. The designer shall detail the base connections.

**UHPC Joint**

The UHPC Joint 2.6.4.3—Type 1 Mechanical Connectors

The basis of this article in the ACI Building Code features a seminar (ACI 335-14, Chapter 1—Earthquake Resistant Structures, of this code includes procedures for "Special Moment Frames and Special Structural Walls." These structures are essentially a type of reinforced concrete structure. Mechanical connectors in special moment frames must be tested and approved for use in those structures. The designer shall detail the base connections. Details of mechanical reinforcing bar connectors shall be selected based on the use conditions and the limits set for ultimate load capacity of the reinforcement. The designer shall detail the base connections.
**Grouted Duct Connections**

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

\[ l_a \geq \frac{f_y}{f_y} \frac{2E_h}{E_h} \left( 3.8.5.5(1) \right) \]

where:
- \( l_a \) = anchored length of longitudinal reinforcing bars into a pipe cap or footing (in)
- \( f_y \) = yield strength of the longitudinal reinforcement (ksi)
- \( f_y' \) = nominal compressive strength of grout (ksi)

Note: There are additional requirements for seismic applications.

---

**Pocket Connections**

Pocket connections may be used to connect columns with a forming member placed at the interface. This method is often used in seismic design to transfer shear and moment to adjacent columns. Typically, each connection is designed to provide the required shear and moment transfer to the adjoining columns.

Note: The AASHTO LRFD ABC Guide Spec has extensive design requirements for these connections.

---

**Pile Cap Socket Connection**

The cylindrical shell area of the socket, \( A_s \), shall be calculated as:

\[ A_s = \pi d_h^2 \times h \] (3.6.5.6-2)

where:
- \( d_h \) = inside diameter of the CMP socket (in)
- \( h \) = effective height of the CMP socket (in)

The effective length of the pocket or socket connections shall be based on the distance of the embedded part, accounting for the pile embedded length and the distance from the top of the pocket to the top of the embedded part.

---

**Pockets and Sockets**

- **Figure C3A.4-1**—Pocket Connection
- **Figure C3A.3-1**—Examples of Socket Connection

---

**Pile Cap Socket Connection**

- **Figure C7A.3-2**—Corrugated Steel Pipe Pocket Vent end in Integral Drummer form

---

**Pile Cap Socket Connection**

- **Figure C7A.3-1**—Corrugated Steel Pipe Pocket

---

**Pocket Connections**

- **Figure C3A.4-1**—Pocket Connection

---

**Pile Cap Socket Connection**

- **Figure C3A.4-1**—Pocket Connection
Part 5:
This Part Covers:
• Design Example:
  • Link Slab

Section 3: Link Slab Connections

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 expansion.

In 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 from the design of normal deck reinforcements. The design of the transverse reinforcement should account for the overall forces including bar, beam, and slab forces.

Section 3: Link Slab Connections

The research conducted on integral endplate pile connections (Wiget, J. J., et al., 2000) 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 corrugations creates. 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 yield resistance of a 24-inch diameter CIP pocket filled with 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.
Section 3: Link Slab Connections

The design bending moment in the link slab is the sum of the moments in the slab due to the live load and the cracking moment in the slab. The moment at any section of the slab is calculated as follows:

\[ M = M_{LL} + M_{cr} \]

where:
- \( M_{LL} \) is the moment due to the live load
- \( M_{cr} \) is the moment due to the cracking moment

The design bending moment can be derived from the moment of inertia of the slab at each point, which is obtained from the cross-sectional properties of the slab. The moment of inertia for a rectangular section is given by:

\[ I = bh^3 / 12 \]

where:
- \( b \) is the width of the slab
- \( h \) is the height of the slab

The design bending moment can be derived from the theory that the slab is loaded at each end of the link slab. A rotation is applied to the slab at each end resulting in a maximum bending moment at the final end. Figure 3.3.5.3-2 shows the free body diagram for this situation.

![Free Body Diagram](image)

Link Slab Design Approach

1. Design adjacent spans as simply supported.
2. Determine the length of the deformed link slab using approximately 5% of each span length.
3. Determine the end rotations of the girders from the beam design under service live loads.
4. Calculate the negative moment in the link slab due to service load rotations, \( M_a \), using the cross section properties of the link slab.
5. Calculate cracking moment of link slab, \( M_{cr} \), per Ref. 1, AASHTO 5.7.3.6.2.
6. If \( M_a \) - \( M_{cr} \) cannot be expected in the link slab, then additional reinforcement is required.
7. Design the reinforcement for the link slab to resist the applied moment using working stress methods and check the control of cracking criteria per AASHTO LRFD Article 5.7.3.4.
8. Use a gamma value, \( 
\gamma \), of 0.75 for Class 2 exposure condition.
9. The tensile stress in the reinforcement, \( f_{ys} \), shall not exceed 0.6fy.

Link Slab Example

- 3 Span Bridge – 2 Link Slabs at the piers
  - Spans = 113.39’-77’-86.57’
  - Beam Spacing = 7.67’
  - Deck Thickness = 8’
  - Fc = 4 ksi

Link Slab Formulas:

- Id = \( b * h^3 / 12 \)
- Ec = \( 33,000 * w^{1.5} \times \sqrt{f’c} \)
- \( M_a = 2 * Ec * Id * \theta / L \)

Modulus of Rupture Formulas:

- \( f_r = 0.24 \times \sqrt{f’c} \)
- \( y_t = d / 2 \)
- \( M_{cr} = f_r * Sd = f_r * Id / y_t \)

Crack Control Formulas:

- \( n = E_s / E_c \)
- \( \rho = A_s / ( \text{bar spacing} \times d_{sy} ) \)
- \( k = \sqrt{2*\rho*n + (\rho*n)^2} - (\rho*n) \)
- \( j = 1 - k/3 \)
- \( f_{ss} = M_a / ( A_s * j * d_{sy} ) \)

Design Link Slab Reinforcement

Span number
Reinforcement design required?
Link slab top bar size, \( n \)
Link slab top bar spacing, in
Cover to face of bar, in
Distance to bar center, \( d_c \), in
Beta factor, \( \beta_s \)
Bar effective depth, \( d_{sy} \), in
Modular ratio, \( n \)
Ratio of tension reinforcement, \( \rho \)
k-value
j-value
Stress in bar, \( f_{ss} \), ksi
Bar yield stress, \( f_y \), ksi
Exposure factor, \( \gamma_e \)
Max. bar spacing, in
Actual bar spacing, in
Part 6:

This Part Covers:

- Design Example:
  - Precast Full-Depth Deck Panel
  - UHPC Connections

Section 3: Full Depth Deck Panel Design

Precast concrete full-depth deck panels can be used to upgrade existing superstructure elements for load transfer or to replace entire superstructure spans. The panels are manufactured in a factory setting and then placed in position on the site.

Example: Precast FDDP Design

Reference:
- FHWA LRFD Design Examples

Figure 2-2: Equivalent Biaxial Equations for Various Parts of the Deck

Example: Precast FDDP Design

Reference:
- FHWA LRFD Design Examples

Figure 2-1: Superstructure Cross Section
Reference

• FHWA LRFD Design Examples
• Results

Closure Joint with UHPC:

- PCI Northeast recommended detail
- www.pcine.org
- Need: Lap length to complete the detail

Dev. Length = 8 \( d_b \)
Bottom Cover = 1.5" 
2\( d_b \) < min clr, Cover < 3\( d_b \)
Therefore add 2\( d_b \)
Total dev. Length = 10 \( d_b \) = 6.25"

Lap length with UHPC:
= 0.75 \( l_{du} \)
= 0.75 * 6.25" = 4.68" say 5"
Closure Joint with UHPC:
• PCI Northeast recommended detail
  • www.pcine.org

\[
W_2 = \text{specified joint width} = L_2 + T + 1.5\" = 5\" + 1\" + 1.5\" = 7.5\"
\]

\[
P_2 = 0.5(W_2 + L_2) + T = 0.5(7.5 + 5) + 1\" = 7.25\"
\]

Similar design for transverse closure joints

Shear connectors

Shear connector pocket options
• PCI Northeast details

This Part Covers:
• Design of Substructures
• Common Substructure Connections

Most common design approach
• Emulative design
  • The same as non-prefabricated element

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
Section 3: Couplers and Design

Sample Section Design with Couplers

The mechanical coupler will be larger than the connected bar. There are several manufacturers of mechanical couplers, however their is no standard guide to the detailing of concrete structures.

Example: Emulative Couplers Design

Example Calculations: Emulative Detailing of Reinforcement with Grooved Splice Couplers

Approach: Calculate equivalent beam sections.

Note: A Beam Section is shown for simplicity. A similar coupler calculation is more complex, but follows a similar approach.

Given: Section shown to the left below.

Factored applied moment = 5000 in-kips

Example: Emulative Couplers Design

Use CMP Socket Connections:

- Corrugations to create a corrugated void
- Similar to aggregate interlock in Interface Shear Applications

Section 3: CMP Socket Connections
Part 8:
This Part Covers:
• Design Example:
  • Integral Abutment

Example: Integral Abutment Design

The following figures show the proposed construction sequence:

Step 1:
Install piles
Set precast cap elements

Step 2:
Set Girders
Cast Deck
Note: Barrier rails not shown
Figure 12.1.2 - 4 Detail of CMP Void and Pile Tolerance

Table 12.1.1 - Specified Pile Width Tolerances

<table>
<thead>
<tr>
<th>Specified Pile Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 ft</td>
</tr>
<tr>
<td>5 ft</td>
</tr>
<tr>
<td>6 ft</td>
</tr>
</tbody>
</table>

Table 12.1.2 - Specified Pile Height Tolerances

<table>
<thead>
<tr>
<th>Specified Pile Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 ft</td>
</tr>
<tr>
<td>5 ft</td>
</tr>
<tr>
<td>6 ft</td>
</tr>
</tbody>
</table>

Table 12.1.3 - Specified Pile Diameter Tolerances

<table>
<thead>
<tr>
<th>Specified Pile Diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 ft</td>
</tr>
<tr>
<td>5 ft</td>
</tr>
<tr>
<td>6 ft</td>
</tr>
</tbody>
</table>

Table 12.1.4 - Specified Pile Orientation Tolerances

<table>
<thead>
<tr>
<th>Specified Pile Orientation</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 ft</td>
</tr>
<tr>
<td>5 ft</td>
</tr>
<tr>
<td>6 ft</td>
</tr>
</tbody>
</table>

Table 12.1.5 - Specified Pile Position Tolerances

<table>
<thead>
<tr>
<th>Specified Pile Position</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 ft</td>
</tr>
<tr>
<td>5 ft</td>
</tr>
<tr>
<td>6 ft</td>
</tr>
</tbody>
</table>

Table 12.1.6 - CMP Pipe Void Size: Tolerances

<table>
<thead>
<tr>
<th>CMP Pipe Void Size Tolerances</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance from outside edge of CMP to top of cap = CMP + 2D + 0.25</td>
</tr>
<tr>
<td>CMP + 0.25 + 0.50 = 0.75</td>
</tr>
<tr>
<td>Note: The assumptions for clear distances were chosen as small values in order to fit the CMP within the 0.50 width section. If larger tolerances were desired, the CMP would need to be larger and the steel width would need to be increased in order to accommodate the void. This would necessitate a redesign of the steel and pipe.</td>
</tr>
</tbody>
</table>

Figure 12.1.3 - Details of the final CMP void

Example: Integral Abutment Design

Figure 12.1.4 - Proposed Precast Integral Abutment Pipe Cap (stem)
Example: Integral Abutment Design

The actual load on the CFP wall is the dead load of the beams, deck, and abutment diaphragm. The load may be resisted by a column or pile if the capacity of the sole support is adequate. To verify this, the engineer must consider the following:

1. The capacity of the sole support
2. The load that the sole support is capable of supporting
3. The effect of the load on the sole support

To verify the capacity of the sole support, the engineer must calculate the load that the sole support is capable of supporting. This can be done using the following formula:

\[ P = \frac{wL^2}{8} \]

where:
- \( P \) is the load supported by the sole support
- \( w \) is the weight of the load
- \( L \) is the length of the load

Once the load supported by the sole support is determined, the engineer must verify that the load is within the capacity of the sole support. If the load is greater than the capacity of the sole support, the engineer must take corrective action to ensure that the load is within the capacity of the sole support. This may involve increasing the size of the sole support or reducing the load.

Example: Integral Abutment Design

The FEMAX design example treated the cap as a simple span. The resulting positive moment caused negative shear. To improve the design, a positive moment is applied to the negative moment section. A consistent beam design would give better results.

In re-design, the positive moment will be calculated using moment diagrams. The negative moment caused by shear is superimposed on the positive moment calculated using moment diagrams. This can be accomplished using the free body diagram below.

Example: Integral Abutment Design

The FEMAX design example treated the cap as a simple span. The resulting positive moment caused negative shear. To improve the design, a positive moment is applied to the negative moment section. A consistent beam design would give better results.

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Example: Integral Abutment Design

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In re-design, the positive moment will be calculated using moment diagrams. The negative moment caused by shear is superimposed on the positive moment calculated using moment diagrams. This can be accomplished using the free body diagram below.

Example: Integral Abutment Design

Step 4: Design of Integral Abutment Diaphragm

The FEMAX example design uses a post-tensioning concrete beam to resist the load of the pile cap and the integral diaphragm. The maximum positive moment is calculated assuming the beam reaction is applied at mid-span between piles taking 80% of the single pile moment.

Note that this approach is conservative since it does not take into account the dead load transferred into the cap beam below the composite action between the cap beam and the diaphragm.
Example: Integral Abutment Design

Check compression control resistance factor based on concrete strain (55, 3.4.2.4)

\[ \begin{align*}
\epsilon & = 0.003/\text{yr} \times \text{cycle} \\
\epsilon & = 0.003 / 32 = 0.00009375 \\
\epsilon & = 0.003 / 32 = 0.09375 \\
\epsilon & = 0.0005176 = 0.005 \text{ OK Tension Controlled Section} \theta = 0.90
\end{align*} \]

Therefore, CIP diaphragm can resist passive earth pressure with existing reinforcement.
Example: Integral Abutment Design

Reverse Engineer FHWA LRFD Design Example

In general, the change of a CIP cantilever abutment to precast concrete involves the connection of the wall stem to the footing. The use of a precast cap beam is also optional. This is done to keep reinforcing above the bridge seat the same.

The abutment is a standard cantilever abutment supported on spread footings. The proposed re-design involves the re-design of the following:

<table>
<thead>
<tr>
<th>Stem</th>
<th>Proposed beam connection type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Stem</td>
<td>Connection to footing by placing the wall stem on temporary supports with pin-ended, reinforcing, dowels, and using tying bars or ties going through the stem.</td>
</tr>
<tr>
<td>Bridge-wall in lieu of cap element</td>
<td>Concrete, Metal, etc. with reinforcing dowels</td>
</tr>
</tbody>
</table>

Parameter 1: Keep reinforcing above bridge seat the same

Parameter 2: Detail a cap element with all the complexities of the bridge seat

- One complex element
- Lower elements are simple and less expensive
- The cap element will be the lower elements together
- Moment demand on cap connection is minimal, which makes design easy and inexpensive
- Connect the two using a pocket connection
- Based on PCI Northeast Recommended Details

Example: Cantilever Abutment Design

Upper portion concept
Example: Cantilever Abutment Design

Calculation of forces: All forces are calculated on a "per foot of wall" basis.

Grout Reaction:
[Equation 7-54 of the FHWA LRFD Design Example [75]]

\[ R_{G,\text{reaction}} = 1.65 \, \text{kF} \]

\[ R_{G,\text{total}} = 2.20 \, \text{kF} \]

Active Soil Reaction (see page 7-51 of the FHWA LRFD Design Example [75])

\[ p = 4.39 \, \text{lb/ft}^2 \]

\[ w_e = 1.5 \, \text{lb/ft}^2 \]

\[ w_o = 0.5 \, \text{lb/ft}^2 \]

\[ w = 0.75 \, \text{lb/ft}^2 \]

\[ p = 2.00 \times 12.0 \times 1.0 = 24.0 \, \text{kF/ft} \]

\[ R_{G,\text{reaction}} = 1.65 \, \text{kF} \]

\[ R_{G,\text{total}} = 2.20 \, \text{kF} \]

\[ M = \text{horizontal and vertical force acting on wall} \]

\[ 3.22 \, \text{lb/ft}^2 \]

\[ 1.19 \, \text{lb/ft}^2 \]

Example: Cantilever Abutment Design

CME Load Calculation:

\[ H = \text{Height of soil} \times \frac{1}{6} \, \text{of slab} \]

Use AASHTO Table 3.9.2.1.1 to obtain equivalent height of soil:

<table>
<thead>
<tr>
<th>Equivalent Height (ft)</th>
<th>( h ) (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>4.8</td>
</tr>
<tr>
<td>4</td>
<td>4.8</td>
</tr>
<tr>
<td>5</td>
<td>4.8</td>
</tr>
</tbody>
</table>

Therefore, for \( h = 4.8 \) ft: \( h_{\text{eq}} = 5.7 \) ft

Change in earth pressure due to LL surcharge:

\[ \Delta L = P_{LL} \times (h_{\text{eq}} - h) \]

\[ \Delta L = 5.7 \times (5.7 - 4.8) \]

\[ \Delta L = 9.6 \, \text{kF} \]

\[ R_{w} = \text{force due to live load surcharge} \]

\[ 5.9 \, \text{kF} \]

\[ 1.23 \, \text{kF} \]

Example: Cantilever Abutment Design

Dead load of pier cap:

\[ P_{D,\text{dead}} = \text{Dead Load} \times \text{Length of Pier Cap} \]

\[ 77.6 \, \text{kF} \]

\[ 218.7 \, \text{kF} \]

Calculate Dead Load Moment:

\[ V_\text{dead} = P_{D,\text{dead}} \times \text{Length of Pier Cap} \]

\[ 440.4 \, \text{kF/ft} \]

\[ M_{\text{dead}} = V_\text{dead} \times \text{Length of Pier Cap} \]

\[ 832,780 \, \text{kF/ft} \]

\[ 1 \, \text{mi} = 5280 \, \text{ft} \]

\[ 440.4 \, \text{kF/ft} = 832,780 \, \text{kF/ft} \]

\[ 1 \, \text{mi} = 5280 \, \text{ft} \]

\[ 440.4 \, \text{kF/ft} = 832,780 \, \text{kF/ft} \]

Calculation of Dead Loading Moment:

\[ V_\text{dead} = P_{D,\text{dead}} \times \text{Length of Pier Cap} \]

\[ 440.4 \, \text{kF} \]

\[ 1 \, \text{mi} = 5280 \, \text{ft} \]

\[ 440.4 \, \text{kF} = 832,780 \, \text{kF} \]

Example: Cantilever Abutment Design

Footing (pile cap) options:
- Precast with CIP pile sockets:
  - Difficult to detail sockets
  - Difficult to manage pile tolerance
  - Reinforcement layout can be problematic
- CIP Concrete
  - Easy to detail
  - Accommodates pile tolerances
  - Almost as fast

Stem Design:
- If CIP is filled, the design is unchanged

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Example: Cantilever Abutment Design

The construction sequence for this concept is as follows:
1. Install the pilings
2. Set the stem elements to line and grade supporting the stem elements on the temporary integral posts. Shim under the posts to provide the proper grade.
3. Install the footing reinforcing and form the footing.
4. Cast the footing and cure the concrete until it attains adequate strength to support the stems.
5. Fill the CMP void with concrete. Fill the void to the elevation of the bottom of the cap dowels.
6. Grout or concrete the vertical connections between the stem elements.

Example: Pier Bent Design

This example:
• Use FHWA LRFD Design Example as basis
• Use emulation for the design
• Detail connections with grouted couplers
  • Place outside of the hinge zone
• Analysis and design of the bent is the same as CIP

Example: Pier Bent Design

Column design
• Same as CIP

Detailing with couplers
• Larger diameter than bars
• Potential conflicts with cap reinforcing

Design
• Type 1: Meet AASHTO requirements
  • Type 1: Meet AASHTO requirements for mechanical splices (125% Fu)
• Type 2: Strong as bar (100% Fu)

Example: Pier Bent Design

CIP column
• Same as CIP

Trial precast column with coupler
• Use fewer larger bars
• Easier to connect
• More cost efficient
Example: Pier Bent Design

Column capacity check
• Centroid of bars is moved inward
• Use column interaction design approach
• Red lines are the interaction diagram for the revised design
• Pink dots are the column demands
• By inspection, the new column is adequate

Example: Pier Bent Design

Precast cap
• The only change is the layout of bars in the bottom mat of longitudinal steel to make room for four couplers
• Use 2 cap elements to facilitate construction
• Detail connection between two cap elements as a reinforced closure joint

Example: Pier Bent Design

Precast footing
• The only change is the layout of bars in the top mat of reinforcing steel to make room for four couplers
• Use flowable fill or grout to seat footing

This example shows the high performance of the column. The column design is based on the interaction diagram for the revised design. The revised interaction diagram shows the new column capacity. The new column is adequate for the design.

Part 11:

This Part Covers:
• Other ABC Technologies
• Deck Beams
• Modular Beams
• Railings and Barriers
• Steel Beams
• Seismic Detail Overview
• Miscellaneous Provisions

Section 3: Adjacent Precast Deck Beam Design

Adjacent deck beams are fabricated in a controlled environment. The beams are designed to minimize the effects of concrete shrinkage and temperature variations. The beams are prestressed to transfer the loads to the support structures. The beams are designed to be supported on beams or by other means.

Section 3: Modular Deck Beam Design

Modular deck beams (MDBs) are also referred to as Modular Steel Deck Beams (MSDB). The modular deck beam is a composite beam that is fabricated in a controlled environment. The modular deck beam is designed to be used in conjunction with other modular or precast elements. The modular deck beam is used to transfer the loads to the support structures. The modular deck beam is also used to provide a smooth surface for the deck system.
Section 3: Precast Railing Design

Provisions are based on FHWA Implementation Guide

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

Section 3: Steel Beam Connections

- The AASHTO LRFD Bridge Design Specification requires that the AASHTO Bridge Design Subcommittee consider the preliminary design of the member. The AASHTO Bridge Design Subcommittee can be used for the preliminary design of the member.

- The AASHTO Bridge Design Subcommittee can be used for the preliminary design of the member.

- Several components of the bridge are required to be designed for the preliminary design of the member.

- The AASHTO Bridge Design Subcommittee can be used for the preliminary design of the member.

Section 3: GRS/IBS Design

Provisions are based on FHWA Implementation Guide

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

Section 3: ABC Seismic Design

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

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

- Several components of the bridge are required to be designed for the preliminary design of the member.

- The AASHTO Bridge Design Subcommittee can be used for the preliminary design of the member.

- The AASHTO Bridge Design Subcommittee can be used for the preliminary design of the member.

Section 3: Seismic – Grouted Ducts

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

- Several components of the bridge are required to be designed for the preliminary design of the member.

- The AASHTO Bridge Design Subcommittee can be used for the preliminary design of the member.

- The AASHTO Bridge Design Subcommittee can be used for the preliminary design of the member.

- The AASHTO Bridge Design Subcommittee can be used for the preliminary design of the member.
Section 3: Seismic Socket Connection

Corrugated Column End

- Based on FHWA Highways for LIFE Research in Washington
- No reinforcing passing from column into footing
- Good for pile supported foundations

\[ P_m < \frac{H_p}{D_c + G_l d + C + G} \]

Section 3: Seismic Socket in CIP Footing

- If \( L_e \geq 1.5D_c \), intentionally roughening (0.25") is required
- Concrete placed against a clean concrete surface
- Except \( \mu \) is limited to 0.5 when \( L_e \leq 1.1D_c \)

Section 3: Seismic Socket in CIP Footing

- Figure 3.6.7.2-1 Placement of Fitting Bars with a Socket Connection

Section 3: Seismic Socket in a Drilled Shaft

- Figure 3.6.7.1-1 Column-in-Shaft Longitudinal Bar Arrangement

Section 3: Seismic Socket in a PC Element

- Figure 3.6.7.4-2-1 Details of Pipe Cap with Socket

Other Provisions in the Guide Spec

- Detailing Requirements
- Durability of ABC Technologies
- Construction Specifications
  - Temporary Works
  - Fabrication and Assembly Planning
  - Layout and Tolerances
  - Concrete Structures
  - Steel Structures
  - Geosynthetic Reinforced Soil–Integrated Bridge System
Conclusions
The AASHTO LRFD ABC Guide Specs cover all aspects of typical ABC Design and Construction:
- Loads and Load Factors
- Element Design
- Connection Design
- SPMT and Lateral Slide Systems
- GRS-BBS
- Construction Specifications
You have all the tools required to design an ABC project
- Let's go build something

Questions?